ANALYSIS OF SEMI-RIGID PAVEMENTS

A Thesis
Submitted in Partial Fulfilment
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in
CIVIL ENGINEERING
(Highway Engineering)

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TO
MY
PARENTS

CERTIFICATE

Certified that the thesis entitled "ANALYSIS OF SEMI-RIGID PAVEMENTS" which is being submitted by Sri KULWANT SINGH in partial fulfilment for the award of the Degree of Master of Engineering in Civil Engineering (Highway Engineering) of the University of Roorkee is a record of the student's own work carried out by him under my supervision and guidance. The matter embodied in this thesis has not been submitted by him for the award of any other degree or diploma.

This is further to certify that he has worked for a period of about five and a half months between March and August 1972 at this University in order to prepare this thesis.

28/8/72

August 28 , 1972.

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Kulwant Singh.

CONTENTS

CHAPTER I

CHAPTER - 1

INTRODUCTION.

1.1 GENERAL :

Pavements are generally classified in two categories. Flexible and Rigid. Flexible pavements consists of a series of layers with the highest quality material at or near the surface(1). The load carrying capacity of Flexible pavements is brought by the load distribution characteristics of the layered system. The materials used in the layers are considered to have negligible flexural strength. The common flexible pavement layers are crushed stone or gravel, Water Bound Macadam (WBM), Bituminous mixes etc.

Rigid pavements are those having considerable flexural strength and high modulus of elasticity, capable of distributing load over a relatively wide area. The major portion of structural capacity of the Rigid pavements is supplied by the slab itself. Portland cement concrete is the only material considered in the category of Rigid Pavements. Hence at present there are two methods of pavement design - Rigid pavement design when considering materials which have considerable flexural strength (like that of cement concrete) and flexible pavement design when considering materials which have negligible flexural strength(2).

Though pavement materials like soil-cement, soil-lime-flyash, Pozzuolanic concrete and lean cement concrete have much low flexural strength than cement concrete, still they possess noteworthy flexural strength. The 28 days flexural strength of cement concrete used in pavement construction is in the range of 120 to 180 Kg/cm^2 whereas that of soil-cement usually used as base-course construction is in the range of 7 to 50 Kg/cm^2. Hence there is a need for an intermediate classification and a suitable method of design for such pavements. This category of pavement material may be classified as semi-rigid or semi-flexible materials.

According to Harris[3] the semi-flexible pavements should have sufficiently low elastic modulus and coefficient of thermal expansion so as to reduce temperature expansion, contraction and warping to the extent that jointing is not necessary. These may be achieved by lowering the strength below that of Portland cement concrete and by a judicious choice of the aggregates[3]. However from the stand point of thickness and economy it is desirable to provide as high a strength as possible.

1.2 PROBLEMS IN SEMI-RIGID PAVEMENT DESIGN :

Upto this time there has not been an accepted method of design for semi-rigid pavements. Soil-cement pavements have been designed by a conventional flexible pavement design method by some agencies like the U.S. Corps of Engineers[4]. British practice is also to consider soil-cement as a flexible pavement[5]. This means soil-cement base-course is considered equivalent to the granular base-course, in thickness requirement. Highway Research Board, U.S.A. has recommended the soil-cement base-course thickness equal to granular base-course on good quality sub-grades; but on weaker subgrades the thickness of soil-cement equal to three-quarters of the required granular base-course has been recommended. Semi-Rigid pavements like soil-cement have been reported[6,7] to deflect upto a considerable distance from the loaded area and thus transmit the load through a wider area to the lower layer. This results in noteworthy reduction in vertical stress transmitted by the semi-rigid layer and lower value of maximum deflection in comparision to flexible pavement layer of equivalent thickness. This means that the thickness requirement of semi-rigid pavement could be less than the granular base-course under identical conditions of subgrade, climate and traffic.

An empirical, flexible pavement design method like the C.B.R. method also seems to be unsuitable for the design of semi-rigid pavements. This is because the C.B.R. value of the semi-rigid material very much exceed 100 % which is designated for a standard crushed stone. The soaked C.B.R. values of soil-cement mixes commonly used in base course after 7 days curing vary from 100 % to over 500 % depending upon the soil type[8]. This means that C.B.R. test itself cannot be adopted to assess the strength characteristics of semi-rigid materials. Moreover the total pavement thickness requirement by C.B.R. method does not depend on the strength characteristics of the pavement component layers.

Thus it may be concluded that by the conventional flexible pavement design methods it may not be possible to arrive at the exact thickness requirement of semi-rigid pavements. It may be too conservative to adopt the semi-rigid pavement thickness equivalent to flexible pavement. Empirical approaches based on load carrying capacity have indicated that the thickness requirement of semi-rigid pavement would be less than flexible pavement.

The thickness of semi-rigid base course designed by rigid pavement design methods is at times even higher than the thickness designed by a flexible pavement design method. As an example thickness of a soil-cement base course by Westergaard method of rigid pavement design works out to about 60 cm (24 inches) when the flexural strength of material is 3.5 Kg/cm^2 (50 psi), subgrade modulus 2.75 Kg/cm^3 (100 pci) for a wheel load of 4050 Kg (9000 lbs). However in practice a base-course thickness of 15cm (6 inches) has been reported to be adequate to carry the wheel load under these conditions(5). Therefore it seems that rigid pavement design methods may not be directly applicable for the design of semi-rigid pavements. Further analysis of combined stresses due to wheel load and temperature changes need careful consideration in the case of semi-rigid pavements as these are likely to be different from rigid pavements. Further formation of shrinkage cracks in semi-rigid pavements and the subsequent behaviour of the slabs under varying climatic and traffic conditions are difficult to be predicted or evaluated thus making the analysis complicated. Thus it may be seen that the design of semi-rigid pavements cannot be fully based either on a flexible pavement design approach or a rigid pavement design approach. This is also because of the fact that the behaviour of semi-rigid pavements laid over soil subgrade under repeated application of loads has not been fully established. In other words the fatigue behaviour of semi-rigid pavements have not been fully evaluated. It is also possible that the fatigue behaviour of different materials falling in the category of semi-rigid pavement materials may have different fatigue behaviour. Hence it is necessary to investigate and arrive at suitable methods for the analysis and design of semi-rigid pavements.

1.3 REVIEW OF LITERATURE.

1.3.1 Introduction :

There has not been an accepted method of design or analysis for semi-rigid pavements. Of the various types of semi-rigid pavements that may be constructed, soil-cement is the most commonly adopted material and hence considerable investigations have been carried out on this material with a view to design the base-course thickness.

The conventional method for the design of semi-rigid pavements are made in two steps. In the first step the mix is designed to fulfil certain criterion of strength or durability or both. In the case of soil-cement there are some standerised methods of mix design whereas for other semi-rigid materials there are no standard methods upto this time. The second step is the thickness design of the semi-rigid base-course material. The thickness requirement is generally decided by a conventional flexible pavement design method and at times suitable modifications in the evaluated thickness values are allowed based on experience. There have also been attempts to design the base-course based on Elastic layer System theory, Westergaard theory and by certain empirical approaches.

1.3.2 Thickness Design by Flexible Pavement Design Methods :

As discussed earlier the first step in the design is the design of mix so as to fulfil the prescribed requirements. Soil-cement mix is commonly designed by PCA mix design method which considers durability in prescribing a design criteria. The durability test consisting of wet-dry and freeze-thaw tests have been standardised by ASTM. Soil-cement mix is also designed by the British method which is mainly based on a compressive strength requirement after 7 days curing. However, standard methods of test and design specifications are not available for other semi-rigid materials such as soil-lime-flyash and Pozzolanic concrete. Whatever be the mix design method the main objective seems to be to obtain a mix which is durable and strong enough to be considered equivalent to the granular base-course. The various thickness design approaches which consider soil-cement as flexible pavement layer are given below.

U.S. Corps of Engineers Method :

The U.S. Corps of Engineers treats soil-cement as a flexible material and the soil-cement pavements are designed using the CBR design procedure[4]. The total thickness is determined based on the CBR value of subgrade and the traffic load. The Corps of Engineers assigned a CBR value 50 to 80 to soil-cement mixtures based on service behaviour.

Some investigators consider CBR test on soil-cement inapplicable because of the hard and brittle nature of the material.

Highway Research Board Recommendations :

The HRB (U.S.A) carried out a nation wide survey and based on the results recommended soil-cement thicknesses for various subgrade soils. Generally equal soil-cement thickness as granular base thickness is recommended on good quality subgrades, whereas a soil-cement thickness equal to three-fourth of the required thickness of granular base-course may be used on weaker soils.

California Highway Department Method :

The California Highway Department designs cement-treated bases by means of Hveem Cohesiometer value and the resistance value of the subgrade. Included also in the design are such variables as traffic intensity and anticipated wheel loads. The California design manual commonly permits soil-cement thicknesses less than the required thickness of granular base(4). It is reported that using the California design method, the required thickness of soil-cement is generally 35-50 % less than that required for a granular base. This procedure is notable in that it recognizes soil-cement as a construction material that can be designed for in the same manner as other common paving materials like asphalt concrete(5). For example, a typical reduction would be 1 inch of class A cement-treated base for 1.72 inches of gravel. This large reduction in thickness is possible because of the use of processed aggregates, rigid quality control, and inspection during construction.

British Practice :

The British practice is to consider soil-cement as a flexible pavement material and to design the thickness by a flexible pavement design method. Generally the soil-cement mix is designed to have 17.5 Kg/cm^2 (250 psi) compressive strength after 7 days curing.

The British have also investigated the applicability of a method based on shear strength of the subgrade. This procedure is applied to subgrades having a strength independent of the overburden pressure (i.e., When $\phi = 0$)(5). In this method the shear strength of the subgrade soil is compared with the maximum shear stress induced at any depth by a given wheel load and tyre pressure, as determined by the theory of elasticity. A thickness of soil-cement is selected such that at any depth greater than the base thickness, the induced shear stresses are less than the shear strength of the subgrade. This method has proven satisfactory for the design of pavements over very weak subgrades.

Thickness Design For Equal Load Carrying Capacity In Terms of Crushed Stone Base- Course :

The load carrying capacity of soil-cement base course for equal deflections is reported to be considerably higher than equal thickness of granular base-course. Nussbaum and Larsen[6] made a comparative study of load capacities at the PCA Research Laboratories. The ratio of thickness of soil-cement to granular base was about 1.5 times for 10 cm (4 inches) and about 3.3 times for 25cm (10 inch) thicknesses for both weak and strong subgrades.

The load capacity of soil-cement bases was evaluated by the Canadian Good Road Association[9] by collecting data of Benkelman Beam rebound deflections. They found that if a design rebound deflection of 0.075cm (0.03 inch) is contemplated, one unit of soil-cement base course is effective as nearly 3 units of crushed granular base-course, when subgrade has a rebound deflection of 0.2 cm (0.08 inch). Some of the typical ratios obtained by them are given below :

Subgrade Deflection		Ratio of depth of crushed gravel to depth of CTB for various design deflections.		
cm	in.	0.075cm(0.03in)	0.1cm(0.04in)	0.125cm(0.05in)
0.10	0.04	2.1	-	-
0.20	0.08	2.9	2.3	2.2
0.40	0.16	2.8	2.3	2.1

Design by Elastic Layer Theory :

Sowers and Vesic[10] showed that the vertical subgrade stress values under soil-cement bases were comparable with those computed by elastic two-layer theory. Vesic[7] suggested that the Elastic Two-layer theory is valid only when the reinforcing layer possesses an appreciable tensile strength as in the case of soil-cement.

Mitchel and Shen[11] suggested that soil-cement base course thickness may be designed by the Three-layer theory, considering the horizontal tensile stress-strain at the bottom of soil-cement layer and a vertical compressive strain at top of subgrade.

1.3.3 Rigid Pavement Design Approach :

Attempts have been made to design semi-rigid pavements using rigid-pavement design approaches also. Some investigators have indicated that rigid pavement design methods are suitable for the design of semi-rigid pavements too, whereas others have indicated that they are not suitable as a thickness requirement works out to be considerably high.

Westergaard's Method :

Baker and Papazian[12] showed that the flexural stress obtained by using Westergaard and Burmister approaches were almost identical for the same loading conditions. Matcalf and Frydman[2] have suggested the use of Westergaard equations for evaluating tensile stresses in soil-cement pavements. They considered the corner loading condition as most critical.

Meyerhof's Method :

Thickness design of Lime-Flyash aggregate bases was investigated by Baranberg[13]. He suggested the use of Meyerhof theory for ultimate failure of slabs. The desired factor of safety may be given based on fatigue studies.

1.3.4 Thickness Design by Experimental Methods :

A comprehensive research programme to develop a thickness design procedure for soil-cement pavements by was initiated at the PCA laboratories[6]. The first part of the study was in establishing relationship between load and deflection for soil-cement slabs by plate load tests. The second part was aimed at evaluation of the fatigue properties of the soil-cement. A best-fit straight line equation was obtained from a non-dimensional logarithmic plot of test data, $\frac{\Delta K}{p}$ vs $\frac{a}{h}$. The equation was of the form :

$$\frac{\Delta K}{p} = \alpha \left(\frac{a}{h}\right)^{\beta}$$

Where

Δ = the deflection

K = modulus of subgrade reaction

p = applied pressure.

a = radius of loaded area

h = thickness of soil-cement base

α = a parameter, the ordinate on the best-fit line corresponding to an abscissa $\frac{a}{h} = 1$

β = slope of regression line.

Nielson[4] has also suggested a similar design equation.

Knowing the values of α and β from the best-fit line and adopting the design values of Δ, K, p and a, it is possible to find the desired thickness h, of the soil-cement base.

Based on the fatigue study, an equation of fatigue function was given as :

$$\therefore \frac{R_c}{R} = V N^{-\delta}$$

Where

R_c = critical radius of curvature which is constant for a given material, but varies with thickness.

R = allowable radius of curvature for N number of load applications.

V = factor varying with thickness h of base,

$$V = \frac{h^{2/3}}{(2.1h - 1)}$$

δ = dimensionless exponent dependent on soil type.

The allowable stress ratio should be determined from fatigue studies for any desired number of stress repititions N.

1.4 SCOPE OF PRESENT STUDY.

1.4.1 Objects

Design of semi-rigid pavements by flexible-pavement design approaches have mainly two drawbacks. The first drawback is in the mix design as the mix is assumed to perform in the same manner as the flexible pavement materials or the crushed stone base. Mixes designed totally based on durability criteria are likely to cause greater variation with respect to the strength and load transmission characteristics of the resulting mixes. The second drawback is in assuming the gravel equivalence during the thickness design process. As reviewed above some agencies assume the thickness requirement of semi-rigid layer to be equivalent to the granular base-course, whereas others allow a reduction in thickness based on some gravel equivalence factor. Various agencies have suggested different values of this factor, some of them based on performance studies and others based on some strength test results. Further the flexible pavement design method itself may be either fully or partly empirical. Thus there are difficulties in arriving at a rational design approach for semi-rigid pavements by flexible pavement design methods. However, based on the studies of Sowers and Vesic(10) and the report of Vesic(7) the elastic layer theory appears to be a probable solution for a rational approach in the analysis of semi- rigid pavements.

The recommendations of Metcalf and Frydman(2) for the use of Westergaard method for analysing soil-cement pavements seems to be worth investigating further. Similarly the suggestions of Barenberg(13) for adopting Meyerhof ultimate load anlysis also needs further investigation with a view to arrive at a rational method in the design and analysis of semi-rigid pavements.

In this investigation it is aimed at evaluating suitability of the various theoretical approaches for the design and analysis of semi-rigid pavements. For this purpose the stiffness ratio[12] E/K is considered to be in the range from 100 to 10,000 to represent a wide range of semi-rigid pavement materials. The suitability of Elastic layer theory, Westergaard's analysis and Meyerhof's ultimate load analysis for the analysis and design of semi-rigid pavements of the above range, was the main objective of the study.

1.4.2 Scope :

Using the Elastic two-layer theory, charts were prepared for the maximum vertical stress on subgrade σ_z , flexural stress due to interior load fi and maximum flexural strain e for various values of E_1/E_2 and thickness h. These charts can also be used for design purpose to limit the values of allowable stresses and strains in the base-course layer. There has been considerable discussion over the deflection criterion in pavement design as it is argued that it is more reasonable to specify the allowable minimum radius of curvature in a pavement slab. Hence the variation in the minimum radius of curvature R_o in the two layer system due to variation in loading characteristics and slab thickness was theoretically analysed. The chart for calculating the radius of curvature of the two layer system given by Odemark[14] was extended for higher ratios of E_1/E_2 upto 500, using an electronic Computor IBM 1620. The theoretically evaluated values were compared with the experimental values of another investigation.

Westergaard's expressions for load stress of pavement slabs are commonly adopted for cement concrete slabs. The validity of these expressions for semi-rigid materials having lower values of elastic modulus E_1, were studied. Charts were prepared for evaluating the load stresses due to interior, edge and corner loading for various stiffness ratios E/K and varying thicknesses h. These charts could also be useful for thickness design by adopting appropriate strength values.

The breaking load for cement concrete pavements estimated by elastic theory is reported to be much lower than actual values and hence the Meyerhof's ultimate load analysis is preferred by some investigators. The applicability of Meyerhof's analysis for the range of semi-rigid materials was studied . Charts were prepared for evaluating the flexural stress in slabs of different values of E and h. The failure loads estimated theoretically by Westergaard and Meyerhof's methods of analysis were compared with the experimental results of another investigation. The results thus obtained were critically analysed and a comparative study was made.

CHAPTER 2

CHAPTER-2.

ANALYSIS BY ELASTIC LAYER THEORY.

2.1 THEORETICAL ANALYSIS :

In the analysis by Elastic Layer theory here the two-layer system is being considered, though generally semi-rigid pavements fall in the category of three-layer system. The semi-rigid base-course material need a bituminous wearing course at the top and hence the pavement system consists of three layers. However, in this Country the bituminous wearing course consists of a bituminous surface treatment of relatively small thickness. The thickness values of these surface courses seldom exceed 2 cm in the case of bituminous surface dressings and 2.5 cm in the case of bituminous carpet. Such a thin bituminous layer cannot be considered to add to the structural strength. The bituminous course may be considered as the first layer of the three-layer system only when the bituminous concrete of thickness greater than 5cm is provided over the semi-rigid base-course. Hence when thin bituminous surface dressings/ carpet is used as a wearing course it is justified to consider the pavement as a two layer system. However, the bituminous wearing surface would slightly reduce the stresses in the pavement system due to the increase in effective loaded area to be considered on the top of the base course.

In a two-layer system increase in the modular ratio E_1/E_2 causes considerable reduction in vertical stress on the subgrade but at the same time the radial tensile stress in the reinforcing layer increases rapidly. Thus the design of a two layer system has to take into account the following factors :

(i) the allowable vertical stress on the subgrade.

(ii) the allowable flexural stress under the pavement slab.

(iii) the allowable deflection of the slab.

(iv) the allowable maximum tensile strain of the slab.

The tensile strain is dependent on the flexural stress and the radius of curvature of the slab and the slab thickness. The radius of curvature is likely to be influenced by the maximumum deflection, loaded area and the values of elastic modulus of the two layers. In a semi-rigid pavement where the modular ratio is fairly high (as compared to flexible pavements) the governing factor for the design would be the flexural stress or the tensile strain and not the vertical subgrade stress.

