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ABSTRACT OF THESIS

entitled

"AN EXPERIMENTAL STUDY OF THE STRENGTH OF THIN ISOLATED COLUMNS IN REINFORCED CONCRETE."

The object of the research described in this thesis was to determine the usefulness of the regulation limiting the punching shear in an isolated column base.

The tests were carried out on thin bases as the punching shear stresses are greatest in thin sections.

The thesis is divided into three parts giving,

1) a survey of methods of design of isolated column bases,
2) tests on bases founded on different soils, and
3) tests on bases with varying percentages of reinforcement.

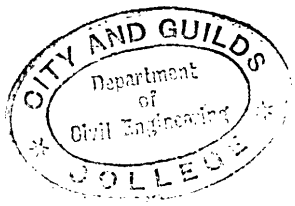
Part I in addition to the standard methods of design also shows the application of Westergaard's methods and discusses the various theories for the distribution of pressure beneath a column base.

Part II describes a series of tests carried out on thin bases which were founded on different beds with a view to determining the effect of the bed on the strength of the base. The tests show that the bed has an appreciable effect on the strength and that a given base can withstand a much greater failing load on a sand bed than on a clay bed.

Part III describes a series of tests carried out on bases with different amounts of reinforcement, the bed being kept constant.

The tests showed that the failure of a thin base is due to flexure and not to shear: in addition they showed that for any given diameter of bar the increase in strength is proportional to the area of reinforcement. The tests also showed the advantage of the use of reinforcement with good anchorage.

A thesis submitted for the Degree of Doctor of Philosophy in the Faculty of Engineering of the University of London.



AN EXPERIMENTAL STUDY OF THE STRENGTH OF THIN ISOLATED COLUMN BASES IN REINFORCED CONCRETE.

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PREFACE.

The experimental work described in this thesis was carried out in the Civil Engineering Laboratories of the City and Guilds College under the supervision of Professor A.J.S. Pippard to whom the Author is greatly indebted.

The thesis is in three parts. Part I gives a survey of methods for column base design, details of the work of other investigators and reasons why this research was carried out.

Part II describes tests carried out on small mortar specimens. Part III gives details of tests carried out on larger concrete bases whilst in the Appendix the design of the test machine and tables of results are given.

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CHAPTER I.

INTRODUCTION.

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INTRODUCTION.

That the stability of a structure depends largely on the efficiency of its foundation is an old and well established truth. The wisdom of the man who built his house upon a rock has never been doubted but the difficulty in the erection of most buildings is that to found them upon rock is usually an impossibility. The majority of buildings are founded on geological formations which are generally gravels, sands, or clay, the latter varying greatly in its nature.

In the case of rock the allowable bearing pressure varies from 30 - 40 tons per square foot, but for the type of ground usually met with in design the allowable bearing pressure varies from $\frac{1}{2}$ - 4 tons per square foot and may even be less than this.

The load from a structure comes down to the foundations either from (1) walls or, (2) columns or stancheons, or (3) by a combination of walls and columns.

In small buildings the load is generally carried by the walls but in large buildings the majority of the load is carried by columns which carry heavy loads much more economically than walls.

The main problem in foundation design is to prevent appreciable settlement of the building or, if this is not possible, to design for equal settlement for all parts of the building. Settlement of structure is a very

complicated problem but with the development of the study of soil mechanics much light is being thrown on the subject and this will doubtless in the near future affect the methods used in foundation design. At present, however, it is accepted for design purposes that appreciable differential settlement will be avoided if the load from the superstructure is spread over an area of foundation so that the allowable bearing pressure of the soil is not exceeded.

The load from the columns is spread over the required area by means of a foundation block or blocks which are made of mass concrete or reinforced concrete or of a combination of these two: the steel grillage foundation is the same in principle as the concrete foundation.

[*] Reinforced concrete foundations may be divided into the following classes: (1) wall footings, (2) independent (or isolated) column bases, square or rectangular in shape, (3) Combined footings carrying one or more columns, (4) raft foundations, (5) piled foundations.

Of these various types of foundations the independent column base of square shape is the most economic (in cases where the soil is capable of resisting

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[*] This and the following paragraph are reproduced from that section of "Modern Concrete Construction" Volume IV, which was written by the Author of this thesis.

a pressure of average intensity, say 3 tons per sq. foot) and if the designer has a free hand in the design of the foundations he will undoubtedly select this type of base.

It can be confidently stated that at least two-thirds of the recently-erected framed structures have used this type of foundation, consequently any research into the strength and method of design of it is of great value to the engineer. In spite of this fact very little research has been carried out on the strength of the isolated column base and as far as can be ascertained the tests described in this thesis are the first that have been carried out in this country.

Two possible reasons may be suggested for this:

1. The foundation is usually bedded below the ground level and then back filled up to the level of the floor immediately above it. Thus any cracks which may develop in the foundation are not noticed and may be the proverb, "out of sight, out of mind", is true to a certain extent of this type of foundation.

2. The isolated column base is such an economical type of foundation. In a recently erected warehouse with four floors designed for super loads of 250 lb. per square foot the cost of the isolated-column-base foundations was 29/6d. per square yard of site area (about 3340 square yards). The structural frame of the warehouse itself which was of reinforced concrete with mushroom floor design cost 6·2d. per cubic foot (the cube of the building

was approximately 1,483,500 cubic feet).

Therefore cost of structure (including foundations)

= 6.2 x 1,483,500 = £38,324.

Cost of foundations alone = £4,930.

Thus it is seen that the cost of the foundations is about 12 per cent of the total cost.

If piled foundations had been used the cost of the foundations would have been about 25 per cent of the total cost. Thus the engineer realising that the isolated c column base represents the most economical type of foundation has probably overlooked the fact that if it were possible to design them more economically with a saving of, say, 10 per cent the saving on the total cost would be about 1 per cent, a figure which is quite appreciable on a large contract.

Failure of an isolated column reinforced concrete base may be due to two possible causes.

1. Excessive settlement of the ground below the base.
2. Failure of the base itself by one of the causes about to be mentioned.

Failure by settlement of the base is a question for the student of soil mechanics and only fleeting reference will be made to such failures in this thesis.

Failure of the reinforced concrete base itself may be caused in the following ways.

1. By crushing of the concrete immediately below the base plate of the stancheon. This can only occur when steel

stancheons are used to carry the load and is avoided by making the base plate of the stancheon of such an area as to transmit the load to the concrete without exceeding the permissible compressive stress in the latter.

2. By rupture of the base due the development of excessive compressive or tensile stresses as a result of the bending moments to which the base is subjected. Such failure can be avoided by keeping the stresses developed in the steel and concrete within the permissible limits for compression and tension respectively.

Failure due to rupture is really caused by excessive differential settlement between the edges and the centre of the base: it is therefore dependant not only on the load to be carried and the base sizes but also the type of soil on which the base rests.

3. By rupture of the base due to the development of excessive shear stresses. The shear stresses set up in an isolated column base are generally known as "punching" shear stresses and according to one school of thought these stresses should be kept within a definite limit whilst another says that these stresses need not be limited.

Figure 1 shows a column of area "a" carrying a load W supported on an isolated column base of thickness d, and area "A", then, if p is the perimeter of the column the value of the unit punching shear, s, is given from the

equation

$$S_p = \frac{W(1-\frac{a}{A})}{pd}.$$

In the special case of a square column of side "b", then

$$S_p = \frac{W(1-\frac{b^2}{A})}{4bd}.$$

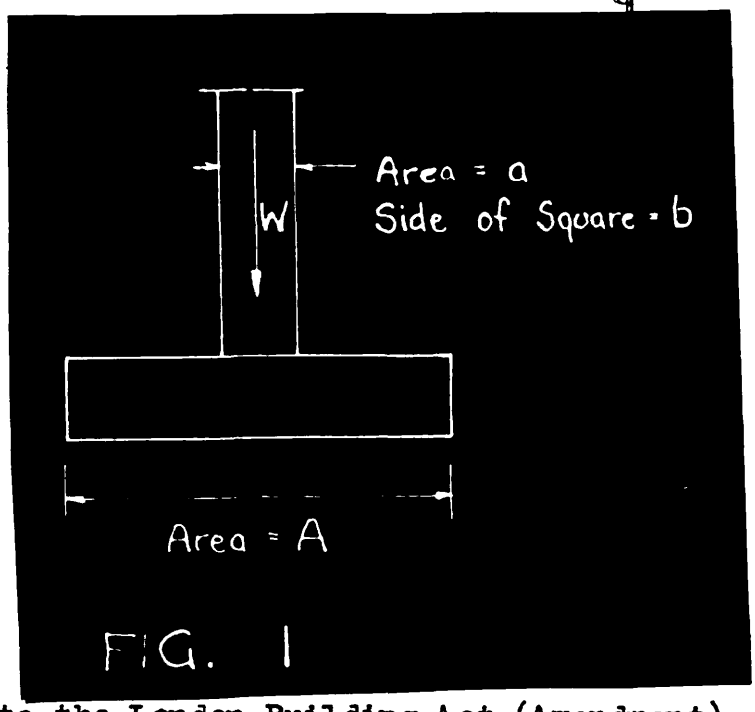


FIG. 1

According to the London Building Act (Amendment) Act, 1935, By-law 99 for reinforced concrete construction the value of s_p must not exceed twice the permissible unit shear for the concrete of which the base is constructed, e.g. in the common case of a 1:2:4 mix ordinary grade concrete the unit punching shear must not exceed 150 lb. per square inch.

The Recommendations for a New Code of Practice for Reinforced Concrete (19), however, do not limit this punching shear in column bases and a designer working to these recommendations is quite justified in using a column base where the punching shear stress is infinite.

It is true that Clause 810 of the Recommendations specifies that the punching shear in the slab over a column in a flat slab floor must be kept within specified limits.

Reynolds (1) in giving examples of the design of buildings according to the Recommendations, has applied this

clause to the design of a column base, restricting the punching shear to the limits suggested therein. It is, however, clearly stated in Clause 801(a) that the Recommendations in Section 8"only apply to the design of rectangular slabs of approximately uniform thickness arranged in three or more rows in each direction": therefore there is no justification for applying Clause 810 to the case of the isolated column base.

There is little doubt that the L.C.C. Regulations and the Recommendations for a New Code are in definite disagreement with regard to the limiting of the punching shear in column bases and the main reason for carrying out the research described in this thesis was to find out which of the two is correct.

As far as the Author is aware no tests with this object in view have been previously carried out, consequently neither school of thought has any experimental evidence on which to base its regulations. The results of the tests should, therefore, be of value in determining whether the punching shear stress should be limited.

Since the punching shear was intended to be the critical factor in the tests they were carried out on bases which were thin in section, thus giving a high value for the punching shear.

The tests described fall into two main classes.

Class I.

A series of tests on small bases 8ins. x 8ins. in plan with nominal thickness of 1in. and 1½ins. The main object of these tests was to determine whether the soil on which the base was founded had any effect on its strength. Other purposes fulfilled by this series were,

1. To observe approximately the increase in strength with varying amounts of reinforcement, noticing whether a similar increase was observed for each type of bed.

2. To study the effect of reinforcement placed in the top of the slab. It is pointed out in Chapter II that some designers use this reinforcement when the punching shear stress is high.

3. Since the specimens used were small the tests are of value in deciding how useful "model" tests can be in the case of reinforced concrete.

Different series of tests were carried out by varying (a) the bed on which the base was founded and (b) the amount of reinforcement in the base.

Three different kinds of bed were used, rubber, sand and wet clay. On each bed tests were carried out on specimens 1½ inches thick with reinforcement varying from 0 to 1.8 per cent. On the sand bed alone tests were also carried out on specimens 1 inch thick with reinforcement varying from 0 to 1.2 per cent.

As the sections were so thin to make them in

concrete was not practical and they were made of a 3:1 sand-cement mortar. The column used was 2 inches square thus giving a value for the ratio between the depth of the base and the side of the column varying between 0.5 and 0.75, both of which are very low values. (In most cases in practice this ratio is between 1.5 and 2.0, this high value usually being necessary in the case where the punching shear is limited).

Because of the thin section it was not practicable to use Mild Steel rods as reinforcement and the lightest section Expanded Metal was used. It is realised that the use of such a material in research is not to be recommended but as the variation of strength with reinforcement was only a minor issue in this series of tests it is suggested that the use is permissible under these circumstances.

In this class of tests 18 different series were carried out each series consisting of four specimens, the average of the four results being generally taken. Thus tests were carried out on 72 specimens.

Class II.

This class consisted of a series of tests on specimens 2 ft. x 2 ft. in plan and 4 ins. thick. It having been found from the experiments in Class I that the strength of a given base varied with the type of bed on

which the base rested it was decided in this series of tests to keep the bed as uniform as possible and to determine how the strength of a thin isolated column base varied with the quantity and type of reinforcement used. The only type of elastic bed which would always be uniform is one composed of steel springs.

Assuming that the maximum load to be reached during the tests is 40 tons a base composed of steel springs would be very heavy (about 28 springs made from ½ in. diameter mild steel bars and with a radius of 1 inch would be required). The alternative, therefore, was to use a bed of one of the types employed in Class I experiments. A rubber bed was not thought advisable for two reasons, first, that rubber is not an elastic material and secondly, that it is not met with in practice as a bed for foundations. Clay, whilst being a material commonly met with in foundations, has the disadvantage that it is colloidal and its properties vary considerably with its water content. Dry sand on the other hand is a granular material whose elastic properties depend largely on the degree of compactness of the grains. It was decided that the latter factor was easier to control than the water content of a clay bed, therefore all the tests in this class were carried out on a bed of dry sand.

Whilst the load was being applied readings of the deflection were taken and from these it was possible to

check the degree of compactness of the sand bed.

The reinforcement in the specimen consisted of mild steel bars placed with equal areas each way. The percentage of reinforcement used varied from 0.1 to 1.15. The diameters of bar used as reinforcement were $\frac{1}{8}$ in., 3/16 in., $\frac{1}{4}$ in., 5/16 in., $\frac{3}{8}$ in. and $\frac{1}{2}$ in; thus giving a comprehensive range. Straight bars of each of these diameters were used but to determine the affect of anchoring the reinforcement by means of hooks a further series of tests were made on hooked bars, $\frac{3}{8}$ in., 5/16 in. and $\frac{1}{4}$ in. diameter. In addition to the reinforced specimens plain specimens were also tested.

The stancheon used for the experiments in this case was 4 inches square except for one series of tests in which case a 7 inch square stancheon was used.

A concrete of 1:2:4 mix was used throughout the experiments, Ferrocrete rapid-hardening cement being used.

In the Class II tests 32 different types of specimen were tested.

In some cases two specimens of each type were made in others more, the total number of specimens tested being 72.

The tests were in all cases carried out after 7 days both in Classes I and II. Strength tests carried out at 14 days have been criticised on the ground that the specimens have not cured sufficiently (2) and "a fortiori",

such criticism applies to seven day tests. The cement used throughout, however, was rapid-hardening which is claimed to reach a strength in seven days equal to that reached by a normal cement in 28 days. Because of this the author feels quite justified in producing results based on 7 day tests.

CHAPTER II.

THE DESIGN OF ISOLATED COLUMN BASES.

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It is proposed in this chapter to give first the generally accepted method for the design of an isolated column base in reinforced concrete together with some criticisms and secondly, the application of the mathematical theory of elasticity to design of column bases. Since the tests were carried out on bases square in plan (the type commonly met with in practice) the theory for this shape only is being considered.

Part I - Practical Method of Design.

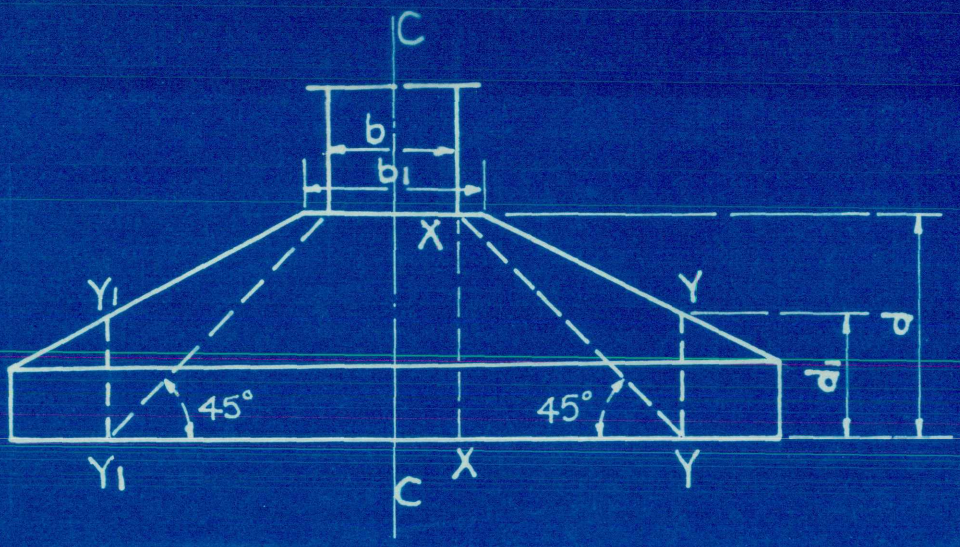
Figure 2 shows a square base which supports a column carrying a load of W, the safe bearing pressure on the ground being P.

