AN INVESTIGATION OF THE

LOAD CARRYING ABILITIES

OF

PRE-STRESSED CONCRETE COLUMNS

A THESIS

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Responsibility, for the shortcomings and possible errors in this thesis, is exclusively that of the author.

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A B S T R A C T

Prestressed concrete columns are not able to sustain axial loads as large as those that can be carried by equivalent reinforced concrete columns. The equivalent reinforced concrete column is roughly ten per cent more efficient in this regard. If the column load becomes eccentric to the vertical axis the efficiency of the prestressed column will increase and eventually surpass that of the reinforced concrete column.

From the tests discussed herein a tentative formula has been suggested for predicting the ultimate load of prestressed concrete columns with $\frac{L}{d}$ of 25. The optimum steel percentage for prestressed columns was found to be approximately one percent and the use of column ties or spirals could increase the load carrying capacity of a prestressed column from five to ten percent.

The prestressed concrete column though giving up load carrying efficiency, for axial loads, to the reinforced concrete column, has better resiliency, is easier to handle and when precast can be cheaper to produce.

INTRODUCTION

The fundamental value of prestressing (pre-compressing) a concrete element is to improve its tensile resistance and ductility. The best advantage of prestressing would therefore obtain to structural concrete members where tensile stresses are predominant. The generally recognized conception is that there is nothing to be gained in applying this process to structural members carrying loads which induce compressive stresses.

The foregoing generalization though basically true, is subject to several positive exceptions from both a theoretical structural and economic point of view.

First of all it is relatively impossible to produce a concrete column without initial curvature or without heterogeneity of the materials from section to section. It is equally unlikely to achieve direct axial loading. Bending occurs at the first application of axial load and therefore pre-stress could be expected to be of some value in offsetting this condition. Certainly in the handling of pre-cast members there is need for resistance to tensile stresses.

There has been some discussion in the proceeding of the American Concrete Institute, that during long term loading of reinforced concrete columns much of the load is transferred to the steel as the concrete creeps. The ultimate failure of these columns may be, in part, due to buckling of the steel bars. In a prestressed column this is an unlikely phenomenon.

Pre-stressed columns are much more resilient than reinforced concrete columns and can recover from minor failures due to overloading without affecting their future ability to carry the working load. They are thus more reliable under impact loads.

Many compression members are from time to time subjected to transverse loads at which time they actually become flexural members. In this case the same advantages as in prestressing a beam would apply.

According to T. Y. Lin (1)* in his book entitled "Design of Prestressed Concrete Structures", prestressing a compression member actually reduces its deflection under transverse loads and that a prestressed pylon could be about 2.5 times as stiff as a reinforced one whose deflection is based on a cracked section. Reducing the deflection at the top of a column would minimize relative movements between building floors and thus effect a saving in the materials to strengthen other parts of the building.

Finally, there may be an advantage to prestressing from a production point of view. In plants that are set up for prestressing, the prestressed columns may be cheaper to produce than reinforced concrete columns and the difference in efficiency would thereby be compensated.

* Number in parenthesis refer to corresponding numbers in Bibliography

GENERALIZATION

The two major considerations for determining the load carrying ability of a column are the compressive strength of the material of which the column is made and the buckling behavior of the column. The part that these two factors play in the ultimate failure of a column is dependent upon its slenderness ratio.

The stresses ordinarily allowed in reinforced concrete column would not be applicable to pre-stressed concrete columns because the compressive stresses due to pre-compression do not contribute appreciable to buckling. The internal load of the prestressed column indirectly has some effect on buckling because it will alter the elastic modulus of the concrete.

To assess the value of prestressing a column and in part to justify this series of tests, it is imperative to examine the probable behavior of a prestressed column.

In short columns where the compressive strength of the material becomes the all important factor pre-stressing would appear to have no value for resisting pure axial load. Prestressing, in this instance, will probably be detrimental in that the internal load will have sapped some of the available compressive strength before the external load is applied (e.g. when the concrete reaches its ultimate strain value about 25% of the prestress will still remain).

Intermediate and long columns where buckling becomes a problem and initial curvature could produce bending stresses, may gain some real advantage from having any possible tension stresses relieved by prestressing.

R. A. Brechenridge at the University of Southern California (2) concluded, after several tests, that, the buckling strength of very slender columns is not reduced by prestressing.

Similar tests to those described in this text were undertaken at the University of Florida in 1956, by A. M. Ozell and A. M. Jernigan (3). This work at the University of Florida was unknown to the author prior to the inception of this research. The similarity between these tests and those at the University of Florida was purely coincidental but fortunate in that it provides some basis for comparison.

Certainly, in order for designers to obtain a basic understanding of prestressed column behavior numerous and varied testing should be undertaken.

O B J E C T I V E

1. To determine the effect of L/d ratio with respect to the ultimate load carrying capacity of prestressed concrete columns.

2. To determine the effect of prestressing force on the ultimate load carrying capacity of a prestressed concrete column.

3. To determine the usefulness of lateral ties and in what locations they are most effective.

4. To draw a comparison between prestressed concrete columns, reinforced concrete columns and plain concrete columns.

D E S C R I P T I O N

Twenty-five full sized columns were tested to failure; all of them loaded axially except one which was loaded with a $1\frac{1}{2}$" eccentricity.

Because of the limits of the testing machine the column lengths were converted to a hinged-end condition. This was done to control the direction of buckling and was accomplished by the use of V-plates top and bottom(see fig.1)

Although the idealized end condition does not accurately represent connections found in practice, it was used in this investigation to obtain a member whose behavior under load might be more readily predictable from a theoretical approach. Any formulas that may be developed from these tests could be applied to any condition by incorporating a compatible multiplication factor.

There were two sets of tests. The first set was the pilot test consisting of 10 specimens which varied as to slenderness ratio and use of lateral ties.

There was a minor variation in the amount of prestress. (see fig. 2)

The second set of 15 specimens were all of equal cross-section and slenderness ratio. The amount of pre-stress was varied and these specimens also included some of reinforced concrete and some of plain concrete (see fig.3)

The following concrete mix design was used for both groups of specimens. The source of aggregate for the final tests was not the same as that for the pilot tests. However, they appeared to be basically similar.

1660#	3/4 stone ($\frac{3}{4}$ to $\frac{1}{4}$ graded)
1340#	sand F. M. 2.80
700#	Portland cement type plain
300#	water

All concrete specimens were low pressure steam cured an average of 13 hours and had the following cylinder strength at destress:

Ap = 4750 psi

Bp = 5000 psi

At = 4200 psi

Bt = 4100 psi

Ct = 4300 psi

Specimens Ap and Bp were stored outdoors at sub-zero temperatures in order to arrest the curing process and thereby restrict the compressive strength of the concrete to a workable level. Strengths of 6000 psi and 7000 psi, which would have otherwise developed, would have resulted in ultimate loads, for the large columns, beyond the capacity of the testing frame. The specimens were placed in the laboratory at normal room temperatures for at least 24 hrs. prior to testing, to ensure testing was performed at normal room temperature.

The prestressing steel consisted of British Wire Rope seven wire strands with an ultimate tensile strength of 251,560 psi and an average modulus of elasticity of 29×10^6 psi. The reinforced concrete columns contained #4 deformed bars with an ultimate tensile strength of 60,000 psi. #2 column ties were used throughout. For data on the test specimens refer to figures 2 and 3.

TEST PROCEDURE

All specimens were tested in a 200, 000# column testing frame with all loads recorded in psi of ram head pressure on the hydraulic loading jack which itself was accurate to ½ of 1% of its load. The gauge was graduated to 200 psi and could be read to the nearest 100 psi. Prior to testing, this gauge was calibrated in a 200,000# Riehle testing machine. Deflections and lateral movements of the columns were recorded to 1/100th of an inch on steel scales fixed to the column at the top, bottom and mid-height, and read by means of a transit.

Several of the specimens were outfitted with SR4 type A3 and A9 strain gauges.

Two standard concrete cylinders were stored with each specimen and their average strength at the time the column was tested was taken as the f'c of the specimen concrete.

The ends of the columns were capped with 5/8" buffalo board and even bearing of the V- plates was provided for as shown in fig. 1. The loading heads on the columns were positioned by means of set screws in the loading heads so that concentricity of the load was achieved to the nearest 1/16 of an inch.

MATHEMATICAL SYMBOLS & ABBREVIATIONS

a - Initial accentricity

Ac - area of gross concrete cross-section

As - area of steel cross-section

E - modulus of elasticity

Eq - modulus of elasticity of concrete based on Portland Cement Association formula where $Eq = 60,000 \sqrt{f'c}$

Es - modulus of elasticity of concrete from Southwell plot

Et - modulus of elasticity of concrete based on average tangent modulus

d - least dimension of column cross-section

f'c - average ultimate concrete cylinder stress

f cu - average ultimate concrete stress due to axial load

f'cu - maximum ultimate concrete stress in extreme fibre due to axial load plus moment due to deflection plus prestress

h - measured lateral deflection due to load P

L - effective length of column

p - As/Ac

P - axial load

P_p - axial load due to prestress

Peq - ultimate load based on Euler formula using Eq

Pes - ultimate load based on Southwell plot

Pet - ultimate load based on Euler formula using Et

Pu - actual ultimate load

S - section modulus

σb - stress due to bending

σc - stress due to axial compression (external load)

σp - stress due to prestress

DISCUSSION OF TEST RESULTS

Table I and Table II contain a summary of the test results. This information is taken from and based upon the test data which is contained in the appendix to this text.

The ultimate strength of the columns varied and in most cases their variation was considerable. These variations were due to differences in concrete cylinder strength, differences in intial curvature and heterogeneity of the concrete. The largest contributing factor appears to be the variation in initial curvature. In any group the column with the greatest lateral deflection failed at the lowest ultimate load.

Fig. 4 shows the plot of ultimate load versus the concrete steel ratio. The variation, which is shown here highlights the difficulty in obtaining a mean curve for these points. An attempt was made to analyze these results by normal methods of experimental analysis. The result was that all differences were significant for any reasonable degree of freedom and similarly an analysis of variance drew an unsuccessful conclusion because a curve could not be accurately fitted. Unfortunately there is an insufficient number of tests to make a useful statistical analysis of the results. For a useful portrayal of the results we were therefore obliged to follow other methods as described below.

Using an average ultimate load value for the various groups a straight line is obtained and shown in Fig. 4. This in itself is unsatisfactory because it does not relate the steel-concrete ratio to the actual stresses in the column. We therefore have Fig. 5, where the above condition is satisfied.

The three stresses involved while a column is being loaded are pre-stress, axial stress and bending stress due to lateral deflection.

As the latter two increase the former decreases. The average stress-strain curves of the concrete cylinders were used to tetermine the loss of prestress.

By trial and error method the maximum stress in the columns was calculated at succeeding stress levels and the elastic tangent modulus from the f'c curves (Fig. 6, 7, 8,) was determined. With this value the loss of prestress was calculated between the various points. These values were exceptionally high and in some cases approached zero before failure occurred. The approach was then modified and the average tangent modulus was used resulting in prestress losses that agreed closely with generally accepted values (approx. - 56%). Fig. 9, 10 and 11 contain the curves from which the fcu and f'cu values were taken. Loss of prestress was also calculated on the basis of Es and found to not be significantly different from that based on Et. In all cases the initial prestress load was taken to be 10% less than the actual load used to stress the strands.

To obtain a more practical approach to the problem it would be better to relate the actual ultimate loads to the Euler loads. Since no definite E value could be selected from the stress-strain curves of the concrete cylinders it was necessary to resort to a Southwell plot as described in " The Analysis of Structures", by Hoff Page 241. (4). (Southwell Plot, devised by Southwell, is an experimental determination of the Euler load. When low yield point, high buckling stress and large initial curvature continue to cause yielding of the material before the Euler load is reached)

The Southwell plots are contained on Fig. 12, 13, 14. In addition to this, Fuler loads wer calculated on the basis of average tangent modulus. A still more practical value for ultimate load could be based on the E value as found by Portland Cement Association formula $E = 60{,}000 \sqrt{f'c}$ and calculated through the Euler formula. The relationship of these values can be seen in Fig. 15. The actual load/strain relationship taken by SR-4 gauges are shown in Fig. 18 to 24 inclusive.

T H E O R E T I C A L   A P P R O A C H

In an effort to develop a basis for a design formula three alternatives were selected for the purpose of this thesis.

1. To fit a curve to information in Fig. 15
2. To adapt the test results to a method described in "The Analysis of Structure" by Hoff page 236 and shown here on Fig. 16. (Method of determining stresses in an initially crooked column axially loaded where the deviation from straightness is assumed to follow the half-sine wave pattern)
3. To adapt the test results to a method described in "Advanced Mechanics of Material" by Seely & Smith (5) page 597 based on the Engesser formula, contained in this text on Fig. 17, a, b and c.

CASE I

St. line variation - use $\frac{Pu}{Peq}$ $Pu = 0.675\ Peq - 8.6\ pFeq$

Reasons: 1. Critical load con be found simply without recourse to stress-strain curves or lateral deflection data which in the vast majority of cases is not available.

2. It corresponds quite closely to plot based on Pes which is very nearly true critical load.

Limitations:

1. There are several variables unaccounted for specifically; initial eccentricity being the most important.
2. Straight line variation has been assumed because of small number of tests.
3. True only for columns of $\frac{L}{d} = 25$ with an initial eccentricity of approximately 0.20" based on visual inspection.
4. From Fig.5 it appears that use of the formula should be restricted to steel percentages between 0.6% and 1.2% which

appears to be the range of the most favorable steel percentages.

CASE 2

The investigation of Case 2 obliged us to introduce a third term into the equation arrived at by Hoff. This third term is necessary to account for the pre-stress, the equation now being: $\sigma_{max} = \sigma_b + \sigma_c + \sigma_p$

$$\text{where } \sigma_b = \frac{M_{max}}{S} = \frac{aP}{S}\left[\frac{Pe/P}{Pe/P-1}\right]$$

$$\sigma_c = \frac{P}{Ac}$$

$$\sigma_p = \frac{Ppi}{A} \left[1 - \frac{\epsilon f - \epsilon i}{\epsilon i}\right]$$

$$\sigma_p = \sigma_{pi} \left[\frac{P + Ppf}{P + Ppi}\right] \begin{matrix}\text{-- final prestress} \\ \text{-- initial prestress}\end{matrix}$$

At any load P, Ppf may be calculated on the basis of an appropriate modulus of elasticity. The equation may be further simplified if the pre-stress at any load is approximated by the equation $\sigma_p = \left[0.90 - .50 \frac{P}{Peq}\right]\sigma_{pi}$

* (Derived from prestress curve shown on figures 9, 10, 11). The resulting equation therefore becomes :

$$\sigma_{max} = \left[\frac{aP}{S} \quad \frac{Peq/P}{Peq/P-1}\right] + \frac{P}{Ac} + \left[0.90 - .50\frac{P}{Peq}\right] \frac{Ppi}{Ac}$$

The extreme fibre stress may also be expressed in terms of f'c as related to the steel percentage in Fig. 5. We now have :

$$Kf'c = \frac{aP}{S} \left[\frac{Peq/P}{Peq/P-1}\right] + \frac{P}{Ac} + \left[0.90 \ .50 \frac{P}{Peq}\right] \frac{Ppi}{Ac}$$

where K is dependent on $\quad P = \frac{As}{Ac}$

* (An equation can be developed for curve of Fig. 5

The factor "a" requires an assumed value and will for the most part vary in accordance with the uniformity of prestress, the heterogeneity of the concrete and the method of casting.

Table III was set up for the purpose of obtaining a reasonable value for "a". The values in column 1 were calculated from the compromised Hoff equation in which was inserted the actual ultimate load values and extreme fibre stresses as found from the test specimens. That is, the "a" values satisfy the actual test specimens. Column 2 was calculated on the basis of the Timoshenko equation given as: $a = \frac{h}{P} Pcr - h.$

In both cases Pcr was taken as Peq.

Peq does not represent the true critical load but it is the most practical value readily available without the use of stress-strain curves etc.

As can be seen in Fig. 16 the use of Peq slightly alters the shape of the graph but for initial eccentricities above 0.15" produces a safe value for ultimate load.

TABLE III a Values

	1	2
$2A_T$	.09	.07
3AT	.148	0.24
$4B_T$	.177	0.22
$5C_T$	.124	0.22
$7C_T$	.075	0.12
$8B_T$	0	0.27

Using a = 0.20" the following test specimens calculate as follows:

TABLE IV

	P	f'cu	Pu	Compared to actual	
3At	.004	1.06f'c	122K	-4%	
4Bt	.0063	1.10f'c	130k	+5%	
5Ct	.0088	1.12f'c	114k	-5%	
8Bt	.016	1.02f'c	100k	-12%	
(3Ap	.0088	1.12f'c	126k	-11%)	Lateral ties
3Bp	.0088	1.12f'c	134k	-8%	

The above (TABLE IV) would indicate that the compromised Hoff equation yields reasonable results for the test columns and also for the columns of the pilot test not included in the original analysis. Actual variations in the initial eccentricity may account for differences.

With appropriately selected "a" and "k" factors this equation should apply to wide selection of $\frac{L}{d}$ values. It is not possible to verify this conjecture because there are not a sufficient number of tests in other $\frac{L}{d}$ ranges. The 8 x 4 columns of the pilot test cannot be predicted by this equation primarily because an approximation of the initial curvature cannot be made nor can it be verified that the extreme fibre stress varies only with steel percentage and is not also affected by $\frac{L}{d}$.

Further, this equation does not make allowance for the use of column ties. Column ties should increase the load carrying ability of the column by increasing the elastic modulus of the column concrete. A reasonable estimate of the increase in load by virtue of #2 ties at 6" o/c might be about 5%. This presumtion is made on the basis of the results in table I though differences in the initial eccentricity could account for the increase as well.

By restricting the lateral strain of the concrete it must by application of Poisson's ratio increase the elastic modulus of the material. It follows then by the Euler equation that an increase in the critical load must result. Full compensation will not be gained because increasing the elastic modulus will result in less loss of prestress due to the axial load.

The prestress strands do have some effect in "containing" the concrete but not nearly so much as the steel reinforcing bars. From table I we can see that the reinforced concrete column could resist extreme fibre stresses up to 10% higher than the pre-stressed columns.

We can also see from table I that, in comparing columns 2At and 3At, four strands are more effective than one strand where the steel percentage is the same.

It is reasonable to expect an optimum steel percentage to exist. This optimum will exist where the strand ability to "contain" the concrete and the existing pre-stress in the column combine to the greatest advantage.

From these tests it may be stated that the optimum steel percentage lies between .006 & .012.

In all the conclusions drawn here it must be understood that they are based on $\frac{l}{d} = 25$ and concrete cylinder strengths in the 5000 - 6000 psi range.

The SR-4 load-strain diagrams indicate that in all cases bending took place in two directions. This would place the altered Hoff equation on the side of safety because it has been considered for bending in one direction only.

CASE 3

Engesser's method, for the third case mentioned above, is based on the tangent modulus formula which is generally regarded as the maximum buckling load that a real column, having slight imperfections, can safely be expected to resist.

A stress strain curve is a basic requirement for this solution. The equation itself is:

$$\frac{P_T}{A} = \frac{\pi^2 E_T}{(L/r)^2} \quad \text{or} \quad P_T = \frac{\pi^2 E_T I}{l^2}$$

In Fig. 17 we have plotted the stress-strain and tangent modulus curves for column 5Ct. The tangent modulus curve was altered with respect to the prestress in the column so that the stress shown on the diagram represents the stress due to axial load only.

The ultimate axial load is obtained by selecting, through a process of trial and error, a set of stress/tangent modulus co-ordinates that will satisfy both sides of the above equation. Solution by this method is shown on Figs. 17a, 17b, 17c.

Besides the impracticability of the Engesser method it makes no allowance for steel percentages. The method assumes the detrimental effect of the prestressing only and therefore predicts a higher ultimate load in proportion to the decreasing amount of prestress. Finally it predicts a plain concrete column to carry the maximum ultimate load.

The information on Fig. 17 would indicate that the Engesser method may give reasonable values for prestressed columns with steel percentage of .01.

No graphical presentation of the load/deflection results from the tests has been included here. As can be seen from the test data, included in the app endix of this text, load/deflection did not follow any obvious pattern. Differences in initial eccentricity contributed to these erratic variations. The tied columns did not deflect laterally as much as the untied columns but we are unable to verify that the difference in initial eccentricity did not contribute to this condition.

The resiliency of prestressed concrete was aptly demonstrated in the tests of the columns having $\frac{L}{d} = 38$. These columns buckled without failing in compression and having a maximum lateral deflection of approximately 6". They developed a series of tension cracks that can be seen in the photographs at the end of this text.

When the load was released the columns recovered to within 1/2" of their original position. At this time they were supported at the mid-height and reloaded to approximately 165 kips before they failed in compression.

CONCLUSIONS

1. Because of limited number of tests the effect of $\frac{L}{d}$ ratio with respect to ultimate load in prestressed concrete columns cannot be established here.