Evaluation of Stresses, Strains and Deflections :

The common range of modular ratios in the case of soil-cement base course have been reported[10,11] to be 100 to 200. Taking into consideration the possibility of having base-courses of higher rigidity, the range of E_1/E_2 considered in this investigation was from 100 to

500 .

For a design wheel load of 4100 Kg (axle load of 8200 Kg or 18000 lbs) and an average tyre pressure of 5.8 Kg/cm^2 (commonly applied tyre pressure in truck tyres), the radius of equivalent circular contact area works out to 15cm. Hence the vertical subgrade pressure (σ_z) expressed as a ratio of contact pressure (p) was evaluated for various modular ratios and slab thickness values (h) ranging from 10 to 30 cm making use of the charts given by Fox[15]. Figure 1 shows variation in vertical subgrade pressure σ_z with slab thickness for modular ratio E_1/E_2 equal to 100, 200, 300 and 500.

Figure 2 shows the relationship between the flexural stress (σ_b) at the bottom of the slab expressed in terms of contact pressure (p) (i.e., σ_b /p) and slab thickness h for various modular ratios E_1/E_2 equal to 100, 200, 300 and 500 for a = 15 cm. These charts have been obtained from the charts presented by Odemark[14]. From Figure 1 and 2 it is seen that both the vertical stress and flexural stress decrease with increase in slab thickness. Thus it is possible to find vertical subgrade stress and flexural stress in the slab for any slab thickness h for the design wheel load of 4100 Kg and a = 15cm provided E_1/E_2 of the two layer system is known. If the allowable values of the stresses with material are decided, it is possible to make use of these charts directly for finding slab thickness required for the design wheel load.

The tensile strain in the slab depend upon slab thickness and deflection under a certain loaded area. The tensile strain under the semi-rigid slab is also a factor to be considered in the design. Hence the maximum tensile strain (e) under the design load (a = 15cm) for varying slab thicknesses were calculated for different E_1/E_2 ratios and deflection levels making use of the tables of combined curvature- deflection function(16) in terms of E_1/E_2 , a/h , poissons ratio of pavement layer μ_1 and poissions ratio of lower layer u_2 , derived from the analyses of Burmister and Odemark. Figure 3 shows the relationship between tensile strain e and slab thickness h for E_1/E_2 100 and 200 at deflection levels (Δ) of 0.01, 0.03, 0.05 and 0.1 cm. The principle of obtaining such a chart has been illustrated in Appendix-I. This chart is thus useful to find the strain for any desired slab thickness and a given set of values of E_1/E_2 and deflection (Δ) for the design load (a = 15 cm). The chart can also be useful for estimating the thickness requirement of the slab for any allowable strain value in a certain case. It may be seen that in most of the cases there is no noteworthy change in the magnitude of the strain by ranging the slab thickness, if the deflection value is mentioned constant.

This is a very useful observation which indicates that a deflection criterion is justified for the design of such slabs.

The deflection values under the design wheel load (P = 4100 Kg, a = 15cm) were calculated for various values of slab thicknesses h, subgrade modulus E_2 and varying ratios of E_1/E_2 using Burmister two-layer theory. Figure 4 shows such a chart of deflection values for E_1/E_2 values varying from 20 to 500 and thickness values ranging from 10 to 30cm. This chart will be of use for finding the deflection of a given slab, for estimating the slab thickness mfor limiting the deflection to any desired value.

Evalution of Radius of Curvature :

As indicated earlier the radius of curvature is considered to be a more reliable measure of stress and strain conditions of a slab subjected to a certain load. Larsen, Nussbaum and Colley[16] consider the minimum radius at which the specimen fails under the static load as a measure of flexural strength. Hence it was decided to find the radius of curvature of slabs of different thickness and modular ratio when subjected to a certain design load. Odemark[14] has given the function F_R for the radius of curvature, for modular ratios E_1/E_2 equal to 2 to 100 for various h/a ratios. Since semi-rigid pavements are considered to have E_1/E_2 values ranging upto 500, it was first decided to extend chart for E_1/E_2 values between 100 and 500.

The equations used in analysing the values of the minimum radius of curvature R_o are given below :

$$R_o = \frac{4 E_1 a}{3 p} \times F_R \quad --- \quad (2.1)$$

Where

R_o = minimum radius of curvature.

a = radius of loaded area.

E_1 = modulus of Elasticity of soil-cement layer.

p = applied pressure

F_R = a function given as

$$F_R = \frac{E_2}{E_1} \times F_1 \quad --- \quad (2.2)$$

Here E_2 = modulus of elasticity of subgrade

F_1 = a function depending upon h/a and E_1/E_2 ratios and is given as.

$$F_1 = \frac{1}{\left(1 - \frac{1}{(\psi\eta_1)}\right)\frac{E_2}{E_1} + \frac{1}{(\psi\eta_2)}} \quad --- \quad (2.3)$$

$$\psi\eta = \frac{(1 + \eta^2)^{5/2}}{1 + 4\eta^2}$$

$$\eta_1 = 0.9 \times h/a$$

$$\eta_2 = 0.9 \times h/a \times \sqrt[3]{E_1/E_2}$$

Where h = thickness of pavement layer.

2.2 **Experimental Verification :**

The theoretical values obtained in this investigation have been compared with some of the experimental results of another investigation[17] carried out in the university using soil-cement slabs.

The values of E_1/E_2 for the soil-cement slabs reported here were evaluated using Burmister's approach. Using a set of dial gauges the deflection profile of the slabs were obtained for different loading conditions upto loads corrosponding to ultimate failure of slabs. The flexural strength of these slabs was determined by testing beams which were sawn out from the slabs.

Using the observations of the above investigation on soil-cement , four slabs of thicknesses 8, 10, 12 and 15cm were selected. Deflection profiles were drawn for various load values and the minimum radius of curvature R_0 of the each slab was found graphically in each case. The experimental values (obtained from these tests) of deflection, Strain, minimum radius of curvature and flexural stress were compared with theoretically calculated values.

Deflection :

The value of load required on 30 cm diameter plate to produce 0.05cm deflection were noted for the four slabs (cement-content 8 %) of thicknesses 8, 10, 12 and 15 cm.

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$$\eta_1 = 0.9 \times h/a$$

$$\eta_2 = 0.9 \times h/a \times \sqrt[3]{E_1/E_2}$$

Where h = thickness of pavement layer.

In order to extend the existing plot given by Odemark[14], for E_1/E_2 values ranging from 100 to 500 the main variable required to be found out is the function F_R. A computor programme was prepared for this function and fed into the electronic Computor IBM 1620. The Computor programme as well as the table showing the values of the F_R for various E_1/E_2 and h/a ratios are given in the Appendix II.

For any given value of h/a and E_1/E_2 the value of the function F_R can be found out using the table (Appendix -- II). Using this function the value of the minimum radius of curvature R_o can be easily calculated using the Equation 2.1.

The variation of theoretical values of minimum radius R_o with variation in contact pressure p for various E_1/E_2 and h/a ratios and E_2 values were graphically represented. Some of the graphs obtained (R_o vs p) for h/a = 1 and E_1/E_2 = 100, 200, 300 for various E_2 values are shown in Figures 5, 6 and 7. These figures can be used to determine the theoretical vales of R_o in a two-layer system for a given set of values of h/a, E_1/E_2 and E_2 for a given contact pressure p of the applied load. Figure 89 shows the variation of R_o with slab thickness h for E_1/E_2 = 100, E_1 = 7000 Kg/cm^2, p = 5.8 Kg/cm^2 and a = 15cm. It is seen that the value of R_o increases with h, other factors remaining unaltered.

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The theoretical values obtained in this investigation have been compared with some of the experimental results of another investigation[17] carried out in the university using soil-cement slabs.

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Deflection :

The value of load required on 30 cm diameter plate to produce 0.05cm deflection were noted for the four slabs (cement-content 8 %) of thicknesses 8, 10, 12 and 15 cm.

The theoretical values of deflection at the above load values were computed using Burmister's two-layer theory. The deflection and load values corresponding to ultimate failure of slabs due to interior loading (When cracks were formed at the top surface of the slab) were noted and the theoretical values of deflection were also evaluated corrosponding to the failure loads. The loads at 0.05 cm deflection were only a fraction of the ultimate load causing failure. The experimental and theoretical values of deflection for the four slabs are given below in Table 2.1 for comparison.

TABLE 2.1

Comparison of Experimental and Theoretical Values of Deflection in Soil-Cement Slabs.

Sl. No.	Ratio of load corresponding to 0.05cm deflection to ultimate load.	Deflection		Deflection at failure	
		Actual cm	Theoretical cm	Actual cm	Theoretical cm
1	0.140	0.05	0.048	0.33	0.28
2	0.240	0.05	0.047	0.38	0.28
3	0.330	0.05	0.051	0.44	0.22
4	0.339	0.05	0.042	0.25	0.18

It may be seen that the load applied at 0.05cm deflection is only about 30 percent or less of the ultimate load corresponding to failure conditions. Hence these loads applied may be considered well within the elastic limits of the slab. It may be seen from columns 3 and 4 of Table 2.1 that the theoretical and experimental values of deflection quite closely tally with each other. However, at failure loads the theoretical deflections (column 5 and 6) are found to be considerably lower than the experimental values of deflection. The higher values of observed deflection at failure may be explained as given below.

When the load on the plate at the interior of the slab is increased, first failure starts from the bottom of the slab at the subgrade interface by forming fine cracks starting from bottom. At this stage cracking is not visible from top whereas deflection increases rapidly with further increase in load. Thus higher values of observed deflection are obtained as the loads are increased upto the stage when the cracks are visible at the top.

Minimum Radius of Curvature :

The deflection profile of the slabs for various loads were plotted as shown in Figure 8 from the deflected shape of slab the minimum radius of curvature corresponding to various values of contact pressure were graphically obtained. The values of minimum radius R_o thus experimentally determined for soil-cement slabs were found to be much lower than the theoretically determined values, of R_o for identical conditions (of E_1/E_2 , h/a and E_2). The variation of minimum radius R_o with contact pressure p obtained for two of the slabs have been plotted in Figures 5 and 7 for comparison with the theoretical values. The probable reason for lower value of R_o in the soil-cement slabs is that the material has been comparatively more flexible with respect to the high modular ratio determined based on maximum deflection criterion. It is also possible that Burmister's two-layer theory is not applicable for the analysis of soil-cement slabs. The value of minimum radius of curvature at failure determined experimentally for the four slabs mentioned above have been given in Table- 2.2 below along with the theoretically determined values for comparison which shows a considerable difference between the two set of values.

Strain at Failure :

The strain at failure of the slabs were determined from the minimum radius of curvature at failure using the relation :

$$e_f = \frac{h}{2\,R_c}$$

Where

e_f = failure strain in flexure

h = slab thickness

R_c = minimum radius of curvature at failure.

The experimentally determined values of minimum radius of curvature at failure were substituted to get the experimental failure strain values. Similarly the theoretically evaluated values of minimum radius at failure were substituted to evaluate theoretical failure strain values. As the theoretical values of minimum radius are higher than experimental values, the theoretical strain values are lower than the experimental values. The theoretical and experimental values obtained for the four slabs have been compared in Table 2.3.

Table- 2.2

Comparison of Theoretical and Experimental values of Minimum radius of Curvature at failure in Soil-Cement Slabs.

Sl.No.	Minimum Radius at failure, 10^3 cm	
	Experimentally determined from deflection profile.	Theoretically determined by Two-layer theory.
1	1.7	3.64
2	2.1	4.40
3	2.6	10.10
4	3.0	11.45

Table 2.3

Comparison of Experimental and Theoretical values of Strain at Failure in Soil-Cement Slabs.

Sl. No	Failure Strain	
	Experimental	Theoretical
1	0.00236	0.00113
2	0.00238	0.00113
3	0.00230	0.00060
4	0.00250	0.00070

Flexural Stress :

The theoretical values of flexural stress for the four slabs were determined for the loads corresponding to 0.05 cm deflection as before. The flexural stress values were calculated using the charts given by Odemark(14). Table 2.4 gives the values of flexural stress calculated for the four slabs and the flexural strength values found experimentally for these slabs.

It may be seen that for Slab 1 the applied load is only 0.14 of the ultimate load and the flexural stress by two layer theory was 9.46 Kg/cm^2 whereas the actual flexural strength of the slab was only 6.8 Kg/cm^2.

Similarly in other slabs too it may be seen that the theoretical flexural stress values exceeded flexural strength values of soil-cement in all the four cases though the applied load was only a fraction of the ultimate load. This means that the flexural stress values found from two layer theory are much higher than the actual flexural stress found in soil-cement slabs. This is contrary to the minimum radius of curvature and failure strain evaluated by the two-layer theory. Hence forther, research is needed to verify the validity of various aspects of the two-layer theory before being used for semi-rigid pavement design.

Table 2.4

Comparison of Theoretical flexural Stress with Experimental Flexural Strength for Soil- Cement Slabs.

Sl.No.	Ratio of Load applied to failure load.	Flexural Stress by two-layer theory Kg/cm²	Actual flexural Strength determined Experimentally. Kg/cm²
1.	0.140	9.46	6.8
2	0.240	11.00	6.6
3	0.330	9.78	6.9
4	0.339	9.78	6.5

CHAPTER 3

CHAPTER-3

ANALYSIS BY WESTERGAARD'S THEORY

3.1 THEORETICAL ANALYSIS :

Westergaard analysis is generally used in the analysis of stresses in the rigid pavements at the interior, edge and corner regions. The application of Westergaard's theory for semi-rigid pavements has been a controversial issue. Whereas Mitchel and Frietag[5] have indicated that the thickness requirement by Westergaard's analysis in soil-cement will work out to be verymuch on the higher side than the flexible pavement design approach, Hogstrom, Chambers and Egonstens[18] have indicated the application of Westergaard's theory for the design of flexible pavements (bituminous concrete). Baker and Papazian[12] have compared flexural stress obtained by Westergaard and Burmister approaches for the same loading conditions and pavement thicknesses and have reported that the stresses were practically the same by the two methods. These reports indicate the possibility of the application of Westergaard's analysis in the design and analysis of soil-cement pavements also. Hence it was decided to investigate the application and limitations of Westergaards theory for semi-rigid pavements. Metcolf and Frydman[2] have also suggested the use of Westergaard's analysis in soil-cement base-course design for a certain range of tensile strength values. He considered corner load stresses to be the most critical one for the analysis. However it is known that the edge load stresses may exceed corner load stresses under certain conditions. Hence it was decided to investigate the critical loading condition for a range of relative stiffness values of the semi-rigid pavements.

Westergaard's radius of relative stiffness value is also considered[12] as "stiffness ratio", which is a ratio of resistance offered by the slab to the resistance offered by the subgrade. The stiffness ratio is represented by E_1/K where E_1 is the elastic modulus of the slab and K is Westergaard's modulus of subgrade reaction. This stiffness ratio E_1/K is also considered by Baker and Papazian[12] to be similar to Burmister's ratio E_1/E_2 .

Considering the variation that may be anticipated in the E_1 values of the semi-rigid base-courses and the K value of soil-subgrade, the stiffness ratio E_1/K considered in this investigation range from 300 to 5000. The load stresses for the interior, edge and corner load positions were calculated for different E_1/K values and various values of slab thicknesses, h. The stress values were expressed as a ratio of contact pressure p.

Figure 10 shows the relationship between stress values represented as a ratio of contact pressure and the slab thickness h for E_1/K ratios 300, 500, 1500 and 5000 and loading at interior, edge and corner. As may be expected the general trend has been a reduction in stress with increase in slab thickness. However, it is seen that at lower range of E_1/K ratios a peak in the stress curve is obtained indicating that with increase in slab thickness, stress first increases and then shows a decrease in trend. The valve of slab thicknesses corresponding to maximum stress for a particular ratio of E_1/K is found out to be different for the different loading conditions such as interior, edge and corner. As an example for E_1/K ratio equal to 300 the maximum stress for interior loading occurs at 12cm, for edge loading at 14.5 cm and corner loading at slab thickness greater than 30 cm. The maximum stress due to edge loading for E_1/K equal to 300 occurs at slab thickness 14.5cm, for E_1/K 500 at 12.5 cm and for E_1/K 1500 at 11 cm. However, it is not logical to expect a reduction in stress if the slab thickness is reduced below the values indicated by the peak stress in these curves. As for example for E_1/K equal to 300 the stress ratio due to edge load at 30, 20, 14.5 and 8 cms slab thickness are (Figure 9) 0.64, 0.86, 1.14 and 0.74 respectively. In this case as the slab thickness decreases from 14.5 to 8 cm stress ratio also decreases from 1.14 to 0.74 as per the Westergaard analysis. The only explanation that can be given for this type of variation is that the theory is not applicable for finding the edge load stresses for E_1/K value less than 300, When the slab thickness is less than 14.5 cm for the assumed loading condition (edge load, a = 15cm). The general finding of this investigation is that Westergaard's analysis is applicable for low E_1/K ratios only when the thickness of the slab is greater than the minimum value which is indicated by the summit of the stress vs thickness curves. At low E_1/K ratios the minimum thickness needed by the corner load formula to be applicable is higher than that for edge load formula. The minimum thickness requirement for any particular E_1/K value is lowest for interior load formula.

Another important observation of the analysis (Figure 9) is that for a range of E_1/K values 300 to 1500, the edge load stresses are highest followed by corner and then by interior load stresses for a considerable range of slab thickness values. However, the stress curves for edge, corner and interior loadings cross each other (for any particular E_1/K ratio) at certain thickness values. At E_1/K equal to 300 for the thickness range 20 to 30 cm the edge load stresses are the highest and the corner load stresses are the lowest;

the interior load stresses fall in between the two. But for thickness values less than 19cm the interior load stresses are the highest. However, at high range of E_1/K in the range of 1500 to 5000 the edge load stresses are the highest and the interior load stresses are the lowest; whereas the corner load stresses fall in the intermediate range. In the case of rigid pavements with very high value of E_1/K, corner load stresses becomes most critical in a considerable range of thickness values.

In semi-rigid pavements with E_1/K values ranging from 300 to 5000 the corresponding base-course thicknesses will also be higher with decreasing stiffness ratios. As such it may be stated that within this range the most critical condition of loading is the edge load. Hence the design and analysis of semi-rigid pavements by Westergaard's analysis may be carried out using Westergaard edge load stress equation. This finding is contrary to the argument given by Metcalf and Frydman(2) that the corner load stresses are more critical, for the analysis of tensile stresses in stabilised soil-cement pavements.

As the edge loading is found to be the critical one for semi-rigid pavements, it is suggested that stress analysis and base-course thickness design may be carried out only for the edge-load condition. This is further justified because of the fact that in semi-rigid pavements expansion and contraction joints are not provided and as such a corner-load condition does not exist. Even if the shrinkage cracks formed across the pavement slabs are through cracks, the interlocking that exists between the two portion are considerable that a corner loading condition seldom occurs. The curves given in Figure 11 may be used for evaluating the edge load stresses and for estimating the thickness requirements for any allowable stress, for the range of semi-rigid materials with E_1/K ratios 100 to 10,000 and base course thickness values upto 30 cm.

3.2 EXPERIMENTAL VERIFICATION :

The experimental values of the tests conducted on the four slabs referred to in article 2.2 were analysed using Westergaard's theory for comparison with the theoretical values. For the plate load tests in the interior region the maximum deflection was calculated using the following relation given by Westergaard :

$$\Delta_i = \frac{P}{8 K l^2} \qquad \text{---} \qquad (3.1)$$

Where

Δ_i = Maximum deflection at interior.