If the column is of steel then the first step is to calculate the size of the baseplate. If c is the safe crushing strength of the concrete, then b, the side of the baseplate is equal to $\sqrt{\frac{W}{c}}$

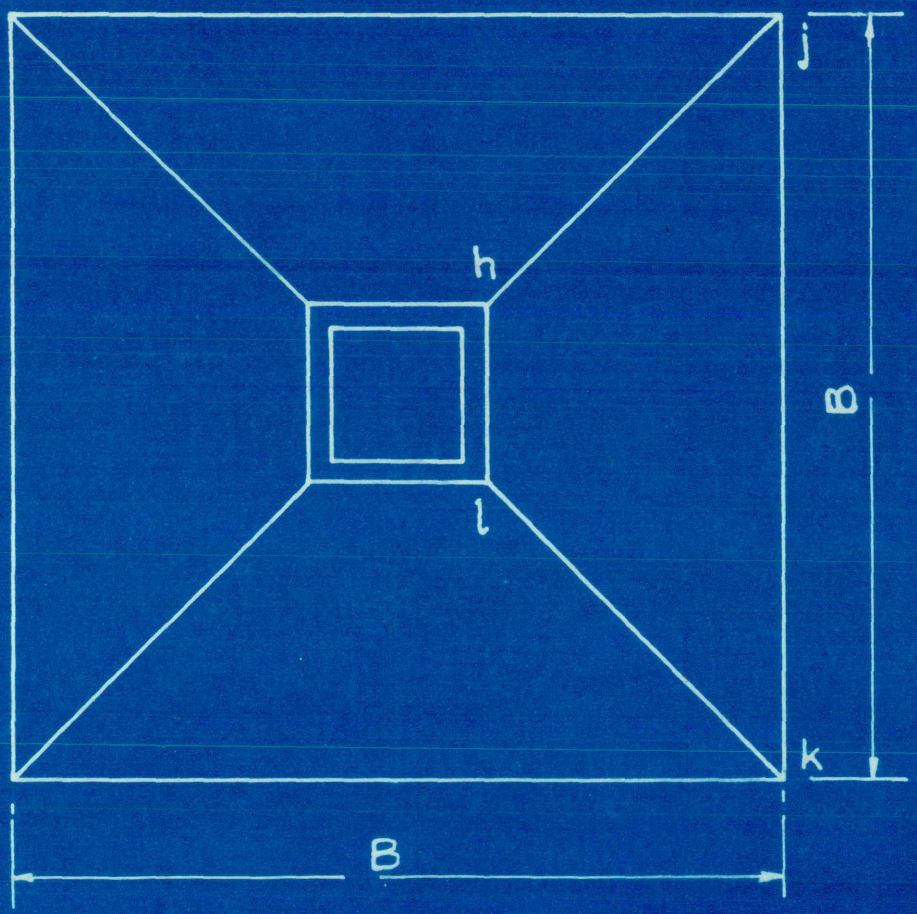
The generally accepted method of design assumes that the ground pressure is uniform under the whole area of the base in which case the size of the base is given from the formula

$$B^2 = \frac{W + W_b}{P}$$

where W_b = the weight of the concrete base.



ELEVATION



PLAN

FIG. 2

(in the Recommendations the weight W_b may be decreased by an amount equal to the weight of the earth displaced).

The depth of the base "d" is governed either from bending moment or punching shear considerations.

Bending Moments.

Most text books state that the critical section for bending moment is at XX: the total upward pressure from the soil is divided into four equal parts each acting on a trapezoidal area similar to hjkl. Thus the moment on XX is the moment of the area hjkl about hl and this can be shown to be equal to $\frac{WB}{24}(2 + \frac{b}{B})(1-\frac{b}{B})^2$.

From this formula the bending moment at XX can be calculated and the required depth to resist this moment can be found. The width of the compression flange is generally taken as b_1 (a very conservative figure).

It is, however, generally found that if d is made sufficient to keep the punching shear within the permissible limits then it is more than sufficient to resist the applied bending moment without the use of compression reinforcement.

The necessary area of tension reinforcement can be calculated in the usual way. There exists some disagreement as to the length of base over which the tension reinforcement should be spread. Some text books limit it to a width equal to the size of the column plus twice the effective depth of the base (approximately $YY_1 = b + 2d$, Figure 2)

whilst others say that this limit is too small and suggest a total increase by the distance from Y to the edge of the base, this increase being divided equally about the centre: thus in the second case the width would be $(b + 2d) + \left(\frac{B-b-2d}{2}\right)$. Neither point of view has any mathematical theory or experimental results on which to base their statement.

The only criticism against this method which the Author wishes to make at this stage is to point out that X-X is only the critical section for bending moment in the case where the column is monolithic with the base. In the case of a steel stancheon the critical section for bending moment is not XX but CC (this fact is proved by the failures which occurred in the tests described in Chapter VIII). Assuming a uniform pressure distribution and dividing the base into four equal parts as before the bending moment on CC is equal to $\frac{W}{12}(B-b)$. Since this formula is much simpler than the one given on page 19 it is suggested that it might be adopted in the case of monolithic structures and insisted upon in the case of a steel stancheon resting on a concrete base.

Shear.

Some text books give as one of the necessary qualities of a column base that the shear across the sections YY (or Y_1Y_1) must not exceed the permissible shear in plain concrete. This is akin to the requirements

of Clause 810 of the Recommendations applying to Flat slab floors. The total shear along this section is equal to $p(B^2 - YY_1^2)$: the depth is d_1, hence if s is the permissible shear, then

$$4s \times YY_1 \times d1 \leq p(B^2 - YY_1^2) \quad (3)$$

Although Taylor and Thompson states that this fact has been proved experimentally the Author has not been able to find records of such experiments

It is of interest to note that this requirement is not mentioned in the L.C.C. Regulations.

Punching Shear.

It has already been stated that the unit punching shear across XX is equal to $\frac{W}{4bd}(1 - \frac{b^2}{B^2})$. If the base is being designed to suit the L.C.C. regulations then this shear must be less than twice the permissible shear in plain concrete, and in almost every text book this requirement is mentioned as necessary in the design of a column base. It is generally found that this factor fixes d, the depth of the base. The Author's criticisms against designing to this requirement are mentioned later in Chapter X, but it is proposed here to give some criticisms against a method which is sometimes used to reinforce a base against punching shear.

It has already been stated that the punching shear must not exceed twice the permissible shear and in some offices if the punching shear exceeds the permissible shear then reinforcement is put in to resist this excess shear. The method of calculating the necessary area is as follows:

Let sp = punching shear stress = $\frac{W}{4bd}\left(1-\frac{h^2}{B^2}\right)$

Let s = permissible shear stress in plain concrete.

Then total excess shear = 4bd(sp-s).

Shear along each column face = bd(sp-s)

Area of reinforcement required = $\frac{bd(sp-s)}{t}$

where t = safe tensile stress in reinforcement.

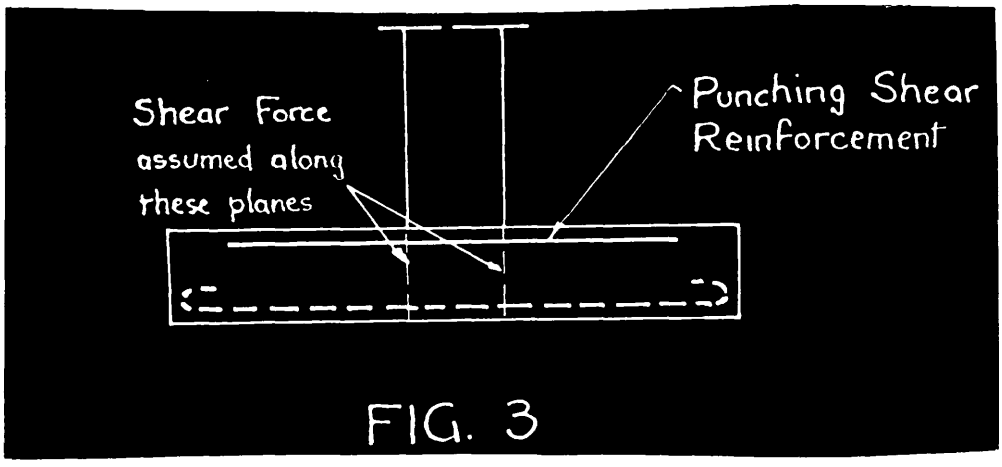


FIG. 3

The necessary reinforcement is then placed in the top of the slab as shown in Figure 3. Although the Author has not seen this practice recommended by any text book he has seen in employed by designers and in at least one recently-erected large reinforced concrete frame building almost every column base was reinforced in this way. In his

opinion it is incorrect and unnecessary because:

1. The stress in plain shear should be calculated on the plane YY (see page 18) since the load from the column will spread at 45 degrees; it is generally found that if sp calculated along XX is less than 2s, then the shear stress calculated along YY is less than s.

2. Since the force on the bars is assumed to be purely shear (see Figure 3) the stress used to calculate the area should not be the tensile stress in steel but the shear stress in steel which is much less (12000 lb. per square inch in shear as compared with 18000 lb. per square inch tension).

3. The load taken by the bars cannot be spread to the surrounding concrete without either (a) exceeding the permissible crushing stress in it or (b) introducing appreciable additional stress in the bars.

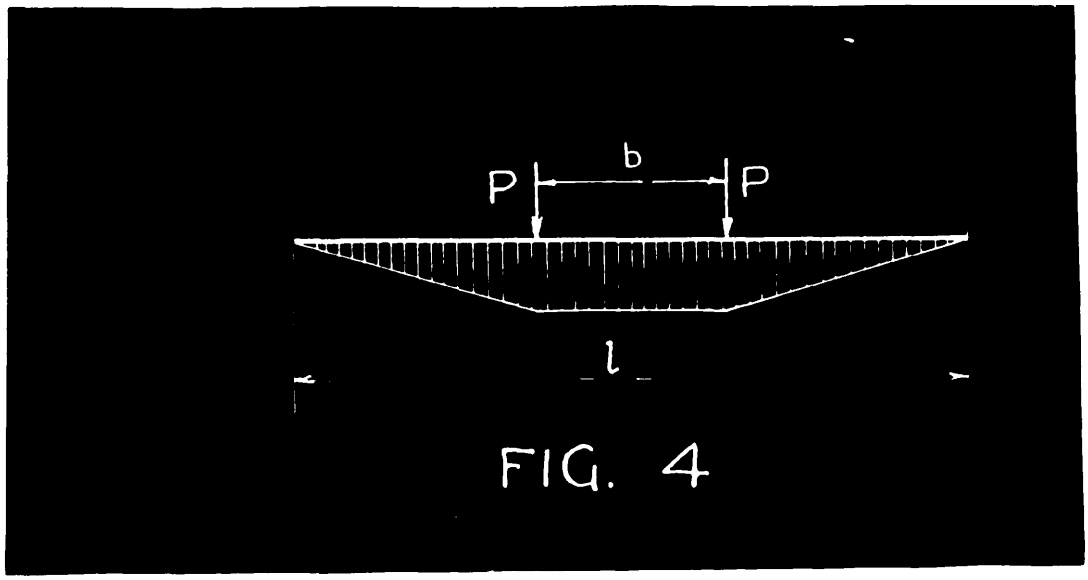


FIG. 4

Figure 4 shows diagrammatically the forces coming on any bar. The loads P represent the punching shears which are resisted by a distributed load acting as shown. This distributed load will be greatest in the immediate neighbourhood of P but it is assumed that the distribution is as shown.

Taking a steel stress = 18000 lb. per square inch and a safe working stress in compression in the concrete as 600 lb. per square inch then l = length over which load must be spread is calculated as follows:

$$2P = 2 \times 18000 \times \frac{\pi}{4} \times d^2$$ (where d = diameter of bar).

Upward load = 600 x d x l.

Therefore $l = 60 \times \frac{\pi}{4} \times d = 47d$ (approximately)

Of this length an amount = b is working at the full 600 lb. per square inch in stress whilst the remainder is assumed working at a stress which varies from zero at the ends to 600 lb. per square inch at P. Hence the total length of embedment must be equal to 2l - b, i.e. 94d-b. In practice the length of the bar is usually made 3.5b and this gives as a critierion that the diameter of bar must be not greater than $\frac{b}{20}$.

The length of bar required to keep the local crushing stresses within the safe lengths having now been found it is of interest to calculate the stress induced in the bar by the load distribution shown in Figure 4.

The bending moment under the load

$$= \frac{600}{2} \times d \times \frac{(l-b)^2}{3}$$

$$= 100d\,(l-b)^2$$

But $l = 47d$ and $b = 20d$ (in general practice).

Hence $M = 70{,}000\ d^3$.

The maximum stress produced by this moment

$$= \frac{M \times 64d}{2 \times \pi \times d^4}$$

$$= \frac{70{,}000 \times 32}{\pi}$$

= 710,000 lb. per square inch.

This figure is very much more than can be taken on a mild steel bar which proves the statement made on page 23 section 3, that the shear reinforcement thus provided is useless because it fails due to the local crushing of the concrete or the excess bending stresses introduced.

Part II - Mathematical Method of Design.

In the design of a column base given in Part I it was assumed that the ground pressure acting on the base was uniformly distributed. Experiments have shown that this is not true and a theory for the design of beams on elastic foundations has been developed on the assumption that the pressure at a given point is proportional to the deflection at that point. Thus if y is the deflection at that point the reactive pressure at that point is ky where k = elastic modulus of the soil.

Figure 5 shows a square plate of elastic material carrying a central point load on to an elastic foundation. At any point x, z; the equations for the loading are as follows:-

$$EI \frac{d^4y}{dx^4} = -pdz = -ky\ dz.$$

and $$EI \frac{d^4y}{dz^4} = -ky\ dx.$$

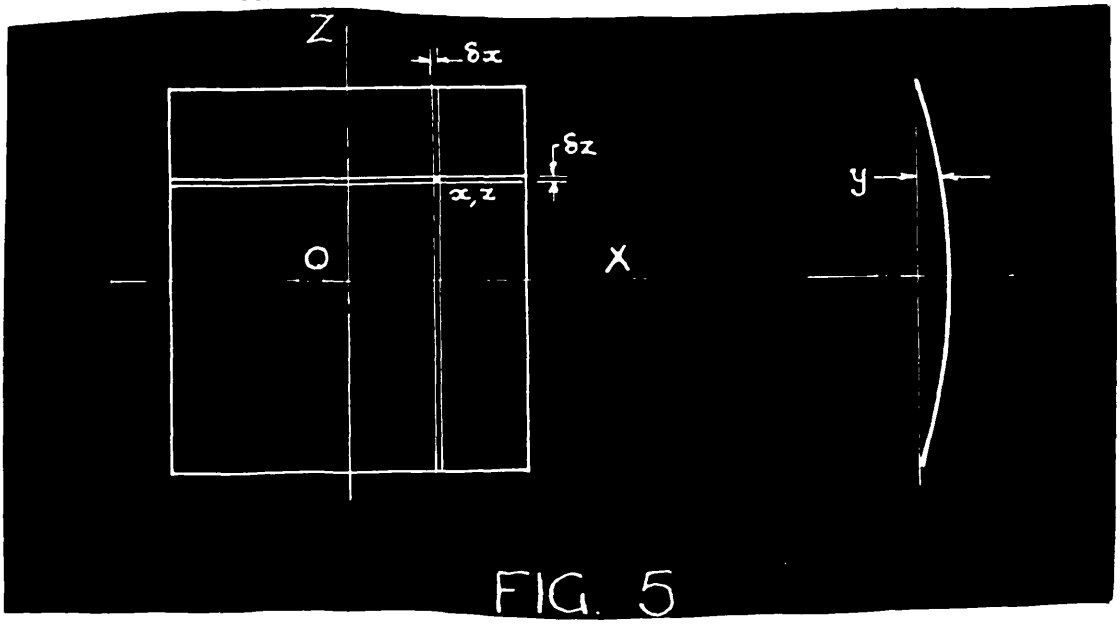


FIG. 5

Unfortunately such equations are not soluble therefore a true mathematical theory for the design of a square column base is not possible.

If, however, the base instead of being square is rectangular with a finite dimension in the direction z and infinitely long in the direction x then the equations become

$$EI \frac{d^4y}{dx^4} = - \frac{dQ}{dx}$$

where Q is the shear force due to the upward pressure and can be shown to be equal to ky

Hence $EI \frac{d^4y}{dx^4} = - ky$

If $\sqrt[4]{\frac{K}{EI}} = \beta$

Then the solution of this equation is shown by Timoshenko and Lessells (4) to be

$$y = C e^{-\beta x} (\cos \beta x + \sin \beta x) \quad \ldots\ldots\ldots (1)$$

where $C = \frac{P}{8\beta^3 EI}$ (P = total load on base)

Hence $M = - \frac{P}{4\beta} e^{-\beta x} (\cos \beta x + \sin \beta x) \quad \ldots\ldots\ldots\ldots (2)$

Under the load, x is zero and M is a maximum thus giving

$$M_{max} = \frac{P}{4\beta} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2A)$$

If the curve given by equation (1) is plotted y is found to be negative for values of βx greater than 2.36. Gray (5) has suggested that this is an impossibility and given a method of design based on his suggestion. In the Author's opinion the negative value for y is a practical

possibility and in his mind there is little of value in the design method which Gray has put forward because; (1) a study of it reveals the fact that it is not consistent, and (2) if this were not so it could only be applied to thin bases where the length is about four or five times the width, a type of base which is not common in practice.

A method of solution of the equations given on page 26 has been developed by Ritz (12) and this was used by Vint and Elgood (6) in their work on the deflection of a boom on an elastic foundation. Ritters method is very complicated and cannot be easily applied but this fact alone should not disqualify it from use in the case of reinforced concrete foundations. The method is definitely for use in the case of thin plates and the degree of similarity between a reinforced concrete column base and a thin plate is so questionable that Ritters' method has not been applied to the work in this thesis.

In the study of the stability of foundations against shear failure in the soil (see Figure 6), the mathematics is similar to that for the beam of infinite length on an elastic foundation as given on page 27 It has been found, however, that the shear resistance of

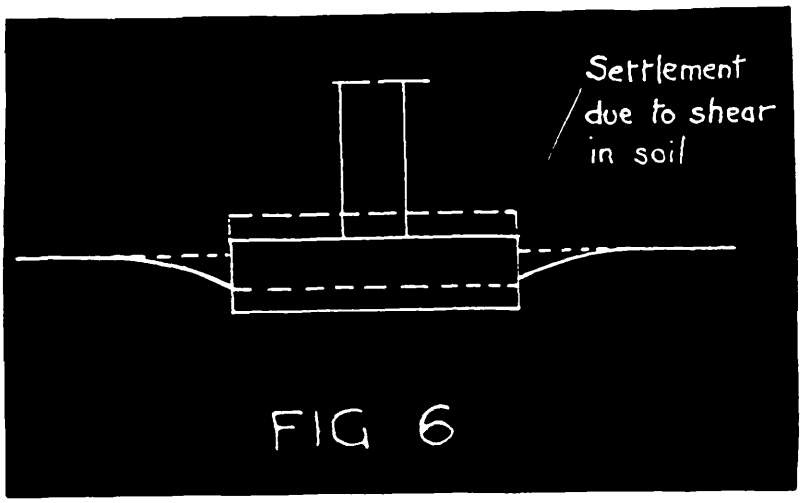


FIG 6

a square or circular base is double that of a strip footing of infinite length and of width equal to the side of the square base or the radius of the circular. It would be of interest to know whether a similar relationship holds in the case of a square column base on an elastic foundation. If it does then equation (2A) page 27 becomes in this case:

$$M_{max} = \frac{P}{8\beta}$$

This formula is applied to the bases under tests and notes are given on it in Chapter VIII.

Analysis due to Westergaard(9).

Westergaard analysed slabs by working on the assumption that the pressure at any point is proportioned to the deflection at that point. He applies his analysis to three cases which are commonly met with in the design of pavement slabs:

1. A load at the corner of the slab.
2. A load in the centre of a slab of infinite dimensions.
3. A load at the edge of a slab of infinite dimensions.