2. From these tests it would appear that the steel percentage does affect the ultimate load carrying ability of a prestressed concrete column. The range of .006 to .01 values for p appears to be optimum. Further, the tests indicate the following equations for ultimate load design :

   (a) $Pu = 0.675\ Peq - 8.6\ p\ Peq$

   limited to columns of $\frac{L}{D} = 25$ and f'c between 5000 and 6000 psi.

   (b) $$Kf'_c = \frac{aPu}{S}\left[\frac{Peq/Pu}{Peq/Pu-1}\right] + \frac{Pu}{Ac} + \left[0.90 - 0.30\,\frac{Pu}{Peq}\right]\frac{Pr\,2}{Ac}$$

   limited to columns of $\frac{L}{D} = 25$ and f'c between 5000 and 6000 psi. There is a further suggestion that the equation is reasonably valid for $\frac{L}{d}$ values between 20 and 30. K varies according to Fig. 5 and a reasonable value for "a" appears to be 0.20.

3. Lateral ties are useful as can be seen from the included photographs however, their exact value cannot be implied from these tests. The suggested increase of ultimate load is about 5% where #2 ties are used at a spacing equal to the least dimension of the column cross-section.

4. Prestressed concrete columns carrying purely axial load are not as efficient as reinforced concrete columns. The stiffness of the reinforcing bars as opposed to the flexibility of the prestressing strand, "contains" the concrete to a greater degree thus increasing the modulus of elasticity of the unit. The preference of one type of column over the other can be established purely on an economic

comparison. Where eccentric loads are involved prestressed concrete columns may well hold the advantage. In these tests the extreme fibre stress of column 1Ap when loaded eccentrically was approximately 1.16 f'_c and that of the axial loaded reinforced concrete column was 1.16f'_c. Initial eccentricity has not been considered.

5. None of the above can be fully substantiated without further testing.

SUGGESTIONS FOR FURTHER INVESTIGATION

Failure of a prestressed concrete column results due to the combined action of the following variables:

slenderness ratio of the column

percentage of steel (amount of prestress)

ultimate compressive strength of concrete

ultimate tensile strength of steel

initial curvature of column

size and spacing of ties or spirals

For any immediate investigation it would seem advisable to limit the variables to the first two shown above.

Holding the latter four factors constant would not limit the results of any future tests because at present, these four factors are fairly standard throughout the industry. The ultimate tensile strength of prestressing strand falls within quite narrow limits. The ultimate compressive strength of the concrete is generally found in the 5000 psi to 6,000 psi range. Initial curvature should be fairly consistent for the production methods generally in use. For the present, tie and spiral, size and spacing can be made to conform to that generally used in reinforced concrete columns.

By reducing the variables to two it will be that much easier to arrive at a formula that will predict the failure of a prestressed concrete column.

The tests presented here indicate that the extreme fibre stress of the column shown as a percentage of f'_c falls within reasonably narrow limits for varying amounts of prestress (steel percentage). Future investigation may be well advised to pursue this further.

If for a certain range of slenderness ratio an ultimate fibre stress taken as a percentage of f'_c can be related to the steel percentage then failure of any column within that range of slenderness ratio can be predicted by calculating the stresses caused by all external and internal loads. This would permit the use of simple elastic theory to predict the failure of any prestressed concrete column.

It will be necessary, however, to select an initial curvature that will be representative of that found in columns manufactured by current methods of production.

For purely axial loads it appears that the reinforced concrete columns are more efficient than prestressed ones. However, for members loaded both axially and transversely it is logical to suspect that the prestressed column becomes the more efficient when the transverse load exceeds a certain percentage of the axial load. Investigation along these lines may prove worthwhile.

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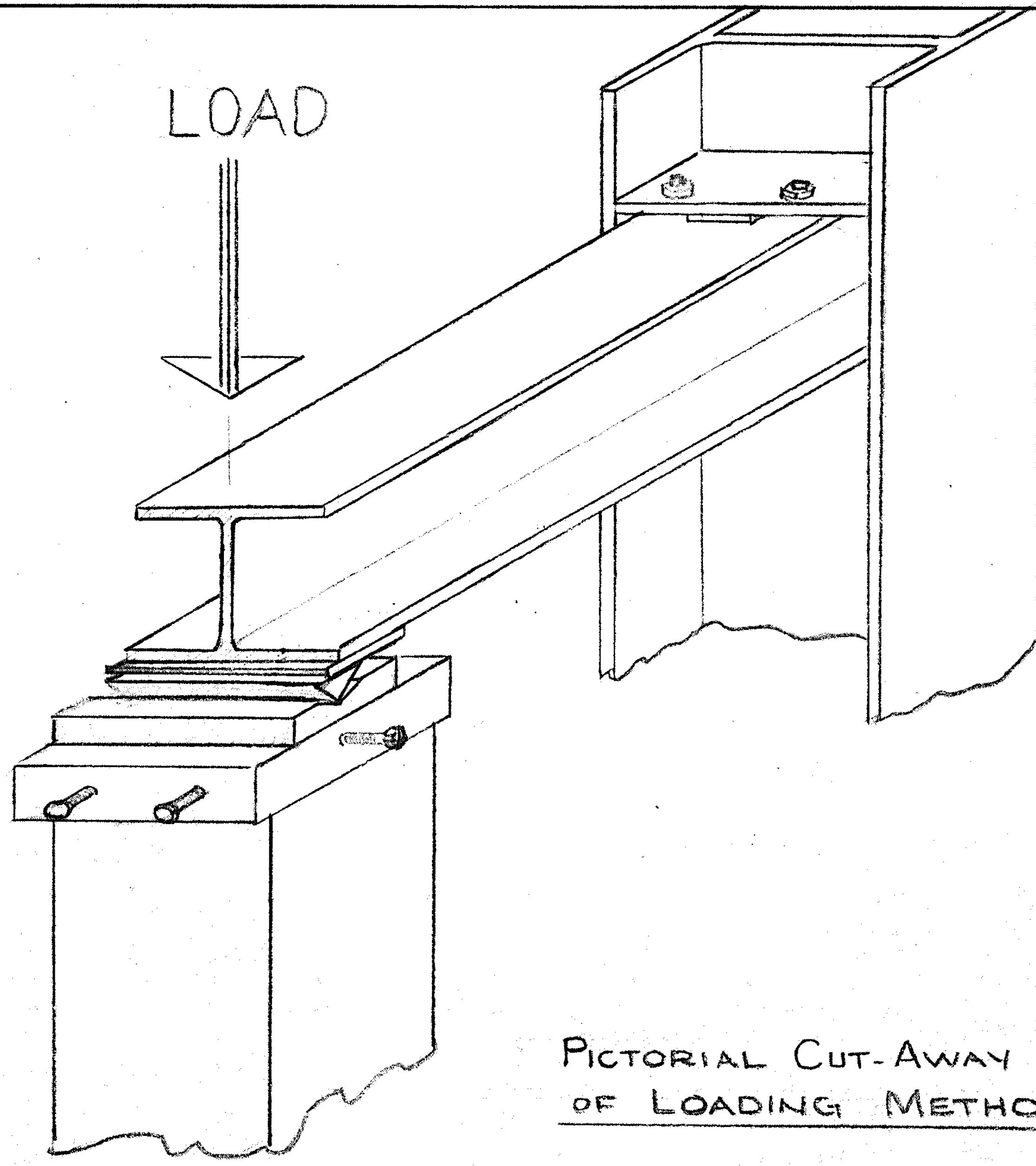
SUMMARY OF TEST RÉSULTS****** TABLE I

Column no.	detail	p	L/d	f'_c	Pre-stress initial	Pre-stress final	Failure	Pu	f_{cu}	f'_{cu}	f_{cu}/f'_c	f'_{cu}/f'_c	P_{et}	Pes	P_{EQ}	P_u/P_{et}	P_u/Pes	P_u/P_{EQ}
$1B_T$	6x6 plain		25	5600			B & C	143	4000		0.71							
$2A_T$	6x6 n.t. 1 - ½ inch strand	.004	25	5200	630	500	"	122 143	3680	5100	0.71	1.02	154 (3.0)	142 (3.07)	200 (4.34)	0.84	0.94	0.67
$3A_T$	6x6 n.t. 4-¼ inch strand	.004	25	5200	630	500	"	121 132	3520	5400	0.68	1.08	154 (3.0)	127 (2.75)	200 (4.34)	0.80	1.00	0.64
$4B_T$	6x6 n.t. 4-5/16 in. str.	.0063	25	5600	1020	820	"	132 116	3450	5650	0.62	1.13	166 (3.6)	133 (2.88)	208 (4.5)	0.75	0.93	0.60
$5C_T$	6x6 n.t. 4-3/8 in. Str.	.0088	25	5000	1380	900	"	121 124	3420	5500	0.68	1.10	129 (2.8)	140 (3.04)	196 (4.25)	0.95	0.88	0.63
$6A_T$	6x6 with ties 4-½ in. bars	.022	25	5200			"	155 155	4320	6000	0.83	1.16	162 (3.5)	167 (3.62)	200 (4.34)	0.93	0.93	0.78
$7C_T$	6x6 with ties 4-3/8 in. str.	.0088	25	5000	1380	850	"	135 131	3710	5450	0.74	1.09	129 (2.8)	150 (3.24)	196 (4.25)	1.03	0.89	0.68
$8B_T$	6x6 nt 4-½ in. str.	.016	25	5600	2540	1900	"	112 116	3180	5100	0.57	1.02	148 (3.2)	120 (2.6)	208 (4.5)	0.77	0.95	0.55
1Ap	8x8 #2 spiral 4-3/8 in. str.	.005	19				Comp. 1½" eccent.	190										

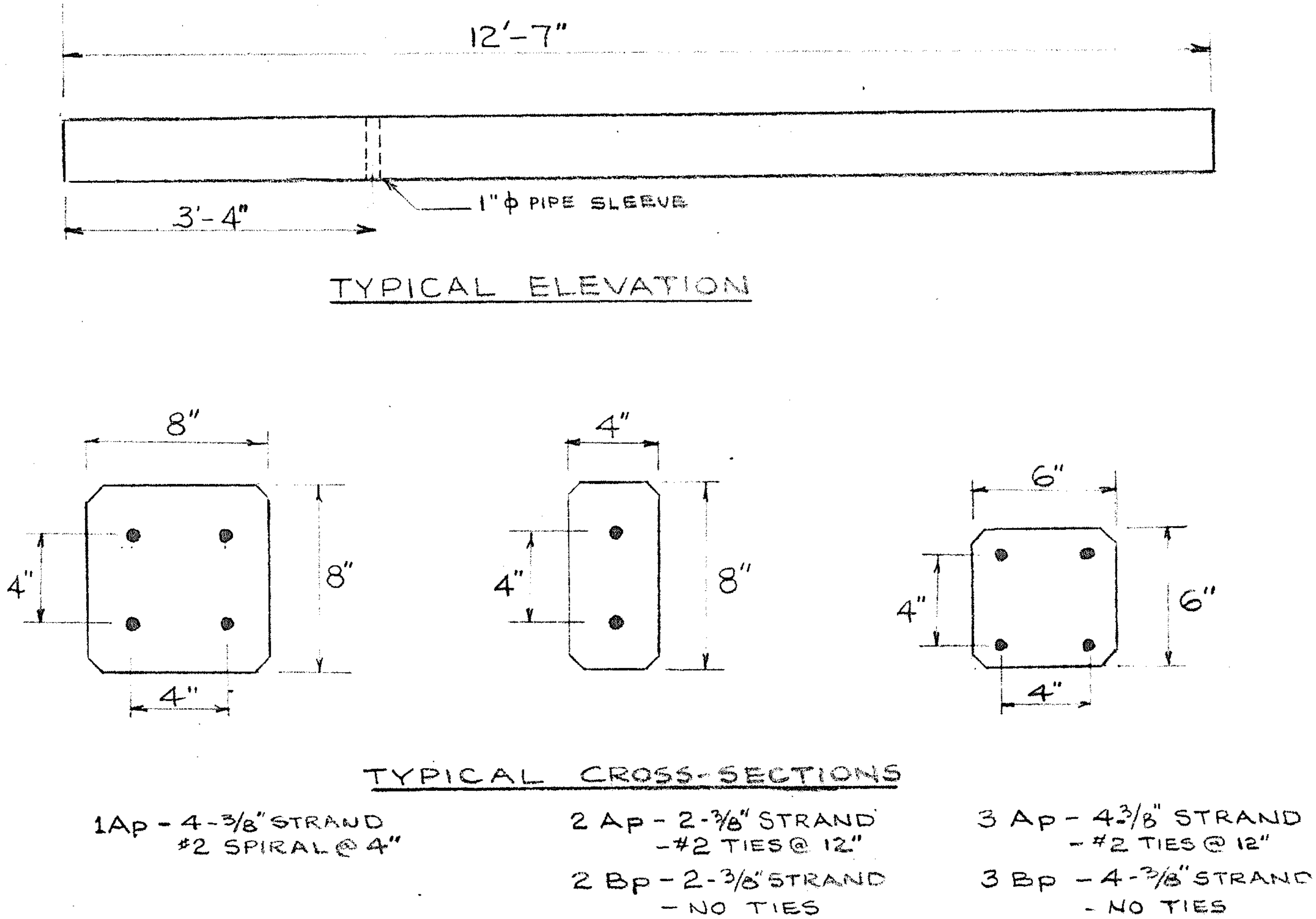
SUMMARY OF TEST RESULTS***** TABLE II

Column no.	detail	p	L/d	f'c	Pre-stress initial	Pre-stress final	Failure	P_u	f_{cu}	f_{cu}/f'_c	P_{EQ}	P_u/P_{EQ}	
1Ap	8x8 #2 spiral 4-3/8 in. str.	.005	19	5000	790		comp.	210	3280				bending due to misalignment of machine
1Ap	8x8 #2 spiral 4-3/8 in. str.	.005	19	6100	790		none	225	3520				
2Ap	8x4 with ties @ 12" 2-3/8 in. str.	.005	38	6400	790		buckle	36 51	1360	0.213	89	0.54	
2Bp	8x4 no ties 2-3/8 in. str.	.005	38	7000	790		buckled	64 64	2000	0.286	94	0.68	
3Ap	6x6 with ties 4-3/8 in. str.	.0088	25	5350	1380		B & C	145 139	3940	0.735	204	0.70	
3Bp	6x6 no ties 4-3/8 in. str.	.0088	25	5700	1380		B & C	165	4050	0.71	209	9.70	



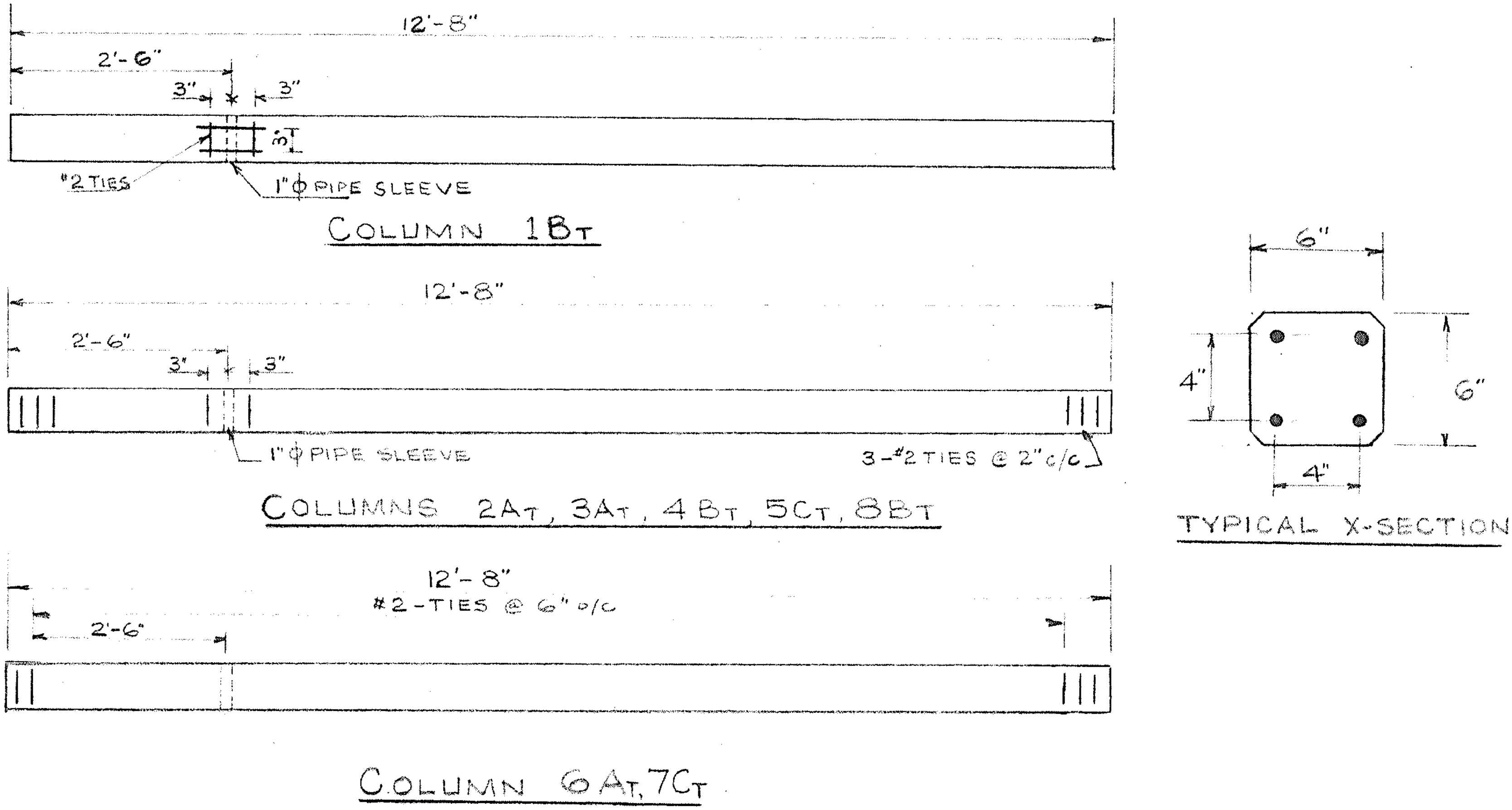
Pictorial Cut-Away View of Loading Method

fig. 1



PILOT TEST SPECIMENS

fig. 2



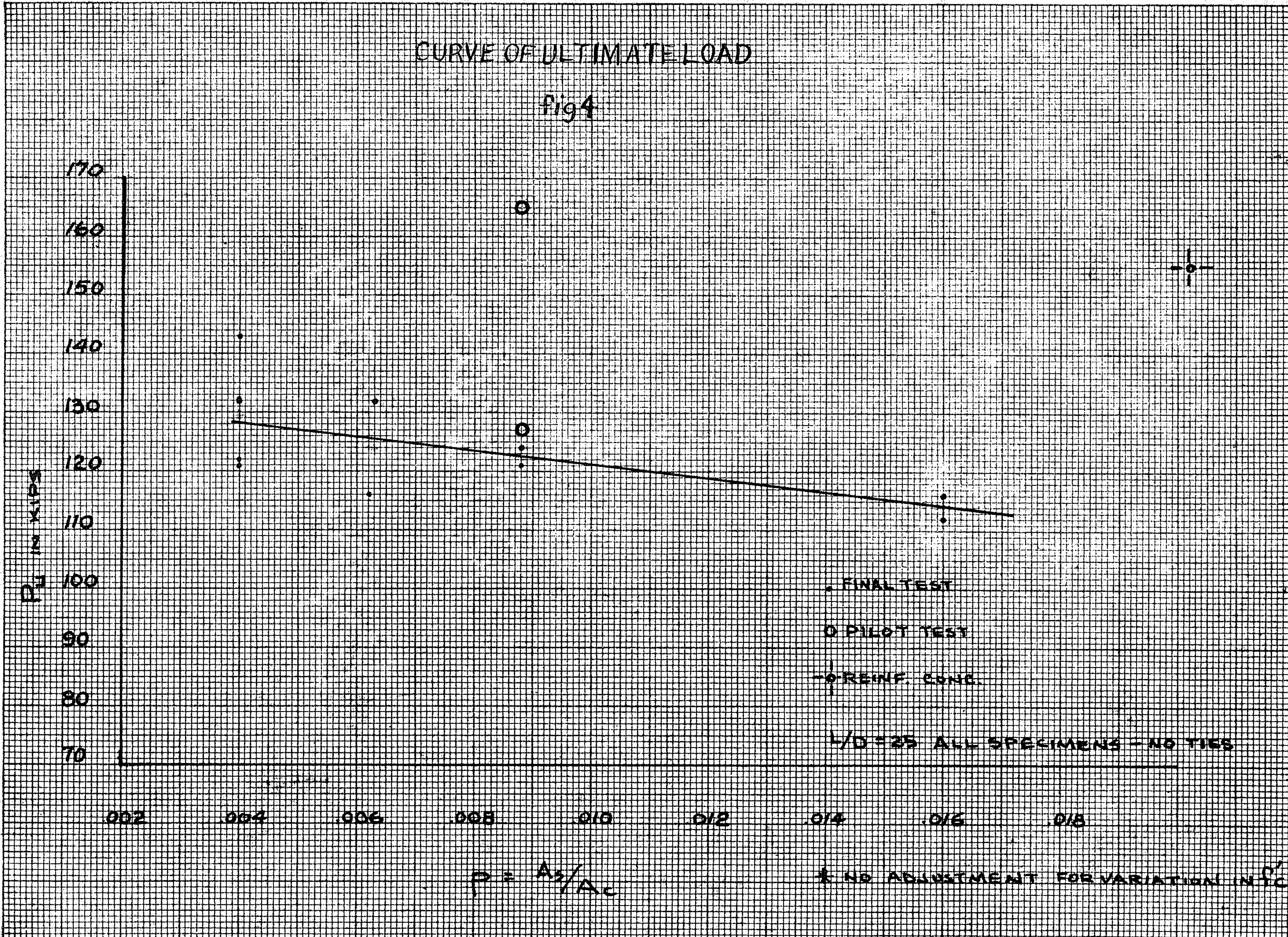
1B - PLAIN CONCRETE
2A - 1-½" STRAND
3A - 4-¼" STRAND
4B - 4-5/16" STRAND
5C - 4-3/8" STRAND
6A - 4-#4 BARS
7C - 4-3/8" STRAND
8B - 4-½" STRAND

TEST SPECIMENS

fig. 3

THIS MARGIN RESERVED FOR BINDING.