P = Total load applied on the plate.

K = Modulus of subgrade reaction.

l = radius of relative stiffness.

$$\ell = \sqrt[4]{\frac{E_1 h^3}{12(1-u_1^2)K}} \quad --- \quad (3.2)$$

Here E_1 = elastic modulus of pavement layer

h = pavement thickness.

u_1 = Poisson's ratio of pavement layer.

The load applied P in the experiment was limited to that corresponding to 0.05 cm deflection. The flexural stress at the interior due to the plate load was calculated. using the following Westergaard's equation :

$$\sigma_I = \frac{0.316P}{h^2}\left| 4 \log_{10} \frac{\ell}{b} + 1.069 \right| \quad --- \quad (3.3)$$

Where b = equivalent radius of resisting section.

The ratio of the plate load to be ultimate load and the flexural stress values of the slabs were also determined as before. The results obtained are given in the table 3.1 below :

Table 3.1

Analysis by Westergaard's Theory for Interior Loading.

S.No.	Load applied for 0.05cm deflection Kg.	Ratio of load applied to failure load.	Max^m deflection by Westergaard's theory cm.	Actual deflection cm.	Flexural by Westergaard's theory, Kg/cm^2	Flexural Strength Kg: cm^2
1	1420	0.140	0.1550	0.05	7.48	6.8
2	2400	0.240	0.1536	0.05	8.58	6.6
3	2150	0.330	0.1610	0.05	7.82	6.9
4	3180	0.339	0.1125	0.05	7.95	6.5

The deflection values calculated by Westergaard's analysis for interior loading as may be seen from the above table are much higher (about 3 times) than the experimental valves. Similarly the stress values evaluated also appear to be on higher side than the actual stress values because the calculated stress in all the four cases exceed the flexural strength values, though the applied load was only 0.14 to 0.339 of the failure load. Hence if the stresses ~~would~~ are calculated at failure load values, stresses would have been several times higher than the actual strength values. The above observation indicate that the deflection and load stress values calculated by Westergaard's theory for interior loading are higher than the actual values in the case of soil-cement pavements. Hence thickness design of base-course by Westergaard's theory would be over safe or too conservative. However, the stress values estimated by Westergaard's theory for the interior loading are lower than those found by Burmister's two layer theory (Refer tables 2.4 and 3.1).

It was shown theoretically in article 3.1 that the edge loading is the most critical condition of the three load positions. Hence the theoretical values of deflection and load stress for edge loading were also compared with the experimental values for the four slabs. Here again the load values corresponding to 0.05 deflection at the edge of the slab were considered for the calculation of edge load stress σ_e and maximum deflection of edge Δ_e by the Westergaard's theory.

The maximum deflection due to edge load was calculated using the following relation :

$$\Delta_e = P \times \sqrt{\frac{2 + 1.2\,u_1}{E_1 h^3 K}} \left| 1 - (0.76 + 0.4\,u_1)\frac{a}{l} \right| \quad \text{-- (3.4)}$$

The edge load stress were determined using Westergaard's stress equation given below :

$$\sigma_e = \frac{0.572P}{h^2} \left| 4 \log_{10} \frac{l}{b} + 0.359 \right| \quad \text{--- (3.5)}$$

The calculated values alongwith the experimental values are reported in Table 3.2.

Table 3.2

Analysis by Westergaard theory for Edge loading

Sl. No.	Load applied for 0.05cm deflection Kg.	Ratio of applied load to failure load.	Max^m deflection by Westergaard theory cm	Actual deflection cm.	Flexural stress by Westergaard theory Kg/cm^2	Flexural Strength Kg/cm^2
1	920	0.29	0.0565	0.05	2.97	6.8
2	990	0.28	0.0468	0.05	2.83	6.6
3	1070	0.24	0.0968	0.05	3.97	6.9
4	1910	0.34	0.1080	0.05	5.31	6.5

The calculated values of deflection due to edge load have closely tallied with the experimental values in two cases, whereas in the other two cases the deflection values calculated by Westergaard's theory are higher by 90 to 100 percent of the actual deflection. However, the deflection values for edge loading have tallied better with the experimental values than those for interior loading. Similarly the stress values calculated by Westergaard's theory are all less than the strength values. In the case of slab 1 the stress is 2.97 Kg/cm^2 when the load applied is 0.29 of the failure load and the strength is 6.8 Kg/cm^2. This indicates that at failure load the calculated edge load stress would have exceeded the strength value by about 26 percent of the strength value. Hence it may be stated that the calculated value of edge load stress is only slightly higher than the actual value but is within reasonable limits. Design of semi-rigid pavements for edge load condition is more justified and thus the use of Westergaard theory for the design seems to be reasonable though the design will be still conservative. For thicker slabs as in the case of slab no. 4 the estimated value of stress seems to be much higher than the actual stress. Hence due consideration has to be given in the design keeping this point in view and also the fatigue aspect of the material. The real problem of thickness design of semi-rigid pavements can be solved only after thorough investigation on fatigue behaviour of these materials, to decide the necessary factor of safety.

CHAPTER 4

CHAPTER 4

ANALYSIS BY MEYERHOF'S THEORY

4.1 THEORETICAL ANALYSIS :

It has been shown[19] in various investigations that the failure load estimated by Westergaard's analysis in the case of rigid pavements are much lower than the actual values. Meyerhof's analysis is based on the ultimate failure of slabs assuming a certain pattern of yield line failure. Barenburg[13] suggested the use of Meyerhof's analysis for the design of semi-rigid pavements consisting of lime-flyash-aggregate bases. In some of the previous investigations carried out on soil-cement slabs (in the Highway Engineering laboratory, University of Roorkee) it was reported that the crack patterns of the slabs failed under plate loads at interior and edge regions were somewhat similar to those assumed by Meyerhof. Hence it was decided to study the possibility of adopting Meyerhof's theory for the analysis of semi-rigid pavements.

The failure loads were calculated in terms of the flexural strength (P/f) for various values of E_1/K ratios (ranging from 100 to 10,000) and various thickness (h) values for the interior, edge and corner loads. Table 4.1 gives the values of ultimate load in terms of flexural strength for the interior , edge and corner loadings

($\frac{P_i}{f}$, $\frac{P_e}{f}$, $\frac{P_c}{f}$) for E_1/K equal to 100, the lowest value in the range.

Table 4.1

Theoretical Values of Failure Load at Interior, Edge and Corner Regions by Meyerhof's Theory.

E/K Ratio	Slab Thickness h cm	$\ell = \sqrt{\frac{Eh^3}{12(1-\mu^2)K}}$ cm	$\frac{M_o}{f} = \frac{h^2}{6}$	$\frac{P_i}{f} = \frac{4\pi M_o}{f(1-\frac{a}{3\ell})}$	$\frac{P_e}{f} = \frac{(4+\pi)M_o}{f(1-\frac{2a}{3\ell})}$	$\frac{P_c}{f} = \frac{4M_o}{f(1-\frac{a}{\ell})}$
	8	8.13	10.66	348	− 331.4	− 50.4
	10	9.60	16.66	437.1	−2768.5	−118.3
100	12	11	24	554.3	1904.5	−264.7
	15	13.02	37.5	764.7	1153.7	−987.1
	20	16.2	66.66	1212	1240	3600
	30	22.4	150	2428	1937	1817

It is found that the corner load equation is not applicable for low values of E/K (of about 100). The P_c/f values are found to be negetive for a considerable range of thickness values. As the thickness value increases, the radius of relative stiffness also increases as may be seen from the columns 2 and 3 of Table 4.1. When the value of 'ℓ' becomes equal to the value of radius of loaded area 'a', the failure load P_c for the corner becomes infinity as P_c is given by the equation :

$$P_c = \frac{4\ M_o}{(1 - \frac{a}{l})} \quad --- \quad (4.1)$$

Here M_o = Bending moment, = $\frac{f\ h^2}{6}$

Here f = flexural strength

h = slab thickness

When the value of h further increases, the value of 'l' also increases with the result the failure load at corner decreases upto a certain limit of thickness value h. Hence it may be concluded that the corner load formula of Meyerhof's theory is not applicable for semi-rigid pavements with low E_1/K ratios, particularly when the slab thickness values are below certain values, depending upon E_1/K ratio.

Figure 12 shows the variation in P/f values with slab thickness h, plotted for E_1/K equal to 500 and 2500 for the interior, edge and corner regions for a = 15cm. Here also it is obvious that for E_1/K equal to 500 the corner load formula is not applicable for slab thickness less than 17cm (as the failure load increases with decrease in slab thickness in this range).

The edge load equation has also similar limitation as discussed above but for a smaller range of thickness values as may be seen from Table 4.1 and Figure 12. For E_1/K equal to 500 the edge load equation is not applicable only if the slab thickness is less than 10cm. From Figure 12 it is evident that for higher E_1/K values of 2500 the critical condition of loading however, is the corner loading.

Variation of P/f values for interior, edge and corner regions with increase in E_1/K values from 100 to 5000 are shown in Figure 13 for a slab thickness h = 15cm and radius of loaded area a = 15cm. From this figure it is obvious that for a = 15cm and h = 15cm the edge load condition is most critical for E_1/K value upto 900; for E_1/K values above 900 the failure load at the corner region becomes the lowest.

From the above discussion it may be concluded that for design of semi-rigid base-courses it is reasonable to consider the ultimate failure load for edge load condition, keeping in view the limitations of the edge load formula for a certain range of thickness values. This is also due to the fact that the failure load at interior is the highest in all cases. Further the corner loading conditions seldom occurs in semi-rigid base-courses because through joints such as expansion joints in cement concrete pavements, are not provided in semi-rigid base-courses.

The edge load values (interms of flexural strength, i.e. P_e/f) for various E_1/K values ranging from 100 to 10,000 have been graphically represented in Figure 14 for various values of slab thickness h. This figure can thus be useful either as a chart for stress analysis or for thickness design by Meyerhof theory for edge loading. For a slab of given values of h and E_1/K it is possible to find the stress corresponding to an edge load P_e. Similarly for any desing wheel load P_e and allowable flexural stress f, it is possible to design the required slab thickness h for a known E_1/K ratio. The portion of the curves k shown by dotted lines, for E_1/K 100, 300, 500 in Figure 14 are the range of thickness values in which the formula is not applicable for the analysis. It is seen that for higher ratios of E_1/K , this range of thickness decreases.

4.2 EXPERIMENTAL VERIFICATION :

The results of the four soil-cement slabs tested (as reported earlier) were analysed using Meyerhof's theory. The load stresses obtained for interior and edge loadings for the applied load causing 0.05cm deflection of 15 cm radius plate are shown in Table 4.2.

The stress values calculated by Meyerhofs' theory for interior loading seem to be higher than the experimental values. Though the calculated stress values for the four slabs and the load values selected are less than the flexural strength values indicated in Column-8 of Table-4.2, these stress values may be considered higher than actual values as the load applied in all cases are less than 30 percent of the failure load. However, the interior stresses calculated by Meyerhof's theory are less than the values by Westergaard's theory and two layer theory reported in Tables 3.1 and 2.4.

The edge load stresses calculated by Meyerhof's theory (Col.-7 of Table 4.2) seem to tally fairly well with actual stress values. However, the calculated stress values seem to be on the higher side in all cases keeping in view the ratio of the load applied to the actual failure load and the ratio of calculated stress to the flexural strength in each case.

Table-4.2

Analysis of Stress by Meyerhof's Theory

Sl. No.	INTERIOR LOADING			EDGE LOADING			
	Load applied for 0.05cm deflection of 15cm radius plate;Kg	Ratio of load applied to experimental value of failure load.	Stress σ_I Kg/cm²	Load applied for 0.05cm deflection Kg	Ratio of applied load to failure load.	Stress σ_e Kg/cm²	flexural strength Kg/cm²
1	1420	0.140	6.32	920	0.29	3.76	6.8
2	2400	0.240	7.87	990	0.28	2.91	6.6
3	2150	0.330	5.52	1070	0.24	3.22	6.9
4	3180	0.329	5.30	1910	0.34	3.82	6.5

CHAPTER 5

CHAPTER-5

COMPARATIVE STUDY OF VARIOUS METHODS OF ANALYSIS

The comparative study in this Chapter has been divided into two parts :

(i) The comparison of theoretical values by various theories studied for the range of values considered for semi-rigid pavements.

(ii) Comparison of experimental results obtained in a previous investigation with the theoretical results.

5.1 COMPARISON OF THEORETICAL METHODS OF ANALYSIS.

5.1.1 Interior Loading :

Non-dimensional plots were made with p_i/f on the Y-axis and h/a on the X-axis using Meyerhof, Westergaard and Burmister's analyses for interior loading. Here p_i is the unit load causing failure due to the interior loading by the above theories, f is the flexural strength of the semi-rigid material, h is slab thickness and a is the radius of loaded area. Figure 15 shows a non-dimensional plot p_i/f vs h/a for E_1/K values 300, 500 and 1500 by the three methods of analysis. In all cases the Burmister's analysis by two layer thoery shows the lowest ratio of p_i/f for the entire range of h/a (0.5 to 2, which are the most common values of h/a). This means that a failure load per unit area determined by Burmister's analysis gives the lowest value.

The values of p_i/f by Westergaard's theory are higher than those of Burmister's analysis but lower than Meyerhof analysis for all values of E_1/K and h/a excluding the range of h/a where the theory is not applicable (for $E_1/K = 300$, Westergaard theory may be considered not applicable when h/a is less than 0.8).

From Figure 15 it may be observed that the rate of increase in p_i/f values with increase in h/a is not same by all the three methods of analysis. Meyerhof's method shows a higher rate whereas Westergaard and Burmister's methods shows somewhat similar trend. Hence for lower values of h/a the percentage difference between the p_i/f values by the three methods are not significant, whereas at higher h/a values they are considerable. In the previous chapter it has been indicated that the theoretical estimation of load stress are higher and value of failure loads estimated are considerably lower than the experimental values. Of the three methods investigated, Meyerhof's method gives results which are somewhat closer towards the actual values. From Figure 15 it may be inferred that the Meyerhof method is likely to give results of ultimate load values which are closer to the experimental results when h/a values are higher. (i.e., in the range of 2 and above).

5.1.2 Edge Loading :

Figure 16 shows a non-dimensional plot p_e/f vs h/a, where p_e is the failure load per unit area due to edge loading. The figure show the relationship by Meyerhof and Westergaard analysis for E_1/K equal to 300, 500 and 1500. Excluding the portion of the curves which are not applicable (shown by dotted lines) it may be seen that Westergaard's analysis gives lower values of p_e/f than Meyerhof's analysis, for the entire range of E_1/K and h/a values. This means that the failure stress values by Westergaard's analysis for edge loading are lower than Meyerhof values. Further it is seen that the rate of increase in p_e/f with h/a values by Meyerhof's analysis is higher than the rate by Westergaard's analysis. This indicates that at higher values of h/a the ratio of failure stress by Meyerhof and Westergaard's analyses would also increase in the case of semi-rigid pavements.

Thus it is likely that in the case of semi-rigid pavements, the Meyerhof's analysis may give failure load results tallying more closely with experimental values for higher range of h/a ratios (similar to the observation in interior loading).

5.2 COMPARISON OF THEORETICAL VALUES WITH EXPERIMENTAL RESULTS :

The flexural stresses calculated theoretically were shown to be higher than the actual values when comparing with the experimental results as already discussed in Chapters 2,3 and 4. It was hence decided to compare the ultimate load values evaluated theoretically for interior and edge loading with the results of ultimate load tests carried out on soil-cement slabs.

5.2.1 Interior Loading :

The results of the plate load tests carried out on soil-cement slab (avkailable in the laboratory) were tabulated with details such as the total load at failure, plate diameter, slab thickness and the actual flexural strength (f) of the slab. From these the values of failure load per unit area p_i (at the interior) were calculated for each load test. The ratio p_i/f and h/a values were calculated and tabulated, indicating also the corresponding E_1/K values of the slabs. The various values obtained have been superimposed in Figure 15 so that the experimental results can be directly compared with the theoretical curves. The points have been divided into two groups of E_1/K values in the range 300 to 500 and 500-1500.

From Figure 15 it is seen that the failure loads determined experimentally at interior are much higher than the failure loads indicated by Burmister, Westergaard and Meyerhof's analyses in all the cases. Of three methods of analysis Meyerhof's method gives the highest failure load value. The closest agreement with the experimental values has been with the Meyerhof's analysis. However, the failure load by Meyerhof's analysis has been found to range between 24 % and 75 % of the experimental failure loads and the average value was found to be about 50 %.

The average failure load estimated by Westergaard's analysis for the interior loading works out to be 40 % of the experimental failure loads. This observation supports the earlier conclusion that the flexural stresses calculated by theoretical analyses are considerably higher than the actual stress values in the case of semi-rigid pavements. For higher h/a values (1.75 to 2) it is seen that the breaking load by Meyerhof's theory is 75 to 80 % of the actual values. This indicates that Meyerhof's theory would give more dependable results for higher value of h/a , in the case of semi-rigid pavements.

Ghosh and Dinakaran[19] have also indicated that the breaking loads by Meyerhof and Westergaard's analyses in the case of plain cement-concrete pavements are also considerably lower than the experimental values. The percentage of theoretical failure loads in terms of experimental values as reported by them are also roughly in the same range as found in this investigation in the case of soil-cement slabs. Hence there is a need for further investigation on the analysis of load stresses and breaking loads. It is desirable to arrive at a method of analysis by which it is possible to estimate the load stresses and breaking loads, tallying closely with the actual values in the case of semi-rigid and rigid pavement slabs.

5.2.2 Edge Loading :

The results of the breaking load tests carried out at the edge of soil-cement slabs were analysed and plotted in the same way as discussed in article 5.2.1 above. The breaking loads determined experimentally have been shown by points superimposed in Figure 16 over the theoretical curves p_e/f vs h/a. Here again it is found that the actual breaking load for edge loading are higher than the theoretical values in all the cases. The theoretical values determined by Meyerhof and Westergaard's analyses were found to be about 60 % and 50 % respectively of the actual breaking loads on the average. There is also and indication that for estimating the breaking load at pavement edge, the Meyerhof's method would be in better agrement with the actual values when h/a ratios are higher. However this is not the case with Westergaard's analysis as the slope of the curve p_e/f vs h/a is rather flat in comparison with Meyerhof's analysis.

5.3 EMPIRICAL APPROACH FOR SEMI-RIGID PAVEMENT ANALYSIS :

Nussbaum and Larsen(6) and Nielson(4) have suggested the use of non-dimensional load response equations to correlate the load-deflection characteristics of soil-cement slabs. Plate load test results are plotted in a log-log scale with $\Delta K/p$ on Y-axis and a/h on X-axis.

Where Δ = deflection of plate

p = applied pressure

K = modulus of subgrade reaction

a = radius of loaded plate.

h = thickness of base course.

Plate load tests were carried out at the interior region of the slab and the results of the load-deflection observations were plotted in the above form. A best fit line was obtained and the equation of the regression line was given in the form :

$$\frac{\Delta K}{p} = \alpha \left(\frac{a}{h} \right)^{\beta}$$

Where α = the ordinate on the best fit line corresponding to an abcissa a/h = 1, (which is reported to be a function of h and K)

β = slope of the regression line (which is a function of h).

Values of α and β were given by the above investigators based on their experimental study.