The first and third cases are not of interest in this work but the second is being studied since it would be analagous to the column base if the dimensions were finite. For this second case Westergaard states that the tension stress, σ_i; immediately below the point of application of the load is given from the formula,

$$\sigma_i = 0.3162 \frac{P}{h^2} \left[\log_{10}(h^3) - 4 \log_{10}\left(\sqrt{1.6a^2 + h^2} - 0.675h \right) - \log_{10} K + 6.478 \right]$$

where P = wheel load

h = thickness of base.

K = subgrade modulus.

A = diameter of circle over which the load P is assumed to act.

This formula is developed by a special theory which Westergaard applies to bases carrying a concentrated load. In it he makes certain assumptions to allow for the fact that under such conditions of loading, plane sections do not remain plane. The stress developed when computed by the ordinary theory is

$$\sigma_i = \frac{3(1+\mu)P}{2\pi h^2} \left(\log_e \frac{l}{a} + 0.6159 \right)$$

where μ = Poisson's ratio.

and $l = \sqrt[4]{\frac{E h^3}{12(1-\mu^2)K}}$.

Westergaard gives a table of the values of for a load of 10,000 lbs. with varying values of h, K and a.

It is not proposed to reproduce the whole of the table but the values for a slab 4 inches thick (i.e. those under test) are given below.

Maximum stress in 4 in. thick slab (Westergaard).

K	0	2 ins.	4 ins.	6 ins.	8 ins.
50	1231	1058	848	693	588
100	1172	998	788	634	528
200	1112	939	729	574	469

The application of Westergaard's theory to isolated column bases can be justly criticised. The theory was developed for use in the design of road slabs where the ratio between the width of the slab and that of the wheel is always very large (say about 40 - 50 for the average case). It is, however, of interest to apply the theory to the design of column bases and this has been done in Chapter VIII.

Westergaard also develops a theory for the deflection of slabs under the loading conditions already mentioned and gives for the second case that the deflection, z_i, under the load is

$$z_i = \frac{P}{8\ kl^2}$$

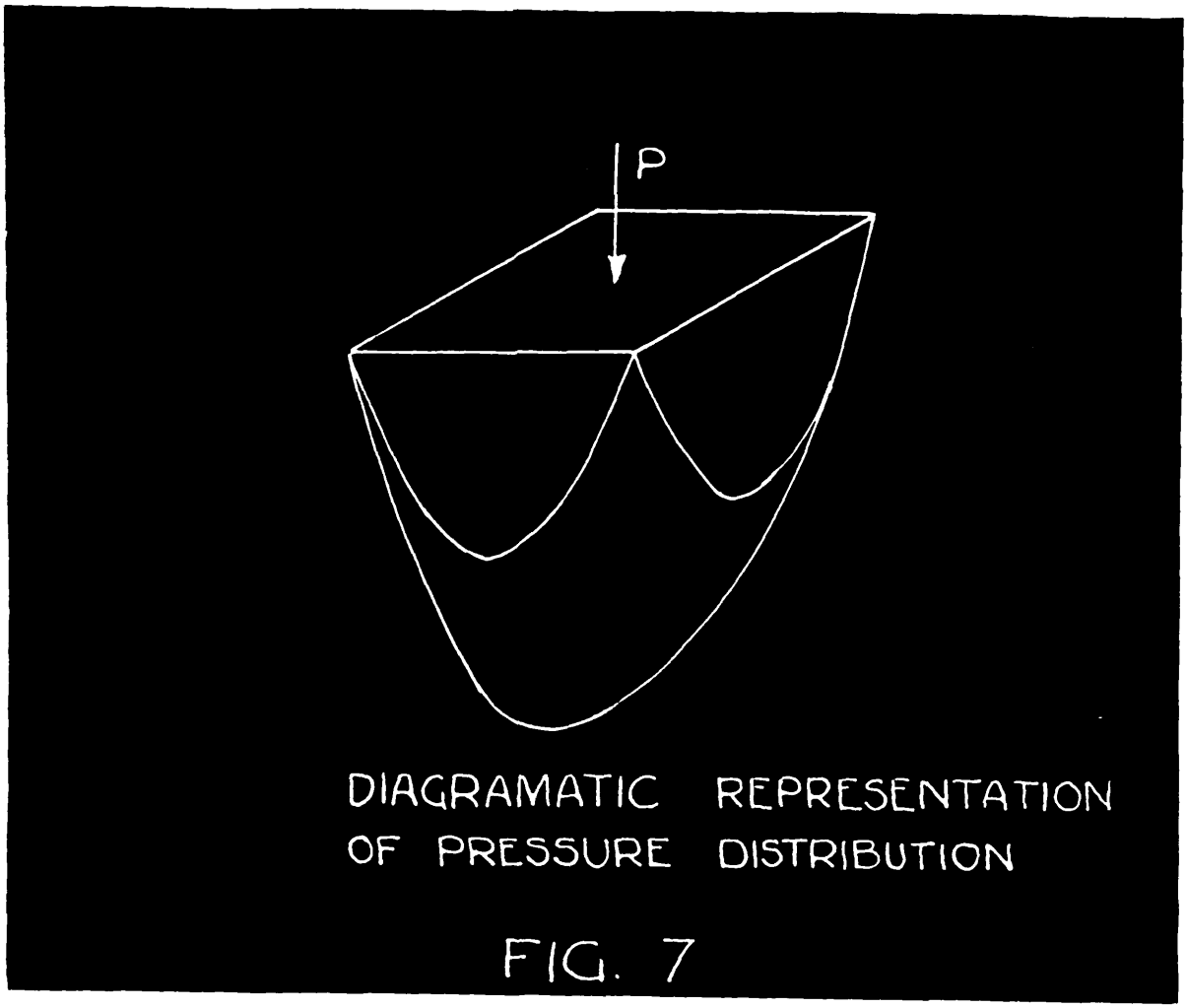
where l has the value given on page 30

Part III - The Distribution of pressure under a column base.

In the general accepted method of design given in Part I of this chapter the distribution of pressure under the base is assumed to be uniform: in the mathematical method given in Part II the pressure is assumed to be proportional to the deflection at any given point.

It has been shown (see Chapter III) that the distribution of pressure in a sand foundation is approximately parabolic in form reaching a maximum at the centre and being zero at the corners. Unfortunately no experiments have been performed to show the magnitude of the pressure at the mid points of the edges of a square base. If it is assumed however that the distribution of pressure is parabolic with a maximum at the centre and having zero value at the four corners then the pressure distribution over the base forms the surface of a paraboloid as shown in Figure 7, a section on the planes $x = o$ and $y = o$, and a section through the plane $x = \pm y$ are as shown in Figure 8

The volume of the paraboloidal solid thus formed is equal to the load coming on the foundation (the weight of the base itself being neglected.



DIAGRAMATIC REPRESENTATION OF PRESSURE DISTRIBUTION

FIG. 7

The volume of this irregular solid is calculated as follows:

Let the equation to be paraboloid be

$x^2 + y^2 = 2az$.

i.e. origin is at the vertex and let h be its height

then $2b^2 = 2ah$ so that $a = \frac{b^2}{h}$

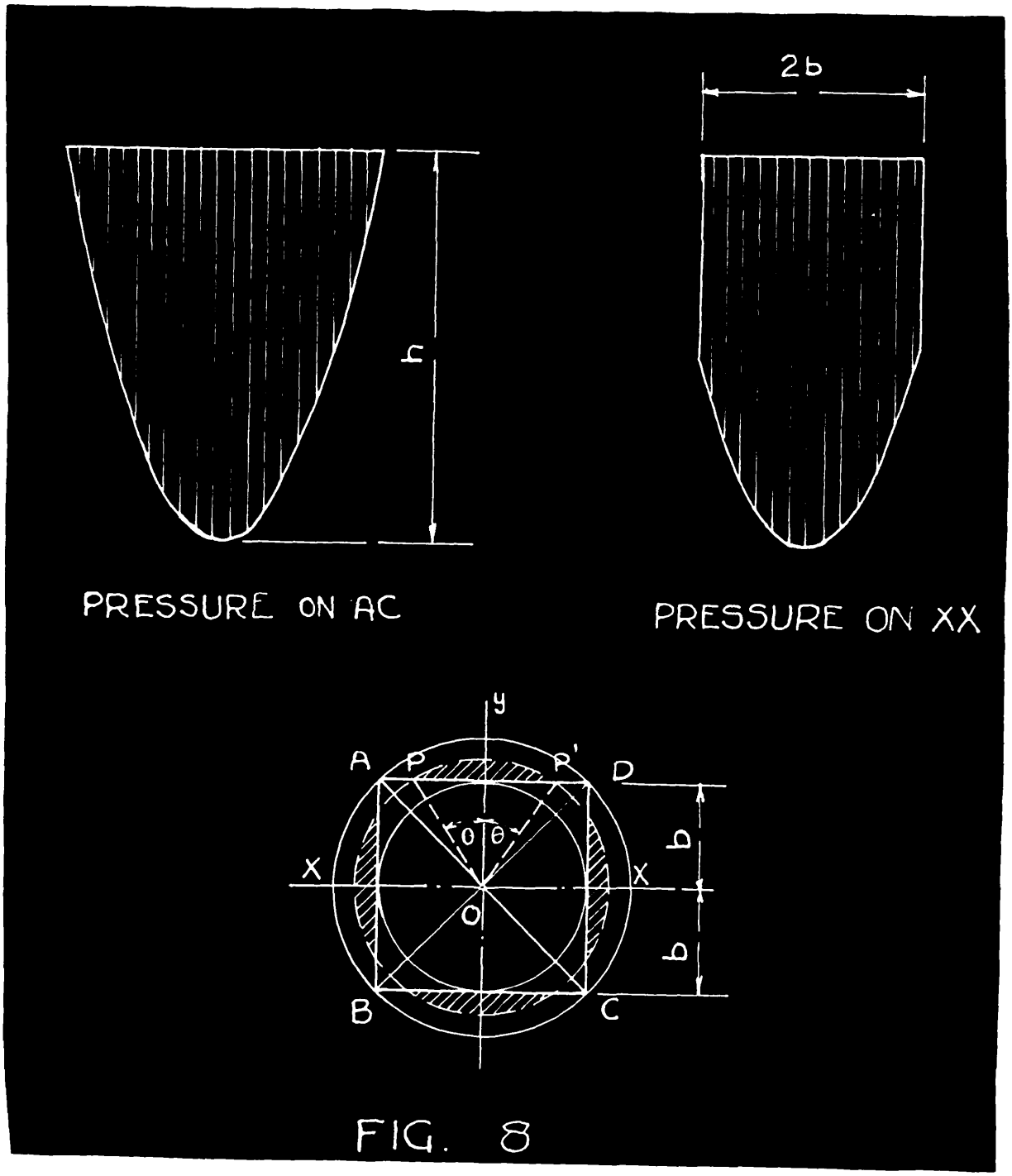


FIG. 8

Any plane z = a constant will cut the paraboloid in a circle and the paraboloid is entire for values of z less than h_o

where $b^2 = 2ah$ o or h o $= \frac{h}{2}$

The radius of the circle at any value of z is given by

$r^2 = 2az.$

Therefore $2rdr = 2adz.$

The volume of the complete paraboloid, V is given by

$$V = \int_0^h \pi r^2 dz = \pi \int_0^h \frac{2b^2}{h} . \; Z \; dz.$$

$$= \pi \frac{b^2}{h} \left[Z^2 \right]_0^h$$

$$= \pi b^2 h$$

The volume of the solid formed by the pressure distribution is equal to this volume less that of the four solid pieces removed. For $h_o < Z < h$ the part shown shaded PP^1 is cut away. Let PP^1 subtend any angle 2θ at the centre, O.

Then A, the shaded area in each quadrant

$= r^2\theta - br \sin \theta$

and $r = b \sec \theta$

Therefore $A = b^2 (\theta \sec^2 \theta - \sin \theta \sec \theta)$

$= b^2 \sec^2\theta (\theta - \sin \theta \cos \theta)$

and $dz = \frac{1}{a} rdr = \frac{h}{b^2} b^2 \sin \theta \sec^3 \theta \, d\theta$

Then V_2 = the volume cut off from each quadrant

$$= b^2\int_0^{\frac{\pi}{4}} (\theta\sec^2\theta - \sin\theta\sec\theta)h \sin\theta\sec^3\theta \, d\theta$$

$$= b^2h\int_0^{\frac{\pi}{4}} (\theta \sin\theta \sec^5\theta - \sin^2\theta \sec^4\theta)d\theta$$

$$= b^2h\int_0^{\frac{\pi}{4}} \left(\theta\sin\theta\sec^5\theta + \frac{\sec^4\theta}{4} - \frac{\sec^4\theta}{4} - \sin^2\theta\sec^4\theta\right) d\theta$$

$$= b^2h\left[\frac{\theta\sec^4\theta}{4} - \frac{\tan^3\theta}{3}\right]_0^{\frac{\pi}{4}} - b^2h\int_0^{\frac{\pi}{4}} \frac{\sec^4\theta}{4}(\sin^2\theta+\cos^2\theta)d\theta$$

$$= b^2h\left(\frac{\pi}{4} - \frac{1}{3}\right) - \frac{b^2h}{4}\left[\frac{\tan^3\theta}{3} + \tan\theta\right]_0^{\frac{\pi}{4}}$$

$$= b^2h\left(\frac{\pi}{4} - \frac{1}{3} - \frac{1}{12} - \frac{1}{4}\right)$$

$$= b^2h\left(\frac{\pi}{4} - \frac{2}{3}\right)$$

$$= .1187\ b\ h$$

The volume of one quarter of the complete paraboloid $= \frac{\pi}{4} b^2h$

Therefore volume of one quarter of the "pressure" paraboloid $= (.7854 - .1187)\ b^2h$

$= .6667\ b^2h$

This volume must be equal to one quarter of the load carried by the base.

Therefore $.6667\ b^2h = \frac{W}{4}$

Hence h, the height of the paraboloid, or, the maximum pressure under the base can be determined.

Maximum pressure, $h = \frac{3}{8}\frac{W}{b^2}$ (approximately).

If the pressure is uniformly distributed then the average pressure, which is also equal to the maximum pressure is $\frac{W}{4b^2}$. Therefore the maximum pressure with the parabolic distribution is approximately 50 per cent greater than when the distribution is assumed uniform.

For calculating the bending moment on the base it can be assumed that the load is divided into four equal parts which produce equal moments about the x and y axes. The bending moment on the x axis is equal to one quarter of the load multiplied by the distance of the centre of gravity of the solid A O D from the axis.

The distance $\bar{x}$ of the centre of gravity is calculated as follows:

Moment of area for any sector

$$= \int_{-\theta}^{\theta} \frac{r^2}{2} \delta\alpha \frac{2}{3} \cos\alpha = \frac{2}{3}\, {}^3\sin\theta$$

for $\theta = \frac{\pi}{4}$ this becomes $\frac{2}{3}\frac{r^3}{\sqrt{2}} = \frac{\sqrt{2}}{3} r^3$

For any segment PSP^1, moment of area

$$= \int_{-\theta}^{\theta} \frac{1}{3} r^3 \cos\alpha \, d\alpha - \frac{2}{3} r^3 \cos^2\theta \sin\theta$$

$$= \frac{2}{3} r^2 \sin\theta - \frac{2}{3} r^3 \cos^2\theta \sin\theta = \frac{2}{3} r^3 \sin^3\theta$$

Moment of area for volume required

$$= \int_0^h \sqrt{\frac{2}{3}}\, r^3 dz - \int_{h_0}^{h} \frac{2}{3}\, r^3 \sin^3\theta \, dz$$

$$= \sqrt{\frac{2}{3}} \int_0^h (2az)^{\frac{3}{2}} dz - \int_0^{\frac{\pi}{4}} \frac{2}{3}\, b^3 \sec^3\theta \sin^3\theta \cdot h \sin\theta \sec^3\theta \, d\theta$$

$$= \sqrt{\frac{2}{3}} \left(\frac{2b^2}{h}\right)^{\frac{3}{2}} \cdot h^{\frac{5}{2}} \quad \frac{2}{5} - \frac{2}{3} h b^3 \left[\frac{\tan^5\theta}{5}\right]_0^{\frac{\pi}{4}}$$

$$= b^3 h \left(\frac{8}{15} - \frac{2}{15}\right)$$

$$= \frac{2}{5} b^3 h$$

Therefore $\frac{-}{x} = \frac{.4\ b^3 h}{.6667\ b^2 h} = .6b.$

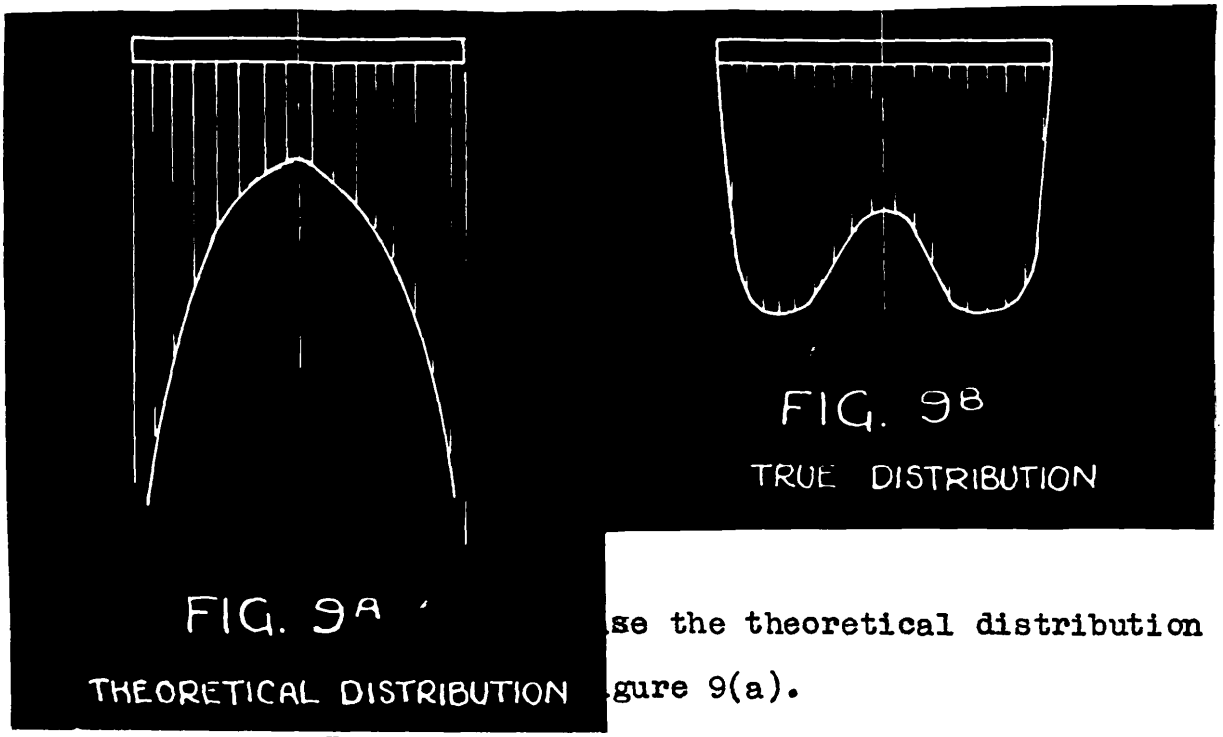
Therefore bending moment on base = $\frac{W}{4}$ x .6b.