IF SHEET IS READ THE OTHER WAY (VERTICALLY), THIS MUST BE LEFT-HAND SIDE.



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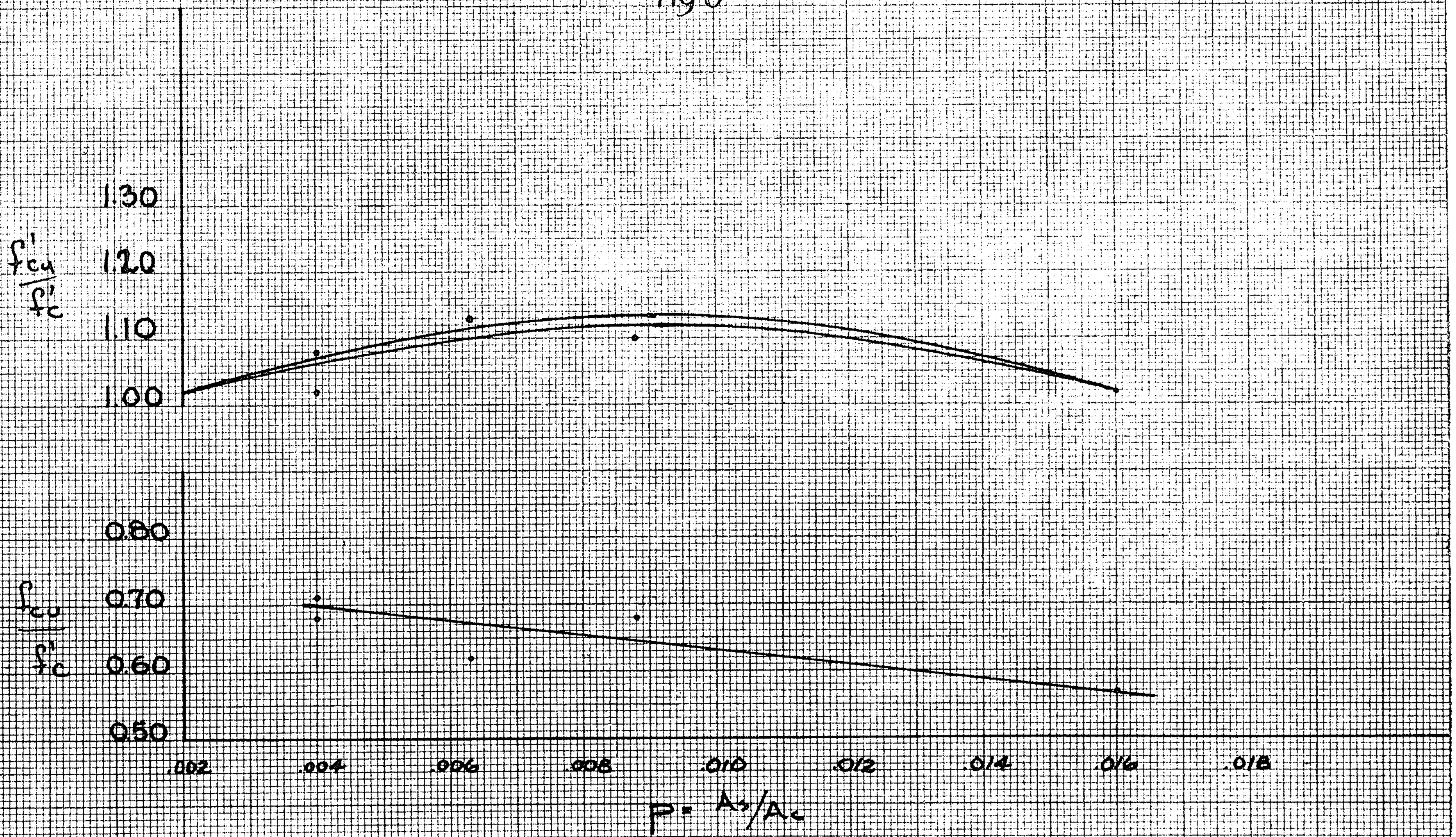
FORM 100

CURVE OF ULTIMATE STRESS

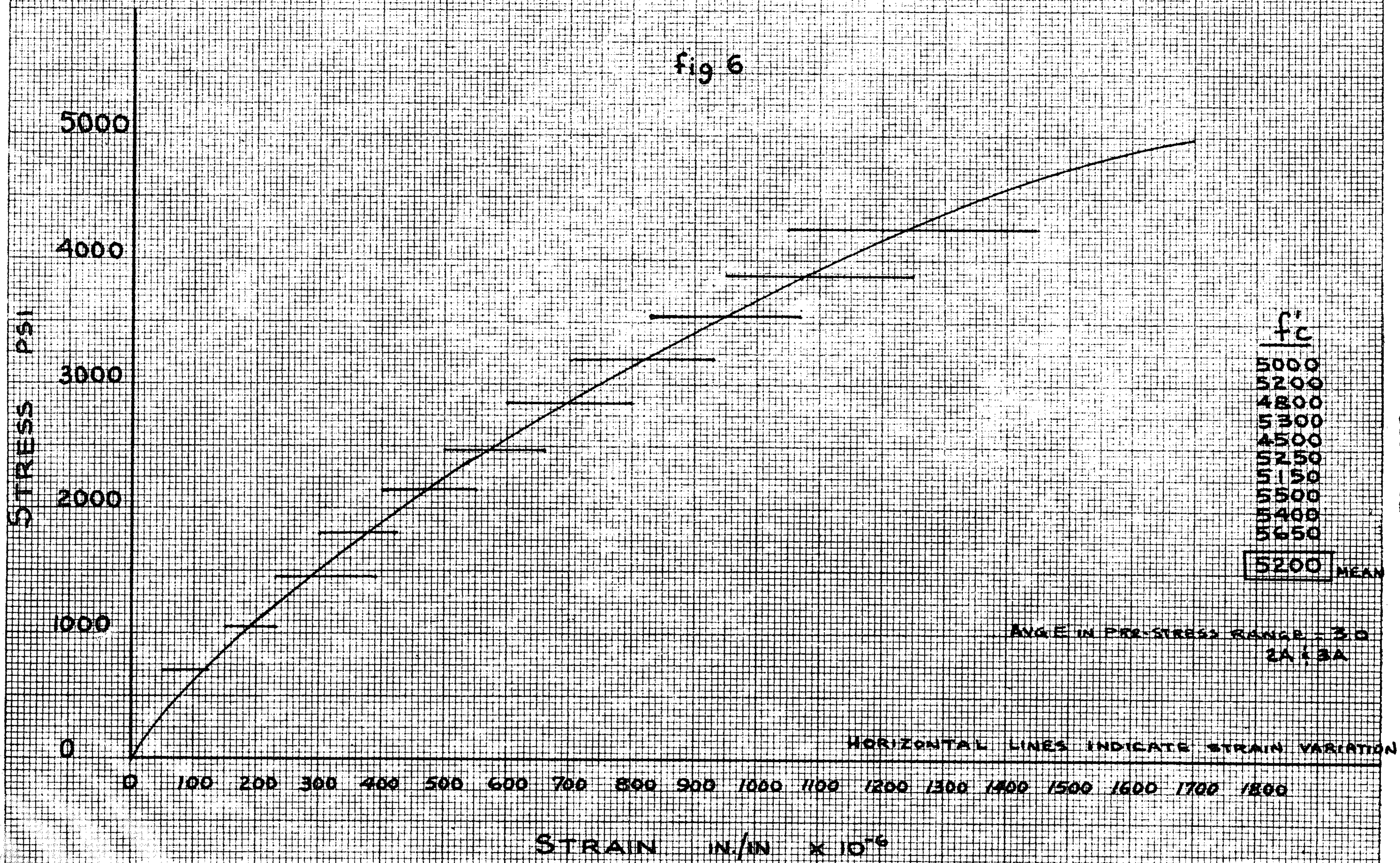
fig 5



STRESS-STRAIN CURVE FOR ELASTIC MODULUS

MEAN OF CYLINDERS BATCH A_T

fig 6



THIS MARGIN RESERVED FOR BINDING.

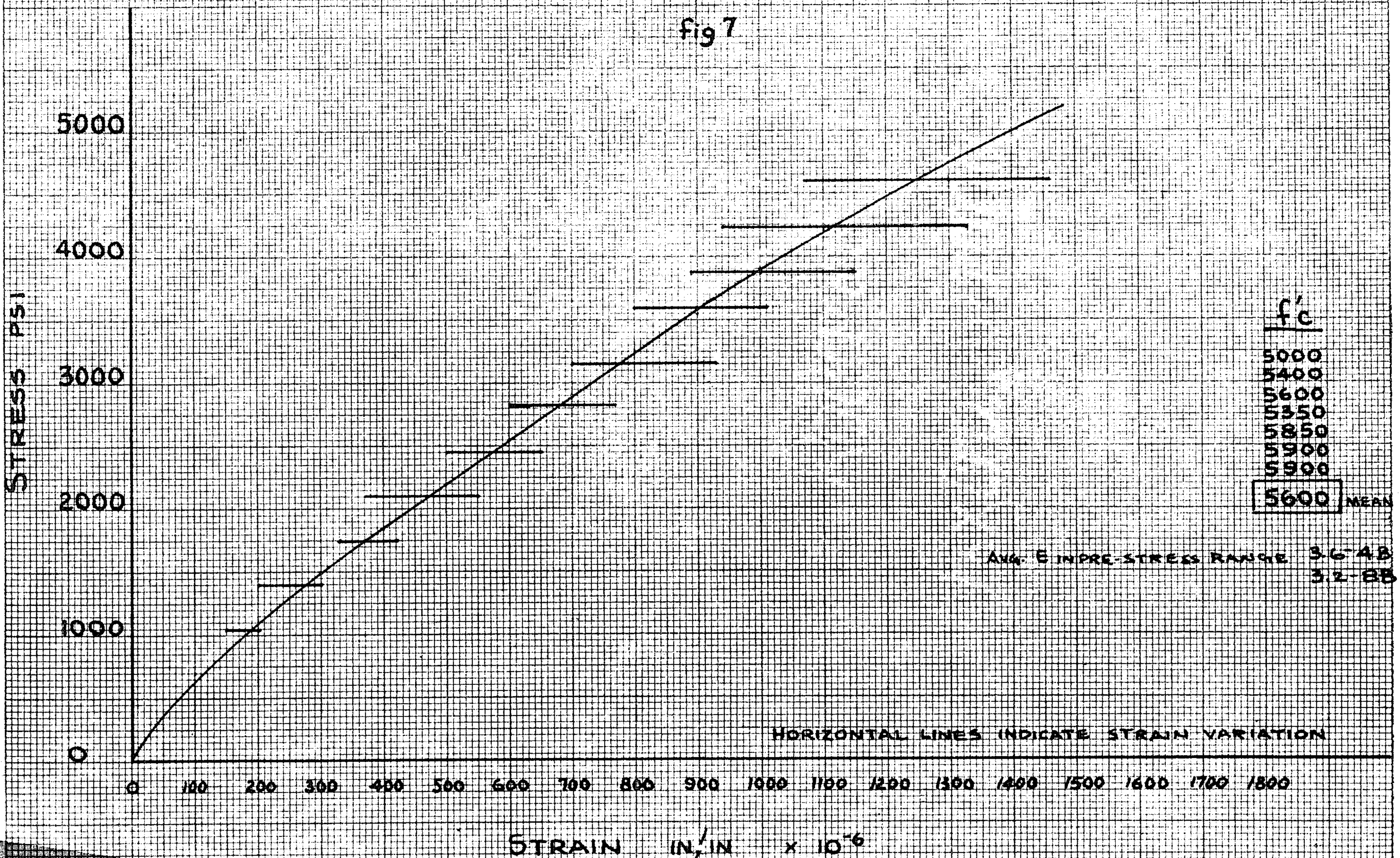
IF SHEET IS READ THIS WAY (HORIZONTALLY), THIS MUST BE TOP.

IF SHEET IS READ THE OTHER WAY (VERTICALLY), THIS MUST BE LEFT-HAND SIDE.

STRESS-STRAIN CURVE FOR ELASTIC MODULUS

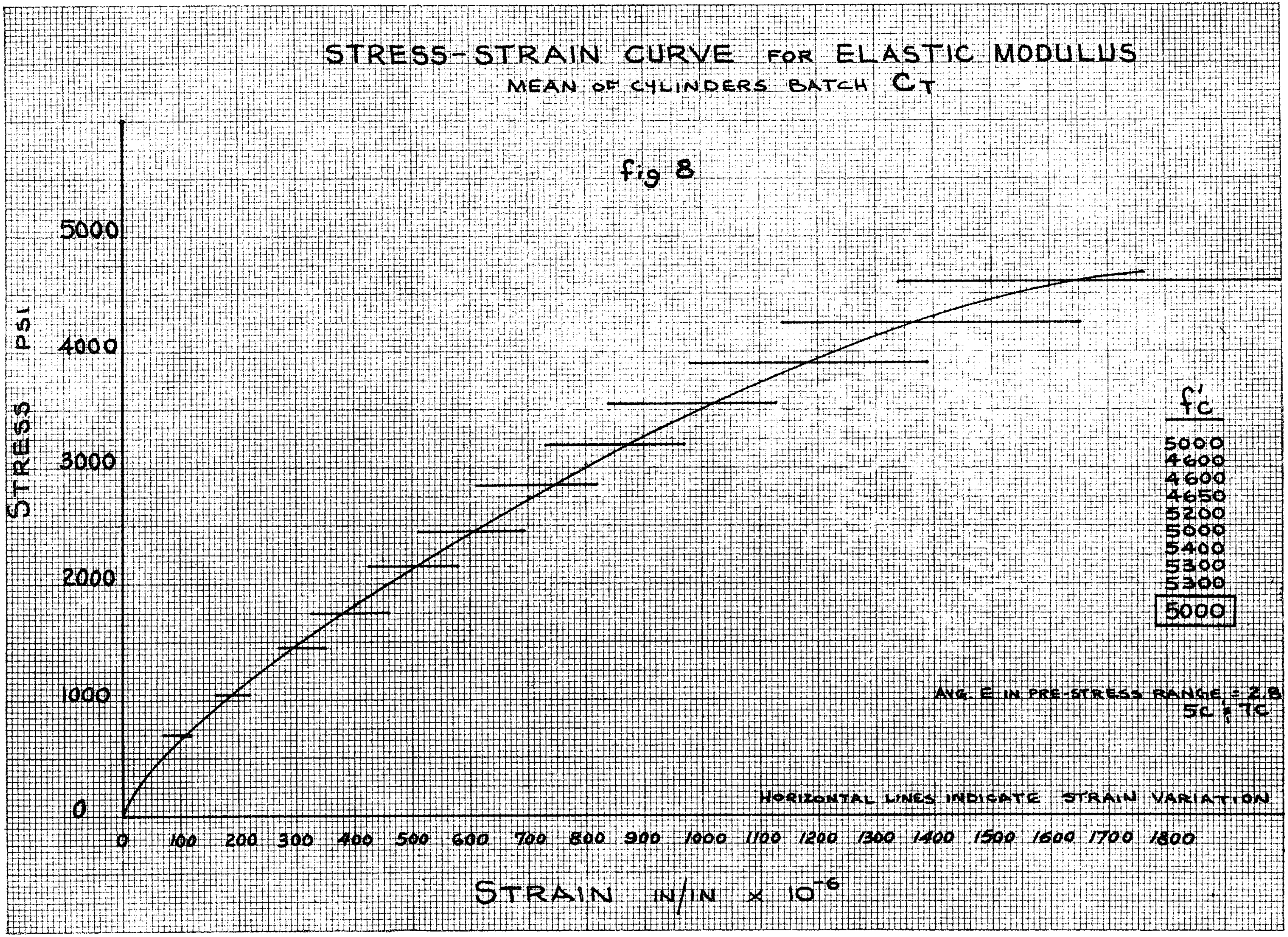
MEAN OF CYLINDERS BATCH BT

fig 7



THIS MARGIN RESERVED FOR BINDING.

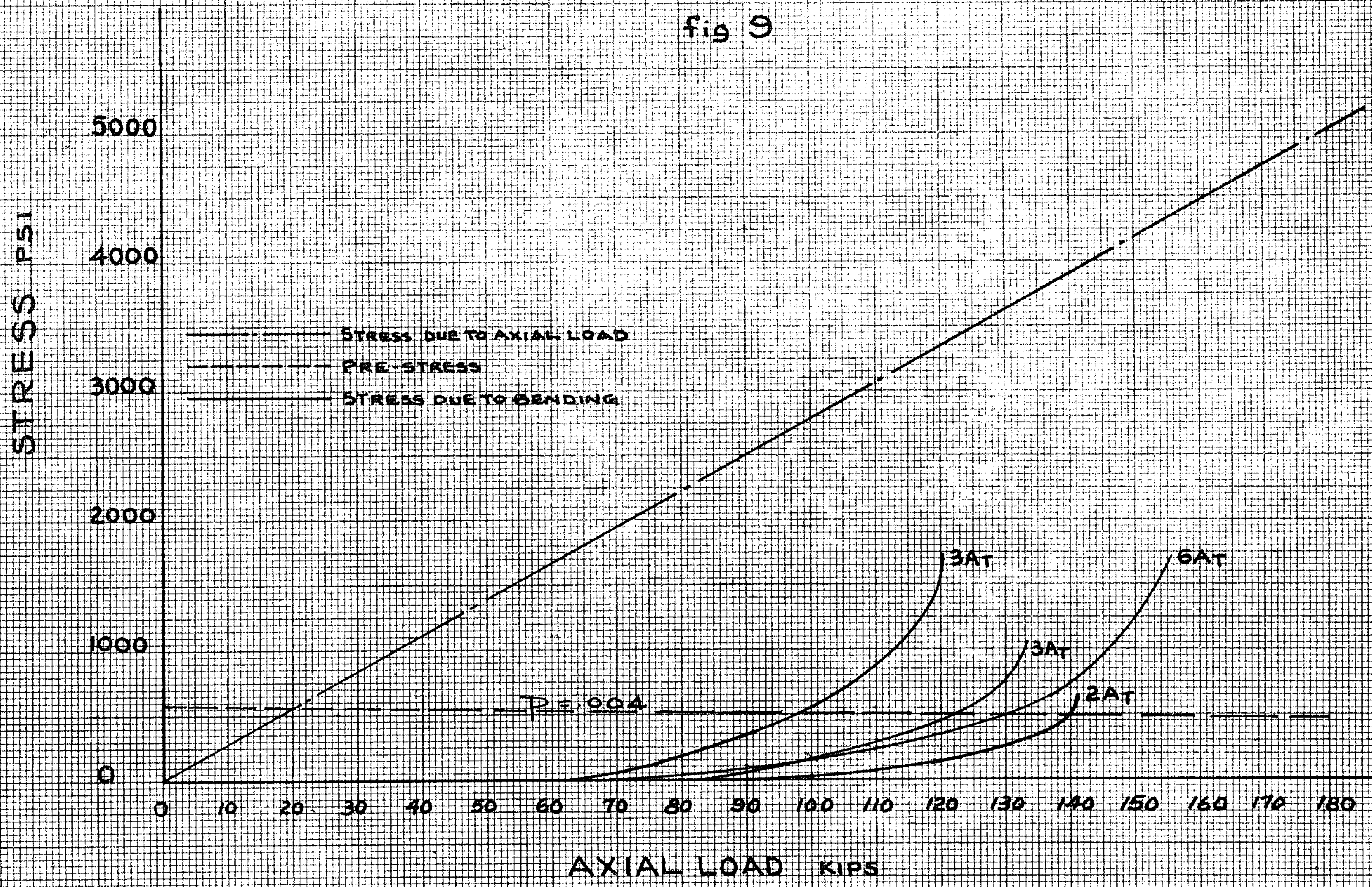
IF SHEET IS READ THE OTHER WAY (VERTICALLY), THIS MUST BE LEFT-HAND SIDE.