Based on the above observations it was considered desirable to compare the empirical non-dimensional approach with the other theoretical approaches for the analysis of semi-rigid pavements. As it has been shown that the most critical condition to be considered for the design of semi-rigid pavements is that of edge loading, the empirical non-dimensional approach may be extended for edge loading condition. The results of the soil-cement slab tests available in the laboratory were made use of for plotting the non-dimensional relation $\Delta K/p$ vs a/h for edge loading condition.

Figure 17 shows the regression line obtained from the above non-dimensional analysis. The details for obtaining the regression line from the load-deflection data are given in Appendix III. Such a relation obtained for semi-rigid base course experimentally would be of great use to compare with the results of theoretical analyses. By Westergaard's analysis for edge loading condition, if the deflection value is determined for a design load or the contact pressure is found for an allowable deflection, it will be possible to compare these values with the results of the non-dimensional plot already obtained for the slab. This will enable the investigator to assess the application and limitations of the theoretical methods in view of the experimental results.

soil-cement slabs, indicate that the calculated stress values are considerably higher than the actual stress values. But the calculated values of minimum radius of curvature are much higher than the actual values; this results in lower strain values than the actual values. Thus the stress and strain calculations contradict with each other when they are compared with the experimental values. However the deflection values fairly tally with each other.

5. There is a certain range of slab thickness when the Westergaards load stress equation will not be applicable for semi-rigid pavements, particularly for low values of E_1/K . These ranges have been fairly illustrated in the charts for the interior, edge and corner loading conditions. In general it is observed that when E_1/K decreases the Westergaards analysis is applicable only for thicker slabs; the thickness value depending upon the position of loading, E_1/K ratio, and radius of loaded area.
6. For semi-rigid pavements of E_1/K values ranging from 300 to 5,000 , edge loading condition is the most critical one and hence may be taken for design. The charts prepared for edge loading conditions may be used for analysing the load stresses as well as the thickness design for semi-rigid base courses.
7. Comparison with experimental results of plate load tests on soil-cement slabs has indicated that Westergaards

CHAPTER 6

CHAPTER-6

CONCLUSIONS

1. Semi-rigid pavements need special consideration for the design and analysis. The theoretical methods for design of rigid or flexible pavements may not be directly applicable for the semi-rigid pavement analysis.
2. The charts prepared using Elastic Two Layer theory in this investigation could be very conveniently used both for the analysis and thickness design of semi-rigid pavements. Subgrade pressure, flexural stress and maximum strain in the pavement, surface deflection and minimum radius of curvature of the semi-rigid pavements can be easily obtained for E_1/E_2 100 to 500 and slab thickness h 10 to 30 cm.
3. From the elastic two-layer theory and the analysis of deflection and strain characteristics it has been shown that it is reasonable to design semi-rigid pavements based on allowable deflection criterion for interior loading condition. The constant design deflection can be applied for a material irrespective of the slab thickness as the flexural strain practically remains constant with increase in slab thickness for a certain radius of loading.
4. The experimental verification of the analysis by two-layer theory by making use of load test results on

soil-cement slabs, indicate that the calculated stress values are considerably higher than the actual stress values. But the calculated values of minimum radius of curvature are much higher than the actual values; this results in lower strain values than the actual values. Thus the stress and strain calculations contradict with each other when they are compared with the experimental values. However the deflection values fairly tally with each other.

5. There is a certain range of slab thickness when the Westergaards load stress equation will not be applicable for semi-rigid pavements, particularly for low values of E_1/K . These ranges have been fairly illustrated in the charts for the interior, edge and corner loading conditions. In general it is observed that when E_1/K decreases the Westergaards analysis is applicable only for thicker slabs; the thickness value depending upon the position of loading, E_1/K ratio, and radius of loaded area.
6. For semi-rigid pavements of E_1/K values ranging from 300 to 5,000 , edge loading condition is the most critical one and hence may be taken for design. The charts prepared for edge loading conditions may be used for analysing the load stresses as well as the thickness design for semi-rigid base courses.
7. Comparison with experimental results of plate load tests on soil-cement slabs has indicated that Westergaards

analysis gives higher stress and deflection values for edge load conditions. The theoretical values of breaking load were approximately one half of the actual breaking load values.

8. Meyerhof's theory is also applicable for the analysis of semi-rigid pavements for a certain range of thickness values. Particularly when E_1/K values are low, the edge load and corner load equations are not applicable for thin slabs.

9. The charts prepared using Meyerhof's theory for edge load are useful for analysing the load stresses or for designing the thickness requirement of semi-rigid pavements for E_1/K values ranging from 100 to 10,000.

10. When the experimental results of plate load tests on soil-cement slabs were compared with the calculated values by Meyerhof's theory, it is noticed that there is fair agreement when h/a ratios are high ($h/a > 1.5$).

11. The Breaking load found by Meyerhof's theory is also lower than the actual breaking load for soil-cement slabs and on the average is about 60 % for edge load conditions.

12. The non-dimensional analysis of load-deflection data for interior loading condition of semi-rigid pavements may be extended for edge loading condition also and a regression equation obtained. This equation or chart could be made use for comparing the experimental results with the results of theoretical analysis.

SCOPE FOR FURTHER STUDY.

1. In the analysis by Elastic Two-Layer theory it has been found that the calculated values of flexural stress are very much higher and those of tensile strain are lower than actual values. These contradict each other. Hence investigations on the limitations of two layer theory for pavement analysis, including on the fundamental aspects of the theory are needed.
2. Based on the actual failure patterns of semi-rigid slabs, it is desirable to arrive at equations to find the breaking loads making use of ultimate load theory, so that the breaking loads may be estimated with greater accuracy.
3. Further study is needed to find cause why Westergaard analysis gives higher load stresses than the actual values.

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APPENDICES

APPENDIX - I

Sample Calculations for Plotting Maximum Strain e vs Pavement thickness h.

The Equations used in finding out maximum strain are given below :

$$R_o = \frac{a^2}{\Delta} \times F_c \qquad --- \quad (A_1\text{-}1)$$

$$e = \frac{h}{2R_o} \qquad --- \quad (A_1\text{-}2)$$

Where

e = maximum strain

R_o = minimum radius of curvature

a = radius of loaded area.

Δ = the deflection.

h = pavement thickness

F_c = a function, depending upon a/h , E_1/E_2 ratios, u_1 and u_2 .

The value of the function F_c can be found out from the tables given by Larsen, Nussbaum and Colley[16] for known values of a/h , E_1/E_2 and u_1 and u_2 .

CALCULATIONS :

For one particular curve we have.

$E_1/E_2 = 100$, $\Delta = 0.03$cm. , $a = 15$ cm.

1. When $a/h = 2$ $h = 7.5$

From tables $F_c = 2.8186$ (for $E_1/E_2 = 100$, $a/h = 2$, $u_1 = 0.2$, $u_2 = 0.5$)

$$\therefore \quad R_o = \frac{15 \times 15}{.03} \times F_c$$

$$= 7500 \times 2.8186$$

$$= 21150 \text{ cm.}$$

Now $$e = \frac{h}{2R_o} = \frac{7.5}{2 \times 21150}$$

$$= 1.773 \times 10^{-4}$$

2. When $a/h = 1.5$, $h = 10$,

$$F_c = 4.7$$

$$\therefore \quad R_o = 7500 \times 4.7 = 35250$$

$$\therefore \quad e = \frac{10}{2 \times 35250} = 1.42 \times 10^{-4}$$

3. When $a/h = 1$, $h = 15\text{cm}$

$$F_c = 8.03554$$

$$R_o = 7500 \times 8.03554 = 60270$$

$$\therefore \quad e = \frac{15}{2 \times 60270} = 1.244 \times 10^{-4}$$

4. When $a/h = 0.5$, $h = 30\text{cm}$

$$F_c = 15.4391$$

$$R_o = 7500 \times 15.4391 = 115800 \text{ cm}$$

$$e = \frac{30}{2 \times 115800} = 1.297 \times 10^{-4}$$

$E_1/E_2 = 100$, $\Delta = .03$ cm , $a = 15$ cm.

Slab thickness, h cm	7.5	10	15	30
Maximum Strain, e	1.773×10^{-4}	1.42×10^{-4}	1.244×10^{-4}	1.297×10^{-4}

APPENDIX...II

EXTENSION OF EXISTING ODEMARKS PLOT FOR E1/E2 VALUES RANGING FROM 100 TO 500

```
C   C  CALCULATE FUNCTION KULWANT SINGH
       A=0.5
    30 C=((1.+(0.9*A)**2.)**2.5)/(1.+(4.*(0.9*A)**2.))
       B=100.
    40 D=((1.+(0.9*A*(B**.33))**2.)**2.5)/(1.+(4.*(0.9*A*(B**.33))**2.))
       X=1./(((1.-(1./C))*1./B)+1./D)
       P=X/B
       PUNCH 100,A,B,X,P
   100 FORMAT(4F18.4)
       B=B+25.
       IF (B-500.) 40,40,50
    50 A=A+0.25
       IF (A-3.) 30,30,55
    55 STOP
       END
```

C   C CALCULATE FUNCTION KULWANT SINGH

A=(h/a)	B=(E1/E2)	X= F_L	P= F_R
.5000	100.0000	3.5088	.0351
.5000	125.0000	4.1257	.0330
.5000	150.0000	4.7330	.0316
.5000	175.0000	5.3329	.0305
.5000	200.0000	5.9268	.0296
.5000	225.0000	6.5159	[illegible]

.5000	250.0000	7.1010	.0284
.5000	275.0000	7.6825	.0279
.5000	300.0000	8.2609	.0275
.5000	325.0000	8.8367	.0272
.5000	350.0000	9.4101	.0269
.5000	375.0000	9.9814	.0266
.5000	400.0000	10.5508	.0264
.5000	425.0000	11.1184	.0262
.5000	450.0000	11.6844	.0260
.5000	475.0000	12.2489	.0258
.5000	500.0000	12.8121	.0256
.7500	100.0000	9.2730	.0927
.7500	125.0000	11.2287	.0898
.7500	150.0000	13.1655	.0878
.7500	175.0000	15.0879	.0862
.7500	200.0000	16.9990	.0850
.7500	225.0000	18.9007	.0840
.7500	250.0000	20.7944	.0832
.7500	275.0000	22.6815	.0825
.7500	300.0000	24.5626	.0819
.7500	325.0000	26.4386	.0813
.7500	350.0000	28.3099	.0809
.7500	375.0000	30.1771	.0805
.7500	400.0000	32.0406	.0801
.7500	425.0000	33.9006	.0798
.7500	450.0000	35.7575	.0795
.7500	475.0000	37.6115	.0792
.7500	500.0000	39.4628	.0789
1.0000	100.0000	19.6506	.1965
1.0000	125.0000	24.0945	.1928
1.0000	150.0000	28.5090	.1901

1.0000	175.0000	32.9010	.1880
1.0000	200.0000	37.2752	.1864
1.0000	225.0000	41.6344	.1850
1.0000	250.0000	45.9811	.1839
1.0000	275.0000	50.3170	.1830
1.0000	300.0000	54.6434	.1821
1.0000	325.0000	58.9614	.1814
1.0000	350.0000	63.2719	.1808
1.0000	375.0000	67.5755	.1802
1.0000	400.0000	71.8730	.1797
1.0000	425.0000	76.1648	.1792
1.0000	450.0000	80.4514	.1788
1.0000	475.0000	84.7331	.1784
1.0000	500.0000	89.0104	.1780
1.2500	100.0000	34.2265	.3423
1.2500	125.0000	42.2530	.3380
1.2500	150.0000	50.2412	.3349
1.2500	175.0000	58.1996	.3326
1.2500	200.0000	66.1341	.3307
1.2500	225.0000	74.0486	.3291
1.2500	250.0000	81.9462	.3278
1.2500	275.0000	89.8292	.3267
1.2500	300.0000	97.6991	.3257
1.2500	325.0000	105.5576	.3248
1.2500	350.0000	113.4056	.3240
1.2500	375.0000	121.2443	.3233
1.2500	400.0000	129.0744	.3227
1.2500	425.0000	136.8965	.3221
1.2500	450.0000	144.7111	.3216
1.2500	475.0000	152.5190	.3211
1.2500	500.0000	160.3206	.3206

1.5000	100.0000	50.3364	.5034
1.5000	125.0000	62.4076	.4993
1.5000	150.0000	74.4367	.4962
1.5000	175.0000	86.4331	.4939
1.5000	200.0000	98.4029	.4920
1.5000	225.0000	110.3504	.4904
1.5000	250.0000	122.2787	.4891
1.5000	275.0000	134.1901	.4880
1.5000	300.0000	146.0870	.4870
1.5000	325.0000	157.9707	.4861
1.5000	350.0000	169.8425	.4853
1.5000	375.0000	181.7034	.4845
1.5000	400.0000	193.5541	.4839
1.5000	425.0000	205.3952	.4833
1.5000	450.0000	217.2283	.4827
1.5000	475.0000	229.0530	.4822
1.5000	500.0000	240.8705	.4817
1.7500	100.0000	64.7788	.6478
1.7500	125.0000	80.5397	.6443
1.7500	150.0000	96.2612	.6417
1.7500	175.0000	111.9517	.6397
1.7500	200.0000	127.6168	.6381
1.7500	225.0000	143.2606	.6367
1.7500	250.0000	158.8858	.6355
1.7500	275.0000	174.4947	.6345
1.7500	300.0000	190.0891	.6336
1.7500	325.0000	205.6709	.6328
1.7500	350.0000	221.2408	.6321
1.7500	375.0000	236.8001	.6315
1.7500	400.0000	252.3493	.6309
1.7500	425.0000	267.8894	.6303

1.7500	450.0000	283.4209	.6298
1.7500	475.0000	298.9444	.6294
1.7500	500.0000	314.4605	.6289
2.0000	100.0000	75.8848	.7588
2.0000	125.0000	94.5201	.7562
2.0000	150.0000	113.1222	.7541
2.0000	175.0000	131.6978	.7526
2.0000	200.0000	150.2516	.7513
2.0000	225.0000	168.7872	.7502
2.0000	250.0000	187.3067	.7492
2.0000	275.0000	205.8119	.7484
2.0000	300.0000	224.3047	.7477
2.0000	325.0000	242.7862	.7470
2.0000	350.0000	261.2572	.7464
2.0000	375.0000	279.7189	.7459
2.0000	400.0000	298.1719	.7454
2.0000	425.0000	316.6167	.7450
2.0000	450.0000	335.0538	.7446
2.0000	475.0000	353.4839	.7442
2.0000	500.0000	371.9075	.7438
2.2500	100.0000	83.6652	.8367
2.2500	125.0000	104.3326	.8347
2.2500	150.0000	124.9735	.8332
2.2500	175.0000	145.5933	.8320
2.2500	200.0000	166.1955	.8310
2.2500	225.0000	186.7827	.8301
2.2500	250.0000	207.3569	.8294
2.2500	275.0000	227.9195	.8288
2.2500	300.0000	248.4720	.8282
2.2500	325.0000	269.0151	.8277
2.2500	350.0000	289.5497	.8273

2.2500	375.0000	310.0765	.8269
2.2500	400.0000	330.5959	.8265
2.2500	425.0000	351.1087	.8261
2.2500	450.0000	371.6152	.8258
2.2500	475.0000	392.1158	.8255
2.2500	500.0000	412.6109	.8252
2.5000	100.0000	88.8732	.8887
2.5000	125.0000	110.9095	.8873
2.5000	150.0000	132.9251	.8862
2.5000	175.0000	154.9242	.8853
2.5000	200.0000	176.9096	.8845
2.5000	225.0000	198.8831	.8839
2.5000	250.0000	220.8463	.8834
2.5000	275.0000	242.8004	.8829
2.5000	300.0000	264.7462	.8825
2.5000	325.0000	286.6847	.8821
2.5000	350.0000	308.6163	.8818
2.5000	375.0000	330.5418	.8814
2.5000	400.0000	352.4615	.8812
2.5000	425.0000	374.3757	.8809
2.5000	450.0000	396.2849	.8806
2.5000	475.0000	418.1894	.8804
2.5000	500.0000	440.0895	.8802
2.7500	100.0000	92.3091	.9231
2.7500	125.0000	115.2526	.9220
2.7500	150.0000	138.1802	.9212
2.7500	175.0000	161.0950	.9205
2.7500	200.0000	183.9989	.9200
2.7500	225.0000	206.8936	.9195
2.7500	250.0000	229.7802	.9191
2.7500	275.0000	252.6596	.9188

2.7500	300.0000	275.5326	.9184
2.7500	325.0000	298.3997	.9182
2.7500	350.0000	321.2615	.9179
2.7500	375.0000	344.1184	.9176
2.7500	400.0000	366.9706	.9174
2.7500	425.0000	389.8186	.9172
2.7500	450.0000	412.6625	.9170
2.7500	475.0000	435.5027	.9168
2.7500	500.0000	458.3393	.9167
3.0000	100.0000	94.5834	.9458
3.0000	125.0000	118.1297	.9450
3.0000	150.0000	141.6636	.9444
3.0000	175.0000	165.1873	.9439
3.0000	200.0000	188.7027	.9435
3.0000	225.0000	212.2108	.9432
3.0000	250.0000	235.7125	.9429
3.0000	275.0000	259.2086	.9426
3.0000	300.0000	282.6996	.9423
3.0000	325.0000	306.1860	.9421
3.0000	350.0000	329.6681	.9419
3.0000	375.0000	353.1462	.9417
3.0000	400.0000	376.6207	.9416
3.0000	425.0000	400.0917	.9414
3.0000	450.0000	423.5596	.9412
3.0000	475.0000	447.0244	.9411
3.0000	500.0000	470.4865	.9410

STOP END AT S. 0055 + 00 L. Z

APPENDIX - III

The values of Δ, K, p, h, and a were obtained from the load-deflection data available in the Highway Engineering Laboratory. These alongwith the procedure of obtaining the Regression line (Figure 17) is given below :

Slab No.	h cm	a cm.	a/h	Δ cm	p Kg/cm²	K Kg/cm³	$\frac{\Delta K}{p} \times 10^{-2}$
1	8	15	1.875	0.05	2.0	5.3	13.25
				0.1	3.13	5.3	16.92
2	10	15	1.5	0.05	1.4	8.0	28.55
				0.10	2.4	8.0	33.34
3	8	15	1.875	0.05	0.85	5.0	29.4
				0.10	1.5	5.0	33.33
4	10	15	1.5	0.05	1.2	6.5	27.1
				0.10	2.175	6.5	29.82
5	12	15	1.25	0.05	1.35	4.5	16.66
				0.10	2.275	4.5	19.8
13	15	15	1.0	0.05	1.8	7.0	19.45
				0.10	4.75	7.0	14.72
15	12	15	1.25	0.05	2.45	6.5	13.26
				0.10	4.0	6.5	16.25
16	15	15	1.0	0.05	3.6	6.4	10.5
				0.10	6.1	6.4	8.9
17	12	15	1.25	0.05	2.8	6.0	10.7
				0.10	5.8	6.0	10.35
18	10	15	1.0	0.05	2.9	6.2	10.7
				0.10	6.2	6.2	11.92

Calculations for finding the Regression line

S.No	a /h = x	x^2	y = ΔK/pz x 10^{-2}	xy x 10^{-2}
1	1.875	3.5156	13.25	24.85
2	1.875	3.5156	16.92	31.74
3	1.50	2.2500	28.55	42.80
4	1.50	2.2500	33.34	50.00
5.	1.875	3.5156	33.33	62.50
6.	1.875	3.5156	29.40	55.10
7.	1.50	2.2500	29.80	44.80
8.	1.50	2.2500	27.10	40.65
9.	1.25	1.5625	19.80	24.74
10.	1.25	1.5625	16.66	20.82
11.	1.00	1.0000	14.72	14.72
12.	1.00	1.0000	19.45	19.45
13.	1.25	1.5625	16.25	20.30
14.	1.25	1.5625	13.26	16.58
15.	1.00	1.0000	10.50	10.50
16.	1.00	1.0000	8.9	8.90
17.	1.25	1.5625	10.35	12.935
18.	1.25	1.5625	10.70	13.38
19.	1.50	2.2500	11.92	17.89
20.	1.50	2.2500	10.70	16.05

$\Sigma x = 28.00$

$\Sigma x^2 = 40.94$

$$\sum y = 375 \times 10^{-2}$$

$$\sum xy = 548.7 \times 10^{-2}$$

Normal equations are

$$\sum y = a \sum x + b.N.$$

$$\sum xy = a \sum x^2 + b \sum x$$

Putting the values we get :

$$3.75 = a \times 28 + b \times 20 \quad \text{---} \quad 1.$$

$$5.487 = a \times 40.94 + b \times 28 \quad \text{---} \quad 2.$$

From equation 1.

$$b = \frac{3.75 - 28a}{20}$$

Putting this value in eq-2

$$5.487 = 40.94\,a + 28\left(\frac{3.75 - 28a}{20}\right)$$

Solving the above equation we get :

$$a = 0.1362$$

Putting the value of a in Eq.1.

$$3.75 = .1362 \times 28 + b \times 20$$

$$20\,b = -0.0636$$

$$\therefore \quad b = -.00318$$

The equation of Regression line is

$$y = 0.1362\,x - 0.00318$$

$$\text{or} \quad \frac{\Delta K}{p} = 0.1362\,(a/h) - 0.0032$$

Calculations for Standard Error of Estimate

$$\text{Standard Error of Estimate} = \sqrt{\frac{\sum (y - y_e)^2}{N_1}}$$

Where $y - y_e$ = deviation of points from the line of Regression.