If the same notation is used as for the formula given on page 19 and allowance is made for the reduction in bending moment due to the spread of the load over the base plate of the stancheon then the expression for the bending moment is $M = \frac{W}{4}\left(.3B - \frac{b}{3}\right)$ which is slightly less than the value obtained when the pressure is assumed uniformly distributed.

The pressure distribution under flexible square base resting on a sand bed has been investigated throughly because a sand bed is used in the tests described in this thesis. There are two other cases of pressure distribution which can be briefly noticed:

(a) Flexible base on clay bed,

(b) Rigid base on any bed.



se the theoretical distribution of
gure 9(a).

Practically, however, infinite pressure at the edges is an impossibility and the distribution of pressure is as shown in Figure 9(b).

In the case where the footing is rigid the distribution is uniform.

CHAPTER III.

PREVIOUS INVESTIGATIONS.

CHAPTER III.

PREVIOUS INVESTIGATIONS.

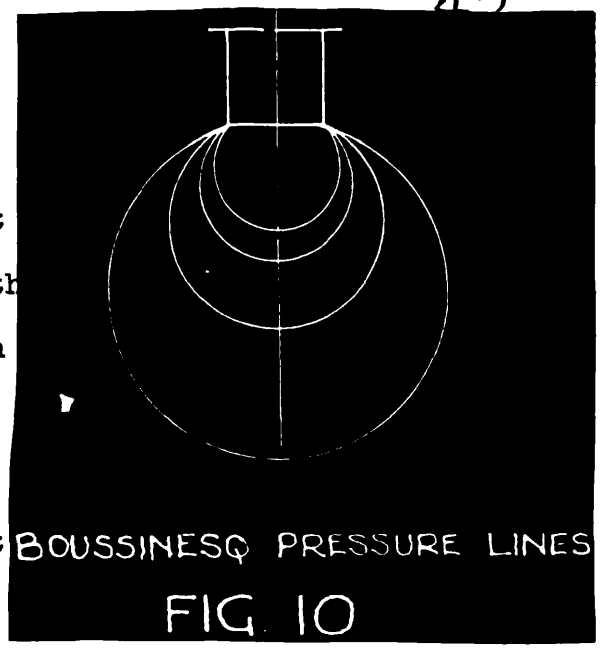
It has already been mentioned that so far as the Author is aware no previous experimental investigations have been carried out on the strength of thin isolated column bases in reinforced concrete. Investigations have, however, been carried out on subjects which have a bearing on the problem and it is proposed in this chapter to give a brief summary of these investigations.

The three subjects on which work has been carried out are: (1) the distribution of pressure under a foundation, (2) the problem of a thin elastic plate supported on an elastic foundation, and(3) the strength of reinforced concrete slabs.

Much work has been done on the first and third of these divisions and only that work which bears on the problem with which this thesis is concerned is being given.

Part I - Investigations on the distribution of pressure

The earliest investigator into this problem was Boussinesq (7) who published his work toward the close of last century. His work is mainly concerned with the distribution of the load at different distances from the footing and enables the calculation of lines of pressure to be made as shown in Figure 10. His theories find



BOUSSINESQ PRESSURE LINES

FIG. 10

useful application in the study of soil mechanics but they are not so useful in th problem under consideration as he did not investigate thoroughly the pressure distribution at the contact surface between the foundation and the soil. In considering the strength of a column base it is this distribution of pressure which affects the bending moments on the base.

More recent investigations into the problem tackled by Boussinesq have been carried out by Burmister (13), Krynine (14) and Cummings (15).

Burmister gives a graphical method for determining the pressure at any depth below a loaded point on the surface of the soil. His method is specially useful in calculating the pressure contours in the ground below a building, the foundations of which consist of a number of isolated column bases. In his paper he gives an example of the pressure contours and the distribution of pressure beneath a building with 24 column bases.

Krynine gives a graphical method, called the "reduced area" method of determining the pressures under irregularly shaped foundations. In addition he gives a

modified form of the Boussinesq equation allowing for differences in the soil. The empiral formula he suggests is

$$p_z = p\left(\frac{n}{2} - 1\right)\left(1 - c^2\right)^{\frac{n}{2} - 2}$$

where p_z is the vertical pressure at any distance c from the centre of the disc (where the intensity of pressure = p) and n is the concentration factor of the soil.

It is the introduction of "n" which makes Krynine's formula differ from Boussinesq's. The concentration factor of the soil varies from 3 in the case of clays to 6 in the case of sands. The substitution of these values in the formula gives pressure distribution curves for these two materials similar to those shown in Figures 9A and 11 respectively.

Cummings (15) also mentions the corrections which must be applied to Boussinesq's equation to allow for the different types of soils and also the distribution of pressure under the footing. He also analyses in his paper the various experiments which have been conducted on the pressure at varying depths below a foundation. These experiments are not given in this thesis as the distribution of pressure with which it is mainly concerned is not the vertical but the horizontal.

Tests by Giesecke, Badgett and Eddy (8).

These tests are of great interest as they were carried out on an actual structure. Beneath a reinforced concrete footing 19 feet square and 2 ft. 6 ins. thick a series of Goldbeck soil-pressure cells were placed. By means of these cells it is possible to measure the earth pressure at the point at which they are situated. The cells were placed at intervals along the diagonal of the footing and readings of the pressures were taken both during and after construction. The results showed that the distribution of pressure was approximately parabolic reaching a maximum at the centre and being zero at the corners. The criticism to be levelled against these tests is that too few cells were used and readings only taken on a diagonal. It is assumed in the calculations given in Chapter 2, Part III, that the intensity of pressure at the mid points of the edges is one half of the maximum pressure. It would have given useful information if a cell had been inserted at these points in the tests.

Tests by Dr Oscar Faber (10).

Faber carried out tests in 1929 on the distribution of pressure on the surface immediately below a foundation block. His experiments were on two types of soil (a) sand and (b) clay. The device for applying the load to the surface of the soil was fitted with extensometers so that the pressure on concentric rings of the surface could be measured. In brief, he showed:

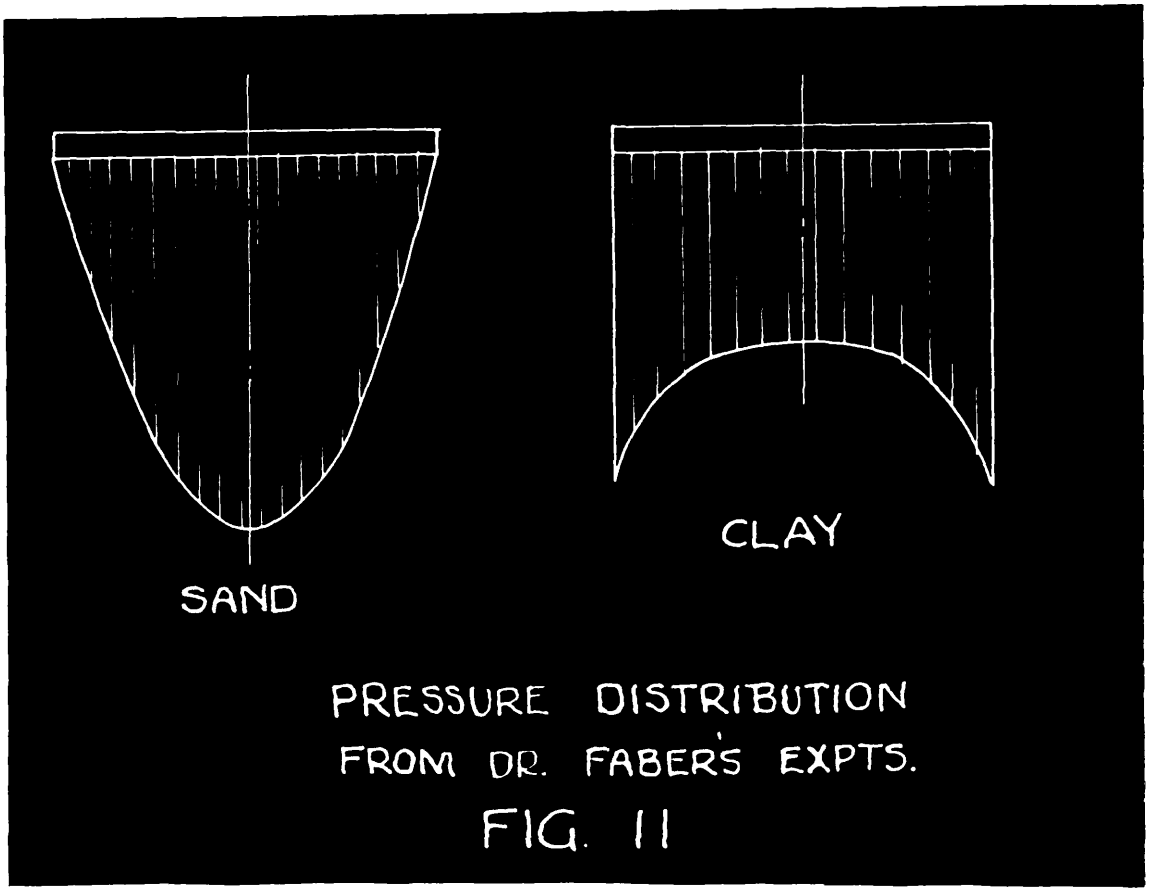
(1) that for sands, where the foundation is near the free surface the distribution of pressure is approximately parabolic reaching a maximum at the centre and being zero at the edges, whilst in cases where the foundation is some distance below the surface the distribution is between parabolic and uniform.

(2) that for clays the distribution (with or without surcharge) approximates to one giving a maximum at the edge and a minimum at the centre, the mean pressure being two thirds of the edge pressure and the distribution between the two being a curve which approximates to an ellipse.

These two types of distribution are shown in Figure 11, and it should be noted that for sands the maximum pressure is twice the mean pressure.

The experiments carried out agreed quite well with these curves.

In the paper describing these experiments Faber shows up the fallacies of the Rankine formula for the depth of foundations. One criticism against these tests is that the circular base tested is seldom, if ever, used in practice: tests on a square base would have been much more practical.



Part II - Tests on plates on elastic foundations.

Tests of this type are useful in that they provide an experimental means for checking up the mathematical solutions to the problem which have been given bu such investigators as Timoshenko and Westergaard. There must, however, be a big difference between thebehaviour under test, say, of a 4 inch thick concrete slab and a steel plate $\frac{1}{4}$ inch thick, consequent ly there is some doubt as to what extent the results of the tests on steel plates can be applied to concrete column bases.

Tests by Glenn Murphy (11).

The object of this paper was to present a method of determining the stresses and deflections in rectangular plates on elastic foundations with special reference to the case where the plates deflect free from the foundation when loaded. The first part gives a mathematical analysis of the problem for this case. The analysis is very complicated but it is of interest to note that strain energy methods are used in attacking the problem. In an appendix tables are given which can be used in the solution of any particular problem but even with these the method is still far too lengthy to be applied to the design of a column base. The experiments carried out by Murphy were on a $\frac{1}{4}$ inch thick steel plate which was placed on a rubber bed and loaded with the three conditions of loading used by Westergaard (9) in his

analysis. The deflections and stresses were measured at many points in the plate. The use of rubber as an "elastic" bed is to be criticised on the grounds that rubber is not a perfectly elastic material but possesses plastic as well as elastic properties. Rubber has also the disadvantage that it is not very consistent in its behaviour under test.

The tests showed that in a centrally loaded square plate the deflection contours are as in Figure 12. This shows that at the corners of the plate there was an uplift and that the plate only remained in contact with the soil over an area approximately equal to $\frac{\pi}{4}B^2$ where B = width of the plate. If column bases behave in this way in practice then it means that if the pressure distribution is parabolic the maximum pressure, i.e. under the load, is equal to $\frac{8}{\pi} \cdot \frac{W}{B^2} = 2.5$ times the average pressure. An increase as great as this means that in most designs the factor of safety is probably reduced to unity.

The experiments verified the theory which Murphy gave to a high degree of accuracy. In some cases the calculated stresses agreed reasonably well with those calculated by Westergaard's formula. In the case of a centrally loaded slab the difference between the maximum measured stress and that calculated from Westergaards formula is only 5 per cent.

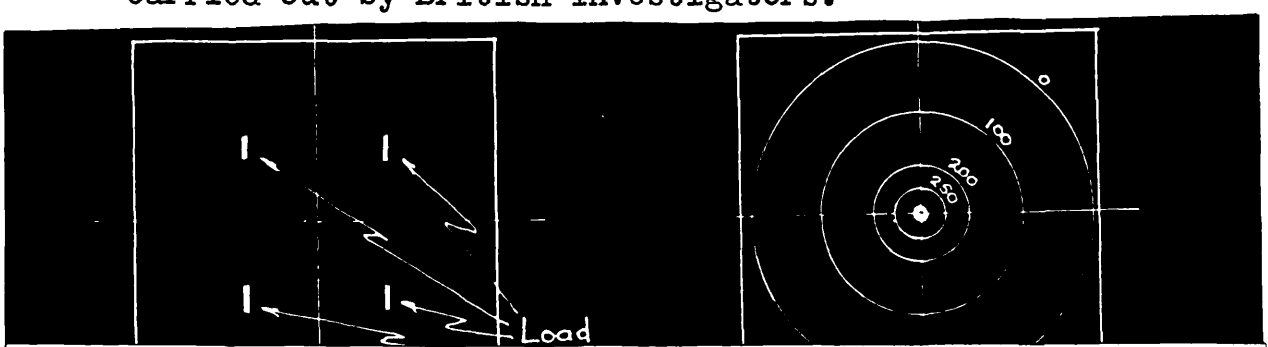
Tests by Vint and Elgood (6).

These tests although not applicable to the design of a reinforced concrete column base are of interest in the design of foundations since they were carried out to determine the behaviour of a bloom plate to which a load is transmitted by means of a stancheon.

The mathematical section of the paper was developed on Ritz's method. The experiments performed to verify the mathematical analysis were carried out on a steel plate 10 inches square and 0.2345 inches thick supported on an elastic foundation consisting of springs. The load was applied at four points as shown in Figure 13 and the measured deflections agreed to within 3 per cent of the calculated.

The elastic base used for these tests is to be commended as representing a definite advance on the rubber base used by Murphy (II).

The tests were carried out at Bristol University and are the only tests on these lines which have been carried out by British investigators.



Part III - Investigations into the strength of Reinforced Concrete Slabs.

It is no exaggeration to say that every technical college and research laboratory has at some time tested reinforced concrete slabs and beams in order to study their behaviour under load. So many observations have been carried out that a thesis could be written on them.

Most experimenters have confined their work to the study of (a) the distribution of stress across a section, (b) the effects of creep, plastic yield and shrinkage, (c) the behaviour of a reinforced concrete section at failure. These investigations have shown that the value of the failing load taken by a reinforced concrete section is generally different from that obtained by calculation using the straight-line, no-tension theory. This fact has led to the production of new theories of design. Reference to these theories is not being given in this Chapter but it is mentioned in the Chapter discussing the results of the tests.

The only tests on reinforced concrete slabs which are described in this Chapter are those in which such slabs are used as isolated column bases.

Although Taylor, Thompson and Smulski (3) of the American, and Manning (16) of the English, text books both state that such tests have been carried out, neither

of them gives any references in support of their statements. A careful study of research bulletins issued during the last 10 years shows that only one series of such tests has been carried out. This series of tests was carried out by P. Lebelle (17).

Tests by Lebelle.

Lebelle gives methods of design for and describes tests carried out on the following types of foundations,

1. Wall footings.
2. Isolated column bases square or rectangular in plan carrying square or rectangular columns.
3. Isolated column bases circular in plan.
4. Isolated column bases octagonal in plan.

The method of design given is based on two assumptions, first, that the ground pressure below the base is uniform and second, that the base is of such a thickness that,

$$d \geq \frac{B-b}{4}$$ (see Figure 2).

It is pointed out in the previous chapter that the first assumption is not true except where the foundation is perfectly rigid: it certainly does not apply to the thin sections tested by the Author but as Lebelle's tests were carried out on thicker sections (in accordance with the assumption given) he has some justification for using it.

The method of design given by Lebelle generally follows the standard method but an interesting feature is the introduction of the effect of the friction between the underside of the base and the subsoil on the strength. He points out that in most cases the friction considerably reduces the stress in the reinforcement.

In the case of isolated column bases which are square in plan three methods of reinforcing are discussed and the design worked out for each case. The methods are:-

1. Reinforcement parallel to the sides, spaced equally across the whole width of the base.
2. Reinforcement parallel to the diagonals equally spaced along them.
3. Reinforcement parallel to the sides but bunched together under the column.

The secondcase whilst economical from the design standpoint has the disadvantage that every bar is of different length consequently it is not good from the constructional standpoint.

In his design Lebelle completely neglects the shearing forces: this is justifiable up to a point on the grounds that the base is so thick that the shearing forces are bound to be within reasonable limits.

In the actual tests carried out Lebelle tested five different types of square base. The tests were carried out in a 500 ton Amsler press, the column base being

supported on a rubber bed. The use of such a bed is criticised in Chapter I (page 13). It also has the disadvantage that the frictional forces between the bed and the soffit of the foundation slab tend to increase the stress in the reinforcement instead of decreasing it as is generally the case.

The tests were carried out after 21 days the concrete used having an ultimate compressive stress of 295 kilogrammes per square centimetre. (4180 lb. per square inch) and a tensile stress of 17.8 kilogrammes per square centimetre (252 lb. per square inch).

The bases failed by cracks at right angles to the direction of the reinforcement.

The main object of the tests seemed to be to compare the actual failing load with the safe working load calculated by Lebelle's method. The results gave a factor of safetyvarying between 3.10 and 4.10 when friction was taken into account whilst when friction was neglected the factor of safety varied from 1.90 to 2.75.

Whilst there is a certain similarity between Lebelle's tests and those described in this thesis the main differences are:

1. Lebelle tested thick bases as opposed to the thin sections of the Author.
2. The use of the rubber foundation instead of the sand

bed used in Chapter VII.

5. The small number of tests carried out by Lebelle.

CHAPTER IV.

EXPERIMENTAL DETAILS FOR MODEL MORTAR BASES.

CHAPTER IV.