AXIAL LOAD - STRESS RELATIONSHIP

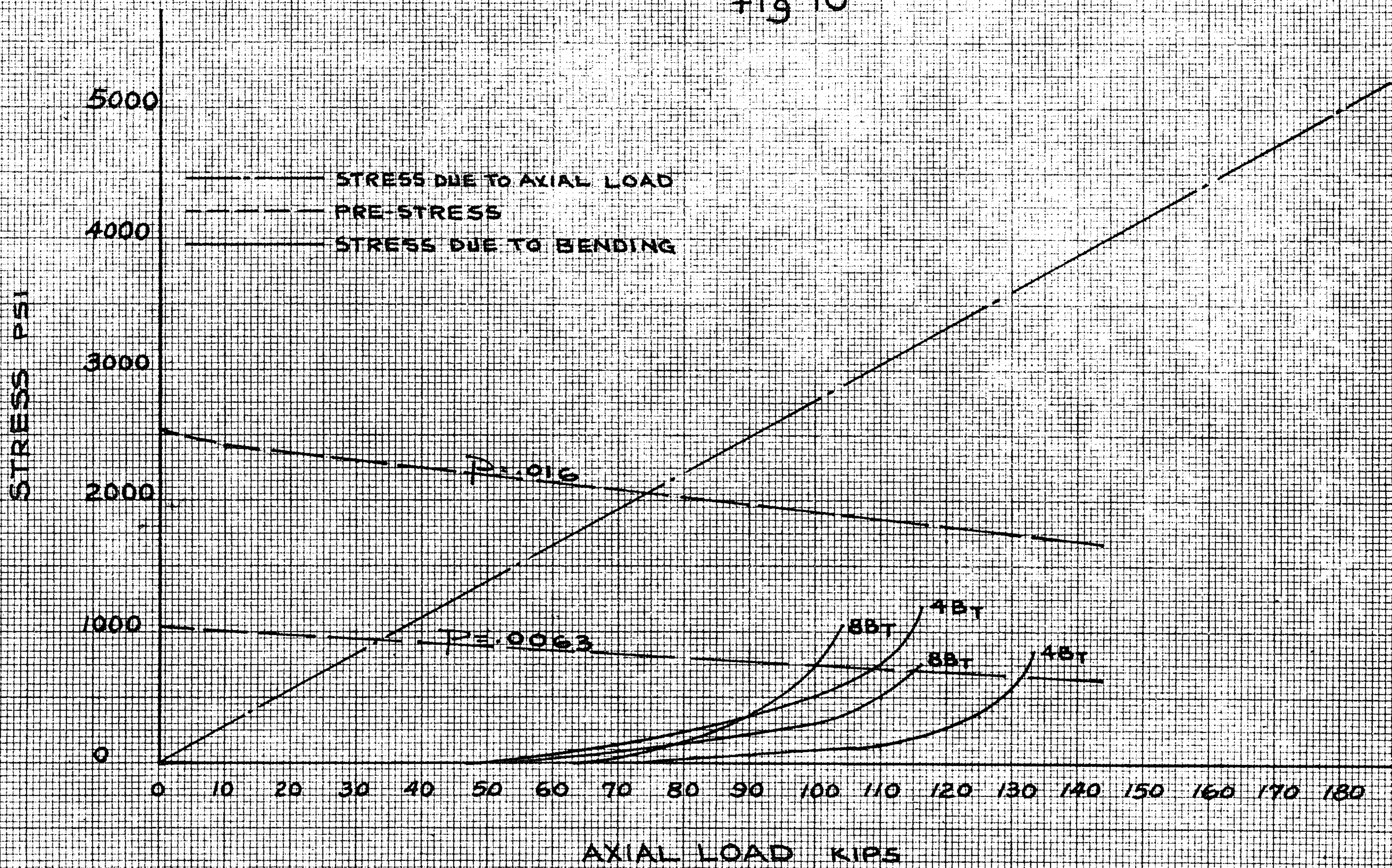
COLUMNS WITH A_T CONCRETE

fig 9



AXIAL LOAD - STRESS RELATIONSHIP
COLUMNS WITH BT CONCRETE

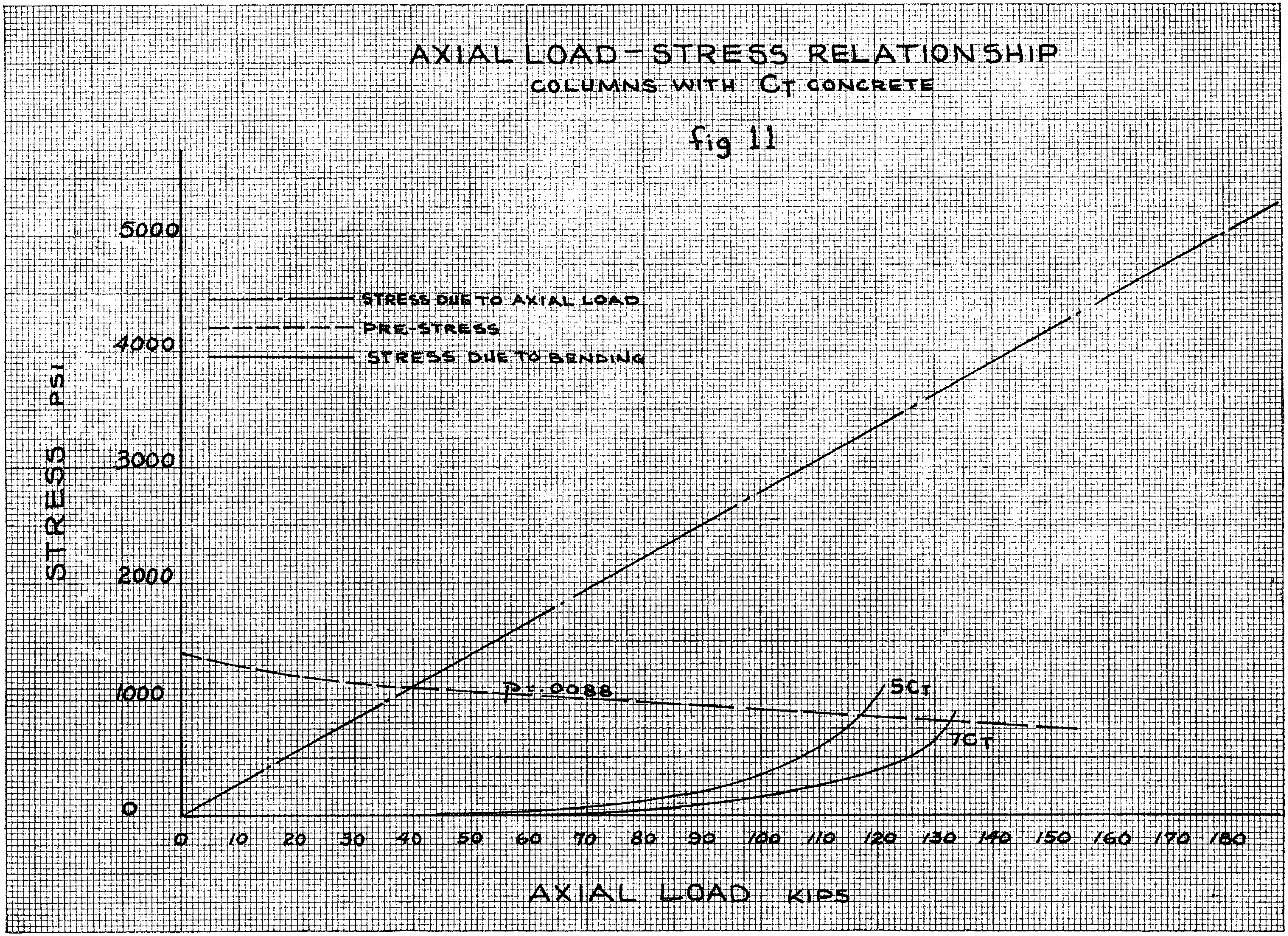
fig 10

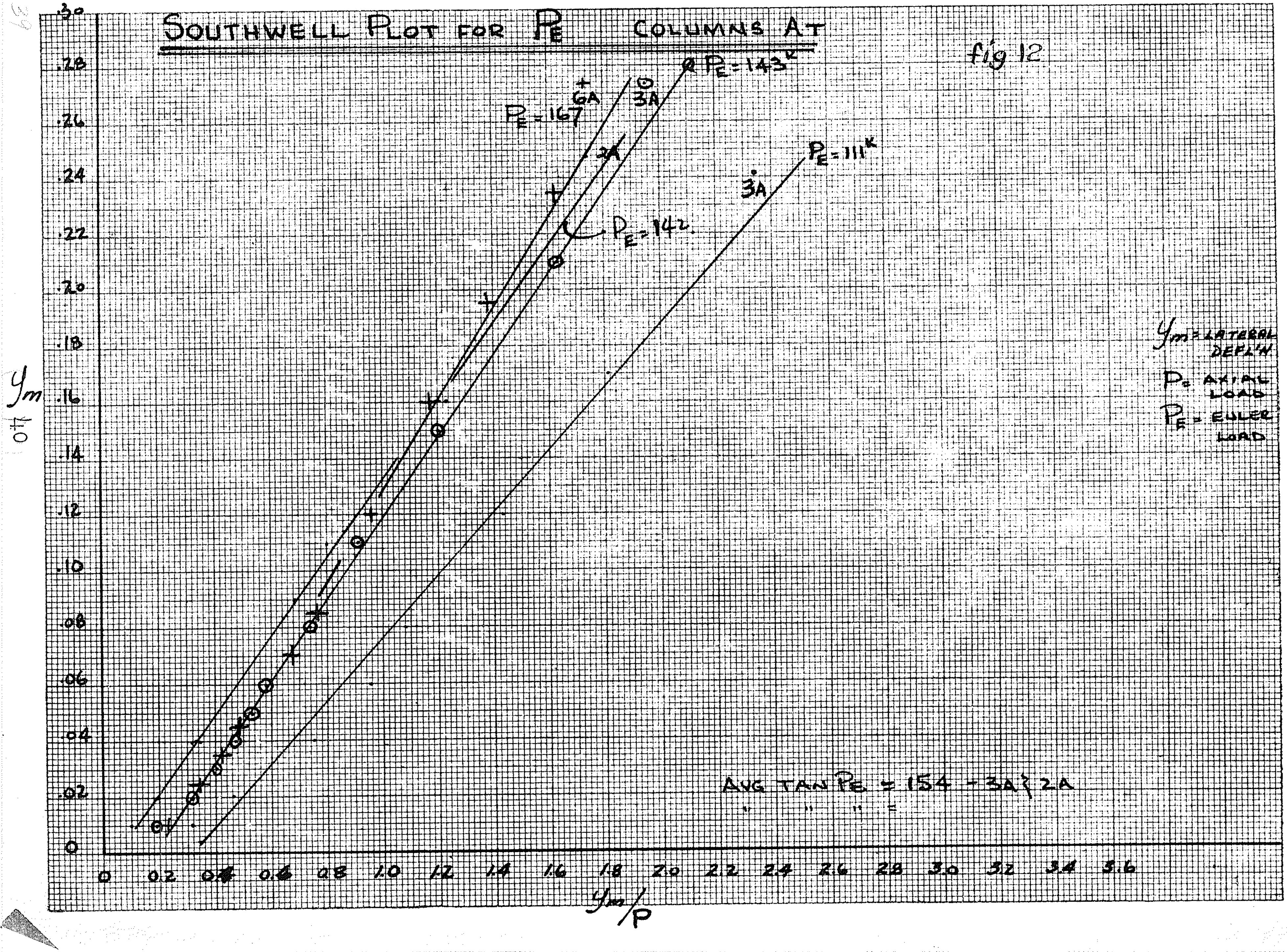


THIS MARGIN RESERVED FOR BINDING.

IF SHEET IS READ THIS WAY (HORIZONTALLY). THIS MUST BE TOP.

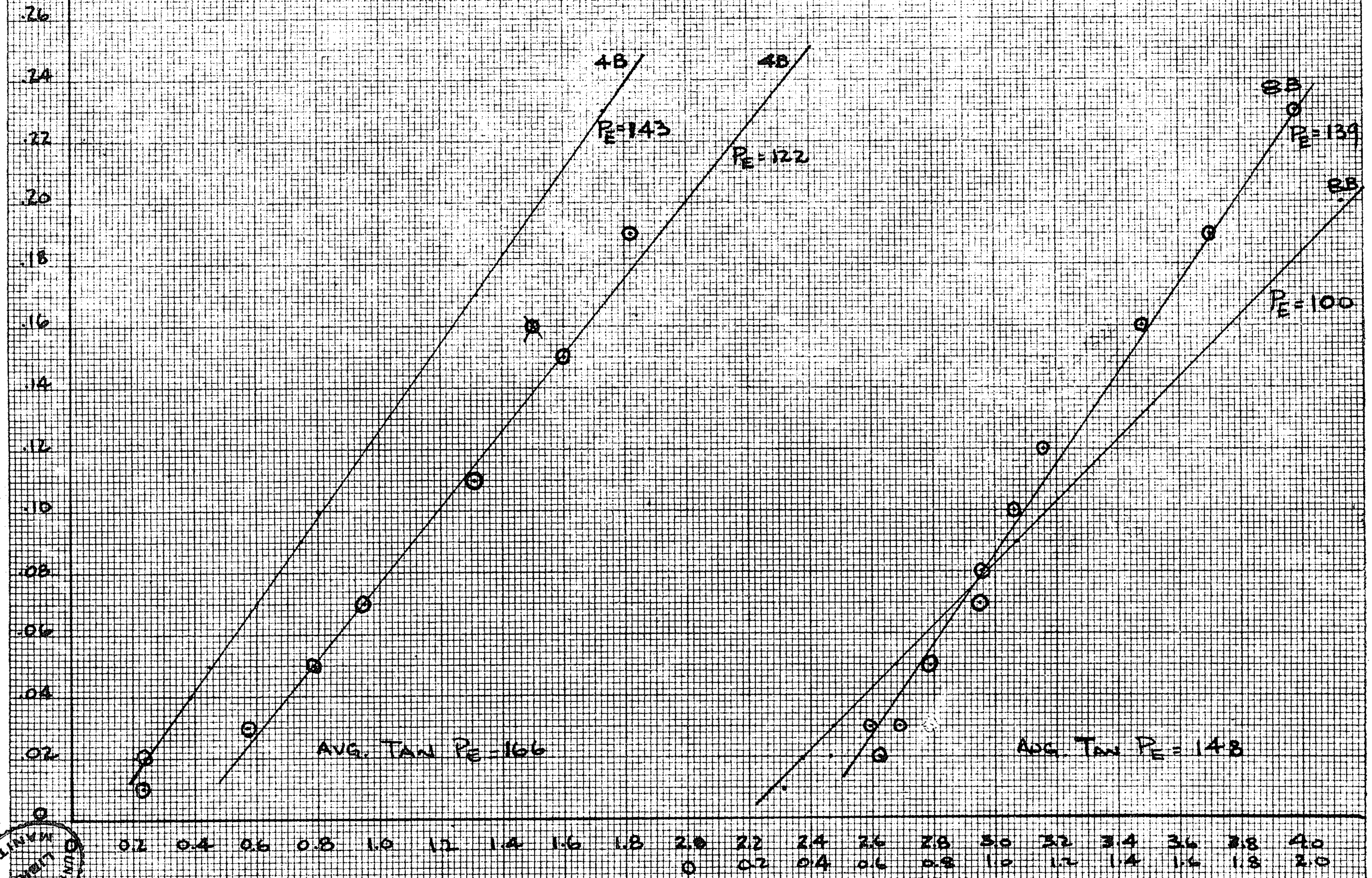
IF SHEET IS READ THE OTHER WAY (VERTICALLY). THIS MUST BE LEFT-HAND SIDE.

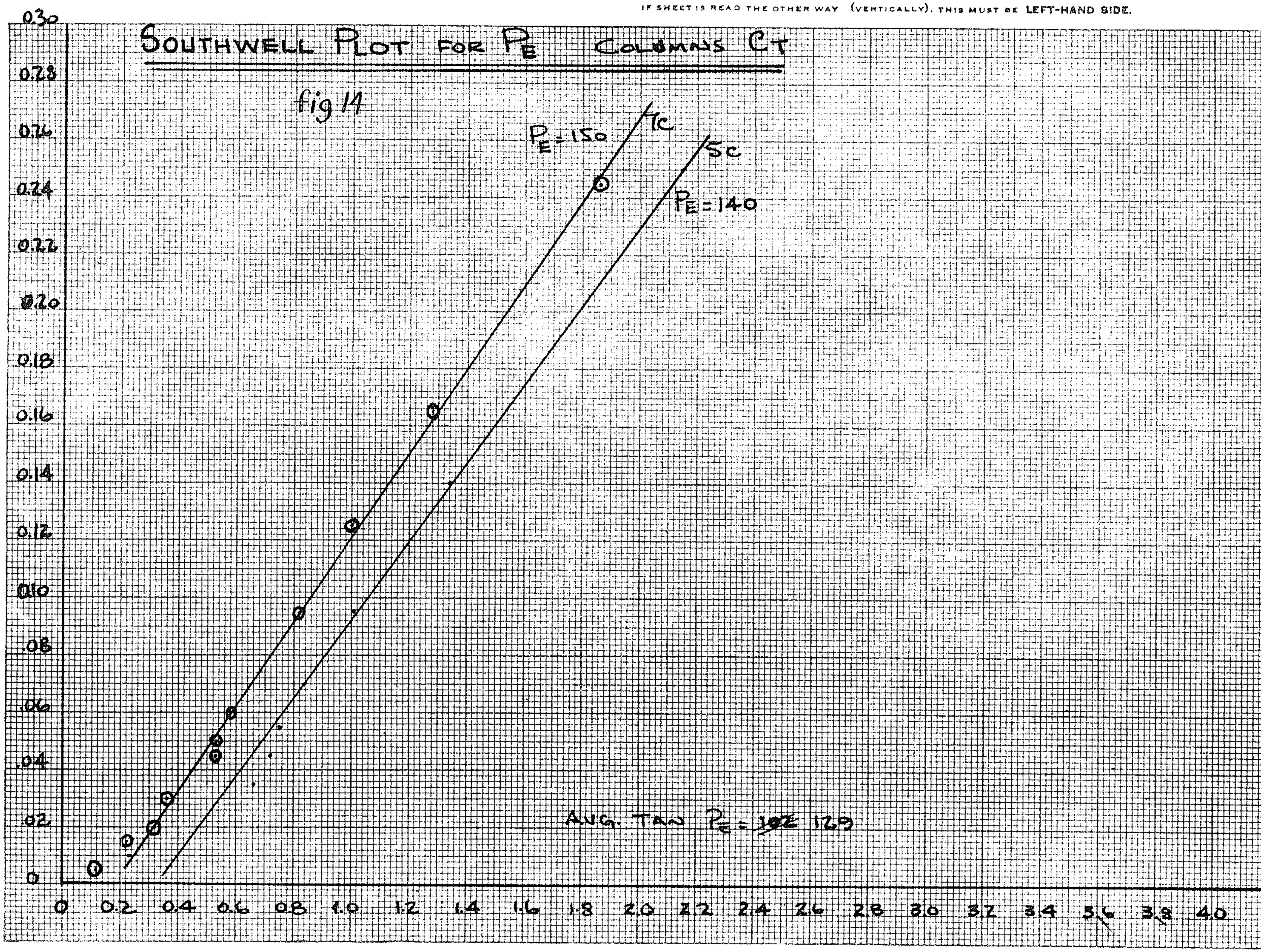



SOUTHWELL PLOT FOR PE COLUMNS AT
fig 12
PE = 143K
PE = 167
6A
3A
2A
PE = 111K
3A
PE = 142.
Ym = LATERAL DEFL'N
P = AXIAL LOAD
PE = EULER LOAD
AVG TAN PE = 154 - 3A & 2A
Ym
Ym/P
0
.02
.04
.06
.08
.10
.12
.14
.16
.18
.20
.22
.24
.26
.28
.30
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6