N_1 = Total number of points.

S.No.	$(y-y_e)\times10^{-2}$	$(y-y_e)^2\times10^{-4}$
1	11.97	143.20
2	8.30	69.00
3	8.44	71.35
4	13.23	175.30
5	8.11	65.90
6	4.18	17.50
7	9.71	94.45
8	7.01	49.25
9	3.10	9.61
10	0.0	0.00
11	1.42	2.02
12	6.12	37.50
13	0.45	0.203
14	3.44	11.83
15	2.80	7.84
16	4.40	19.36
17	6.35	40.40
18	6.00	36.00
19	8.19	67.15
20	9.41	88.70

$\Sigma(y-y_e)^2 = 1006.563\times10^{-4}$

Standard Error of

$$\text{Estimate} = \sqrt{\frac{\Sigma(y-y_e)^2}{N_1}}$$

$$= \sqrt{\frac{1006.563\times10^{-4}}{20}}$$

$$= 10^{-2}\sqrt{50.33}$$

$$= 7.09\times10^{-2}$$

$$= 0.0709$$

APPENDIX - IV

LIST OF SYMBOLS USED

1. h = thickness of soil-cement base.
2. a = radius of loaded area.
3. Δ = the deflection.
4. K = modulus of subgrade reaction.
5. p = applied pressure.
6. E_1 = modulus of Elasticity of soil-cement layer.
7. E_2 = modulus of Elasticity of subgrade layer.
8. R_o = minimum radius of curvature.
9. e = maximum tensile strain.
10. σ_z = vertical subgrade pressure.
11. σ_b = flexural stress.
12. u_2 = Poission's ratio of lower layer.
13. f = flexural strength.
14. P = total load applied.
15. u_1 = Poission's ratio of pavement layer.
16. σ_e = edge load stress.
17. σ_i = interior load stress
18. l = radius of relative stiffness.
19. b = equivalent radius of resisting section.
20. R_c = minimum radius of curvature at failure.
21. M_o = bending moment
22. α = empirical constant.
23. β = empirical constant.

24. e_f = failure strain in flexure.

25. N = number of stress repititions.

26. δ = dimensionless exponant dependent on soil type.

27. F_1 = A function.

28. F_R = A function.

24. e_f = failure strain in flexure.

25. N = number of stress repititions.

26. δ = dimensionless exponant dependent on soil type.

27. F_i = A function.

28. F_R = A function.

GRAPHS

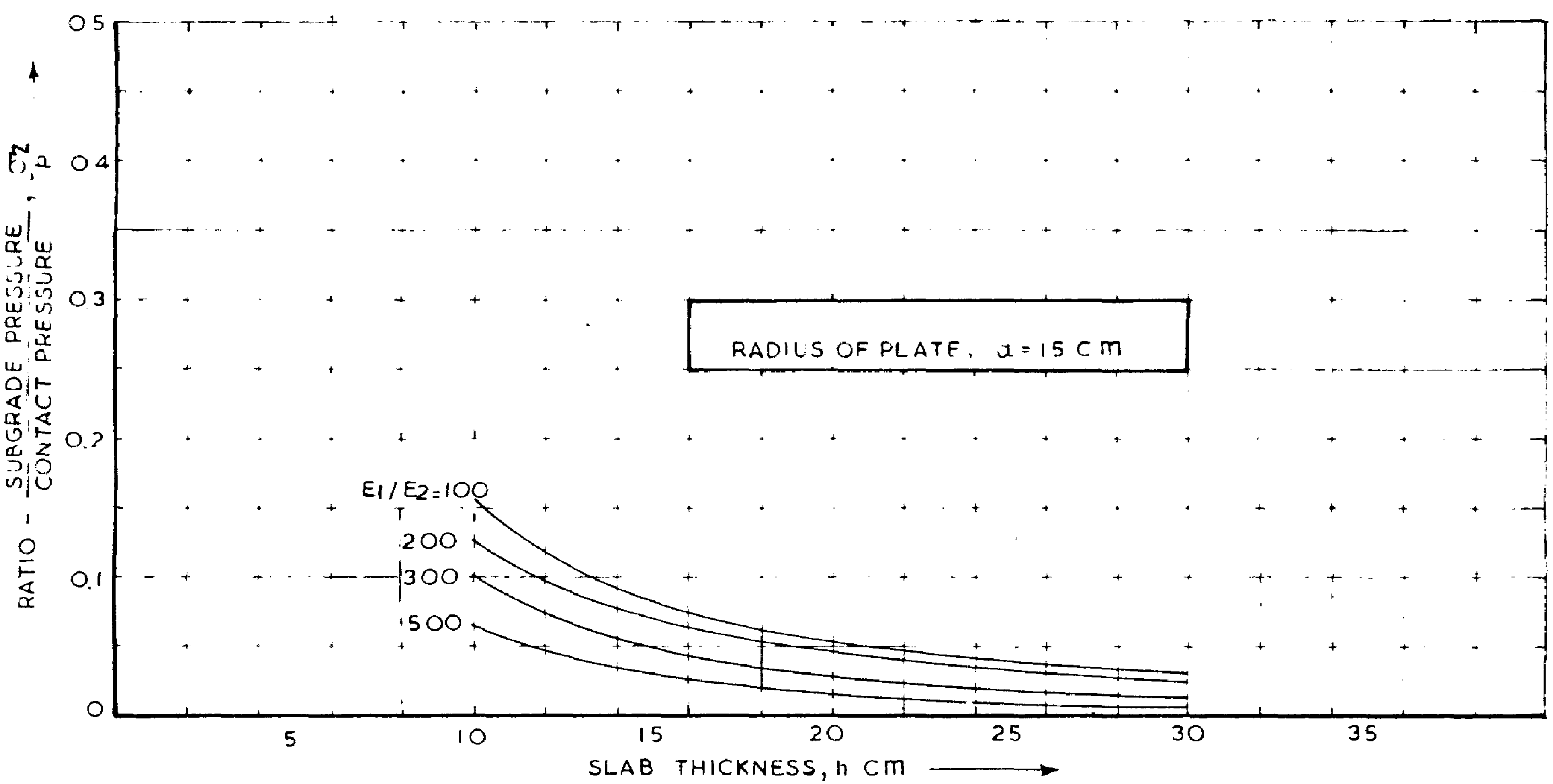

FIG. I VARIATION OF SUBGRADE PRESSURE WITH SLAB THICKNESS (2-LAYER THEORY)

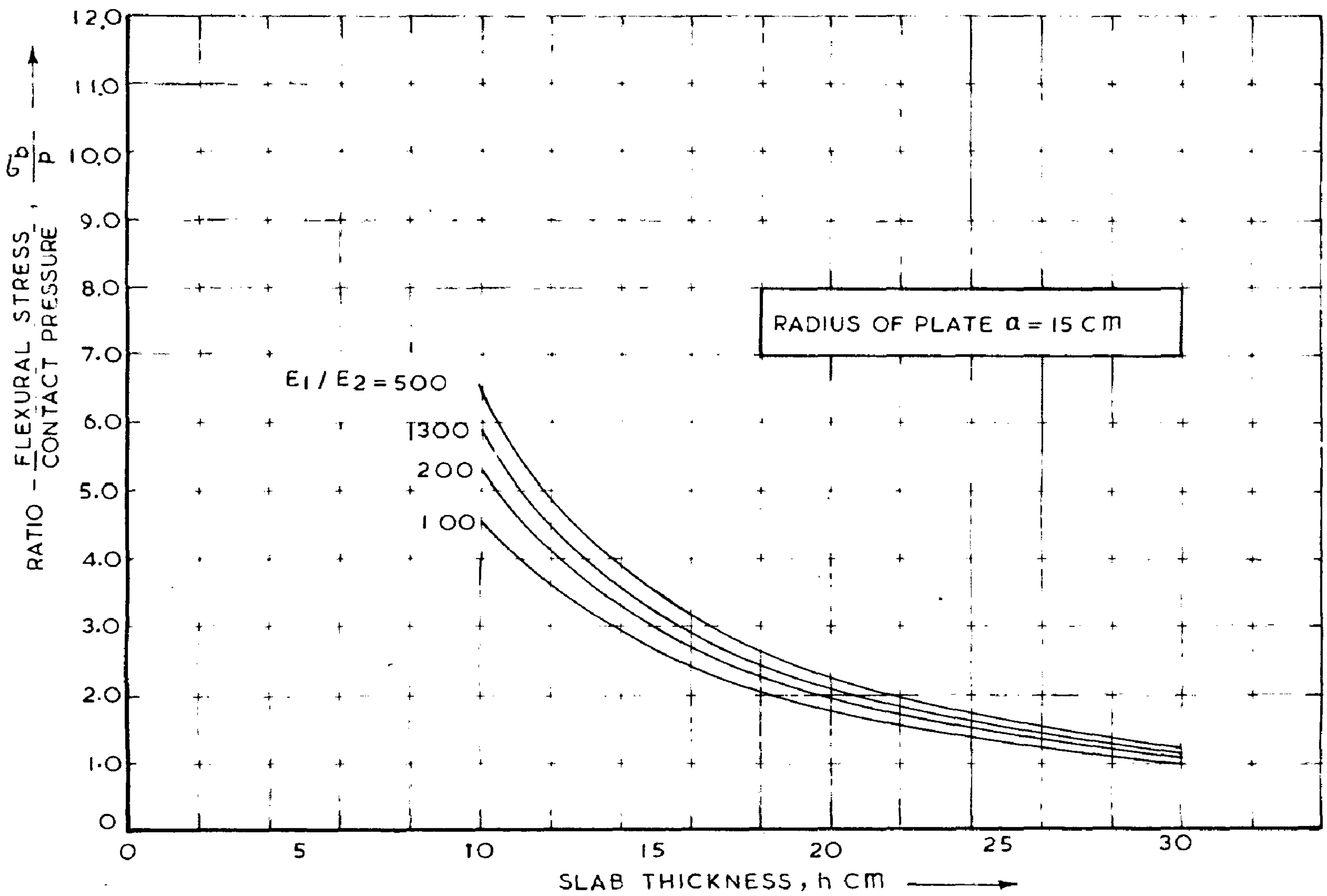

FIG. 2 VARIATION OF FLEXURAL STRESS WITH SLAB THICKNESS (2-LAYER THEORY)

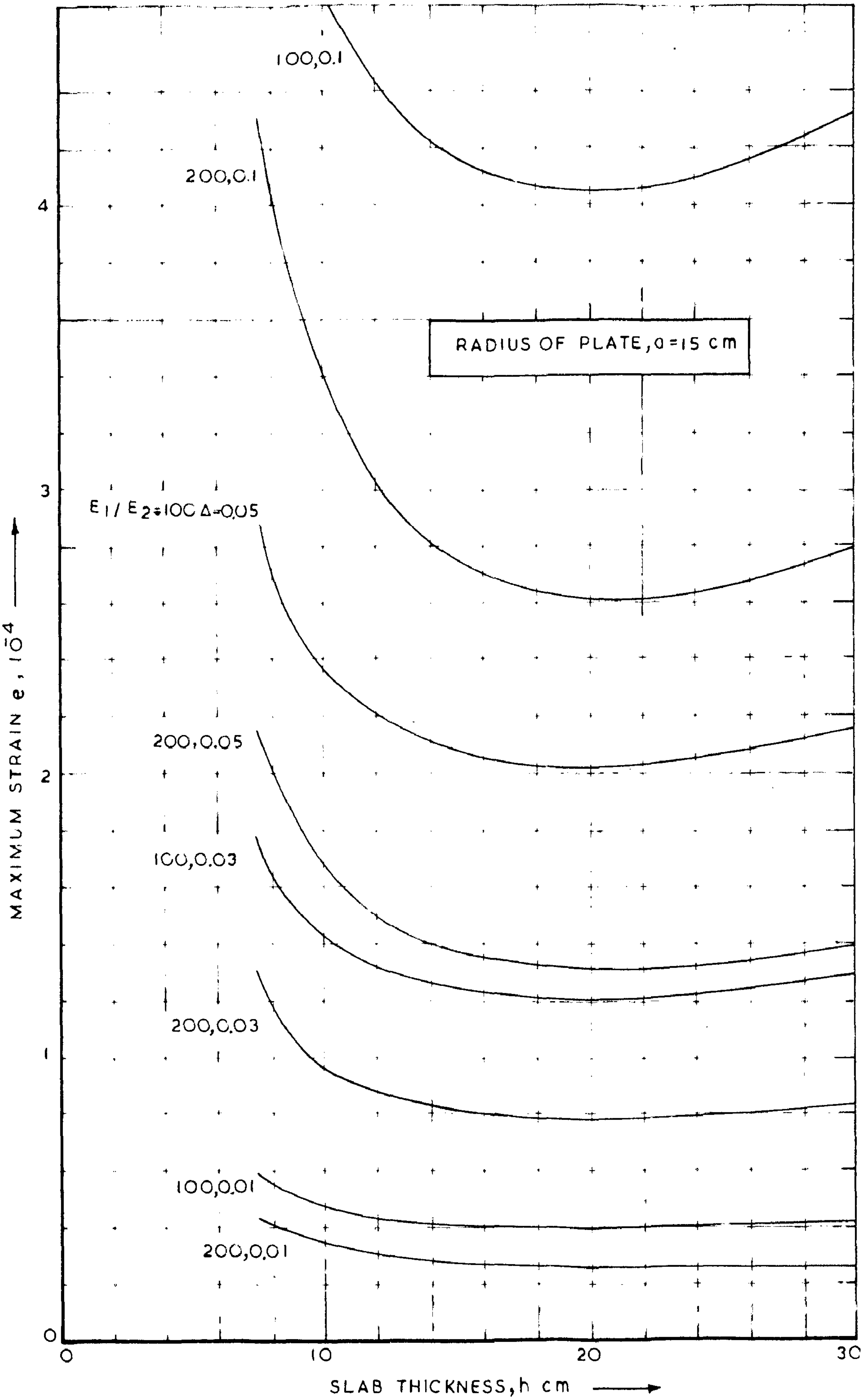

FIG. 3 VARIATION OF STRAIN WITH SLAB THICKNESS FOR DIFFERENT DEFLECTION LEVELS.

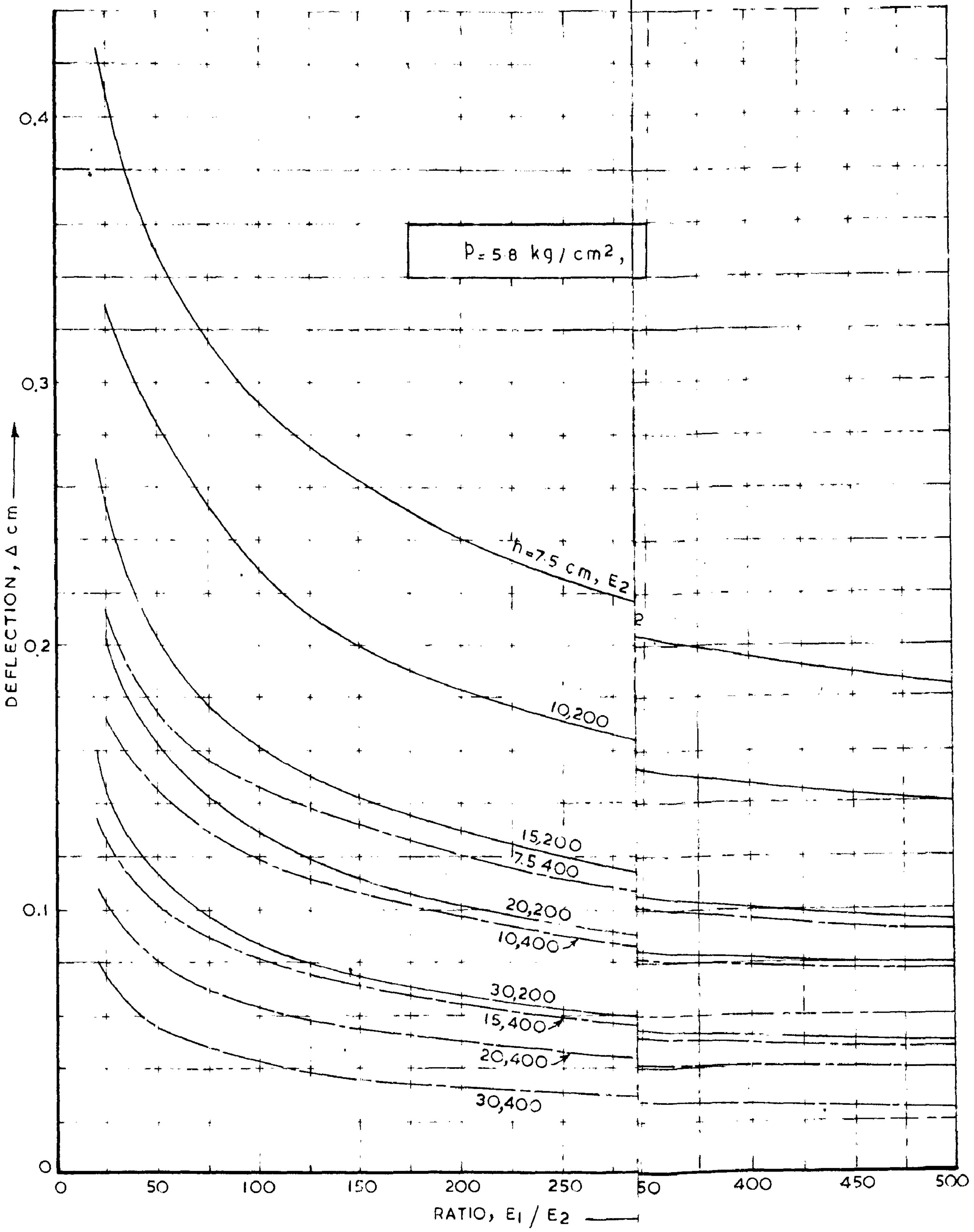

FIG. 4 VARIATION OF DEFLECTION / E2 .