EXPERIMENTAL DETAILS FOR MODEL MORTAR BASES.

It is intended in this chapter to give particulars of the method of manufacture and testing for the specimens in Class I (see page 11). The results of the tests are given in the following chapter whilst the conclusions from the experiments are given in Chapter 6.

Method of making the specimens.

The specimens were all 8in.x 8in. in plan, series 1 being 1in. nominal thickness and series 2, 1½ins. The specimens were made in sets of four from a cement - sand mortar which proportions were 1 : 3 by weight. "Ferrocrete" rapid-hardening portland cement and Stone Court sand were the materials used. The water-cement ratio was kept constant at 45% for all the specimens.

The forms used for making the slabs were very simple and are shown in Figure 14.

It is pointed out on page 12 that it was necessary to use Expanded metal as reinforcement for the specimens.

Particulars of the expanded metal used are as follows:

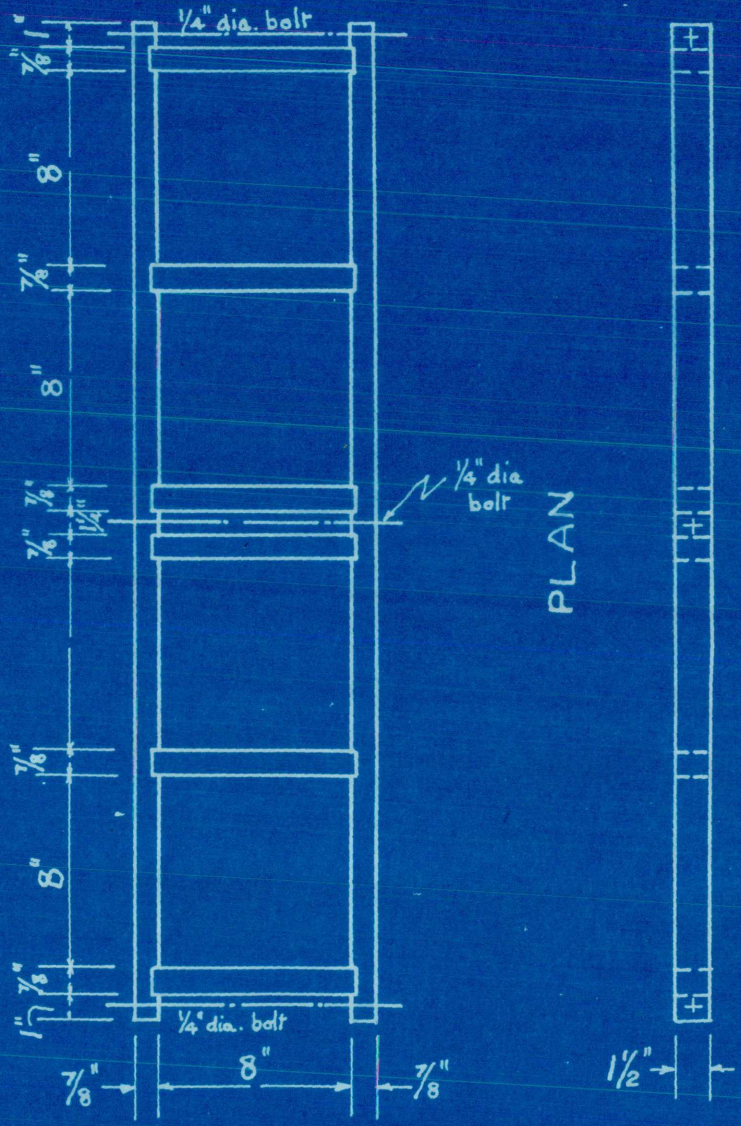
Thickness: No.22 gauge.

Cross section (short way of mesh) $\frac{3}{32}$in. at $\frac{7}{32}$" pitch.

Cross section (long way of mesh) $\frac{9}{32}$in. at 1½in. pitch.

Ultimate Stress = 80,000 lb. per sq.in.

FIG. 14



FORMS FOR MODEL MORTAR SLABS

SCALE : 1/8 F.S.

The sheets used as reinforcement in the bottom of the slab were 7½ins. square, those used in the top were 4ins. square. Where two or more sheets of reinforcement were used they were placed with the mesh in one sheet perpendicular to that in the adjacent sheet.

The sheets were given approximately ⅛in. cover both top and bottom.

The quantities of reinforcement used in the various types of specimens made are given in the table below.

Specimen Mark.	Thickness. (Actual).	Reinforcement in bottom.	Reinforcement in top.
1.0	1⅛"	-	-
1.1		1 layer	-
1.2		2 layers	-
1.3		1 layer	1 layer
1.4		2 layers	1 layer
2.0	1⅝"	-	-
2.1		1 layer	-
2.2		2 layers	-
2.3		2 layers	1 layer
2.4		4 layers	-
2.5		2 layers	2 layers.

The mortar was consolidated by ramming and the upper surface of the specimen finished off smooth.

The specimens were kept in the forms for 24 hours and then immersed in water until ready for test.

In order to determine the compressive and tensile stresses of the mortar used cubes and briquettes were also made. The same mix was used as for the slabs the specimens were stored in the same way.

The cubes were 2¾ins. and the briquettes were of the standard type given in the British Standard Specification for Portland Cement.

Method of testing specimens.

One of the objects of this series of tests was to determine the effect of different beds on the strength of the concrete slab.

The different kinds of bed used were (1) rubber, (2) sand, (3) wet clay.

In the case of the rubber bed 4 sheets of rubber each ¼in. thick were placed on top of each other on the bed of a 200 ton Riehlé test machine. For the other beds a stout timber box adequately reinforced with steel straps was placed on the bed of the machine, the box being filled with the appropriate material. The sand used for the sand bed was dry Stone Court sand: the clay was obtained from the excavation for the pit of the test machine described in Chapter 7. The clay was kept moist during the tests.

The stancheon used was a steel block 2in. sq. in cross section and 3ins. long. The centre lines being marked on each face enabled it to be placed centrally on the

slab. Between the steel stancheon and the mortar slab a layer of felt paper was placed to even out any surface irregularities.

The slab and stancheon were placed on the top of the bed, levelled up, and the load applied to the stancheon. The load was applied until the slab was tested to destruction.

The four slabs in each series were tested following each other and the average of the four results taken.

The set-up of the apparatus for this series of tests is shown very clearly on plate 1.

The specimens were in every case tested after 7 days.

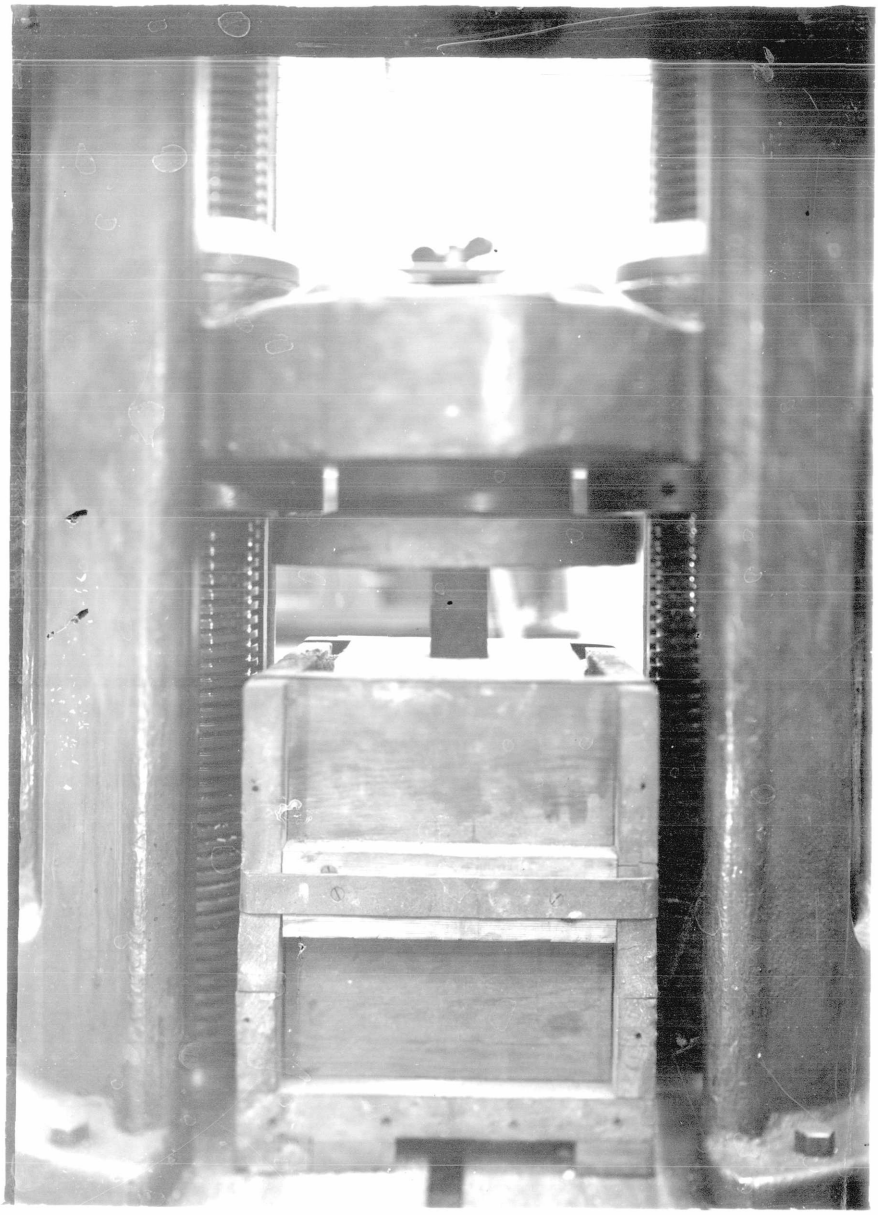


Plate 1 – General Arrangement for Model Tests

CHAPTER V.

TESTS ON MODEL MORTAR BASES.

CHAPTER V.

TESTS ON MODEL MORTAR BASES.

The tests carried out on the mortar specimens can be divided into three main groups (a) tests on cubes and tension specimens, (b) tests on $1\frac{1}{8}$ in. thick specimens, (c) tests on $1\frac{5}{8}$ in. thick specimens. The third group is the largest and most important, the second gives tests only on one type whilst the first although the smallest is necessary to give information about the strength of the mortar.

Tests on cubes and tension specimens.

Six cubes of side $2\frac{3}{4}$ ins. were made and the average crushing load was 32380 lb. This gives the compressive strength of the mortar as 4280 lb. per square inch.

Six briquettes of the size and shape recommended by the British Standard Specification were made and the average failing load was 355 lb., thus the ultimate tensile strength of the mortar was 355 lb. per square inch.

Tests on $1\frac{1}{8}$ in. thick slabs.

These slabs were all tested on a sand bed. Five different series of specimens were tested, the reinforcement for each series being given in the table on page 58 The results of the tests are summarised in the table below whilst a graph showing the relationship between failing load and reinforcement is given on page 65

It should be noted that in all cases four specimens were tested in each series and the results given in the table are the average of the four specimens.

Table 1 - Tests on $1\frac{1}{8}$ in. thick specimens.

Reinforcement (see page 58)	Failing Load (lb.)	Bed
1.0	1600	
1.1	4130	
1.2	7625	Sand
1.3	4380	
1.4	6890	

The types of failure occuring in the different series were as follows:

Series 1.0 Failure occurred by tension cracks along the diagonals (see Figure 15).

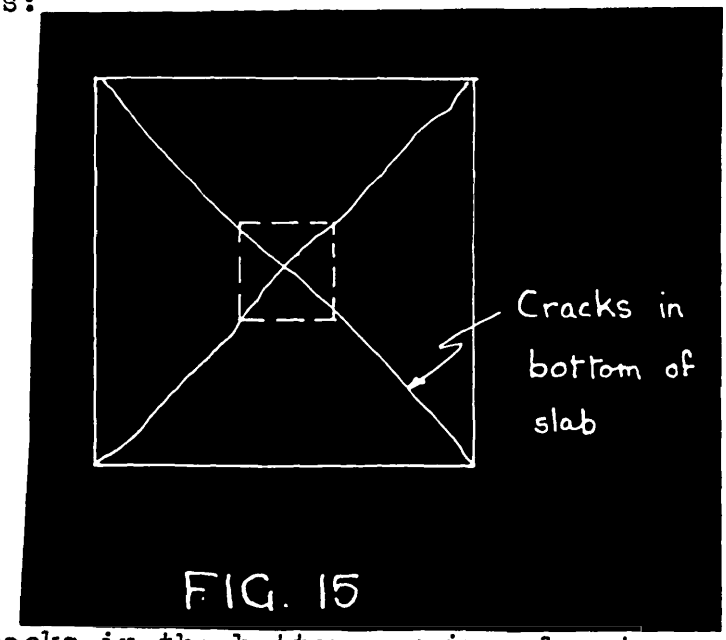
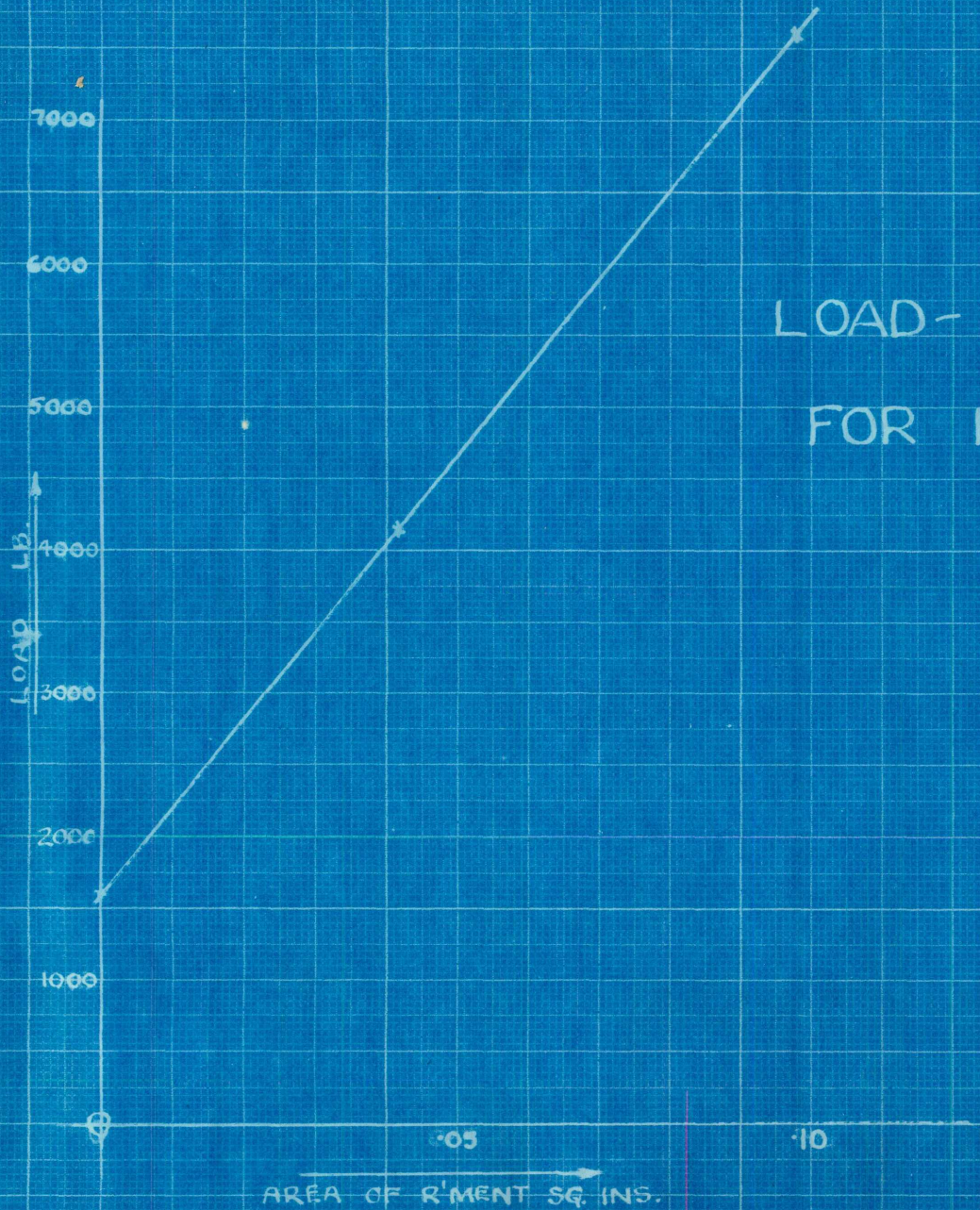


FIG. 15

Series 1.1 The failure is illustrated in Plates 2 and 3. They show the main cracks in the bottom running almost straight across the centre of the slab whilst there is a faint crack in the top of the slab. Failure was probably

LOAD - AREA OF REINFORCEMENT FOR 1⅛" THICK SPECIMENS



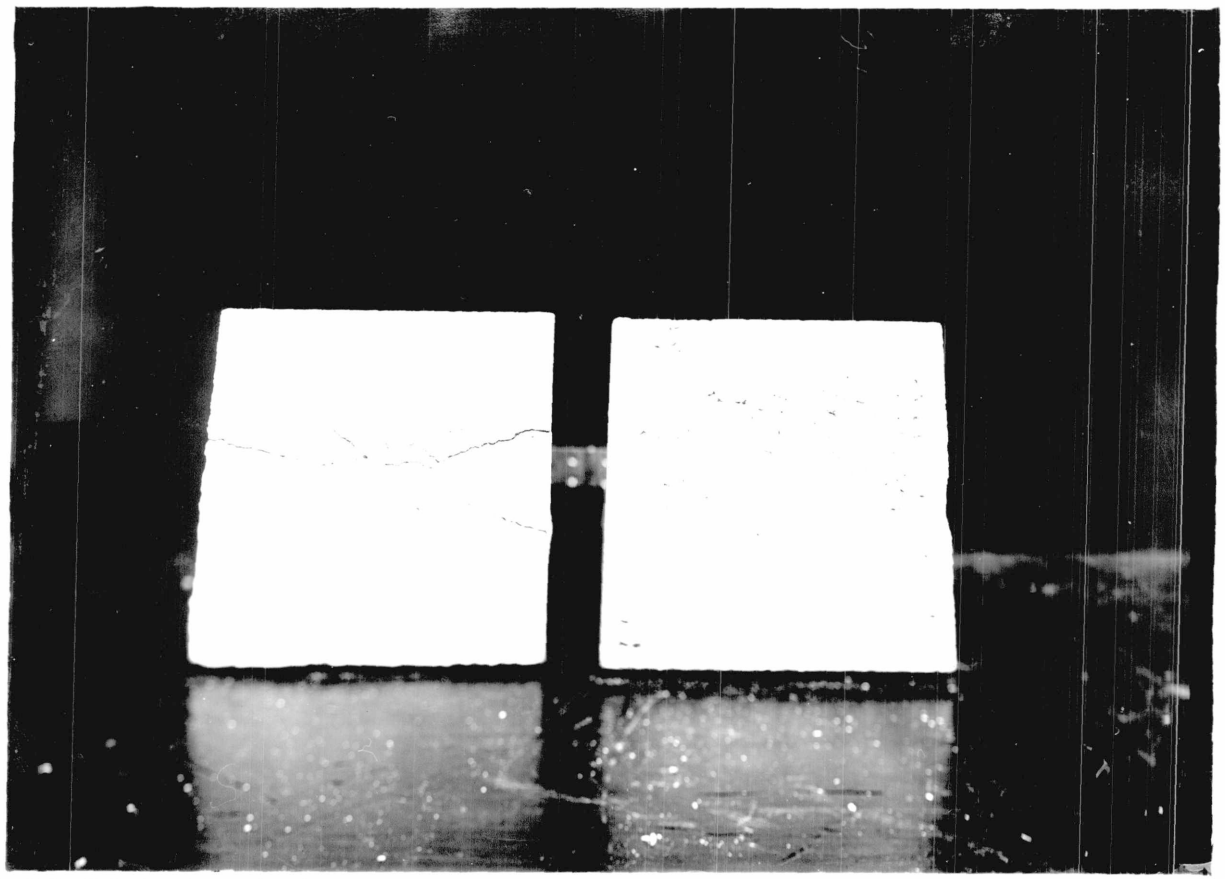


Plate 2 - Failure of Specimens 1.1 & 1.2 (underside)