SOUTHWELL PLOT FOR P_E COLUMNS B TO

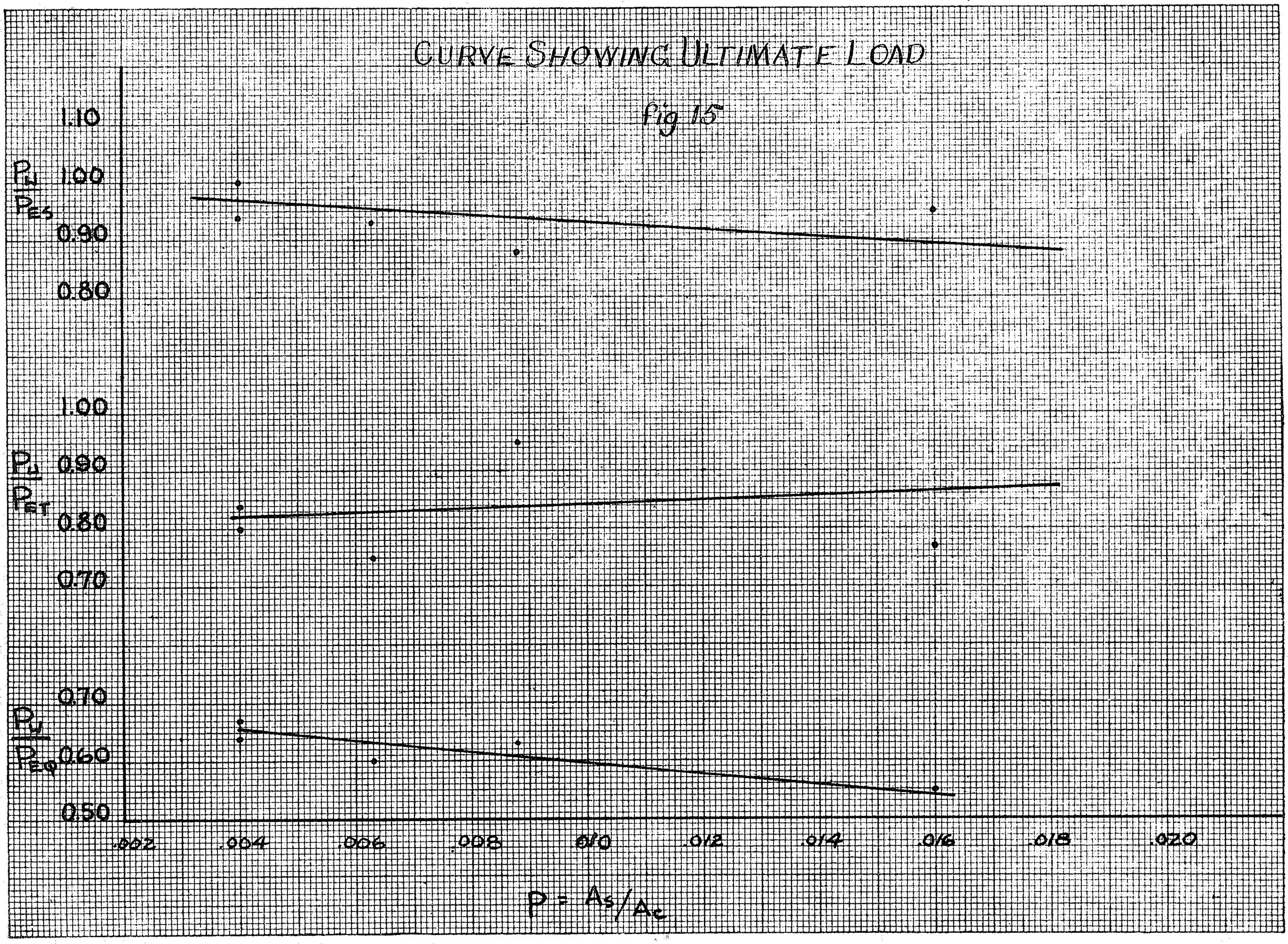
fig 13

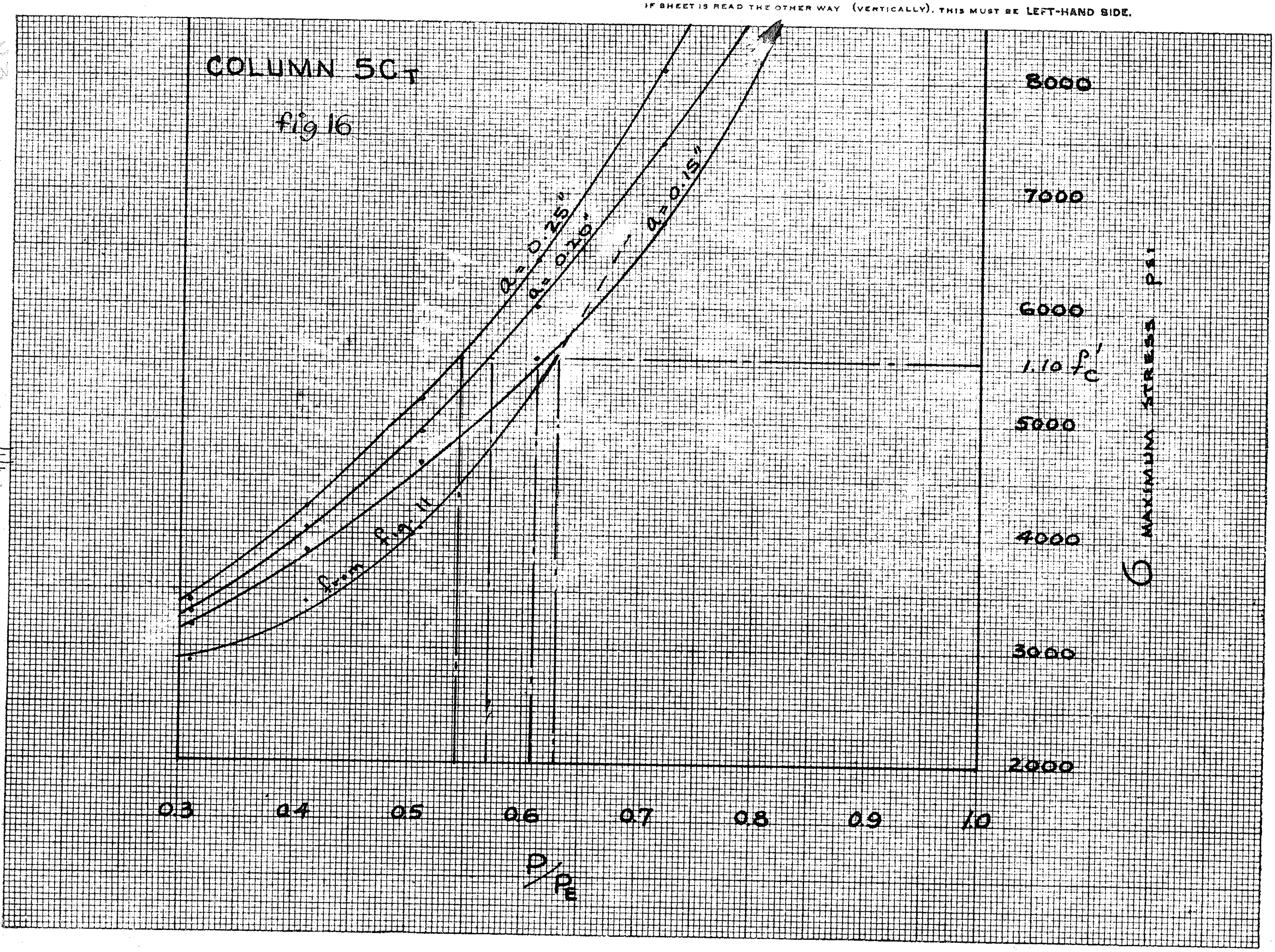


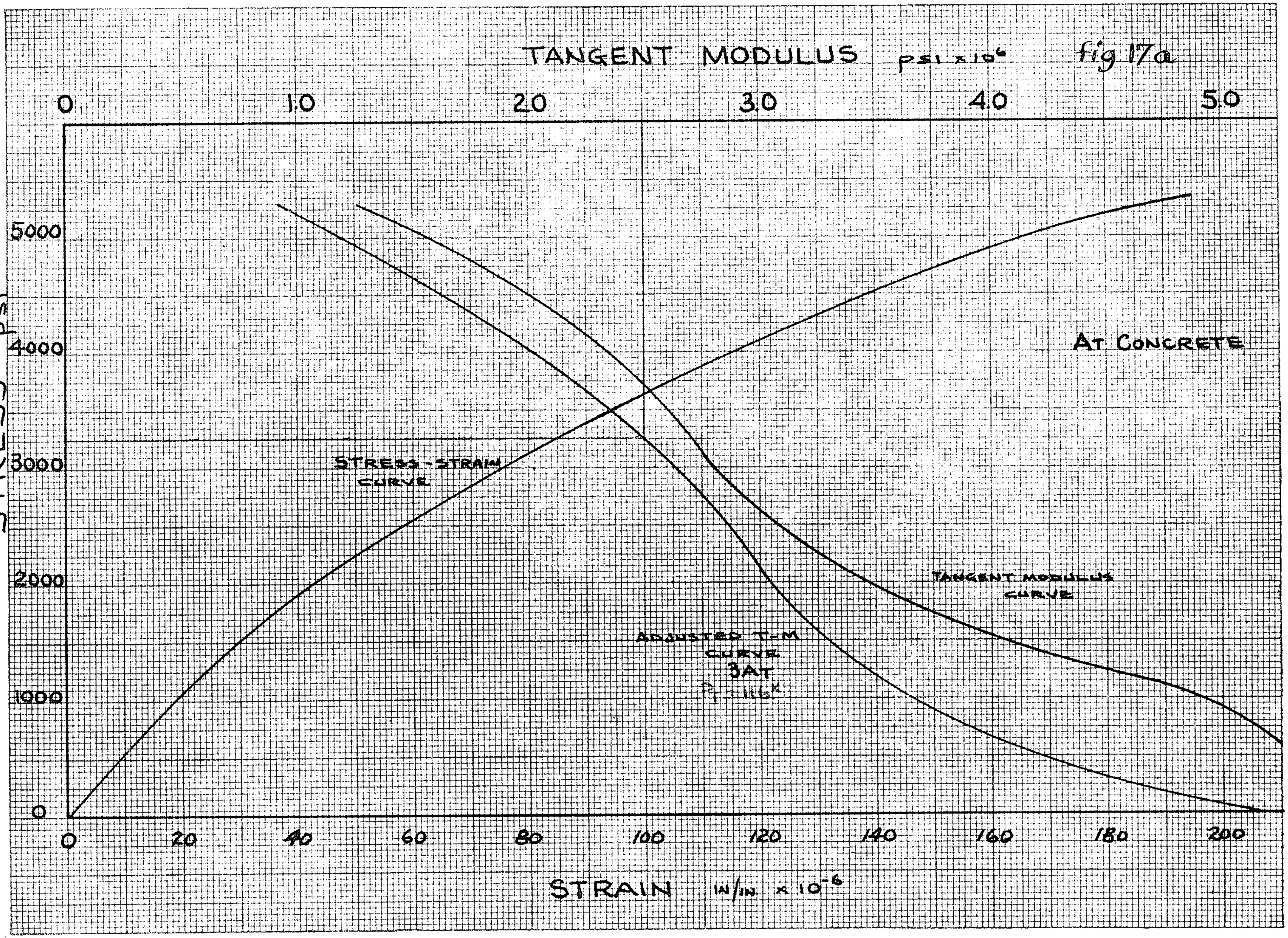

SOUTHWELL PLOT FOR P_E COLUMNS CT
fig 14
$P_E = 150$
7C
5C
$P_E = 140$
AVG. TAN P_E = ~~102~~ 129
0.30
0.28
0.26
0.24
0.22
0.20
0.18
0.16
0.14
0.12
0.10
.08
.06
.04
.02
0
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
4.0

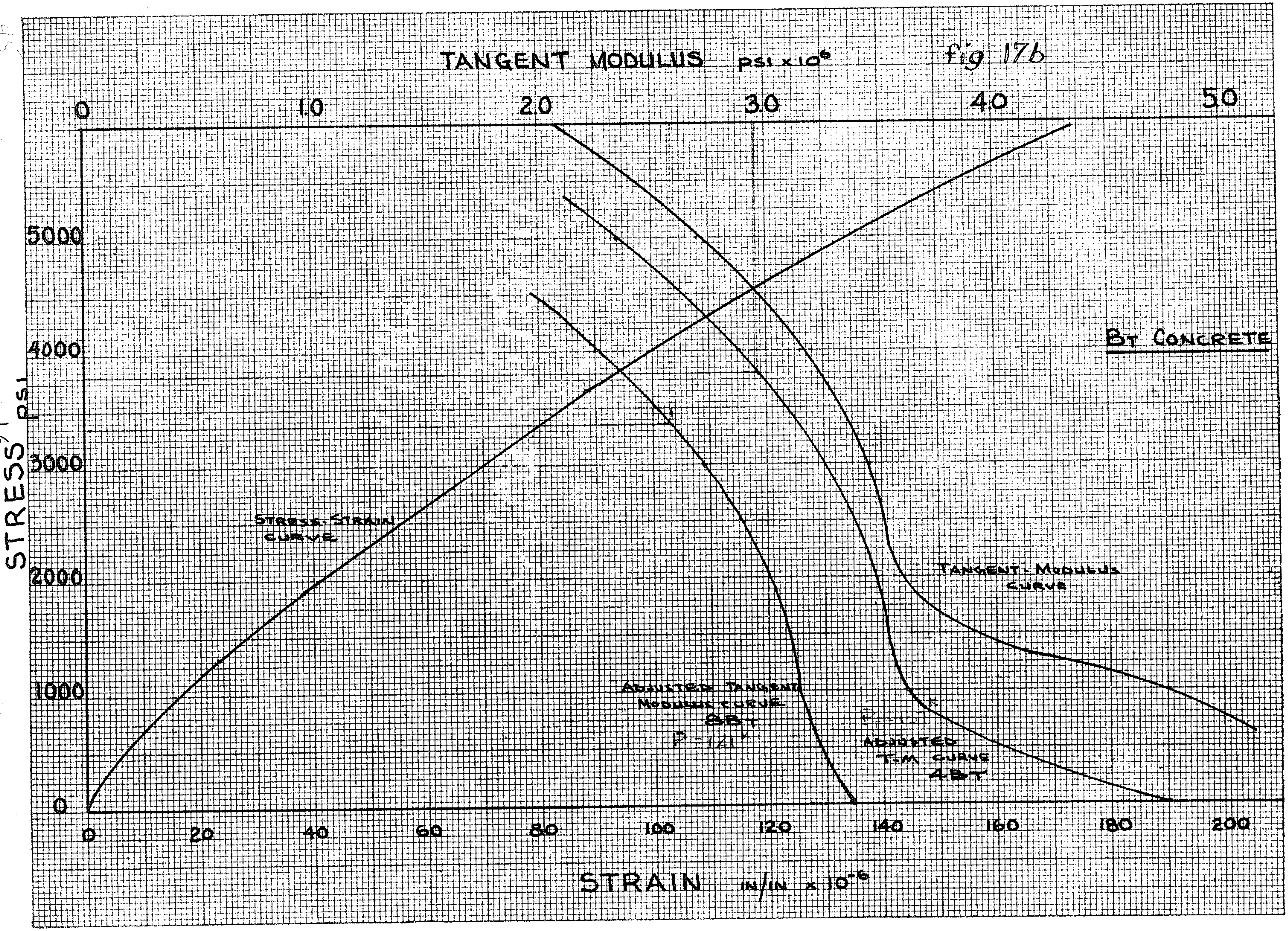
THIS MARGIN RESERVED FOR BINDING.

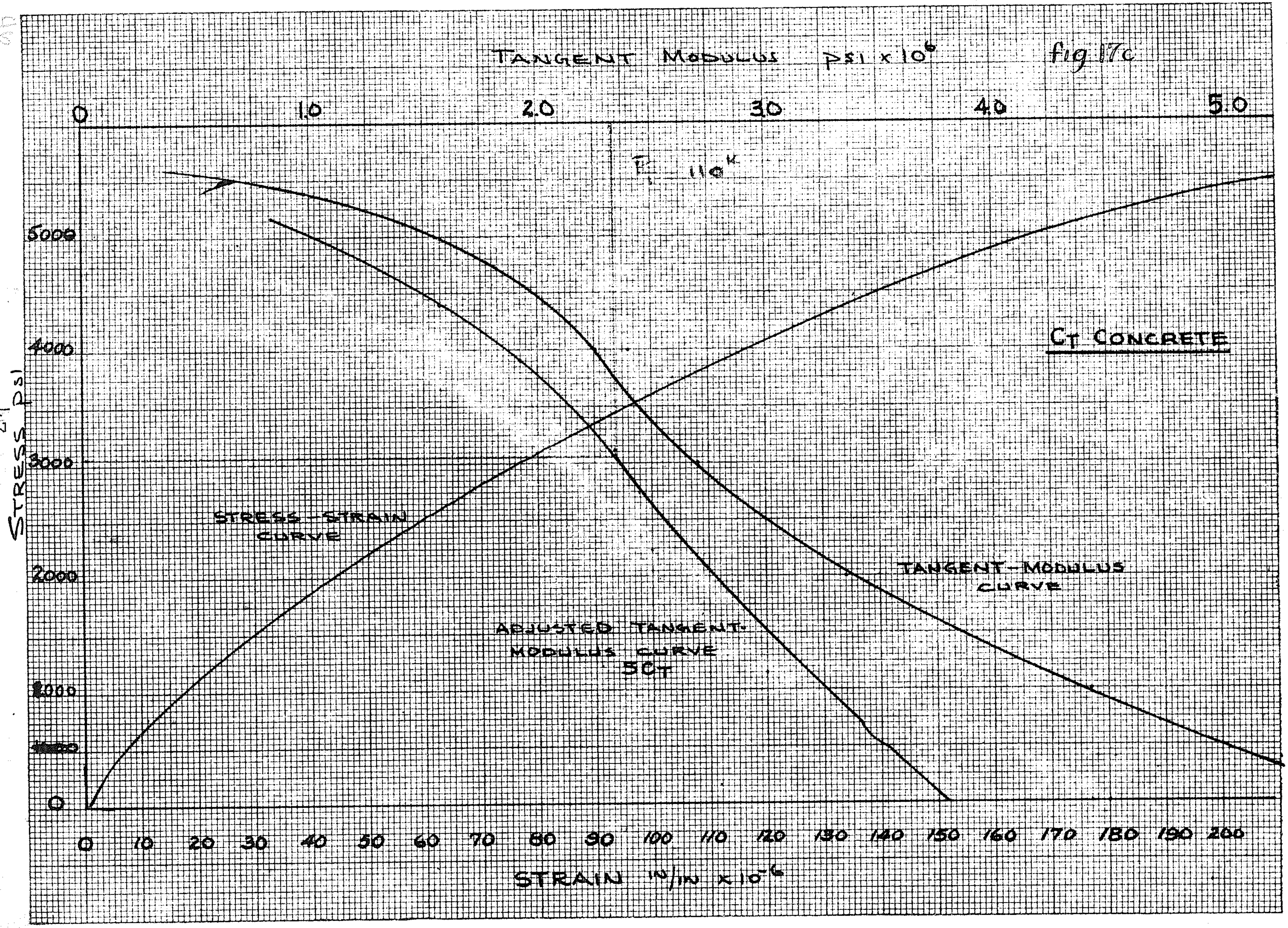
IF SHEET IS READ THE OTHER WAY (VERTICALLY), THIS MUST BE LEFT-HAND SIDE.