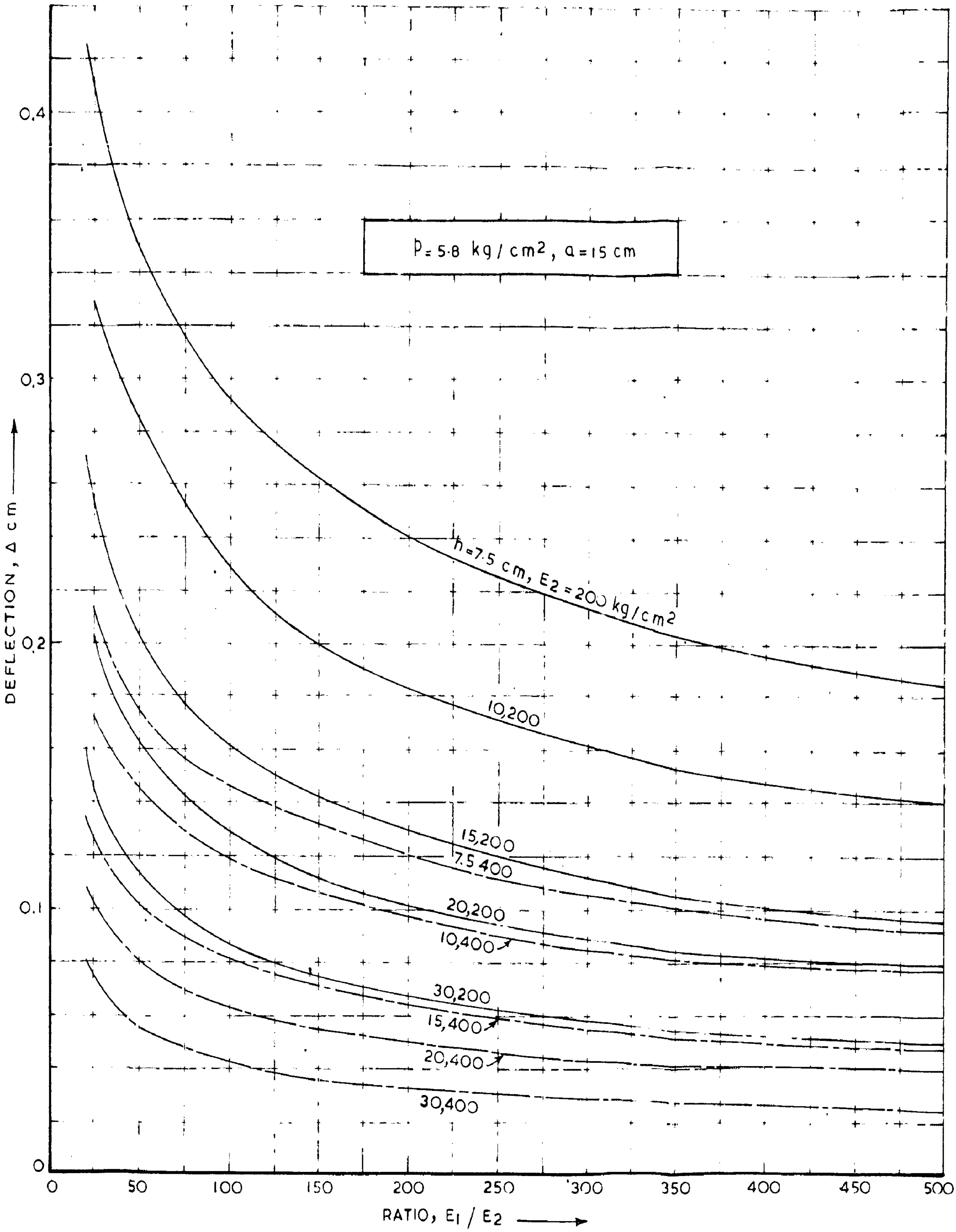

FIG. 4 VARIATION OF DEFLECTION WITH E_1 / E_2 .

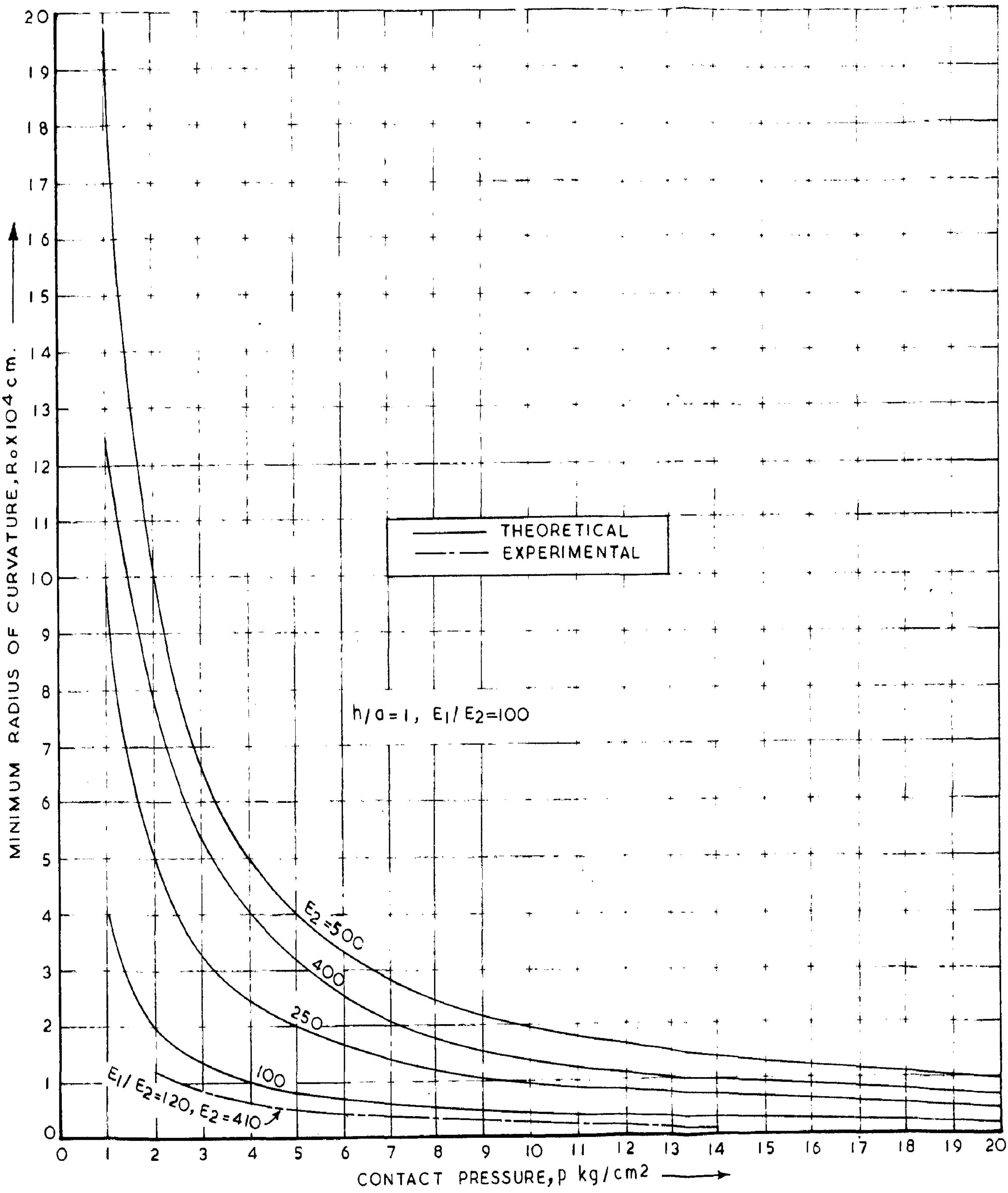

FIG. 5 VARIATION OF MINIMUM RADIUS WITH CONTACT PRESSURE

(2-LAYER THEORY)

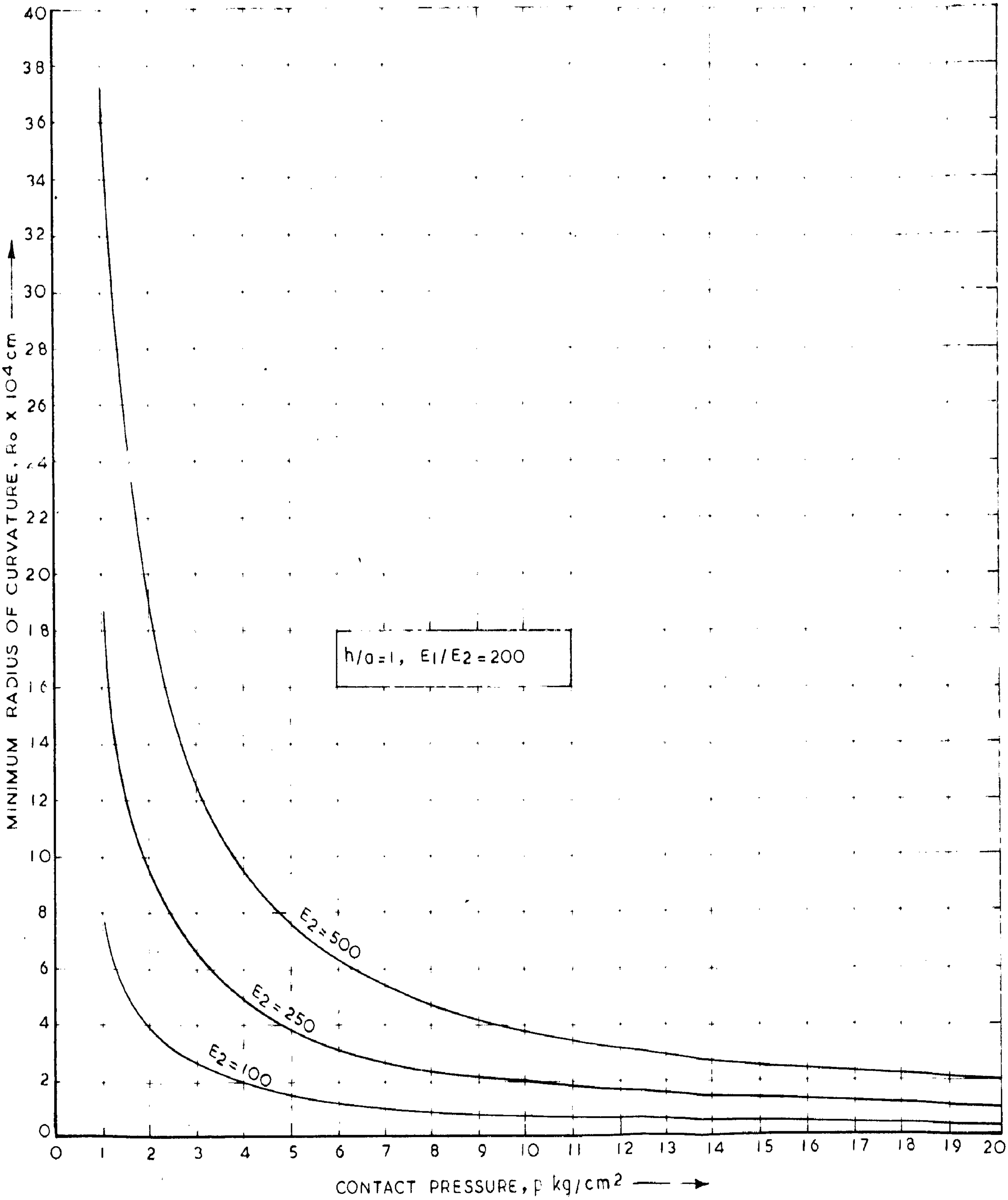

FIG. 6 VARIATION OF MINIMUM RADIUS WITH CONTACT PRESSURE

(2-LAYER SYSTEM)

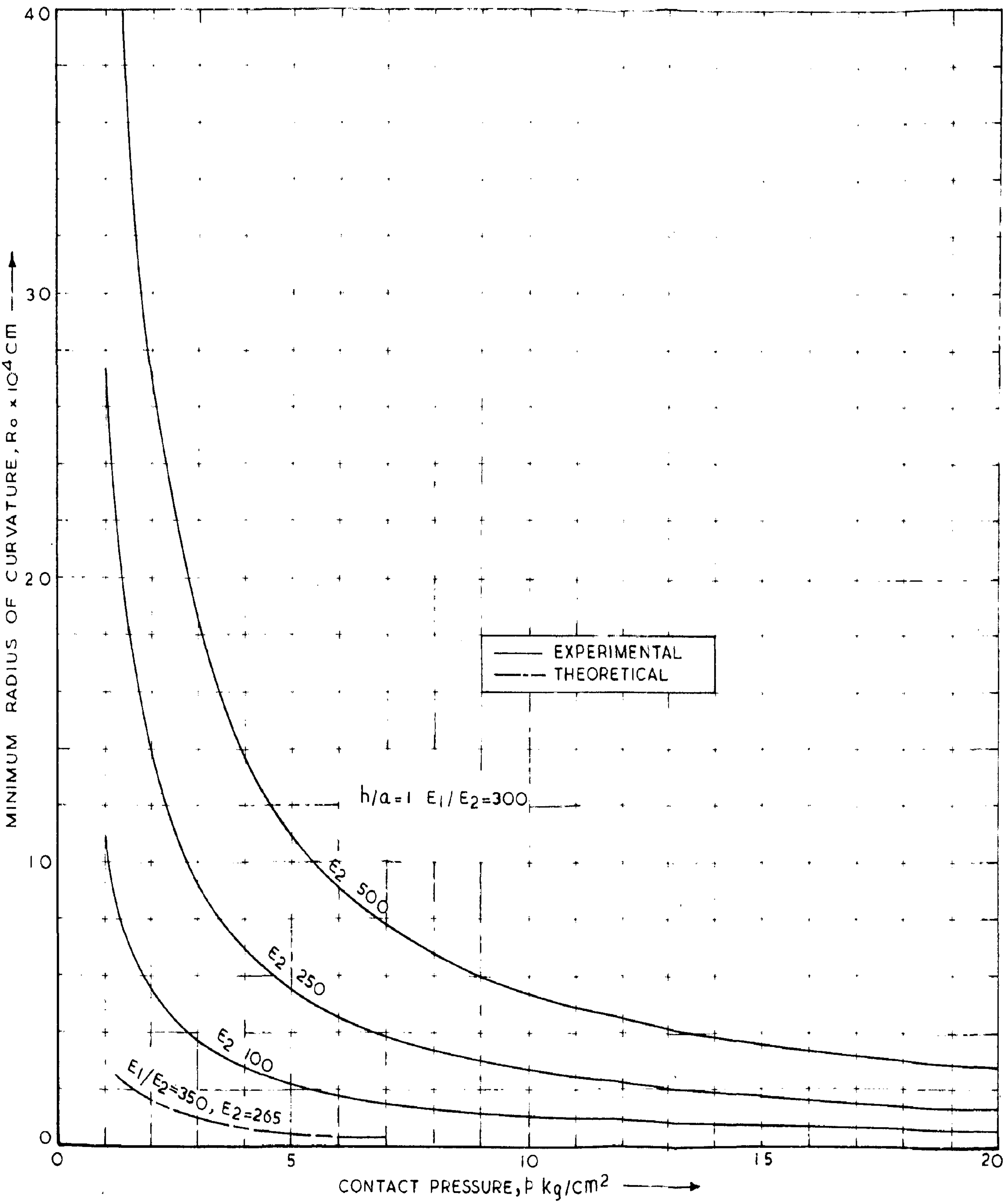

FIG. 7 VARIATION OF MINIMUM RADIUS WITH CONTACT PRESSURE

(2 LAYER SYSTEM)

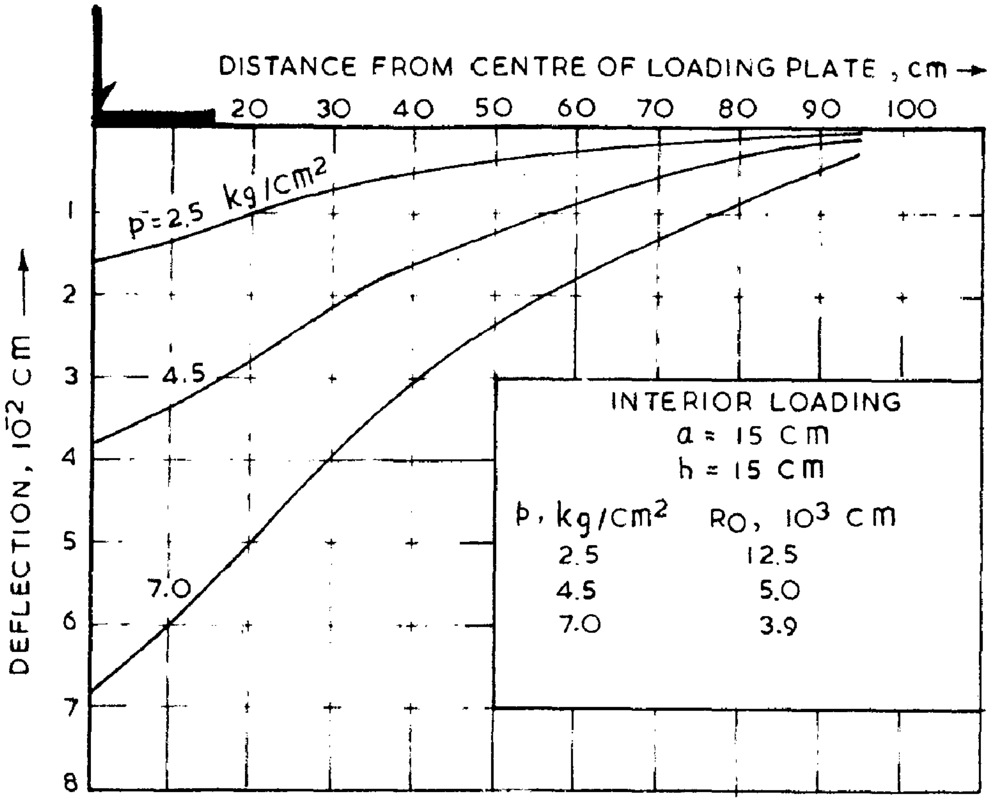

FIG. 8 DEFLECTION PROFILE AND MINIMUM RADIUS OF CURVATURE WITH VARIOUS CONTACT PRESSURES.