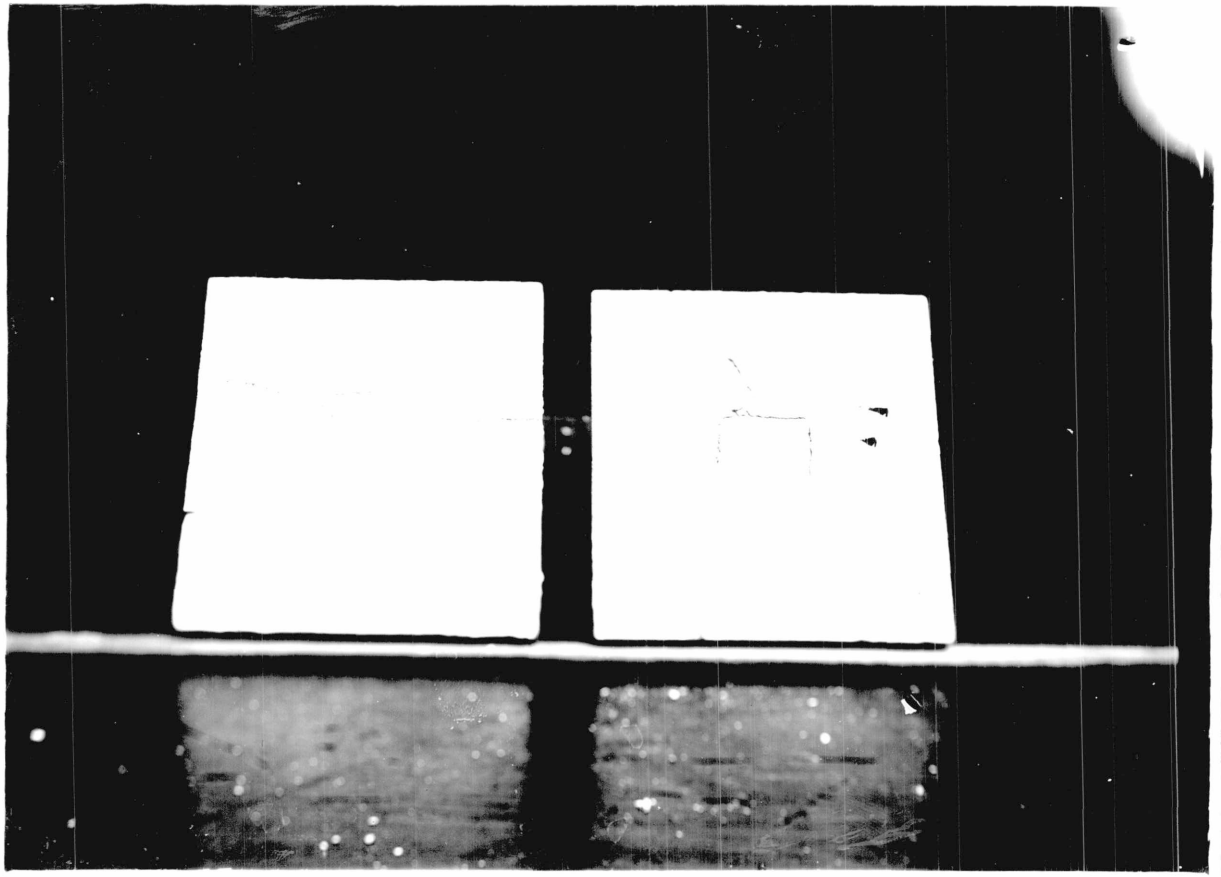


Plate 3 - Failure of Specimens 1.1. & 1.2 (top side)

by tension.

Series 1.2. The failure is illustrated by Plates 2 and 3. These show that the cracks on the underside are small and evenly distributed whilst on the upper surface there is cracking round the edges of the stancheon and also the diagonals. An examination of the cracking round the stancheon showed that it appeared to slope away at about 45 per cent as shown in Figure 16. Failure was due to shear or tension probably the former.

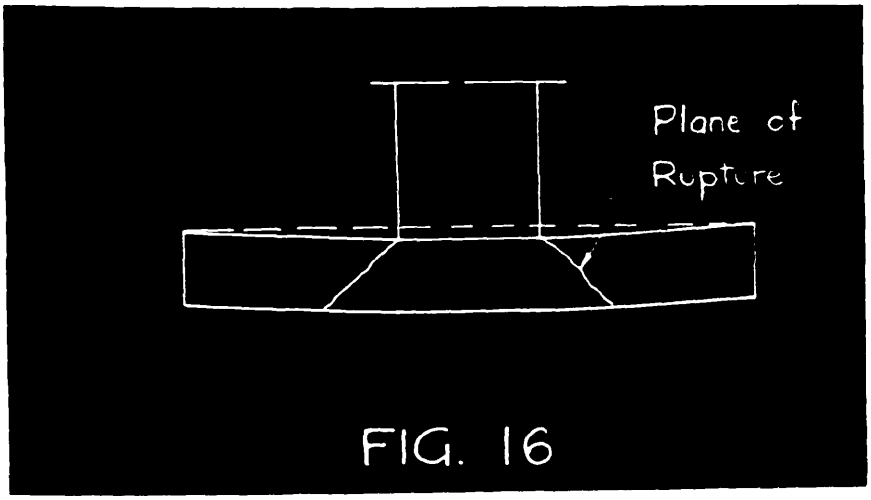


FIG. 16

Series 1.4. The failure is shown in Plates 4 [illegible]. These also show evenly distributed cracks on the underside and very pronounced cracks along the edges of the stancheon on the upper surface. It is noticed that these cracks drop at 45 per cent (approximately) until reaching the top reinforcement, they then run along the surface of this reinforcement until they reach its edge after which they again fall at 45 degrees until the bottom reinforcement is reached. Failure was due to shear or tension probably the former.

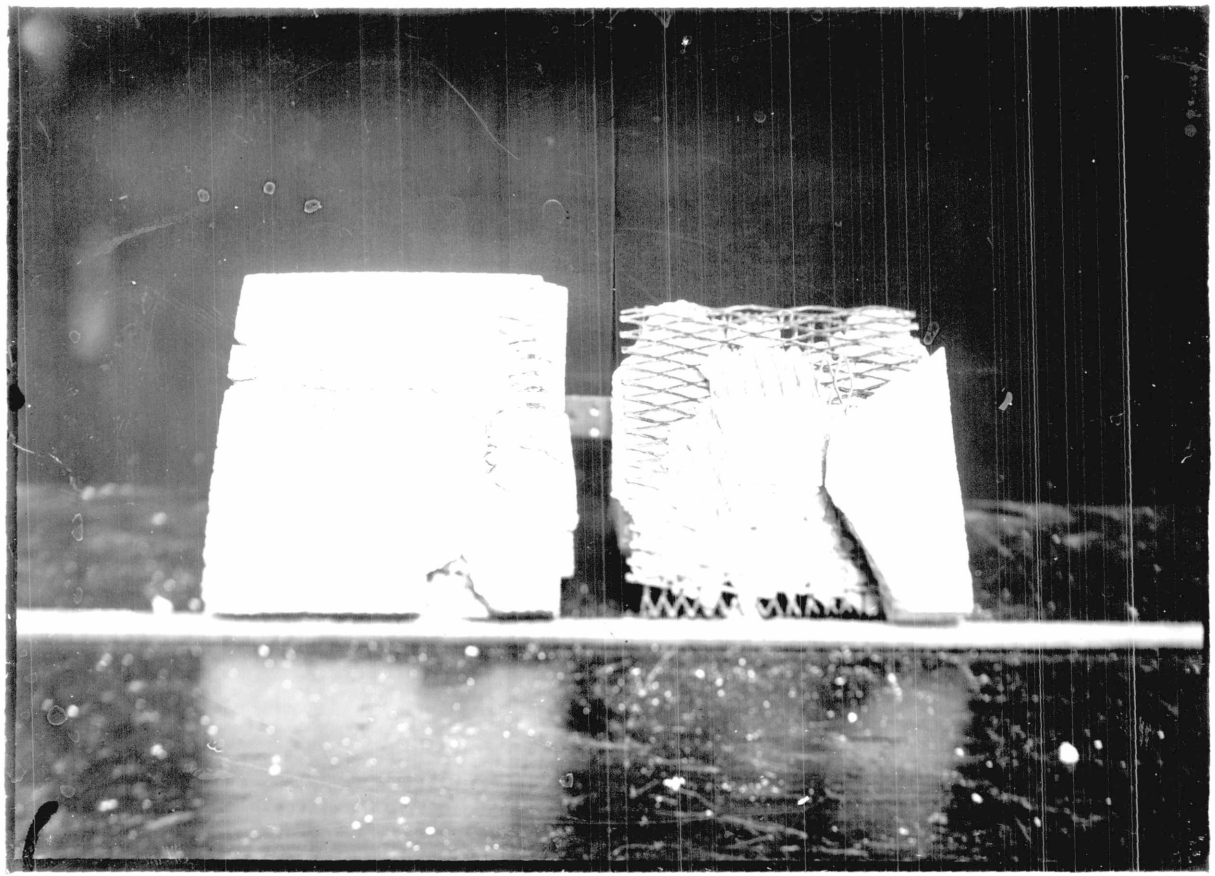


Plate 4 - Failure of Specimen 1.4

On examining the plates 3 and 4 it appears that the failures of 1.2 and 1.4 were both due to shear but it must be remembered that the load was applied for some time after the maximum load had been reached. Although the photographs after test indicate shear failure it cannot be definitely stated whether failure in the first instance was due to shear or tension.

Calculation of Theoretical failing load.

1. Failure due to crushing of mortar under stancheon.

Crushing strength of mortar = 4280 lb. per sq. in.

Area of stancheon = 4 sq. ins.

Therefore Failing load = 4 x 4280 = 17120 lb.

In no case was the failing load as high as this therefore failure was not due to crushing.

2. Failure due to excessive tension or compression.

Series 1.0. In this case failure was definitely due to excessive tension in the mortar. Resistance Moment of section

$$= 355 \times 8 \times \frac{9^2}{8} \times \frac{1}{6} = 600 \text{ in.lb.}$$

Applied Bending Moment (see page 38)

$$= \frac{1600}{4}\left(x - \frac{2}{3}\right)$$

where x is the distance of the centre of gravity of the pressure quadrant from the centre.

Hence $x - \frac{2}{3} = 1.50$

Therefore $x = 2.17$ ins.

It is shown (page 38) that in the case of

parabolic pressure distribution

$x = .3B$

If this is true for the case under consideration then

$x = 2.4$ ins.

Thus it is seen that the pressure distribution is such as to give a greater pressure at the centre and less at the sides than is given by a parabolic distribution.

Series 1.1.

A_t = .042 sq. ins. L.W.M. and .056 sq. in. S.W.M.

Failure takes place across the L.W.M. direction

Resistance moment of section is calculated on the ordinary theory as follows:

$d = .938$ in. $\quad B = 8$ in.

$p = \frac{.042 \times 100}{.938 \times 8} = .56$ per cent. Therefore Steel fails first.

$m = \frac{40,000}{4,280} = 9.4$ (Code of Practice)

$n1 = \sqrt{.0028 + .104} - .052 = .275$

$d - \frac{n}{3} = .82$ in.

Therefore R.M. = 80,000 x .82 x .042 = 2750 in.lb.

It is shown (page 70) that the applied bending moment

$= \frac{W}{4}(2.17 - .67) = .375W$

Therefore if failure takes place due to flexure the failing load carried by series 1.1 specimen

$= \frac{2750 \times 8}{3} = 7340$ lb.

Series 1.2

A_t = .098 sq. inch each way.

It can be shown that in this case failure is due to the mortar reaching its maximum stress first. The resistance moment of the section is 4780 in. lb. giving a value for the failing load (if failure is due to flexure) of 12,700 lb.

3. Failure due excessive shear.

An examination of the failure of the Series 1.2 specimens shows that if failure were due to shear it occurred along a surface of a frustrum of a pyramid as shown in Figure 17.

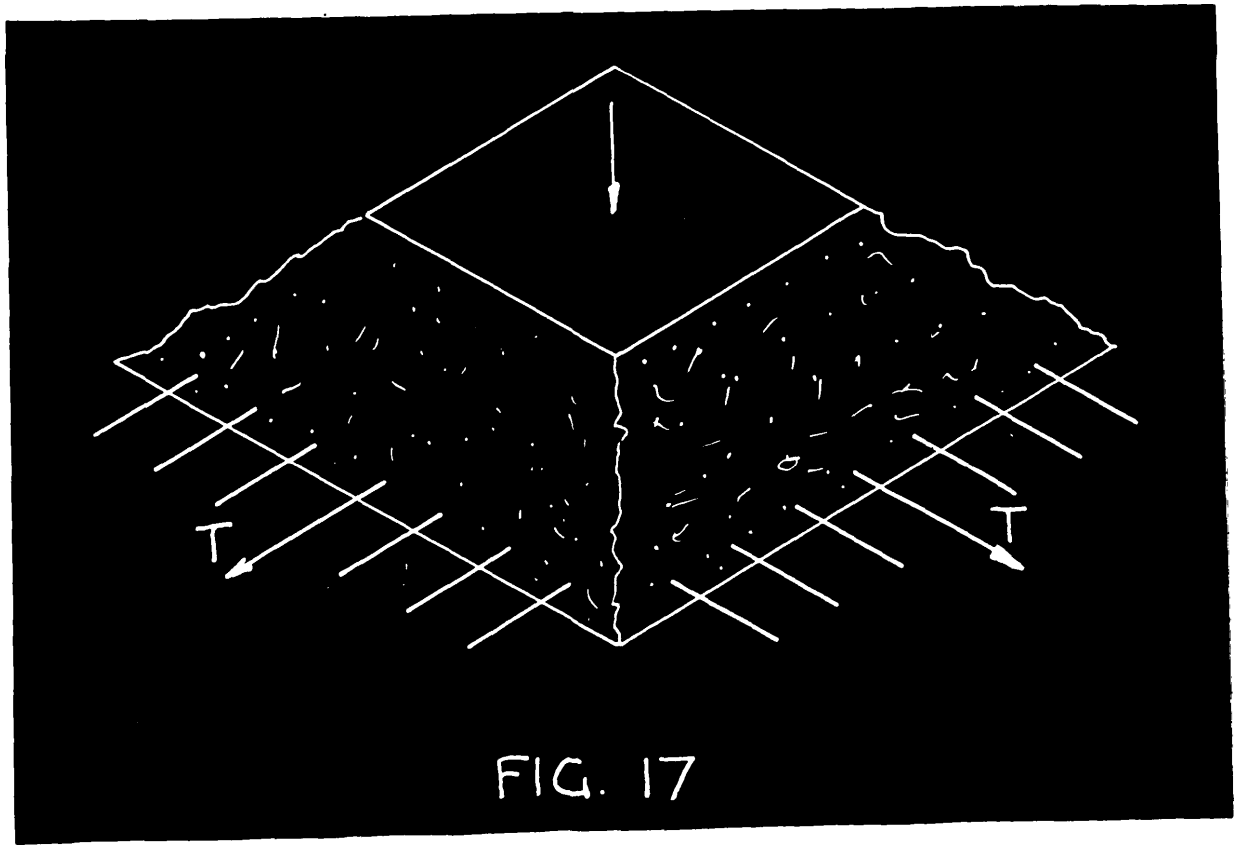


FIG. 17

Therefore in calculating the ultimate load taken if failure is due to shear it is proposed to make the following assumptions.

1. That the shear strength of the plain mortar is equal to the tension strength.

2. That this stress acts on an area equal to the surface of the frustrum of the pyramid.

3. That the effect of reinforcement is to increase the resistance by an amount equal to T sin 45 degrees, where T is the direct tension taken by the reinforcement cut by the surface of the frustrum.

4. That the ground pressure is uniform.

Series 1.1.

Area of frustrum = 4 x 1.6 x 3.125

= 20 sq. inches.

Area of reinforcement = .021 + .028 = .049 sq.in

Shear Resistance = 20 x 355 = 7100 (mortar)

+.049 x 80,000 x .707 = 2770 (steel)

= 9870 lb. total.

The failing load is the vertical component of this shear resistance plus the upward reaction on the underside of the pyramid.

$$\text{Failing load} = \frac{8^2}{8^2 - 4\frac{1}{4}^2}(9870 \times .707) = 9700 \text{ lb.}$$

Series 1.2.

In this case the area of reinforcement is equal to .098 sq. in.: the shear resistance then becomes 12640 lb. and the failing load 12500 lb.

Series 1.3 and 1.4

The effect of the top reinforcement is to increase the size of the frustrum along which failure takes place (see Plate 4). This means a considerable increase in the value of the shearing resistance if calculated by the methods used for 1.1 and 1.2: the actual results (see Table 1) do not show this considerable increase and it is obvious that such a method is of little use.

It is seen that the calculated failing loads differ greatly from the actual. These differences are discussed in the following Chapter.

4. Failure due to punching shear.

According to standard text books on design the punching shear strength of a base is independant of the amount of reinforcement present. If this is true then the strength of the Series 1 specimens can be calculated by making the following assumptions:

1. That the ultimate punching shear strength is twice the ultimate tension stress.
2. That the upward pressure distribution is uniform.