COLUMN $5C_T$
fig 16
a = 0.25"
a = 0.20"
a = 0.15"
from fig. 11
8000
7000
6000
1.10 f_c'
5000
4000
3000
2000
σ MAXIMUM STRESS PSI
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
P/P_E

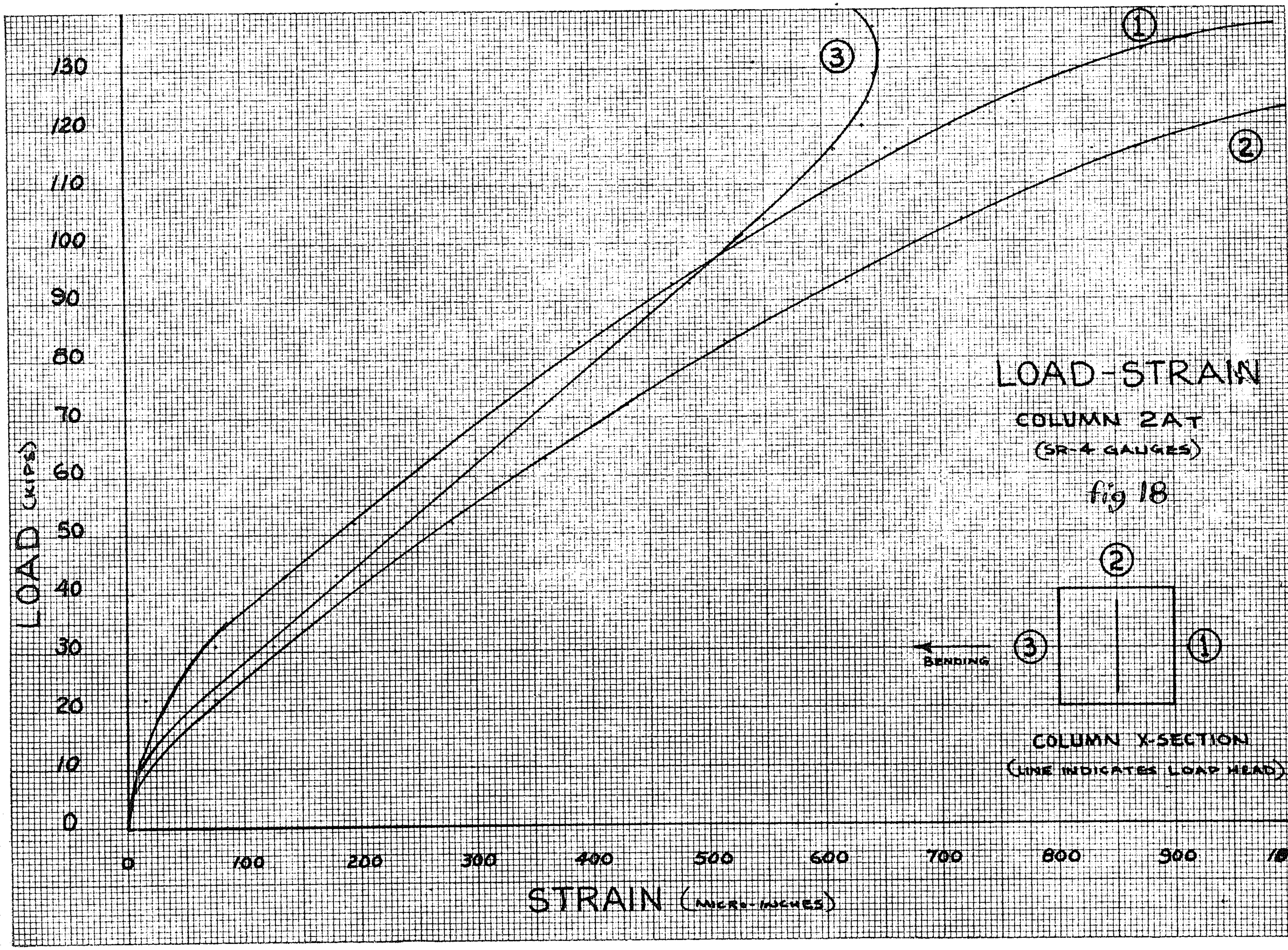

TANGENT MODULUS psi x 10^6
fig 17a
0
1.0
2.0
3.0
4.0
5.0
STRESS PSI
5000
4000
3000
2000
1000
0
AT CONCRETE
STRESS - STRAIN CURVE
TANGENT MODULUS CURVE
ADJUSTED T-M CURVE
3AT
0
20
40
60
80
100
120
140
160
180
200
STRAIN IN/IN x 10^-6


TANGENT MODULUS PSI x 10^6
fig 17b
0
1.0
2.0
3.0
4.0
5.0
5000
4000
3000
2000
1000
0
STRESS PSI
97
BT CONCRETE
STRESS-STRAIN CURVE
TANGENT-MODULUS CURVE
ADJUSTED TANGENT MODULUS CURVE
8BT
ADJUSTED T-M CURVE
4BT
0
20
40
60
80
100
120
140
160
180
200
STRAIN IN/IN x 10^-6


TANGENT MODULUS psi x 10^6
fig 17c
0
1.0
2.0
3.0
4.0
5.0
E 110 K
5000
4000
3000
2000
1000
0
STRESS psi
CT CONCRETE
STRESS-STRAIN CURVE
TANGENT-MODULUS CURVE
ADJUSTED TANGENT-MODULUS CURVE
5CT
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200
STRAIN in/in x 10^-6

THIS MARGIN RESERVED FOR BINDING.

IF SHEET IS READ THE OTHER WAY (VERTICALLY). THIS MUST BE LEFT-HAND SIDE.



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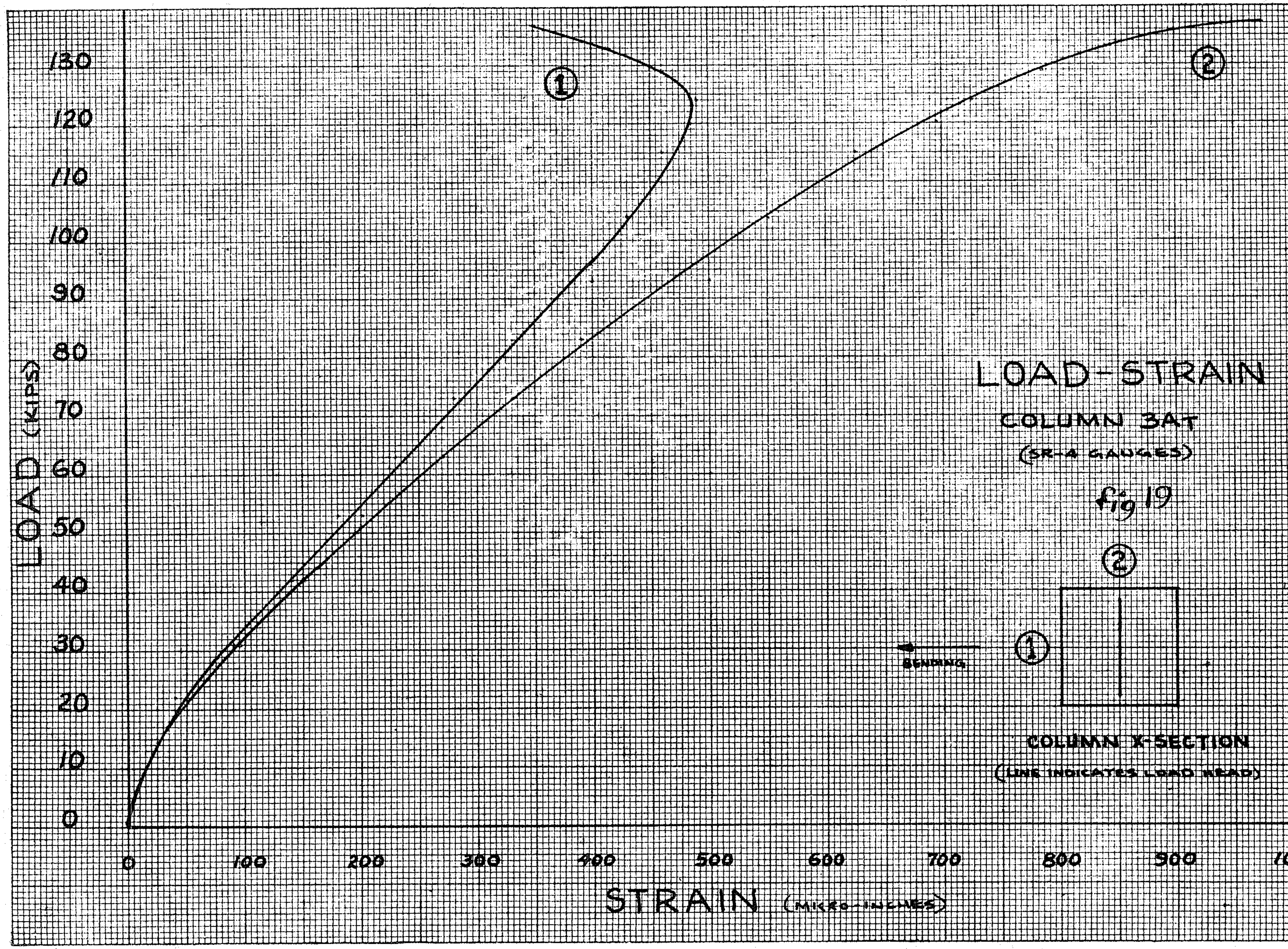
FORM 100

THIS MARGIN RESERVED FOR BINDING.

IF SHEET IS READ THE OTHER WAY (VERTICALLY), THIS MUST BE LEFT-HAND SIDE.



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FORM 100

67

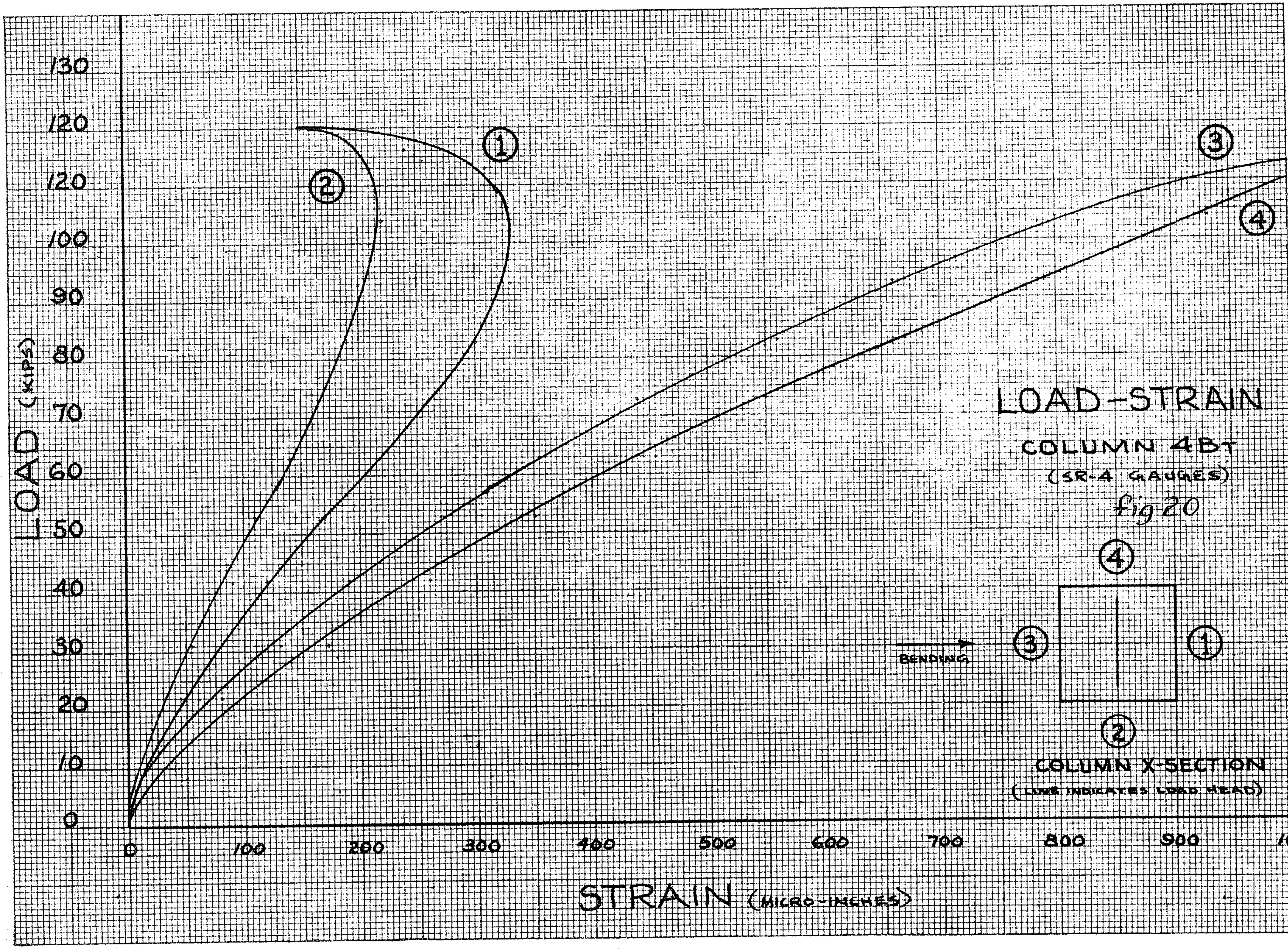
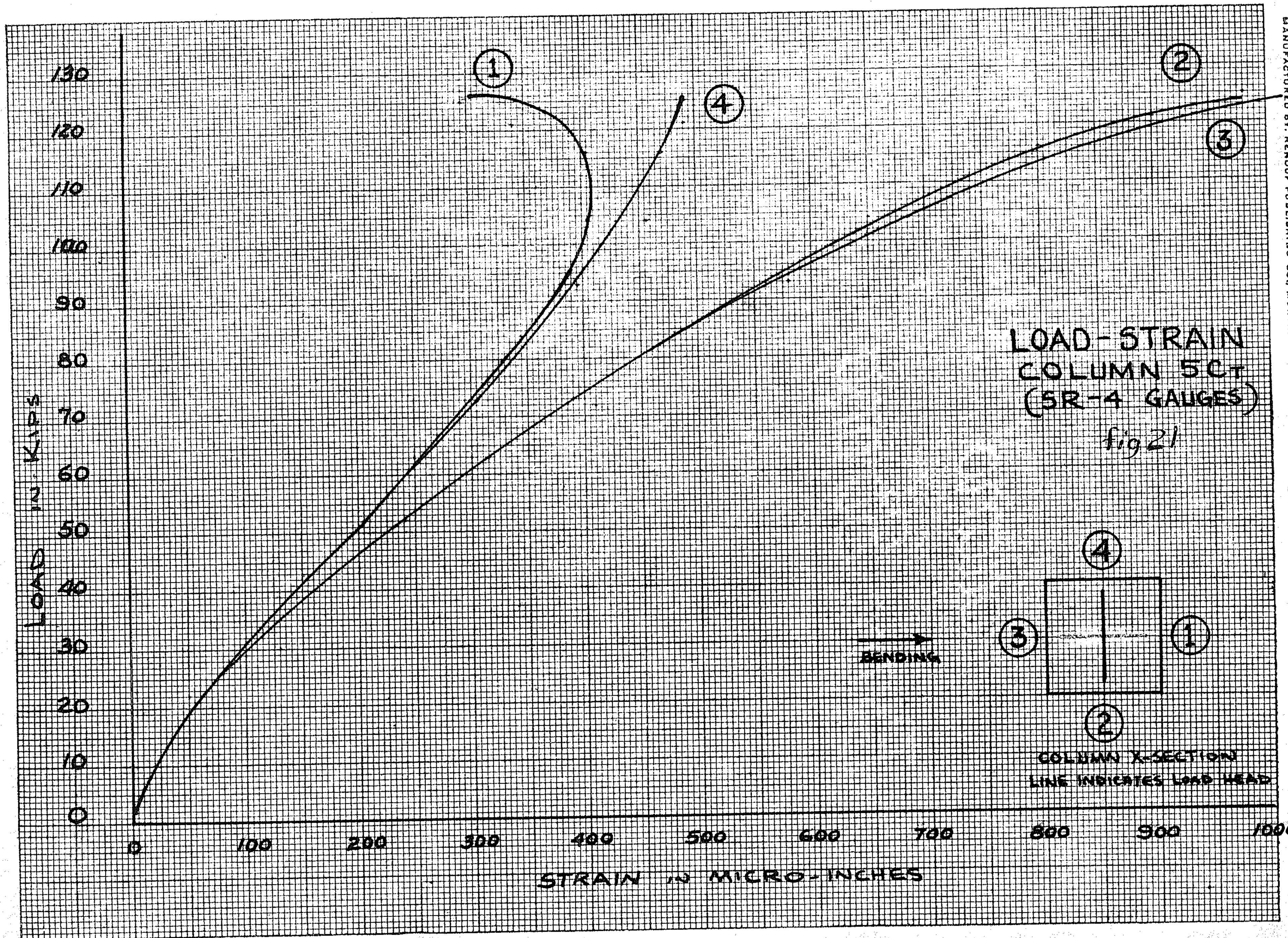

LOAD (KIPS)
130
120
120
100
90
80
70
60
50
40
30
20
10
0
STRAIN (MICRO-INCHES)
0
100
200
300
400
500
600
700
800
900
1000
1
2
3
4
LOAD-STRAIN
COLUMN 4BT
(SR-4 GAUGES)
BENDING
COLUMN X-SECTION
(LINE INDICATES LOAD HEAD)

fig 20

THIS MARGIN RESERVED FOR BINDING.

IF SHEET IS READ THIS WAY (HORIZONTALLY), THIS MUST BE TOP.

IF SHEET IS READ THE OTHER WAY (VERTICALLY), THIS MUST BE LEFT-HAND SIDE.



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FORM 100

50

51

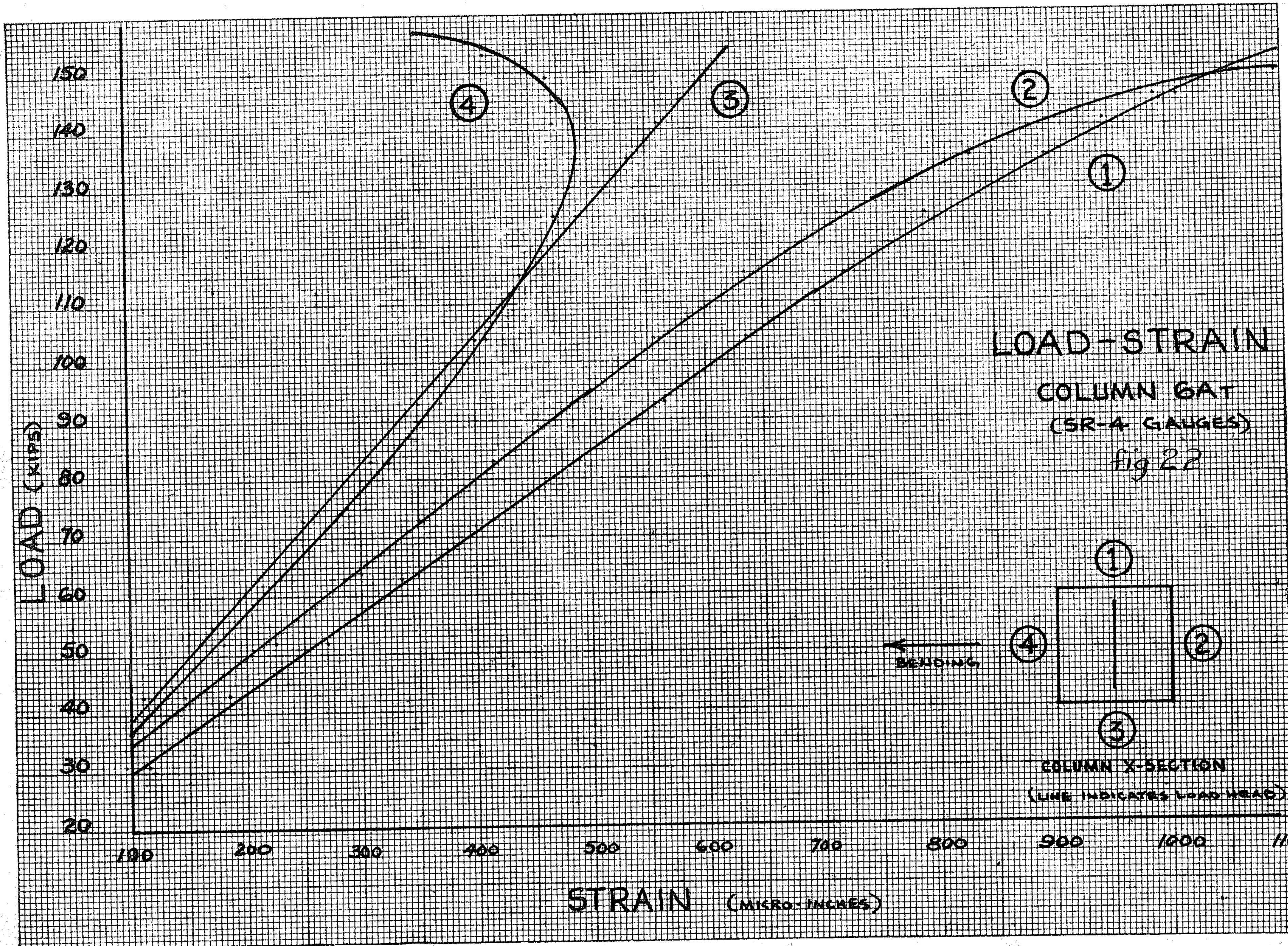

LOAD-STRAIN
COLUMN 6AT
(SR-4 GAUGES)
LOAD (KIPS)
STRAIN (MICRO-INCHES)
150
140
130
120
110
100
90
80
70
60
50
40
30
20
100
200
300
400
500
600
700
800
900
1000
1100
1
2
3
4
BENDING
COLUMN X-SECTION
(LINE INDICATES LOAD HEAD)

fig 22

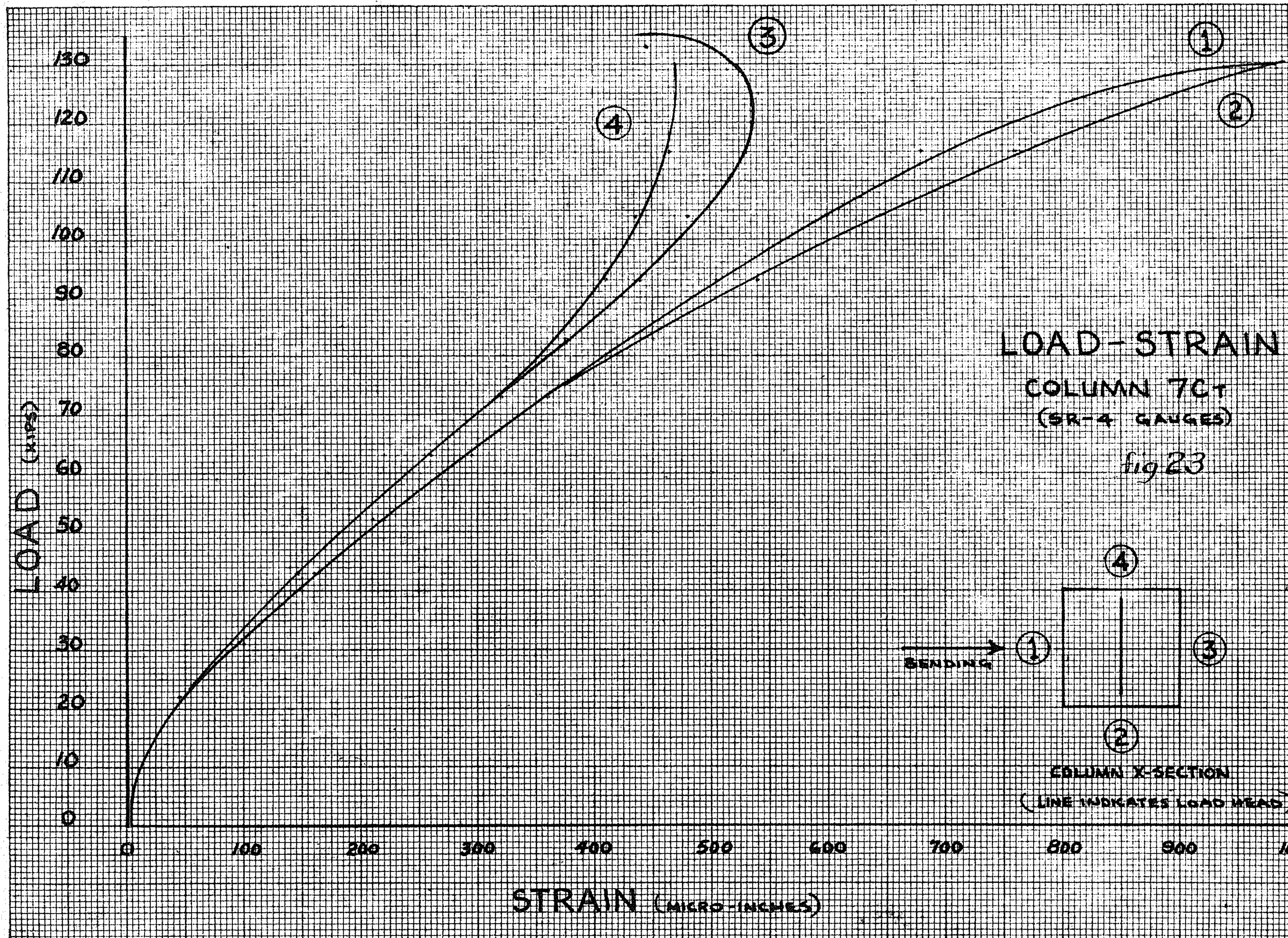
52

THIS MARGIN RESERVED FOR BINDING.