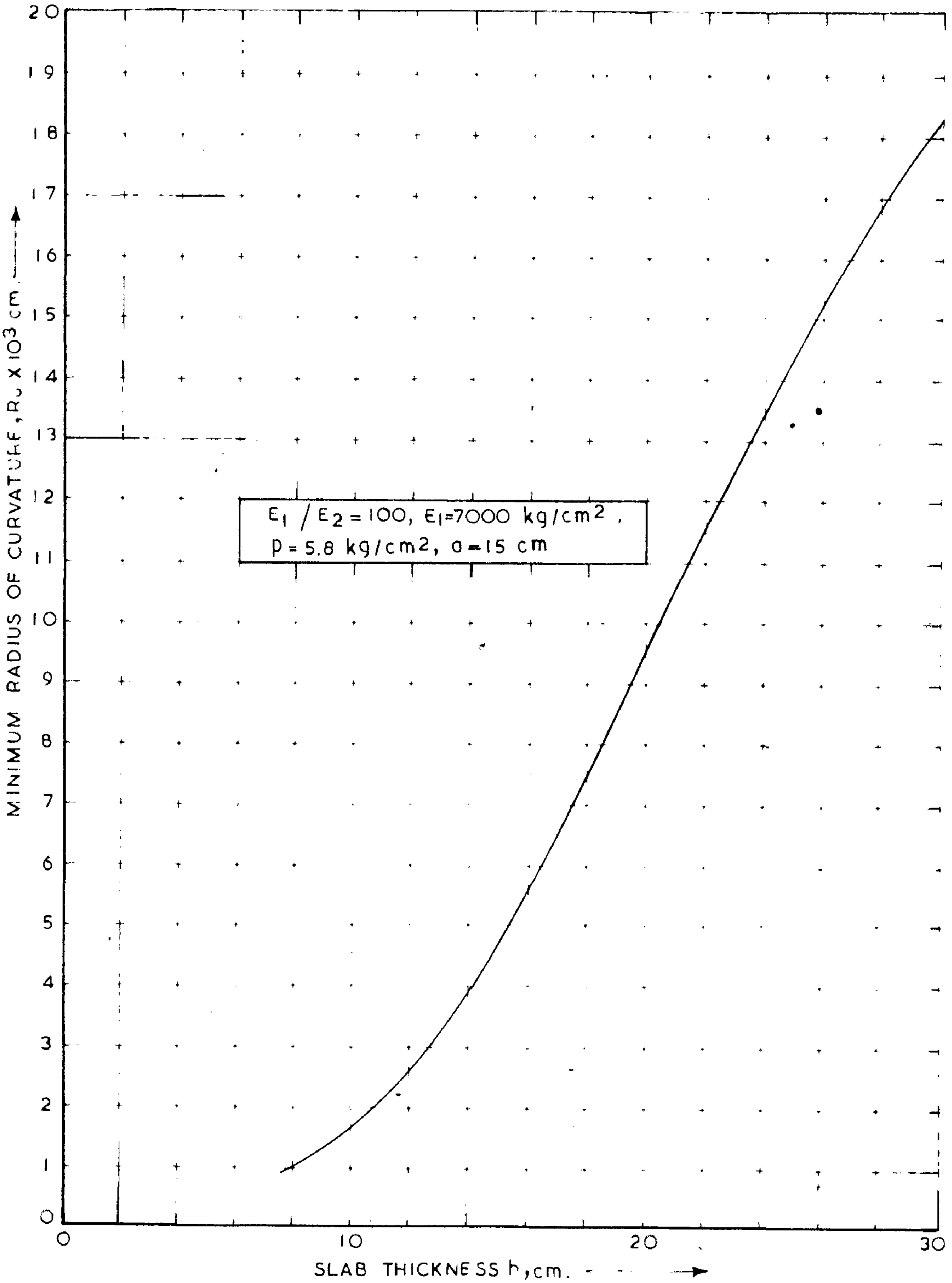

FIG. 9 VARIATION OF MINIMUM RADIUS OF CURVATURE R_o WITH SLAB THICKNESS h.

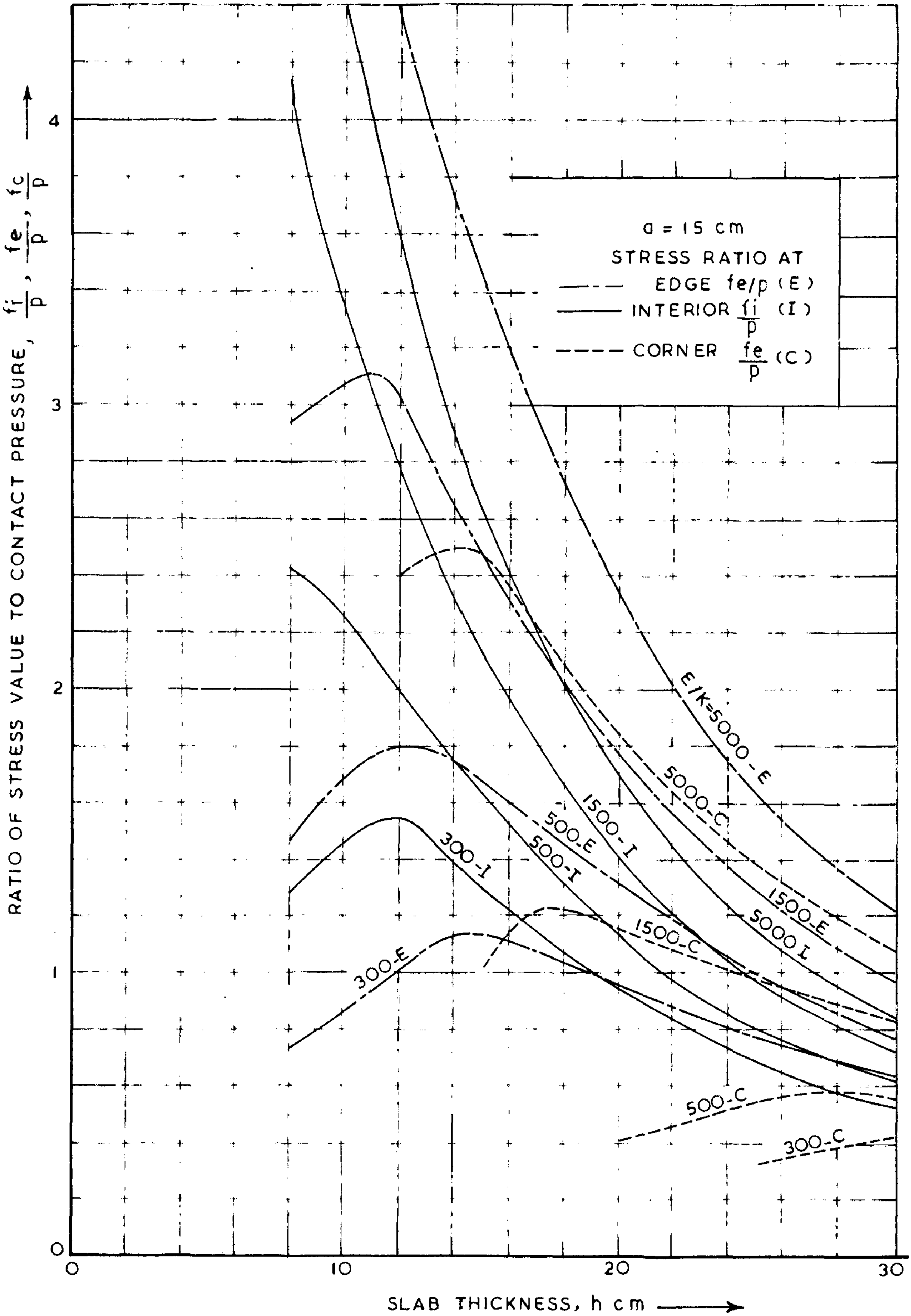

FIG. 10 COMPARISON OF LOAD STRESSES AT INTERIOR, EDGE AND CORNER BY WESTERGAARD'S THEORY.

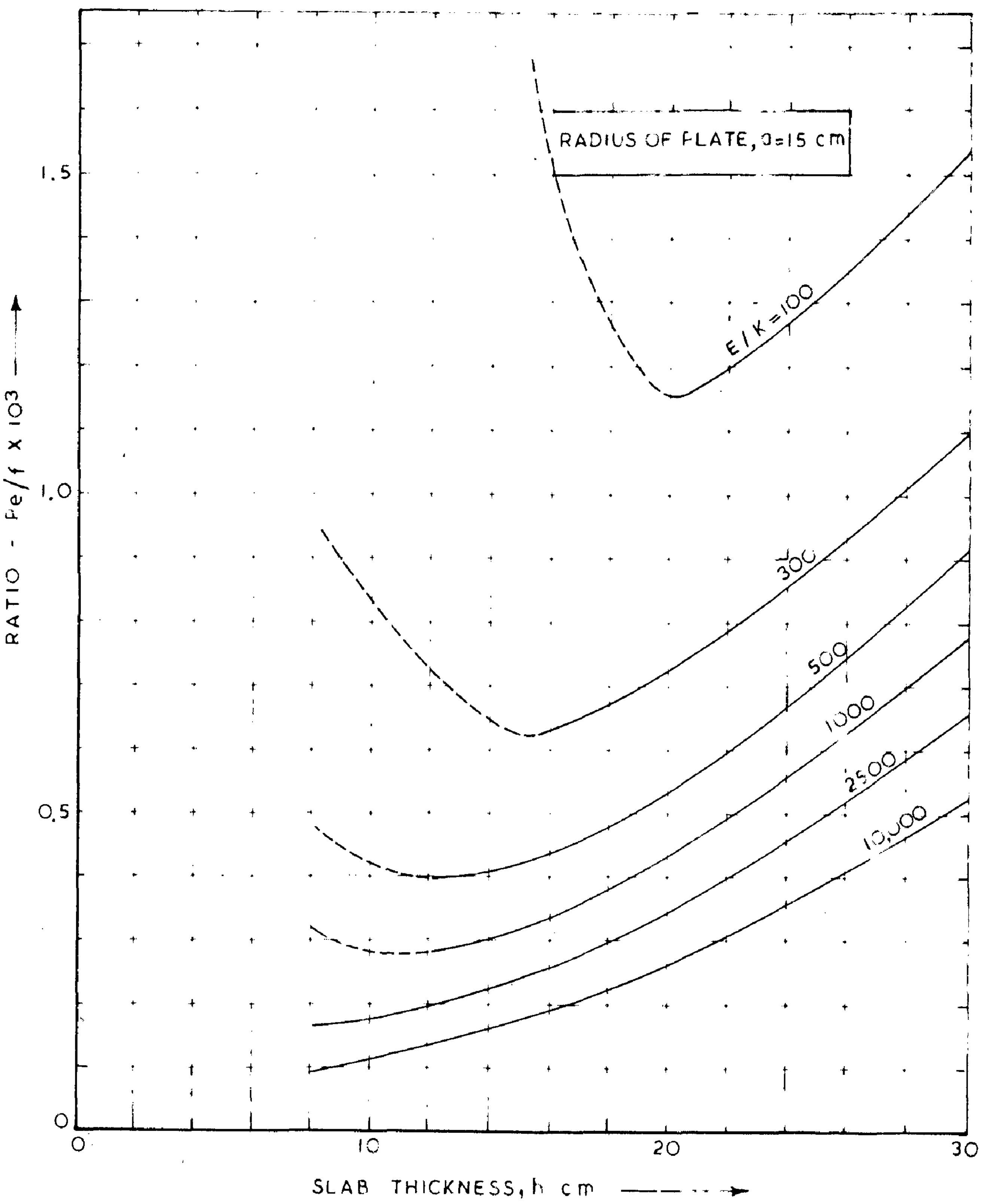

FIG. 11 WESTERGAARD'S ANALYSIS FOR EDGE LOADING.

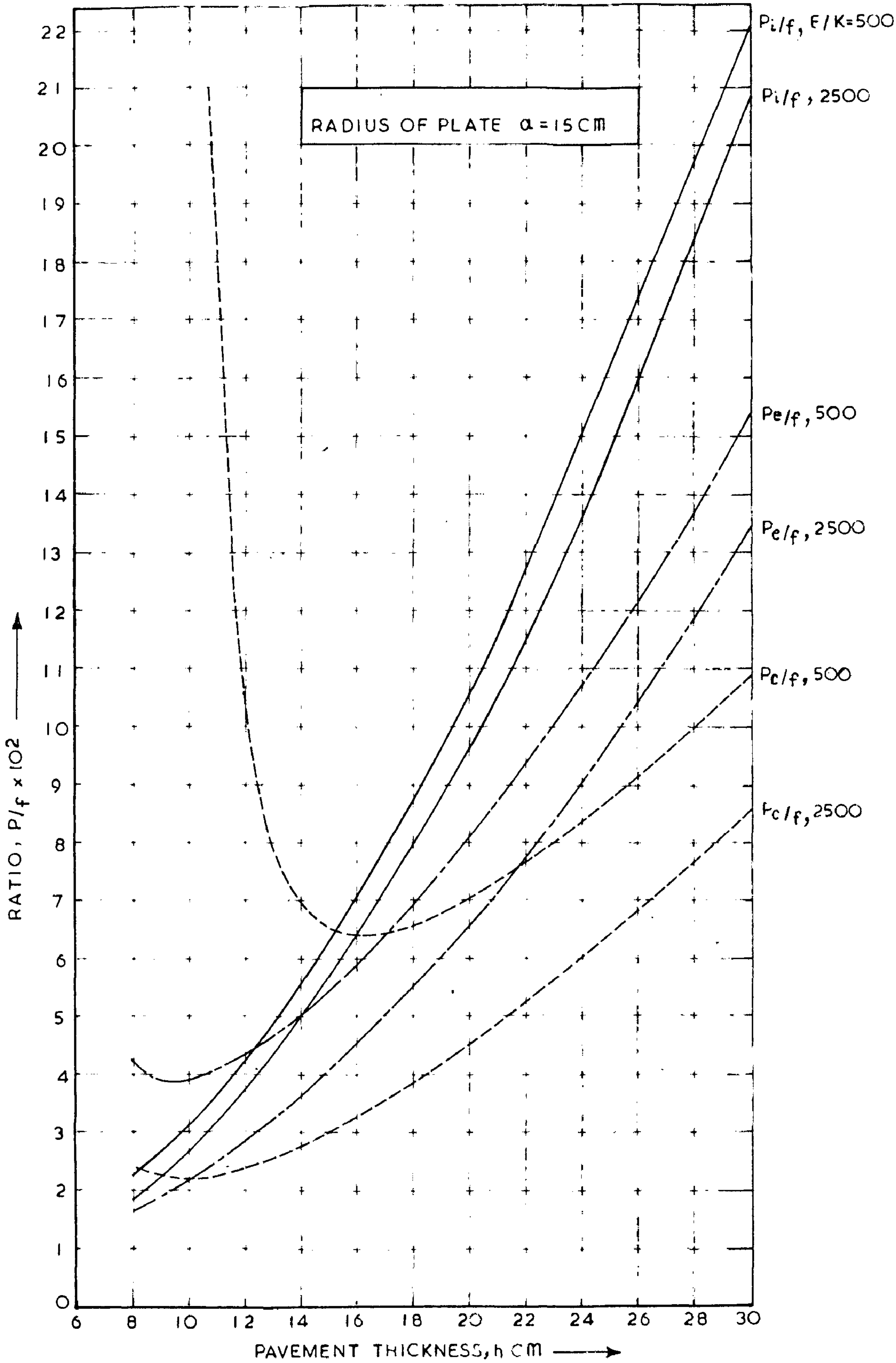

FIG. 12 EFFECT OF STIFFNESS RATIO E/K AND SLAB THICKNESS ON FAILURE LOAD BY MEYERHOF ANALYSIS.

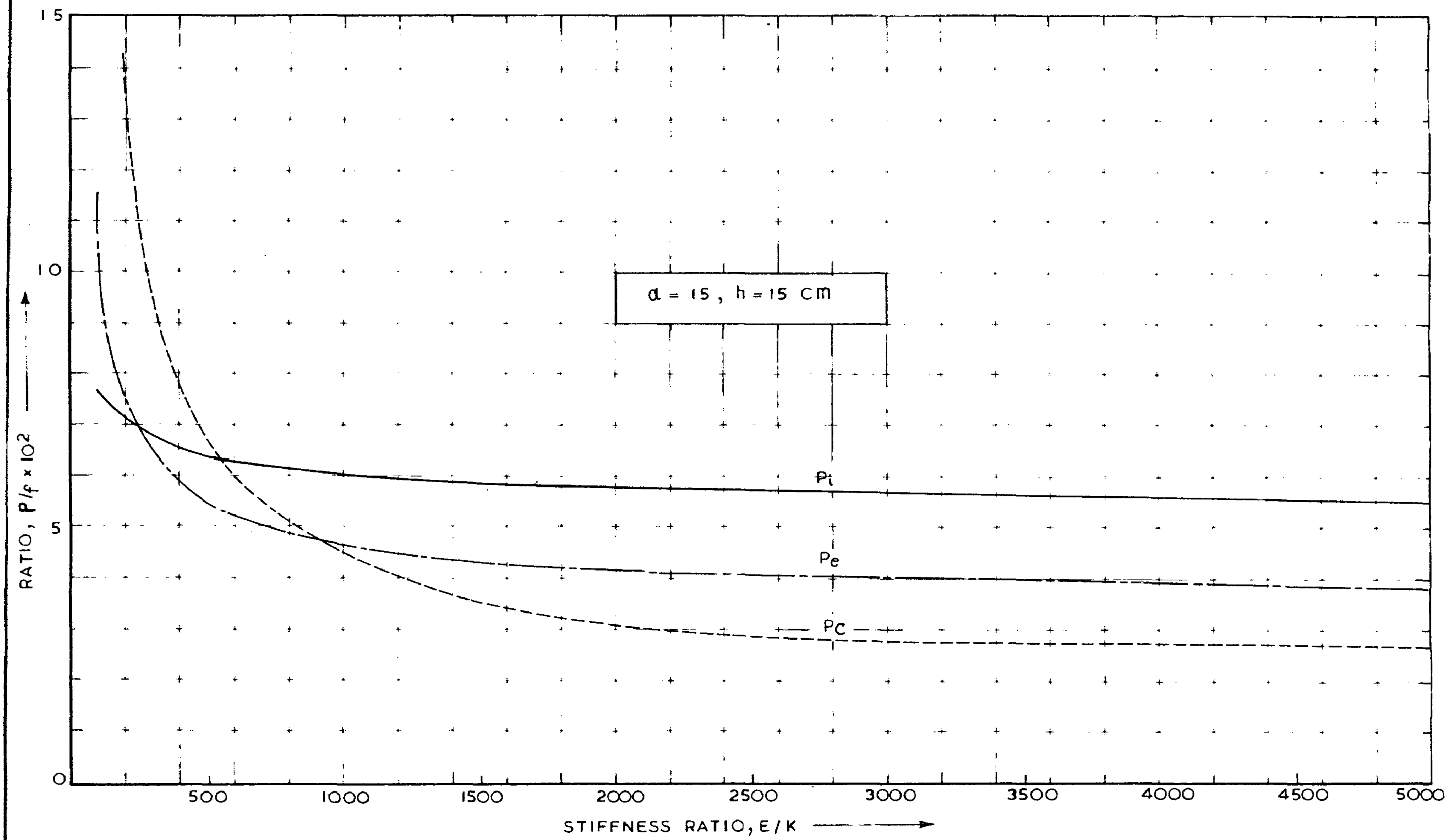

FIG. 13 VARIATION OF FAILURE LOAD WITH STIFFNESS RATIO E/K FOR INTERIOR EDGE AND CORNER LOADING (MEYERHOF ANALYSIS)