The strength of the series is then

$$8 \times 1\tfrac{1}{8} \times 2 \times 355 \times \frac{8^2}{8^2 - 2^2} = 8820 \text{ lb.}$$

Tests on 1⅝ in. thick specimens.

The main object of these tests was to determine the effect of the type of bed on the failing load. Three different types of bed were used, sand, clay and rubber. Different quantities of reinforcement were used, particulars of which are given on page 58 The results of the tests are summarised in the table below and shown graphically on page 76 The results given are generally the average of four specimens.

Table 2 - Tests on 1⅝ in. thick specimens.

Type of Reinforcement	Failing loads on different beds (lb.)		
	Rubber	Sand	Clay
2.0	3230	3060	1970
2.1	3340	4410	2670
2.2	7220	8010	4700
2.3	7860	-	-
2.4	10000	-	-
2.5	-	9400	5300

The types of failure for these specimens were similar to those for the Series I specimens. Plates 56 show the failure in a Series 2.5 specimen tested on a sand bed.

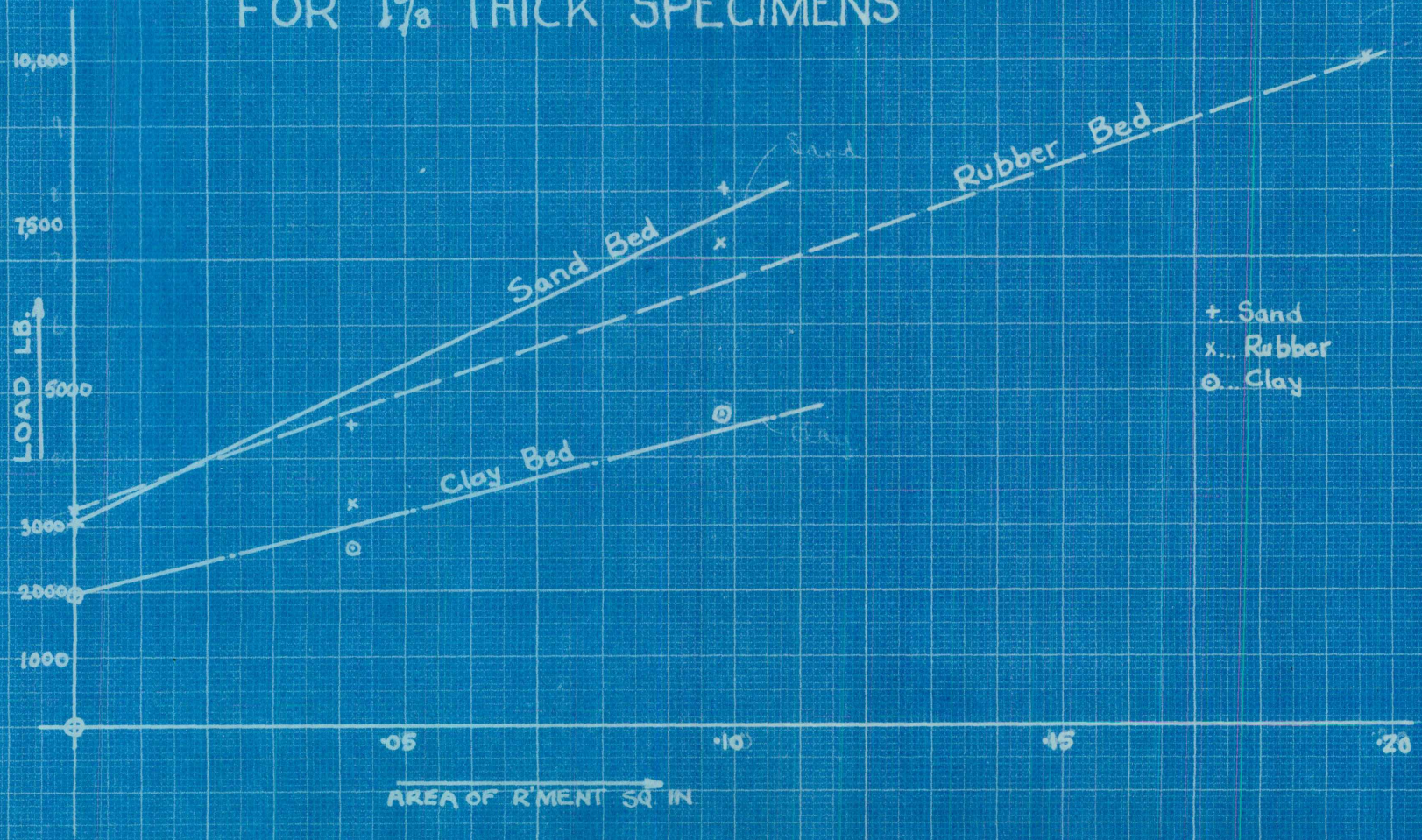
A glance at the results shows the big part played by the bed on the value of the failing load.

Calculation of Theoretical failing loads.

1. Failure due to crushing under stancheon.

Failing load = 17120 lb. (see page 70)

LOAD - AREA OF REINFORCEMENT FOR 1 5/8" THICK SPECIMENS



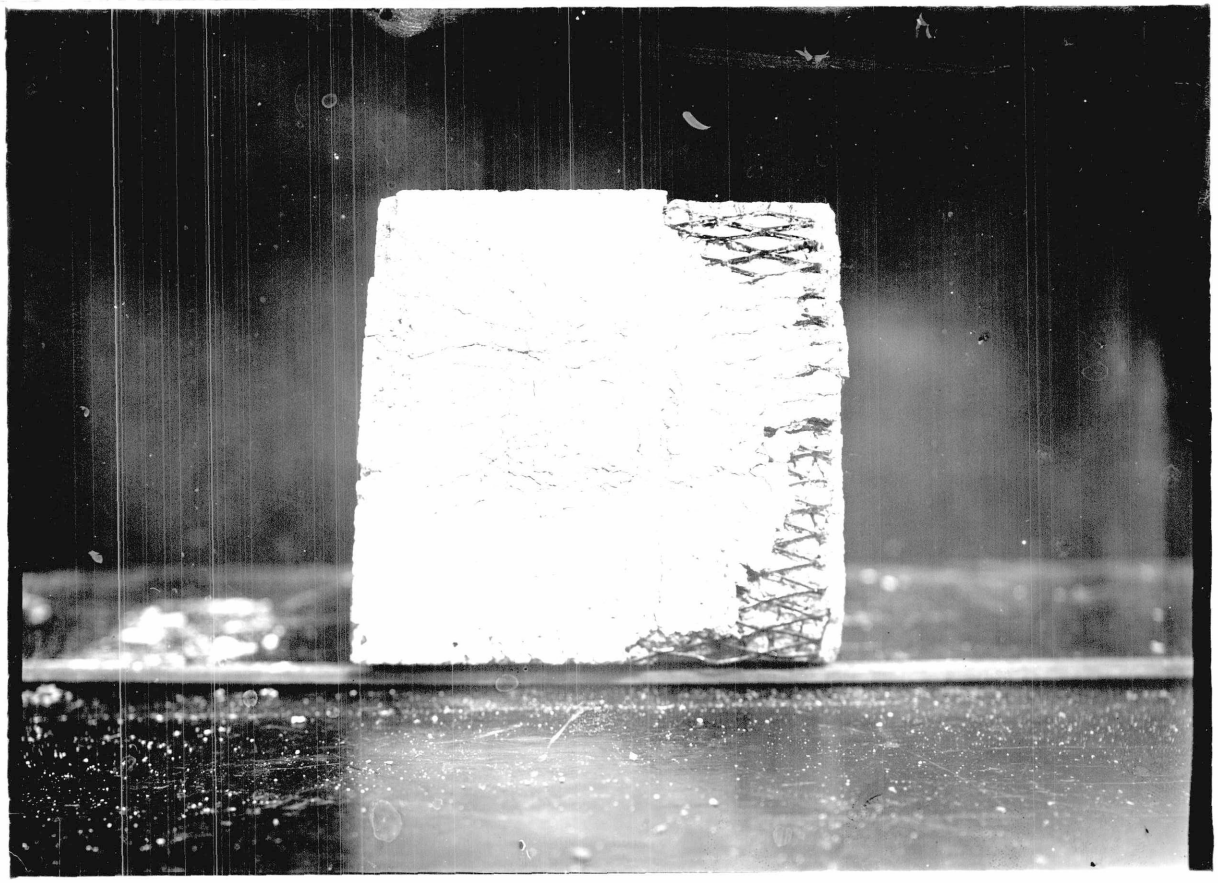


Plate 5 - Failure of Specimen 2 (underside)

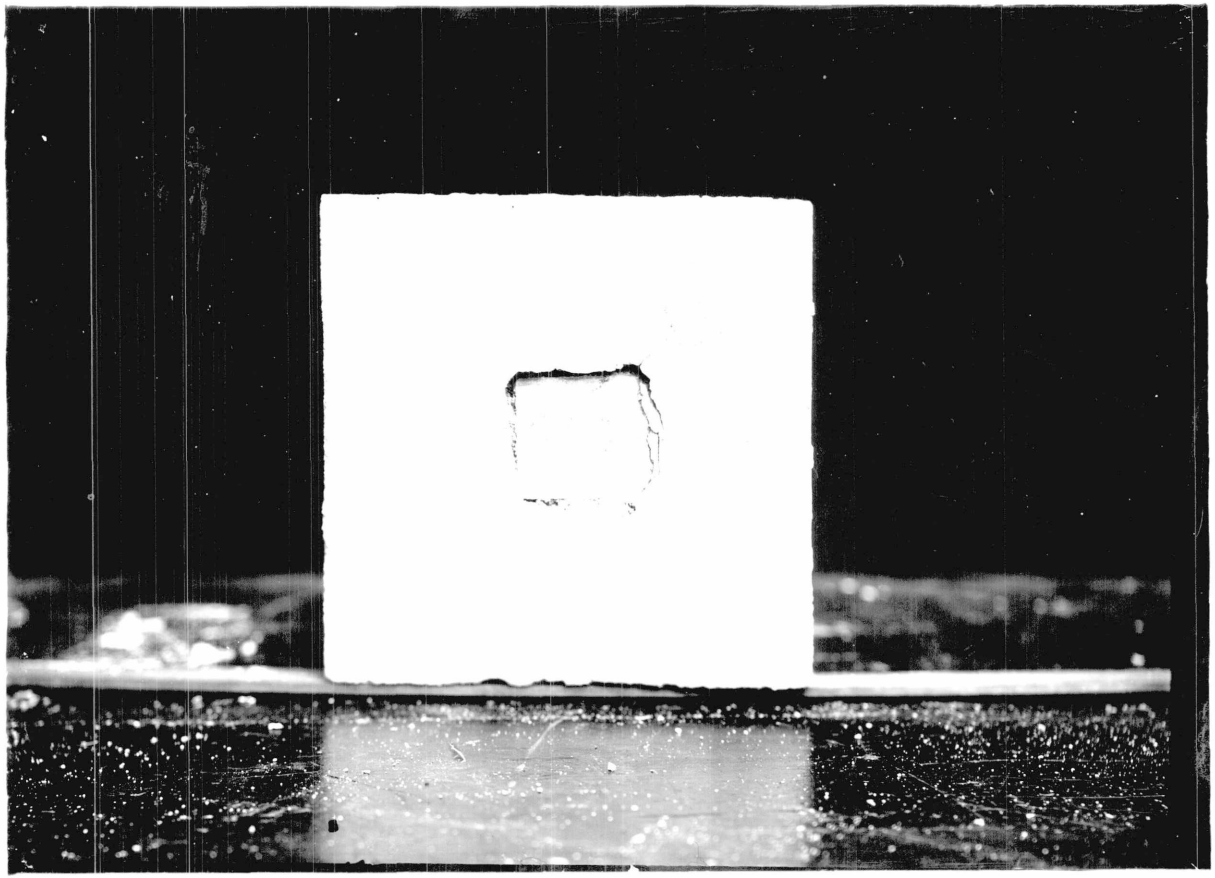


Plate 6 - Failure of Specimen 2 (top view)

2. Failure due to excessive tension or compression.

Series 2.0.

In this case failure was definitely due to excessive tension in the mortar.

Resistance Moment of section

$$= 355 \times 8 \times \frac{13}{8}^2 \times \frac{1}{6} = 1250 \text{ in.lb.}$$

Applied Bending Moment

$$= \frac{W}{4}\left(x - \frac{2}{3}\right)$$

where W = failing load and x = distance of the centre of gravity of the pressure quadrant from the centre.

Using the experimental values of W, the following values are obtained for x -

Rubber bed x = 2.20 ins.

Sand bed x = 2.30 ins.

Clay bed x = 3.21 ins.

For parabolic pressure distribution x = 2.4 ins. and for uniform pressure distribution x = 2.67 ins. The fact that for clay the pressure distribution is much greater at the edges than at the centre (causing the high value of x) is in keeping with previous observations on this point.

It is not proposed to give details of the calculations for the theoretical flexural failing loads; values are summarised in the table below

Table 3 - Theoretical failing loads in flexure

Type of Reinforcement	Theoretical flexural failing load (lb)		
	Rubber	Sand	Clay
2.1	11500	10810	6950
2.2	25900	24400	15700
2.4	29600	-	-

3. Failure due to excessive shear.

Using the assumptions given on page 43 the theoretical failing loads in shear (which must be independant of the type of bed since the upward pressure is assumed uniform) are as follows:-

Series 2.1 . 19000 lb.

Series 2.2 23200 lb.

Series 2.4 31800 lb.

The fact that the pressure distribution is not uniform means that for a clay bed the failing load in shear is less than that given above and for a sand bed it is greater.

4. Failure due to punching shear.

By making the assumptions given on page 44 the failing load for Series 2 specimens is calculated as follows:-

$$W = 8 \times 1\tfrac{5}{8} \times 2 \times 355 \times \frac{8^2}{8^2 - 2^2}$$

$$= 9850 \text{ lb.}$$

CHAPTER VI.

CONCLUSIONS FROM TESTS ON MODEL BASES.

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CONCLUSIONS FROM TESTS ON MODEL BASES.

It is intended in this chapter to summarise the conclusions from the results given in the previous chapter and to discuss any interesting features.

The main object of these tests was to determine the effect of different beds. It is seen from the table on page 75 that the type of bed has an appreciable effect on the load carried by the base. On a clay bed a base carries less than two-thirds of the load carried by a similar bed on sand. This fact is very important and has always been overlooked in design.

The explanation for this is as follows:

1. The distribution of ground pressure under a footing on clay is such that the maximum pressure occurs at the edges thus giving a bigger lever arm for the load producing the bending moment.

2. On a clay bed the base deflects under load much more than on a sand bed. This increase in deflection doubtless causes an increase in the flexural stresses to which the base is subjected.

It is of interest to note that as the load carried by the base increases the ratio between the clay and sand bed gets smaller as shown in the table below

Type of Reinforcement	2.0	2.1	2.2	2.4
Ratio between Clay and Sand loads.	.645	.605	.587	.564

This is possibly explained by the increased deflection as the load increases.

The settlement of bases on clay is always greater than that on sand and the clay soils generally present difficulty from the deflection point of view. These tests show that they also present difficulty from the design point of view.

The Author suggests that when a column base rests on a clay soil the factor of safety in the design should be increased from the standard figure of 4 to a new figure of 6, and the ground pressure assumed uniform. By so doing a design would be obtained where the true factor of safety is between 3 and 4, a figure which is about the average for a reinforced concrete structure.

With regard to the secondary objects of this series of tests (mentioned on page 11), the following general conclusions can be drawn.

1. The graph on page 65 shows that the load carried is approximately directly proportional to the amount of reinforcement. From the rather scanty information given on the graph on page 76 it appears that the rate of increase in strength with the reinforcement added varies for each type of bed used.

2. The effect of a layer of reinforcement in the top of the slab is to strengthen its power of resistance. The increase in strength obtained from the experiments was inconsistent. Series 1.3 showed an increase of 6 per cent for 0.56 per cent reinforcement, Series 2.3 an increase of 9 per cent for 0.37 per cent reinforcement and Series 2.5 an increase of 17.5 per cent in the case of a sand bed and 13 per cent in the case of a clay bed with0.74 per cent of reinforcement. These increases are not as great as those obtained by the use of the same quantity of reinforcement in the bottom of the slab. It is pointed out on pages 23-25 that top reinforcement is useless when considered from the design point of view and the combined evidence of experiment and analysis suggests that top reinforcement is of little or no value.

3. The experiments show that for approximate purposes the use of mortar models adequately reinforced provide useful information with regard to the behaviour of a reinforced concrete structure.

The experiments also show that the fact that the punching shear is within the permissible limits does not mean that the base is safe. For the quality of mortar used (c = 4270 lb. per square inch) the safe ordinary shear can be taken as 100 lb. per square inch (Code of Practice, Clause 301). The permissible punching shear stress is then 200 lb. per square inch. This gives a critical load value

of 1920 lbs. for the Series 1 specimens and 2780 lbs. for the Series 2 specimens. For series 2.1 on a clay bed the failing load is less than the safe load in punching shear. On the other hand where the section is heavily reinforced as in 1.2 or 2.4 the failing load is about four times the safe punching shear load.

The most unsatisfactory feature of the tests is the big discrepancy between the actual failing loads and the calculated. For this reason it is difficult to say whether the actual failure was due to flexure or to shear or a combination of them. The difference between the theoretical and experimental is due in some measure to the fact that the former have been obtained by the use of the straight-line no-tension theory and the extent to which this applies in the case of a slab 1⅛ inches thick is not known although it is very doubtful if it is true.

These experiments were not carried out from the quantitative but the qualitative standpoint, therefore discussion on the difference is not necessary. A full discussion on the difference between the theoretical and the experimental is given in Chapter IX, which gives the conclusions of the experiments carried out on the large specimens. The tests on these specimens were definitely quantitative.

CHAPTER VII.

EXPERIMENTAL DETAILS FOR CONCRETE BASES.

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EXPERIMENTAL DETAILS FOR CONCRETE BASES.

In this chapter the method of manufacture and testing for the specimens in Class II (see page 12) are given together with a description of the test machine. The results of the tests are given and discussed in the two following chapters whilst the structural calculations for the test machine are given in the Appendix.

Method of making the specimens.

The specimens were all 2ft. by 2ft. in plan and 4ins. thick. The concrete mix used was 1 : 2 : 4 by weight: the cement was Ferrocrete rapid-hardening Portland cement which complied in all respects with the requirements of the British Standard Specification; the sand was Stone Court river sand and the coarse aggregate was Thames ballast with a maximum size of $\frac{3}{4}$in. In an attempt to keep a consistent concrete, a constant water-cement ratio of 55% was used throughout the tests. Due to the fact that the tests were spread out over six months and that several fresh supplies of cement and two fresh supplies of aggregate were obtained during this time the attempt at consistent concrete strength was not absolutely successful.