IF SHEET IS READ THIS WAY (HORIZONTALLY), THIS MUST BE TOP.

IF SHEET IS READ THE OTHER WAY (VERTICALLY), THIS MUST BE LEFT-HAND SIDE.

53



53

54

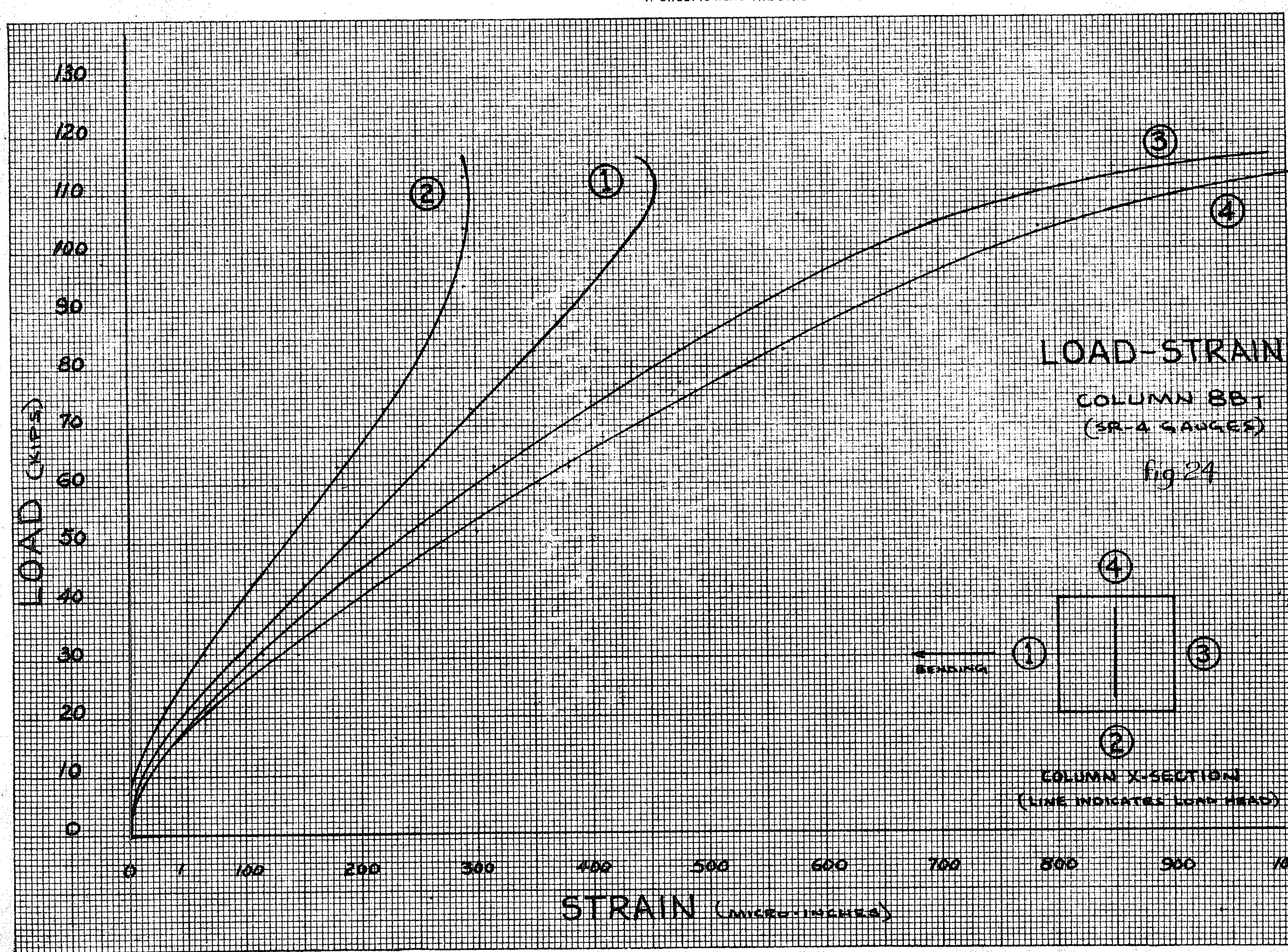
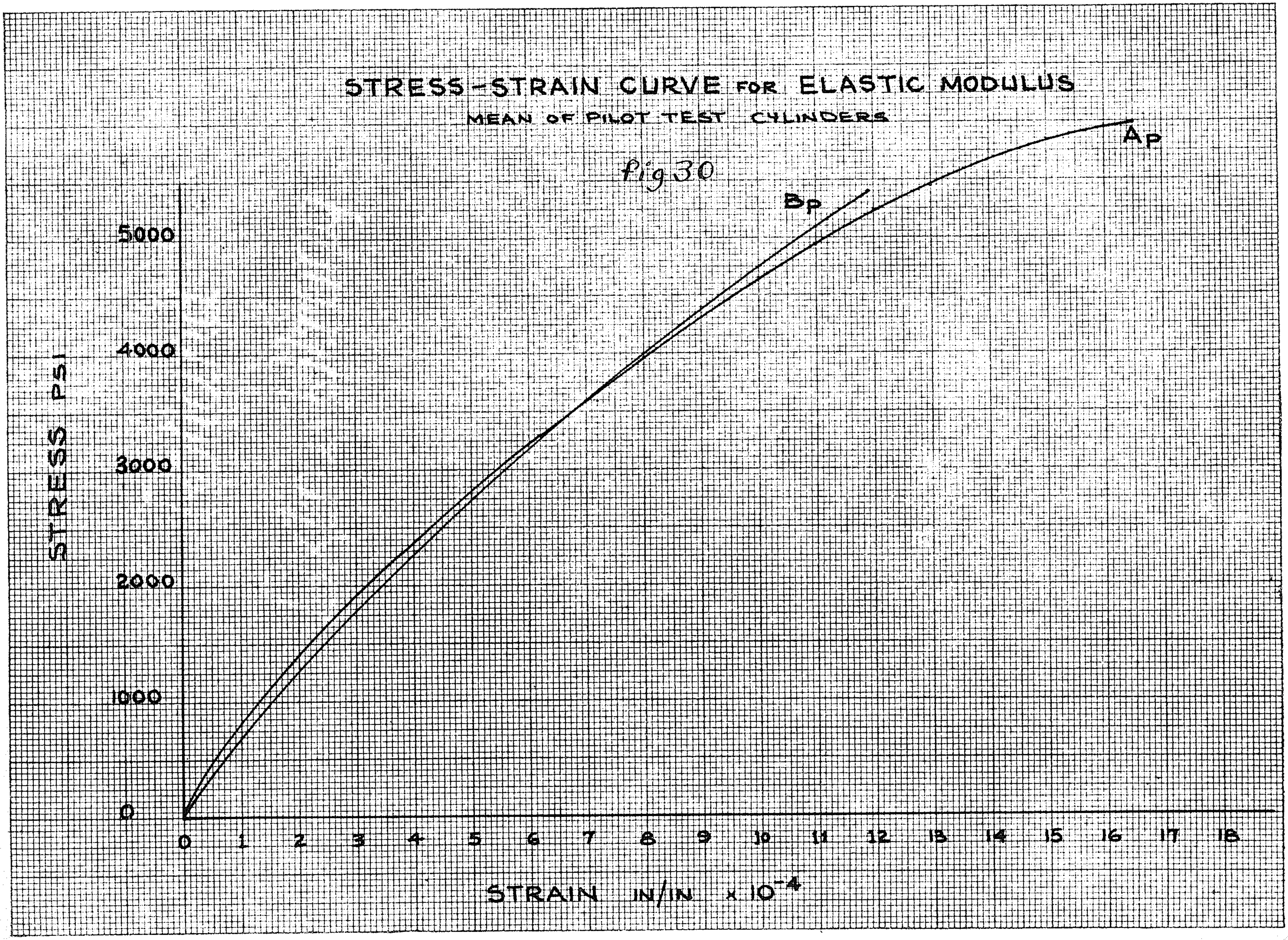

LOAD-STRAIN
COLUMN 8BT
(SR-4 GAUGES)
LOAD (KIPS)
STRAIN (MICRO-INCHES)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
0
100
200
300
400
500
600
700
800
900
1000
1
2
3
4
BENDING
COLUMN X-SECTION
(LINE INDICATES LOAD HEAD)

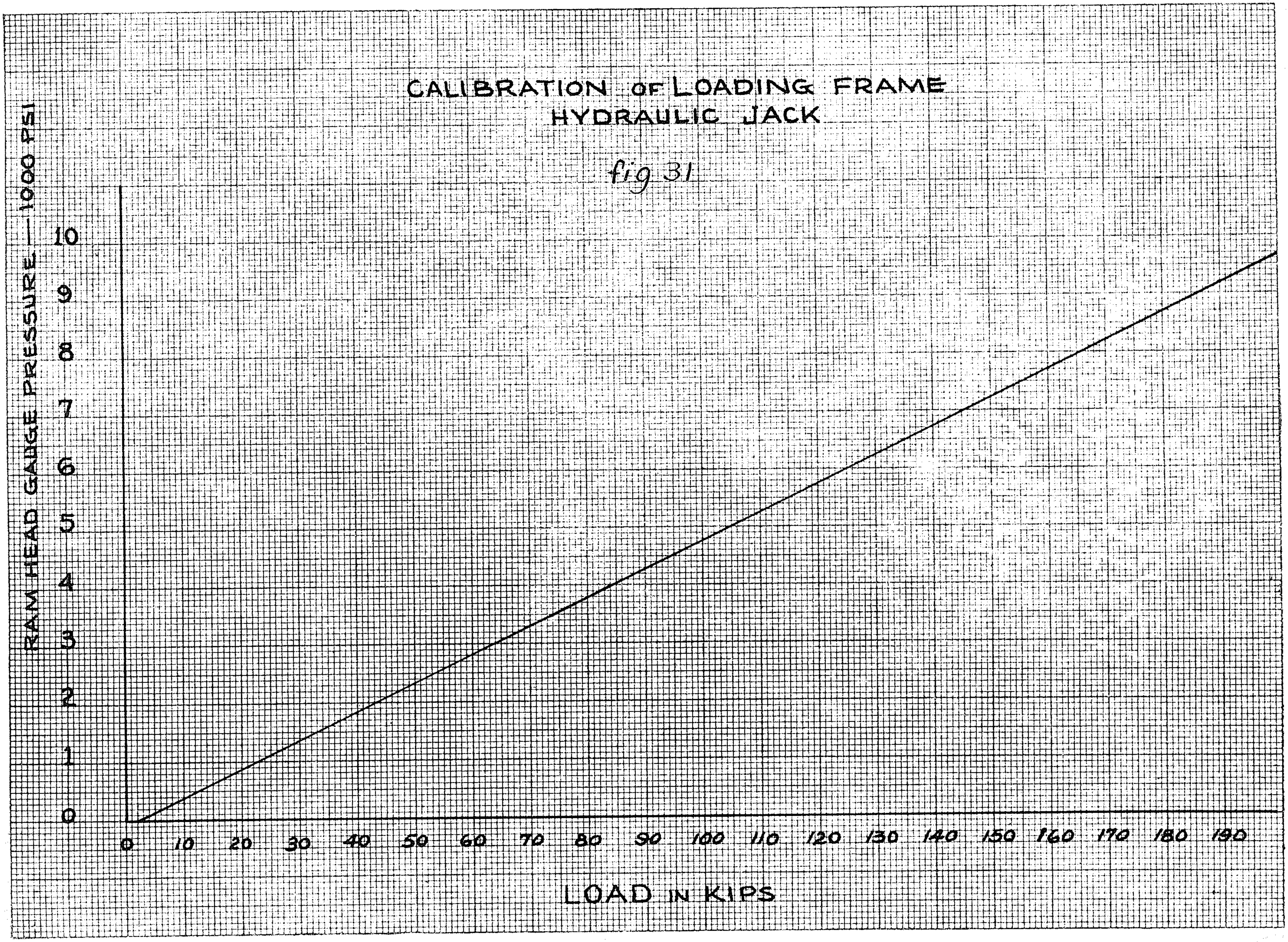
fig 24

54

55

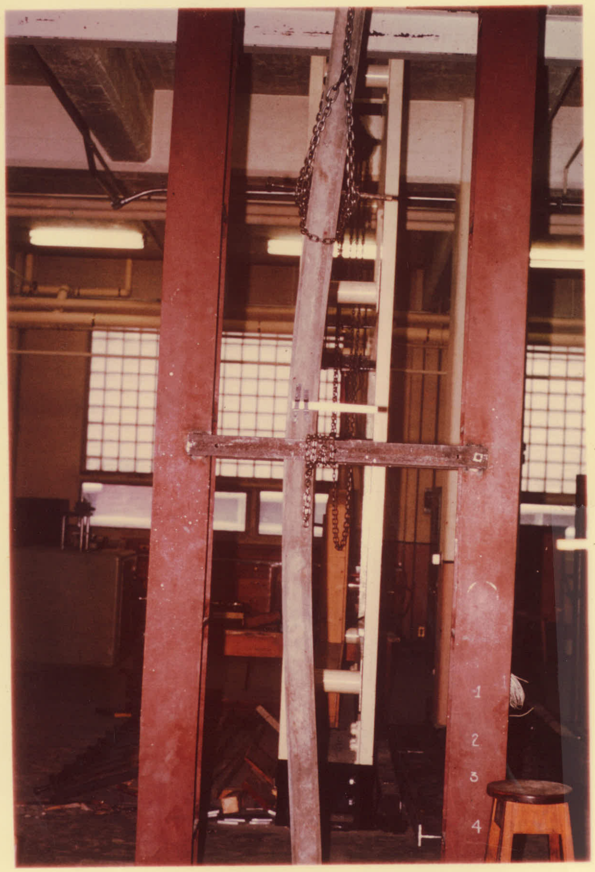


STRESS-STRAIN CURVE FOR ELASTIC MODULUS
MEAN OF PILOT TEST CYLINDERS
Fig 30
Ap
Bp
STRESS PSI
5000
4000
3000
2000
1000
0
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
STRAIN IN/IN x 10^{-4}


CALIBRATION OF LOADING FRAME
HYDRAULIC JACK
fig 31
RAM HEAD GAUGE PRESSURE — 1000 PSI
0
1
2
3
4
5
6
7
8
9
10
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190
LOAD IN KIPS

56

57

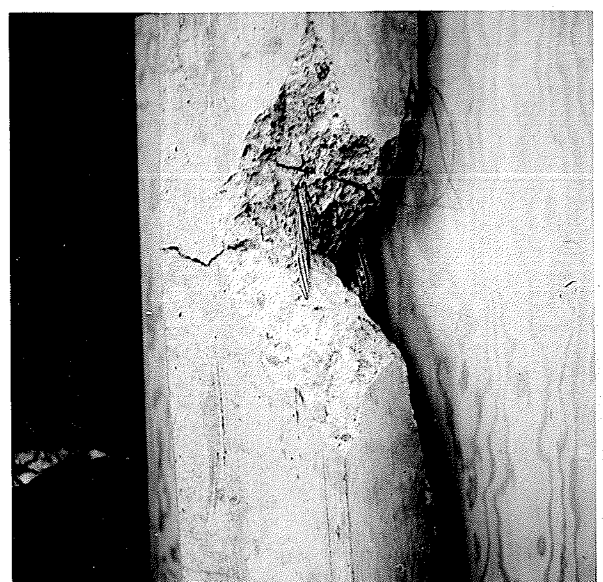


Column 2 B_p
Buckling failure
64 kips

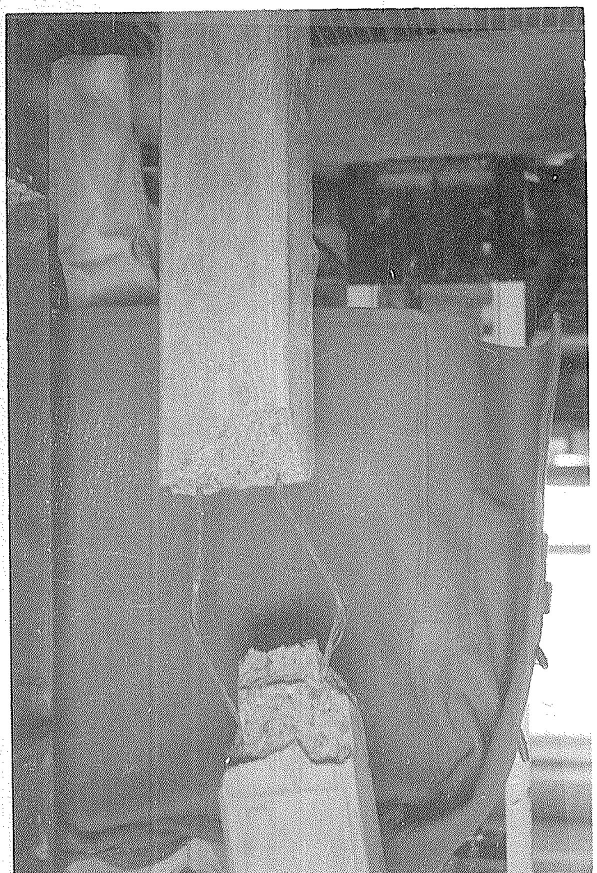
Column 2 B_p
Showing tensile cracks
after buckling failure

Column 2 B_p
Compression failure after buckling
load had been released and column
supported at center
175 kips