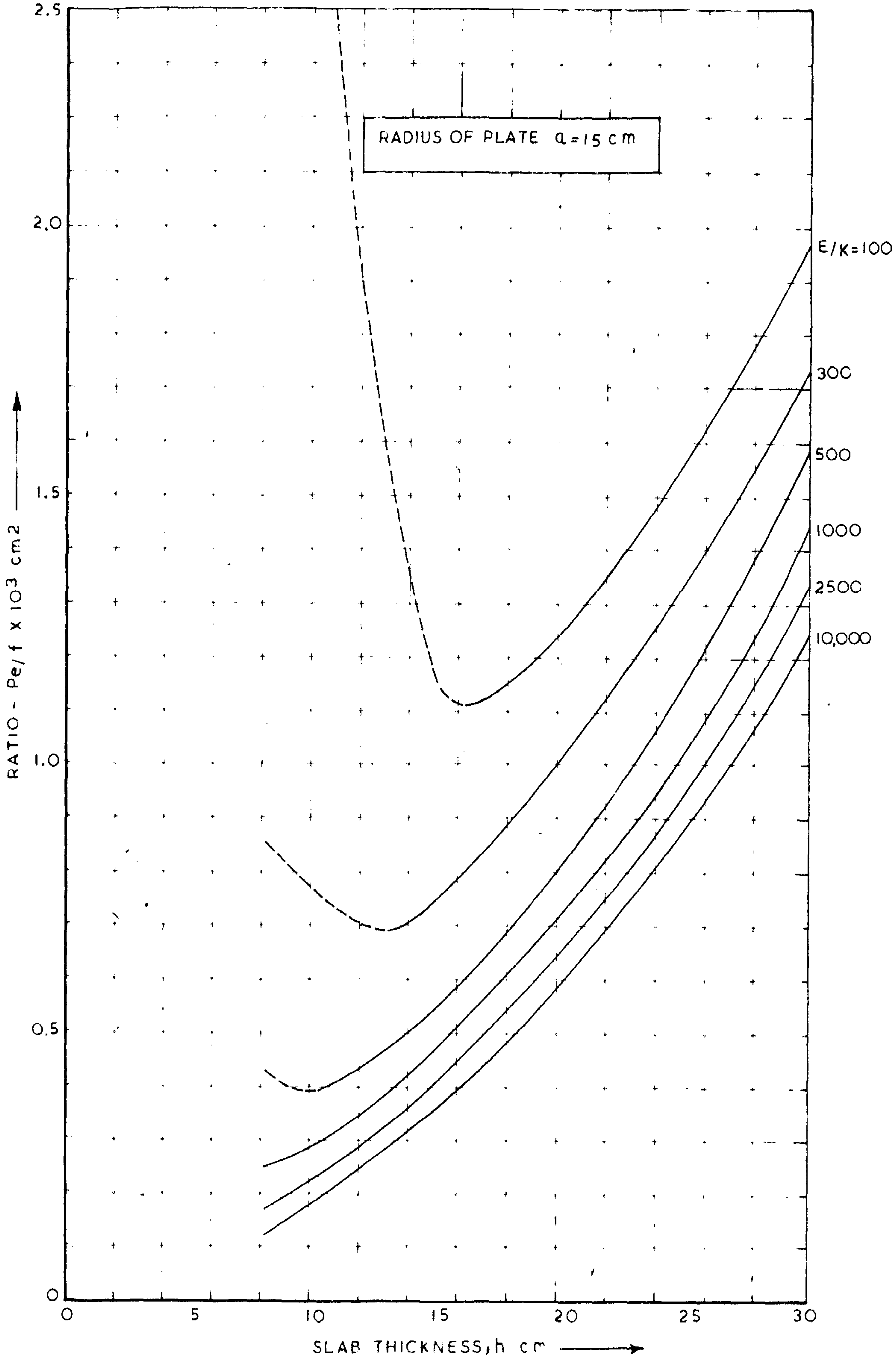

FIG. 14 MYERHOF'S ANALYSIS FOR EDGE LOADING.

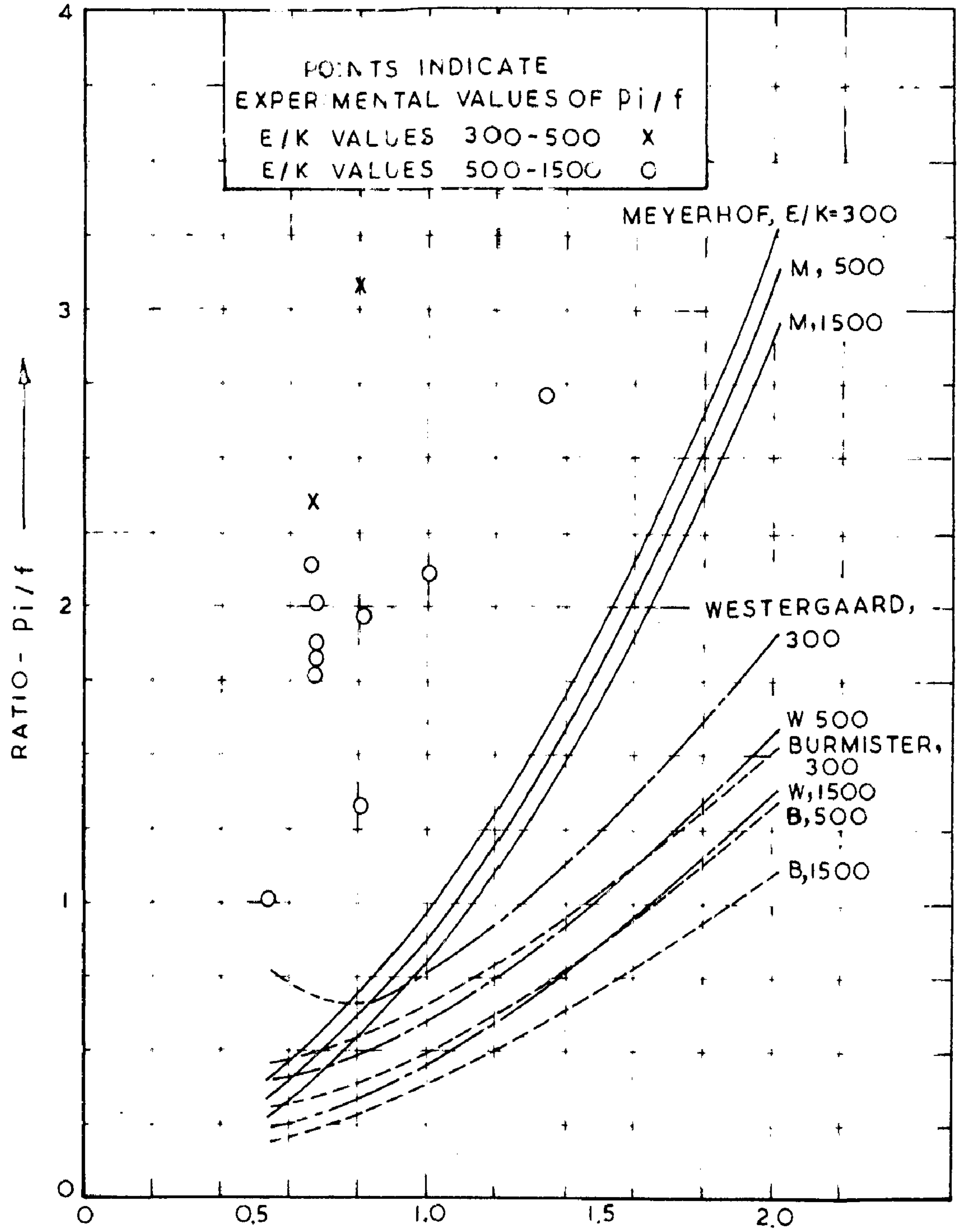

FIG. 15 COMPARISON OF THEORETICAL AND EXPERIMENTAL FAILURE LOADS AT INTERIOR.

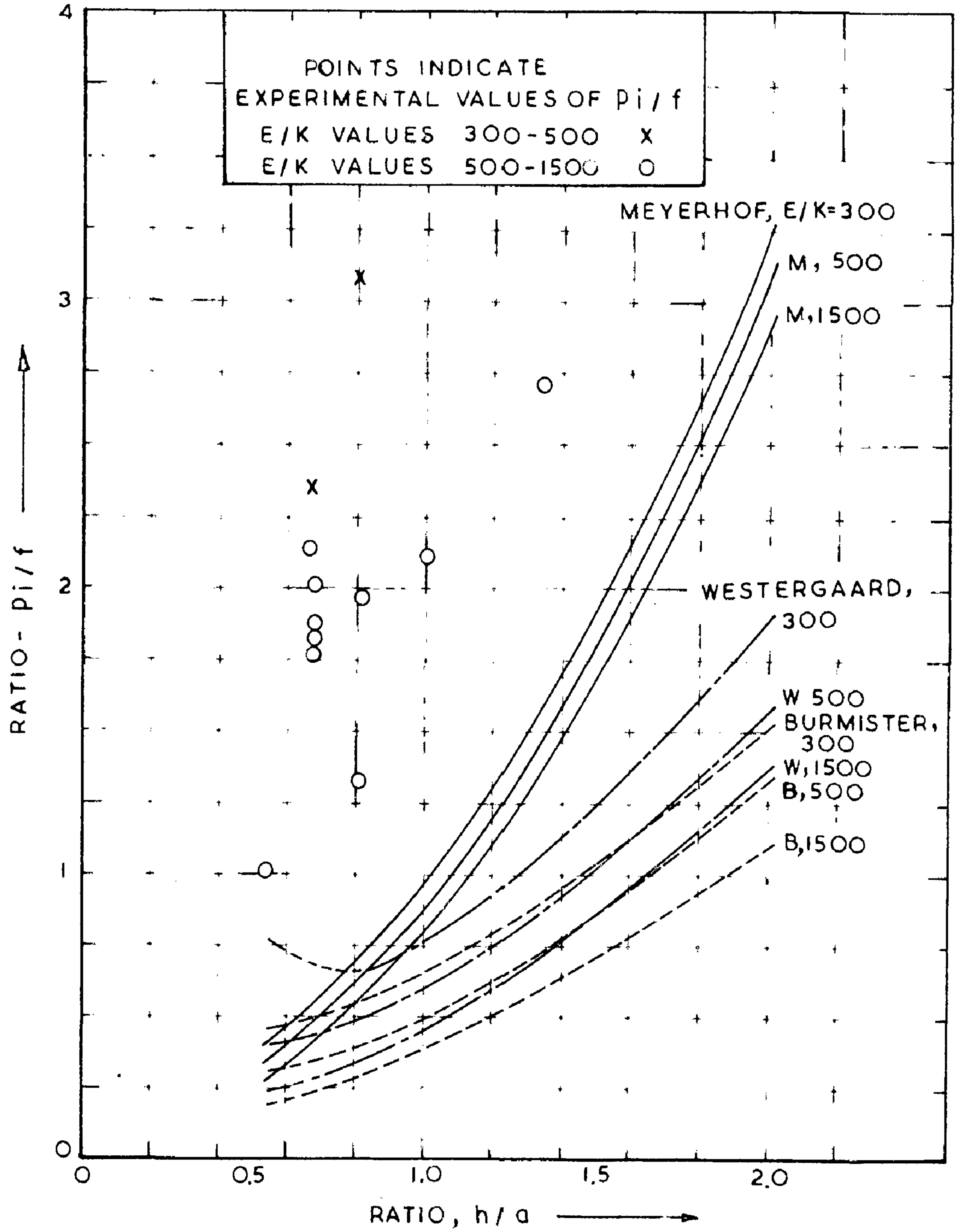

FIG. 15 COMPARISON OF THEORETICAL AND EXPERIMENTAL FAILURE LOADS AT INTERIOR.

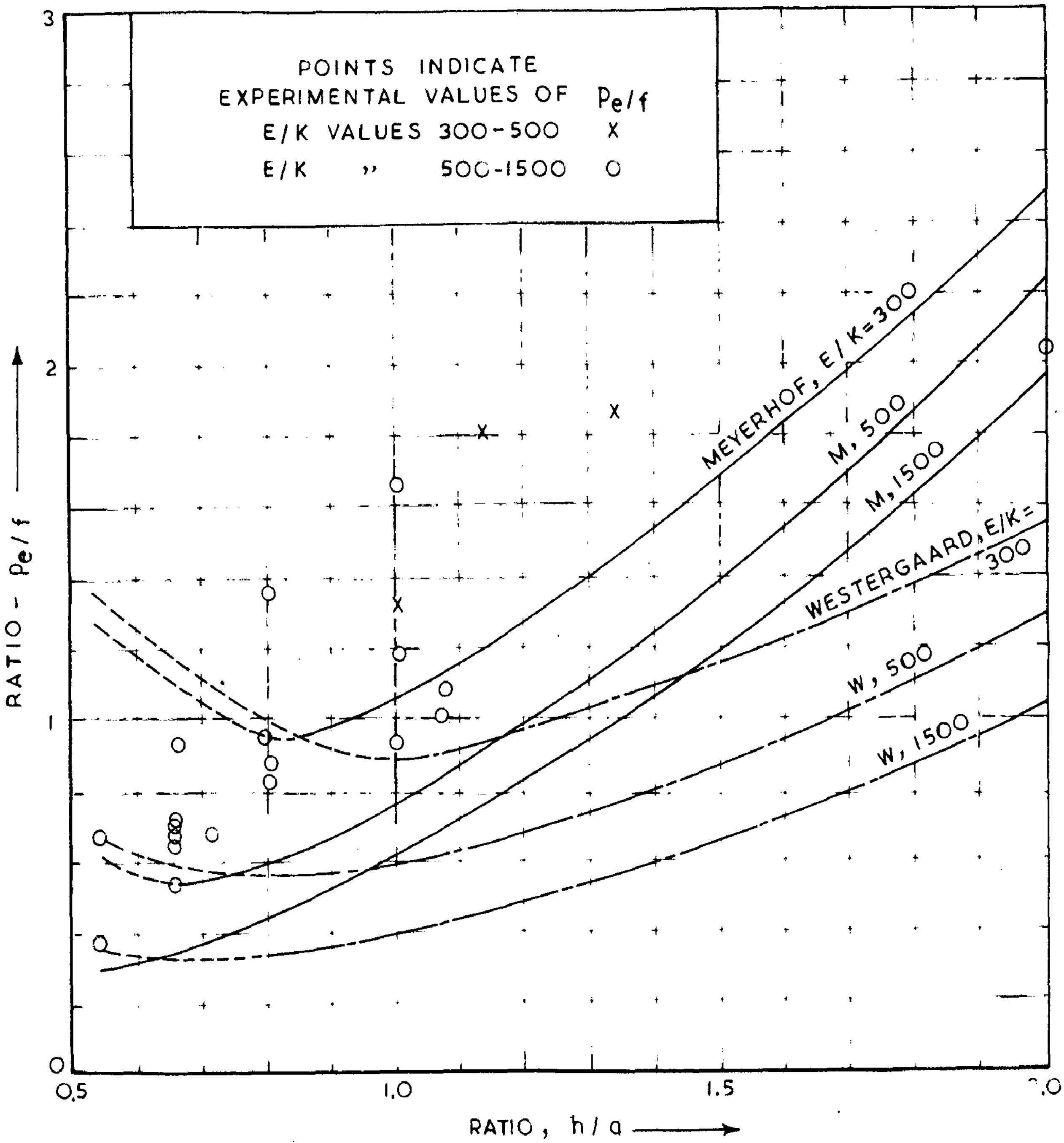

FIG. 16 COMPARISON OF THEORETICAL AND EXPERIMENTAL FAILURE LOADS AT EDGE.

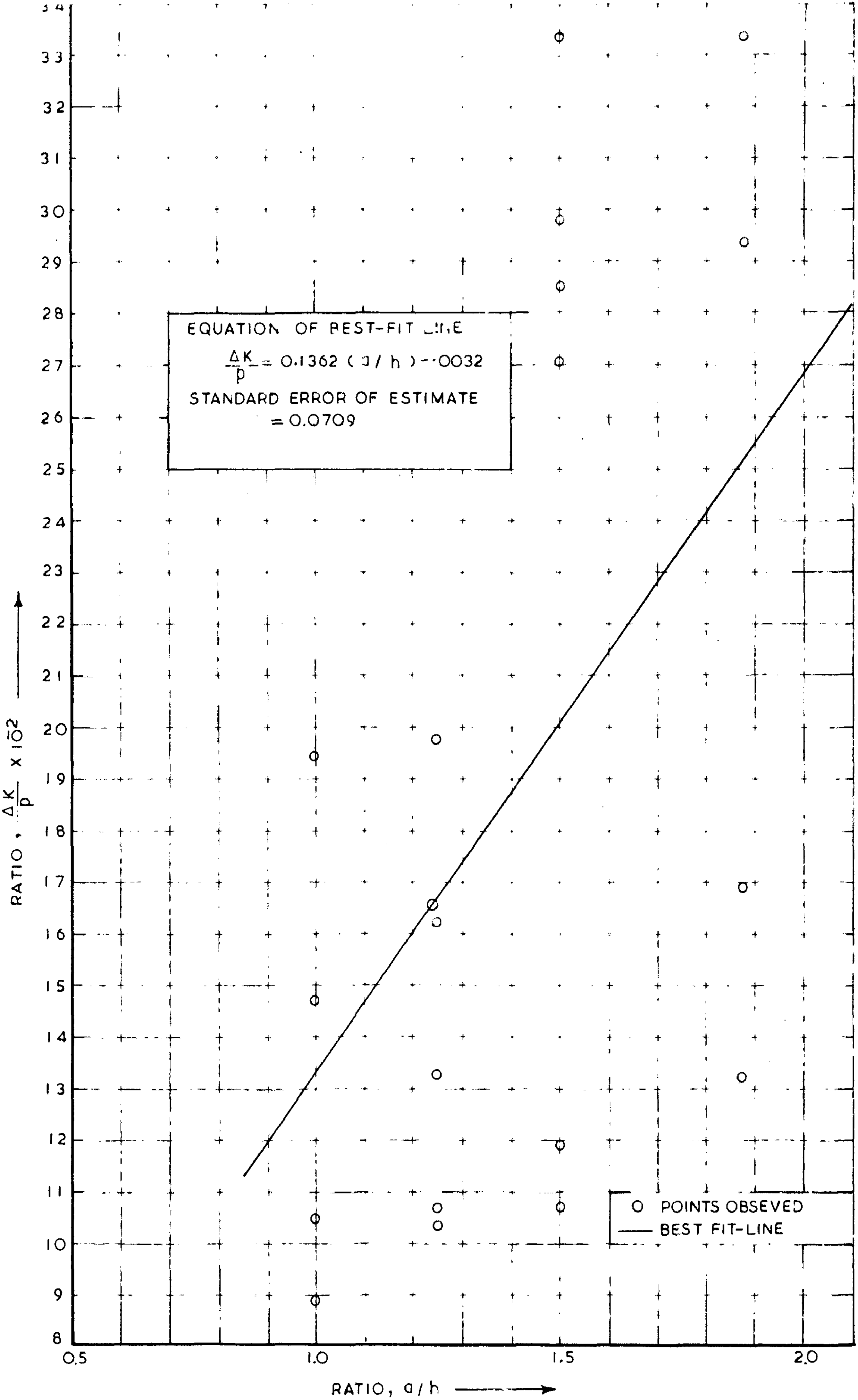

FIG. 17 NON DIMENSIONAL RELATION $\frac{\Delta k}{p}$ VS $\frac{a}{h}$ FOR EDGE