Column 1 A_p

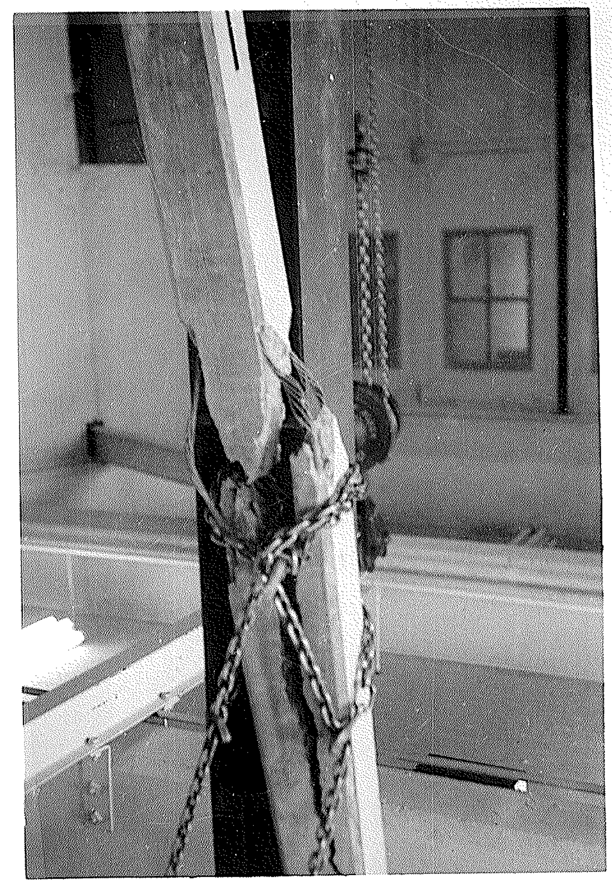


Column 1 A_p
Showing failure due
to eccentric load

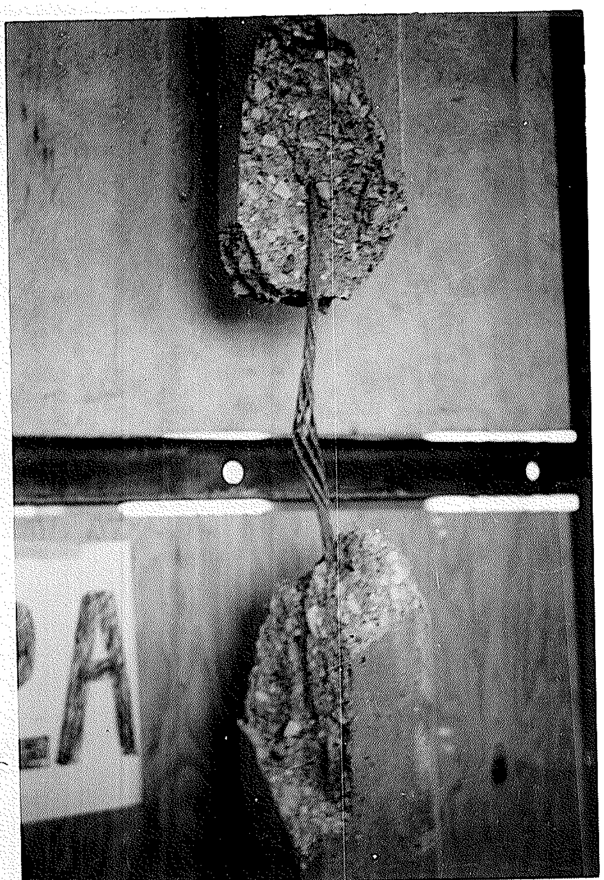


Column 3 B_p

Column 3 A_p



Column 2 B_p
Failure when
supported at
center

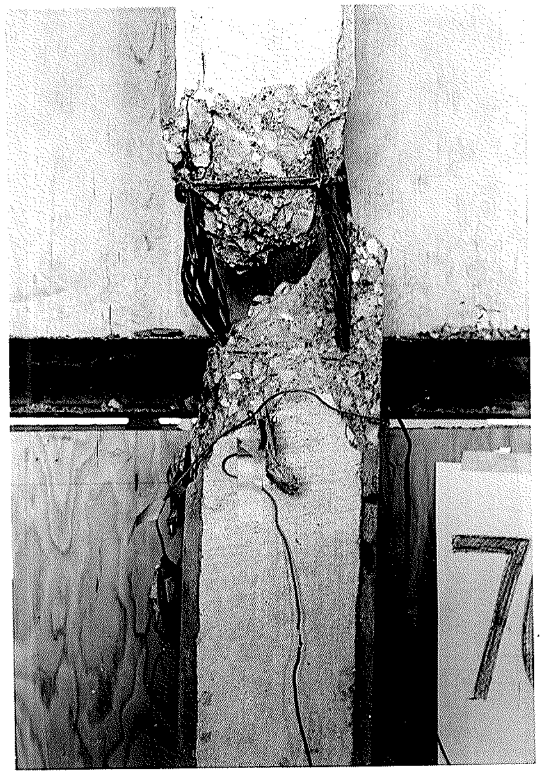


Column 2 A_T
p= .004

Column 3 A_T
p= .004

Column 8 B_T

Column 5 C_T
p= .0088
no ties



Column 7 C_T
p= .0088
# 2 ties @ 6"

Column 6 A_T
Reinforced Concrete
2 ties @ 6 "

APPENDIX of TEST RESULTS

COLUMN IAP - STRAIN/LOAD READINGS - SR-4 Gauges/Type A-

Load Gauge Pressure	Axial Load (K)	Gauge 1	Strain 1	Gauge 3	Strain 3	Gauge 2	Strain 2	Gauge 4	Strain 4
400	10	6158	0	9870	0	4660	0	2835	0
1000	22	6195	37	9800	70	4620	40	2792	43
2000	43	6252	94	9674	196	4535	125	2695	140
3000	64	6332	174	9535	335	4440	220	2600	235
40000	84	6465	307	9440	430	4350	310	2515	320
5000	104	6540	382	9330	540	4260	400	2438	397
6000	124	6625	467	9205	665	4168	492	2360	475
7000	145	6715	557	9074	796	4060	600	2270	565
8000	166	6810	652	8690	1180	3950	710	2180	655
8500	176	6880	722	8490	1380	3875	785	2120	715
9000	186	6935	777	8135	1735	3810	850	2065	770
9500	196	6990	832	7800	2070	3735	925	2012	823
10000	206	7050	892	7420	2450	3640	1020	1955	880
10500	216	7200	1042	6435	3435	3495	1165	1850	985
	after 15 hours with load left on								
9000	186	7340	1182	7760	2110	3270	1390	1828	1007
9500	196	7365	1207	7738	2132	3238	1422	1793	1042
10000	2061	7412	1254	7655	2215	3180	1480	1752	1083
11000	226	7535	1377	7410	2460	3042	1618	1660	1175

COLUMN 1AP STRAIN/LOAD READINGS - SR-4 Gauges/Types A- $1\frac{3}{8}$ inches eccentricity

Load Gauge Pressure	Axial Load (K)	Gauge 1	Strain 1	Gauge 2	Strain 2
1000	22	4/905	0	2/1540	0
2000					
2000	42	4/905	0	2/1550	10
3000	62	4/920	15	2/1565	25
4000	83	4/935	30	2/1580	40
5000	104	4/965	60	2/1600	60
6000	124	4/1005	100	2/1625	85
7000	145	4/1080	175	2/1660	120
7600	157	4/1145	240	2/1690	150
7800	161	4/1175	270	2/1705	165
8000	165	4/1210	305	2/1720	180
8200	169	4/1240	335	2/1740	200
8400	173	4/1275	370	2/1760	220
8600	177	4/1310	405	2/1785	245
8800	182	4/1365	460	2/1815	275
9000	186	4/1430	525	2/1850	310
9200	190	4/1580	675	2/1875	335
9400	194	<u>4/2000</u>	1095	2/1930	390
9600	failure by compression due to bending				

* Underlining indicates movement without additional load

COLUMN $2A_T$ – STRAIN/LOAD READING – SR-4 Gauges/Type A.

Load Gauge Pressure	Axial Load (K)	Gauge 1	Strain 1	Gauge 2	Strain 2	Gauge 3	Strain 3
500	11	10/1415	0	6/1435	0	12/45	0
1000	22	10/1400	15	6/1360	75	10/1980	65
1500	31	10/1340	75	6/1300	135	10/1920	125
2000	43	10/1280	135	6/1225	210	10/1865	180
2500	52	10/1220	195	6/1155	280	10/1805	240
3000	63	10/1150	265	6/1075	360	10/1745	300
3500	73	10/1090	325	6/1000	435	10/1680	365
4000	83	10/1025	390	6/920	515	10/1620	425
4500	93	10/945	470	6/815	620	10/1560	485
5000	104	10/865	550	6/715	720	10/1500	545
5600	116	10/760	655	6/565	870	10/1435	610
5800	120	10/705	710	6/490	945	10/1410	635
6000	124	10/650	765	6/420	1015	10/1395	640
6200	129	10/600	815	6/360	1075	10/1380	645
6400	133	10/540	875	6/230	1205	10/1375	650
6600	137	10/460	955	6/140	1295	10/1380	645
6800	141	10/300	1115	4/1855	1580	10/1430	615

Failure occured by buckling and compression

* Underlining indicates movement without additional load

April 1, 1962

COLUMN $3A_T$ – STRAIN/LOAD READINGS – SR-4 Gauges/Type A –

Load Gauge Pressure	Axial Load (K)	Gauge 1	Strain 1	Gauge 2	Strain 2
500	11	10/760	0	10/1160	0
1000	22	10/710	50	10/1120	40
1500	31	10/670	90	10/1075	95
2000	42	10/620	140	10/1020	150
2500	53	10/560	180	10/970	200
3000	63	10/510	230	10/900	270
3500	73	10/460	280	10/840	330
4000	83	10/410	330	10/780	390
4500	93	10/340	400	10/670	500
5000	104	10/310	430	10/610	550
5500	115	10/280	460	10/535	625
6000	124	10/260	490	10/425	725
6200	129	10/280	460	10/365	785
6400	133	10/325	415	10/330	840
6600	137	10/390	350	10/210	940

Failure occured by buckling and compression

※ Underlining indicates movement without additional load April 2, 1962

COLUMN $4B_T$ STRAIN/LOAD READINGS - SR - 4 Gauges / Type A-

Load Gauge Pressure	Axial Load (K)	Gauge 1	Strain 1	Gauge 2	Strain 2	Gauge 3	Strain 3	Gauge 4	Strain 4
500	11	12/410	0	10/1400	0	10/1770	0	10/1470	0
1000	22	12/365	45	10/1370	30	10/1710	60	10/1390	80
1500	31	12/320	80	10/1345	55	10/1640	130	10/1300	170
2000	42	12/280	120	10/1310	90	10/1580	190	12/1210	260
2500	53	12/230	170	10/1290	110	10/1500	270	10/1120	350
3000	62	12/180	220	10/1260	140	10/1415	355	10/1015	455
3500	73	12/140	260	10/1235	165	10/1330	440	10/920	550
4000	83	12/100	300	12/1210	190	10/1210	560	10/800	670
4500	93	12/70	330	10/1200	200	10/1110	660	10/680	790
5000	104	12/70	330	10/1180	220	10/970	800	10/540	930
5400	113	12/90	310	10/1190	210	10/750	1020	10/345	1025
5600	116	12/160	240	10/1210	190	10/590	1180	12/255	1115
5800	120	12/250	150	10/1240	160	10/250	1520		

Failure due to buckling and compression

* Underlining indicates movement without additional load

April 2, 1962

COLUMN 5C_T - STRAIN/LOAD READINGS - SR -4 Gauges/Type A-

Load Gauge Pressure	Axial Load (K)	Gauge 1	Strain 1	Gauge 2	Strain 2	Gauge 3	Strain 3	Gauge 4	Strain 4
500	11	12/1670	0	16/330	0	14/470	0	16/60	0
1000	22	12/1620	50	16/280	50	14/420	50	16/20	40
1500	31	12/1570	100	16/220	110	14/360	110	14/1960	100
2000	43	12/1520	150	16/170	160	14/305	165	14/1915	145
2500	52	12/1465	205	16/90	240	14/225	245	14/1860	200
3000	63	12/1420	250	16/15	315	14/160	310	14/1810	250
3500	73	12/1375	295	14/1940	390	14/75	395	14/1755	305
4000	84	12/1325	345	14/1850	480	12/1990	480	14/1715	345
4500	93	12/1290	380	14/1770	560	12/1890	580	14/1665	395
5000	104	12/1265	405	14/1680	650	12/1785	685	14/1630	430
5600	116	12/1265	405	14/1530	800	12/1570	800	14/1590	470
5800	121	12/1330	340	14/1430	900	12/1435	935	14/1580	480
6000	125	12/1375	295	14/1350	980	12/1175	1195	14/1570	490

Small increase in load produced failure by buckling and compression

* Underlining signifies movement without additional load

March 30, 1962

COLUMN 6 A_T – STRAIN/LOAD READINGS – SR –4 Gauges/Type A–

Load Gauge Pressure	Axial Load(K)	Gauge 1	Strain 1	Gauge 2	Strain 2	Gauge 3	Strain 3	Gauge 4	Strain 4
700	16	4/1700	0	6/1295	0	6/390	0	6/50	0
1000	22	4/1660	40	6/1265	30	6/370	20	6/20	30
1500	31	4/1585	115	6/1200	95	6/325	65	4/1970	80
2000	43	4/1515	185	6/1150	145	6/280	110	4/1920	130
2500	52	4/1430	270	6/1080	225	6/230	160	4/1870	180
3000	63	4/1365	335	6/1020	285	6/180	210	4/1820	230
3500	73	4/1285	415	6/955	350	6/140	250	4/1775	275
4000	83	4/1210	490	6/890	415	6/80	310	4/1725	325
4500	93	4/1135	565	6/820	485	6/40	350	4/1680	370
5000	104	4/1065	635	6/750	555	4/1990	400	4/1640	410
5500	115	4/980	720	6/670	635	4/1950	440	4/1605	445
6000	125	4/910	790	6/590	715	4/1900	490	4/1580	470
6500	135	4/805	895	6/475	830	4/1855	535	4/1560	490
6800	141	4/755	945	6/400	905	4/1830	560	4/1580	470
7000	145	4/690	1010	6/315	990	4/1810	580	4/1580	470
7200	149	4/635	1065	6/210	1095	4/1790	600	4/1585	465
7400	153	4/580	1120	6/110	1195	4/1770	620	4/1640	410
7600	157							4/1700	350
7800	161							4/1780	270

Failure occurred by buckling and compression.

* Underlining indicates movement without additional load.

March 31, 1962

COLUMN 7CT - LOAD/STRAIN READINGS - SR-4 Gauges/Type A -

Load Gauge Pressure	Axial Load (K)	Gauge 1	Strain 1	Gauge 2	Strain 2	Gauge 3	Strain 3	Gauge 4	Strain 4
500	11	14/260	0	16/410	0	10/920	0	12/875	0
1000	22	14/215	45	16/365	45	10/880	40	12/830	45
1500	31	14/160	100	16/ 315	95	10/820	100	12/770	105
2000	43	14/100	160	16/260	150	10/775	145	12/725	150
2500	52	14/40	220	16/190	220	10/710	210	12/660	215
3000	63	12/1965	295	16/125	285	10/660	260	12/610	265
3500	73	12/1900	360	16/50	360	10/600	320	12/560	315
4000	83	12/1820	440	14/1960	450	10/545	375	15/510	365
4500	93	12/1755	505	14/1870	540	10/480	440	12/465	410
5000	104	12/1660	600	14/1765	645	10/440	480	12/440	435
5500	115	12/1580	680	14/1650	760	10/390	530	12/410	465
6000	125	12/1415	845	14/1470	840	10/380	540	12/405	470
6200	129	12/1275	985	14/1355	955	10/405	515	12/405	470
6400	133	12/1100	1160	14/1200	1110	10/475	445	12/410	465
6600	rapid failure due to observed buckling and compression								

* Underlining indicates lateral movement without additional load

March 31, 1962

COLUMN $8B_t$ - STRAIN/LOAD READINGS - SR -4 Gauges/Type A-

Load Gauge Pressure	Axial Load (K)	Gauge 1	Strain 1	Gauge 2	Strain 2	Gauge 3	Strain 3	Gauge 4	Strain 4
500	11	10/1755	0	10/1650	0	10/1145	0	12/215	0
1000	22	10/1705	50	10/620	30	10/1090	55	12/150	65
1500	31	10/1660	95	10/1585	65	10/1030	115	12/70	145
2000	43	10/1605	150	10/1545	105	10/965	180	10/2000	215
2500	52	10/1560	195	10/1510	140	10/900	245	10/1915	300
3000	63	10/1500	255	10/1470	180	10/830	315	10/1840	375
3500	73	10/1460	295	10/1430	220	10/750	395	10/1745	470
4000	83	10/1405	350	10/1395	255	10/670	475	10/1650	565
4500	93	10/1360	395	10/1370	280	10570	575	10/1540	675
5000	104	10/1325	440	10/1360	290	10/460	685	10/1430	785
5200	109	10/1310	455	10/1355	295	10/380	765	10/1315	900
5400	113	10/ 1310	455	10/1360	290	<u>10/290</u>	855	<u>10/1225</u>	990
5600	116	10/1325	440	10/1360	290	<u>10/175</u>	970	<u>10/1140</u>	1075

Failure occured by buckling and compression

* Underlining indicates movement without additional load

April 1, 1962

MID-HEIGHT LATERAL DEFLECTIONS DUE TO AXIAL LOAD IN INCHES

Load Pressure	Load Kips	2AT	3AT	3AT	4BT	4BT	5CT	5CT	6AT	6AT	7CT	7CT	8BT	8BT	1AP ($\frac{1}{2}$" eccentricity)
500	12	0	0	0	0	0	0	0	0	0	0	0	0	0	
1000	22	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1500	32	0	.01	.01	0	.01	0	.01	.01	0	0	0	.01	.02	
2000	43	0	.02	.01	0	.01	0	.02	.01	0	.01	0	.02	.03	.04
2500	53	0	.04	.01	0	.03	.05	.02	.02	0	102	.01	.02	.03	
3000	63	0	.06	.02	0	.05	.05	.04	.03	.01	.02	.02	.03	.05	.09
3500	74	-.01	.08	.03	0	.07	.06	.05	.04	.01	.03	.03	.05	.07	
4000	84	-.01	.12	.04	.02	.11	.08	.06	.04	.03	.04	.05	.09	.08	.15
4500	94	-.01	.17	.05	.02	.15	.10	.09	.06	.03	.04	.06	.20	.10	
5000	104	.02	.22	.06	.04	.19	.16	.12	.09	.05	.05	.07	.37	.12	.22
5200	108			.08										.16	
5400	112		.33		.05	.29								.19	
5500	115								.11	.06	.08	.11			
5600	116	.04	.40		.07	.38	.25	.25						.23	
5800	121	.04	.53	.11	.09		.34	.36							
6000	125	.04		.15	.10				.14	.10	.11	.14			.28
6200	129	.06		.21	.17						.14	.19			
6400	133	.09		.28	.23						.24	.25			
6600	137	.11							.19	.13					
6800	141	.16							.22	.17					
7000	145								.27	.20					.33
7600	156														.39
7800	161														.43
8000	165														.45
8200	169														.48
8400	173														.51
8600	177														.55
8800	181														.59
9000	185														.64
9200	189														.70
9400	193														.77

CONCRETE CYLINDER TESTS

Concrete A_T

Load (K)	Stress	Strain 1	2	3	45	5	6	7	Avg
20	707	50	75	75	100	90	85	110	83
30	1060	150	150	190	200	175	200	215	183
40	1414	225	225	320	300	275	285	385	288
50	1767	300	325	390	400	375	395	420	372
60	2120	400	425	495	490	480	500	550	477
70	2474	500	525	560	590	585	605	660	575
80	2828	600	625	685	710	700	720	800	691
90	3181	700	725	800		865	830	930	808
100	3534	825	850	920	910	1005	965	1070	935
110	3887	950	975	1060	1010	1150	1120	1245	1073
120	4240	1050	1125	1210	1215	1365	1275	1445	1241
130	4594	1250	1275	1460					1420
Ultimate stress		5000	5200	4800	5300	4500	5250	5150	5200
						5500*	5400*	5600*	

* no strain readings taken

Concrete B_T

20	707		100	95	100	90	95		96
30	1060		150	190	200	160	185		177
40	1414	200	250	300	325	250	295		270
50	1767		325	410	420	340	395		380
60	2120	375	400	545	530	425	500		463
70	2424		500	640	650	510	630		585
80	2828	600	600	745	775	600	745		678
90	3181		700	855	925	690	860		806

C O N C R E T E   C Y L I N D E R   T E S T S

Concrete B_T

Load (K)	Stress	Strain 1	2	3	4	5	6	7	Avg
100	3534	800	800	1010		790	995		879
110	3887		975		1020	840	1145		995
120	4240	1050	1050	1335	1115	940	1295		1131
130	4594		1200			1065	1465		1243
Ultimate stress		5000	5400	5600	5350	5900	5900		5600
							5850*		

* no strain readings taken

Concrete C_T

Load (K)	Stress	Strain 1	2	3	4	5	6	7	Avg
20	707	85	100	70	90	70	105	95	88
30	1060	185	210	160	180	175	225	190	189
40	1414	270	325	270	290	275	350	290	296
50	1767	325	435	380	390	375	465	410	383
60	2120	425	550	495	490	470	580	520	504
70	2474	515	685	620	605	570	695	625	616
80	2828	610	815	745	725	670	815	745	732
90	3181	725	965	895	840	790	930	870	859
100	3534	840	1125	1070	990	935	1060	985	1000
110	3887	980	1325	1270	1100	1115	1205	1135	1161
120	4240	1140	1600	1530	1340	1315	1390	1300	1374
130	4594	1340	2025				1580	1500	1611
Ultimate stress		5000	4600	4600	4650	5200	5400	5300	5000
							5000*	5300*	

* no strain readings taken