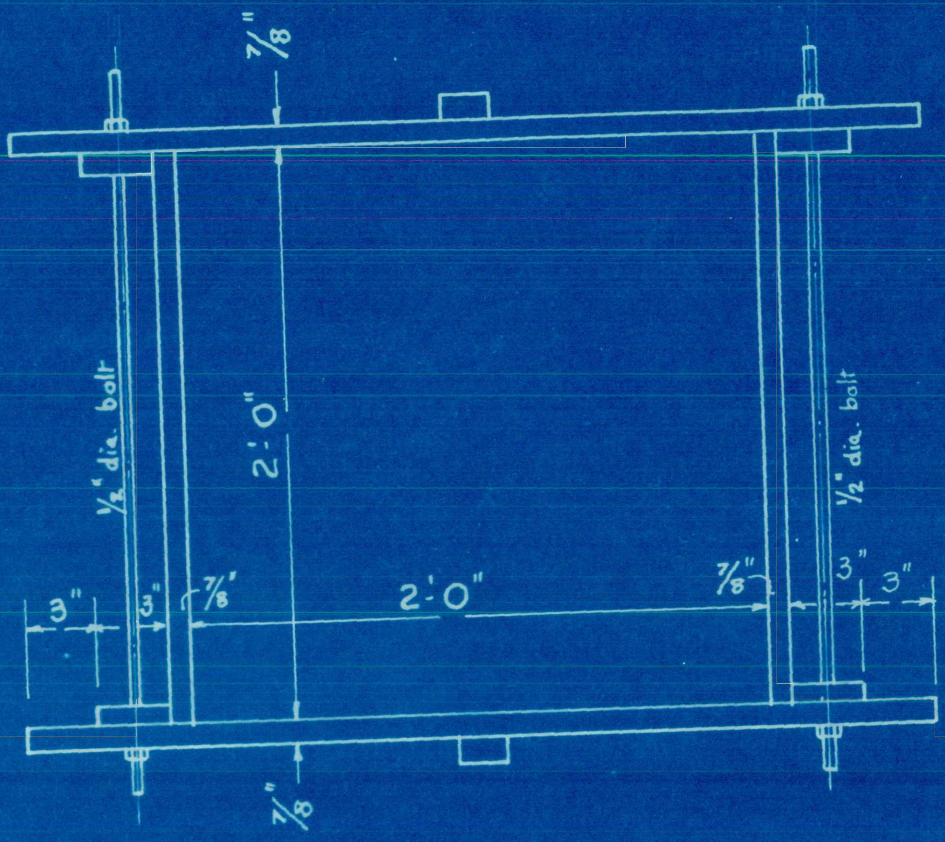
The concrete was machine mixed, the time of mixing for each batch being $1\frac{1}{2}$ minutes. Two batches each consisting of 14 lb. cement, 28 lb. sand and 56 lb. aggregate were required for each specimen.

The forms used are shown in Figure 18. The form was placed on a zinc surfaced board and both clamped to a vibrating table on which the specimen was made. The vibrating table was vibrated by means of a "Sinex" electrical vibrator which gave 6,000 blows a minute. The top surface of the specimen was finished smooth and flush with the upper edge of the forms.

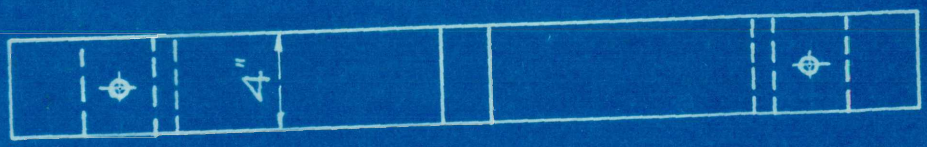
As each specimen weighed nearly 2 cwt. their transport presented difficulties. After 24 hours the forms were removed and the specimen slid from the vibrating table on to a trolley the top of which was level with the vibrating table. The specimen was then wheeled to the storage bed and lowered by means of a grab and pulley.

The specimens were covered with damp sacks until the time of testing.

The main object of this series of tests was to determine the effect of the reinforcement on the failing load, therefore as many different systems of reinforcement as possible were used. With one exception the reinforcement used was ordinary mild steel with a yield point stress of 40,000 lb. per sq.in. The different combination of bars used together with particulars of the specimen number, and area and perimeter of reinforcement used are given in Tables 4 and 5. The reinforcement was in each case wired together with soft steel tying wire to form a mesh and the cover given on the underside was $\frac{1}{2}$in. The bars were in every case 1ft. 9ins. long thus giving $1\frac{1}{2}$ins. end cover.



PLAN



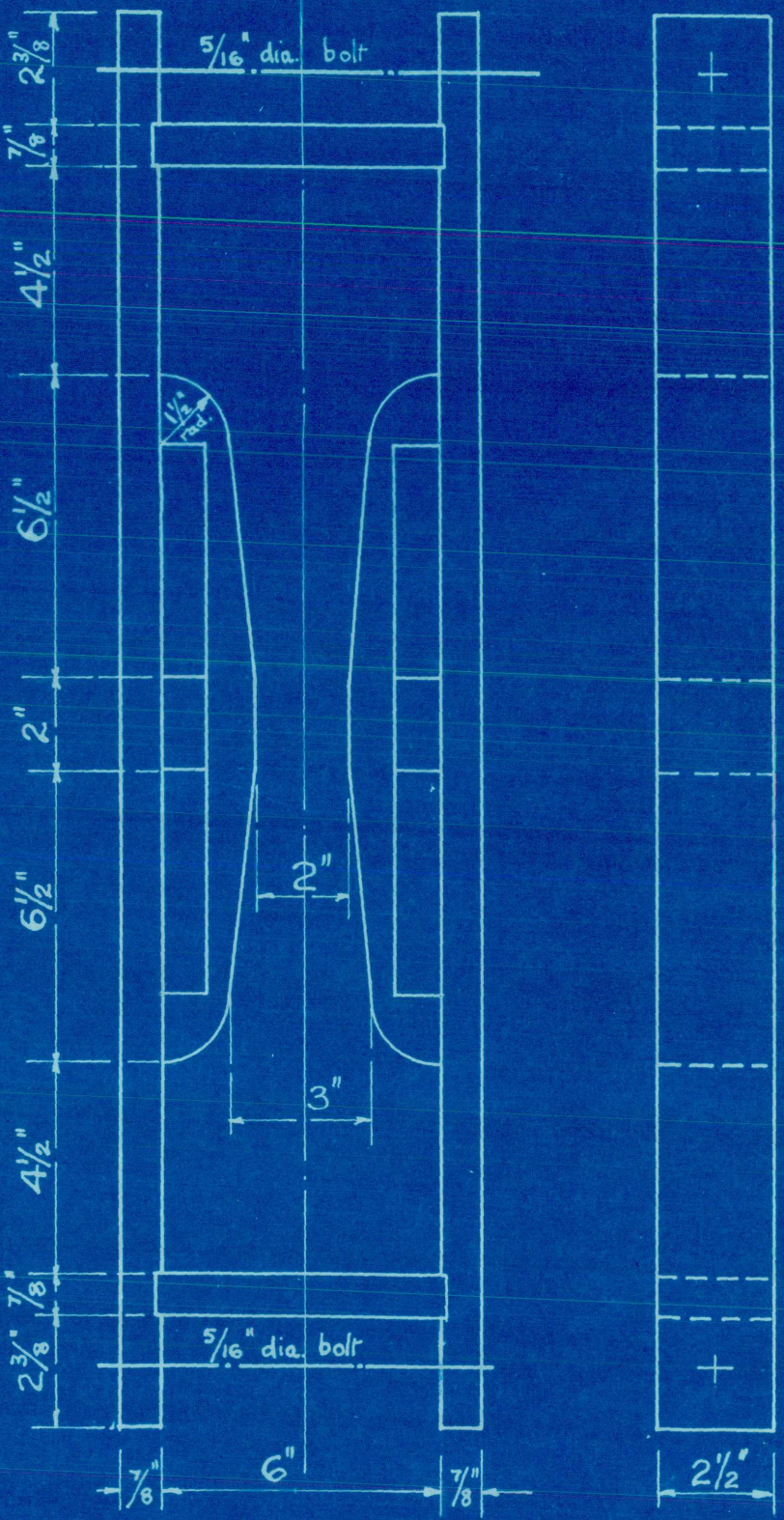
FORMS FOR CONCRETE SLABS

SCALE : 1/8 F.S.

FIG. 18

FIG. 19

SCALE: 1/4 F.S.



FORMS FOR TENSION SPECIMENS

In addition to these slabs, cubes and tension specimens were also made. The cubes were 6 in. standard type. The forms used for the tension specimens are shown in Figure 19. They were designed with a "waist" at the centre to ensure the concrete breaking at that section, it having been previously found that without the waist the local stresses at the bearing surfaces caused failure: this failure was not a true tension failure and was therefore to be avoided. Both cubes and tension specimens were consolidated by vibration.

The problem of bond stresses must also be considered in column bases and tests were carried out to determine its value.

A $\frac{3}{4}$ in. diameter rod about 2 feet long was embedded in a 6 inch cube of concrete (of the same mix as the slabs); the rod was held vertically and the concrete rammed round it. The specimen was cured in the same way as the slabs and tested after seven days by pulling the bar out of the concrete. This test was carried out in the Riehle machine. Four specimens were made and the average result taken.

The Test Machine.

The tests were carried out in a 100 ton testing frame designed by the Author in collaboration with Professor Pippard. The details of the machine are given in Figures 20 to 23 and plate 7, whilst the structural calculations are given in Appendix I.

The frame consists of two stancheons, designed as tension members, A, between which span two cross beams the upper of which B, is designed to carry a 100 ton concentrated load whilst on the lower beams this load is assumed uniformly distributed. To the lower flanges of beams B, a plate carrying 100 ton jack is bolted (the jack was fixed to the plate by shrinking and finally welded up for security).

To enable the ram of the jack to return to its original position after test two 1 in. square rods C, are fixed to the head and through these run two $\frac{1}{2}$ in. diameter rods D; to the upper end of these rods a wire is fixed which runs over two pulleys on the top of girder B (see Figure 20) and then back alongside stancheon A: it carries weights sufficient to overcome the frictional resistance of the ram against the cylinder. By this means the ram of the jack returns to its original position after test. An attachment on the end of the rods D acts as a safety device telling when the ram is projecting for too great a a distance.

Excavation was necessary to enable the stancheons and lower cross beam to be placed: the earth was backfilled up to a certain level and then concrete was put in up to a level 6 inches below the floor level, the concrete is reinforced near the surface in order to carry the load from the floor to the cross beam.

On the reinforced concrete bed a box 3ft. 6 ins. square inside, of stout timbers strapped with steel channels rests. This box contains the sand on which the foundation slab is placed. The slab is lowered on to the bed by means of the tackle shown (E); the surface being previously levelled. When lowering the slab care is taken to ensure that it is placed centrally on the bed (guide lines on the slab and centre marks on the sides of the bed facilitate this). The slab is checked for horizontality by means of a spirit level. A 4 inch square steel block, 6 inches long with machined edges is placed centrally on the slab, a layer of felt paper being between them to even out surface irregularities.

Since the total punching shear depends on the perimeter of the stancheon a simple test to determine the part played by the punching shear is to use two stancheons of different diameters. Therefore two specimens type L were tested using a 7 inch square stancheon instead of the 4 inch square type generally used.

To the steel block a scale, F, is attached: by

means of a telescope, G, focussed on to this scale the deflection of the slab is measured. This scale is of such a length that it is approximately horizontal with the Ames dial on the load-measuring ring: this enables both scale and dial to be seen simultaneously in the telescope

The load is applied by the ram of the jack to the steel block through a ball and socket (to ensure axial loading) and a load-measuring ring.

The load-measuring ring is shown in Plate 8. It is designed by the Sheffield Testing House and is similar to those which are used by the Building Research Station for their bridge tests.

The ring is made of a special chromium steel, is 12 inches overall diameter, 3½" wide and 1¾" thick. The decrease in diameter of the ring is directly proportional to the applied load, therefore by measuring the change in diameter it is possible to calculate the load. This decrease is measured by means of an Ames dial reading to one ten thousandth of an inch. An electric tapper is fixed to the dial to ensure a steady reading.

Before carrying out a test the ring is securely tied to the rods C as a safety measure.

Calibration of ring.

Before being used in the foundation tests the ring was calibrated. The test was carried out in the Riehle machine shown in Plate 1. Readings of the applied load and the corresponding deflection (as given on the Ames dial)

were taken. These are shown in a graphical form on page 104

It has been found that there is very little fatigue in these rings consequently only the one calibration test was carried out.

Method of carrying out tests.

The apparatus, etc. was set up as described and then hydraulic pressure is applied to the ram. By careful control of the inlet valve it is possible to apply the load quite slowly and steadily. Up to a pressure of 700 lb. per square inch the water is taken from the high pressure mains, for pressure greater than this an intensifier is used. As the load is applied readings of the load-measuring dial and the deflection of the slab are taken as frequently as possible until the slab fails. The load at failure is calculated from the dial reading but the pressure gauge reading provides a method of checking this load. After failure the slab is removed from the bed and the cracks on the lower surface studied. The bed is levelled off and is now ready for another test.

Table 4 - Details of Reinforcement in Slabs

Straight Bars.

Type of Specimen	No. Thus	Reinforcement each way	Area square inches each way	Perimeter inches ÷ π each way
A	7	None	-	-
B	4	6 - ½ in. diameter	1.14	3.0
Q	3	5 - "	0.98	2.5
N	2	4 - "	0.78	2.0
C	2	8 - ⅜ in. diameter	0.88	3.00
S	2	7 - "	0.77	2.62
D	2	6 - "	0.66	2.25
R	2	4 - "	0.44	1.5
E	2	8 - 5/16" diameter	0.61	2.50
U	2	7 - "	0.53	2.19
F	2	6 - "	0.45	1.88
T	2	4 - "	0.30	1.25
G	2	8 - ¼ in. diameter	0.39	2.00
V	2	7 - "	0.34	1.75
H	2	6 - "	0.29	1.50
J	2	4 - "	0.19	1.00
W	3	10 - 3/16" diameter	0.27	1.87
X	3	8 - "	0.22	1.50
Y	2	6 - "	0.16	1.13
Z	2	4 - "	0.11	0.75
α	1	20 - ⅛ in. diameter	0.25	2.50
β	2	16 "	0.20	2.00
γ	2	12 "	0.15	1.50
δ	2	8 "	0.10	1.00

Table 5 - Details of Reinforcement in Slabs

Hooked Bars.

Type of Specimen	No. Thus	Reinforcement each way	Area square inches each way	Perimeter ind ÷ π each way
K	2	8 - $\frac{3}{8}$ in. diameter	0.88	3.00
L	4	6 - "	0.66	2.25
P	2	8 - 5/16" diameter	0.61	2.50
O	3	6 - "	0.45	1.88
M	2	6 - $\frac{1}{4}$ in. diameter	0.29	1.50
∈	1	1 layer No.22 gauge expanded metal		

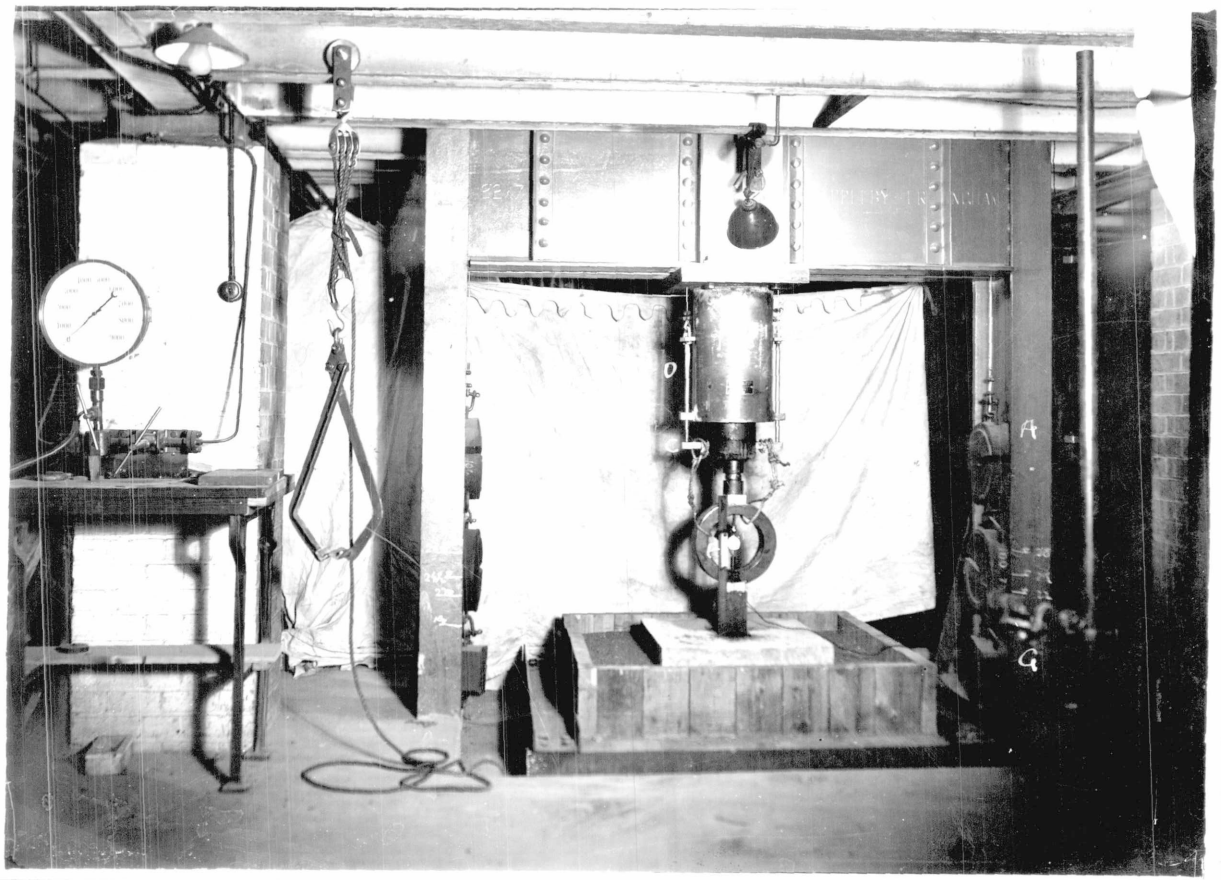


Plate 7 - Test Machine for Concrete Bases

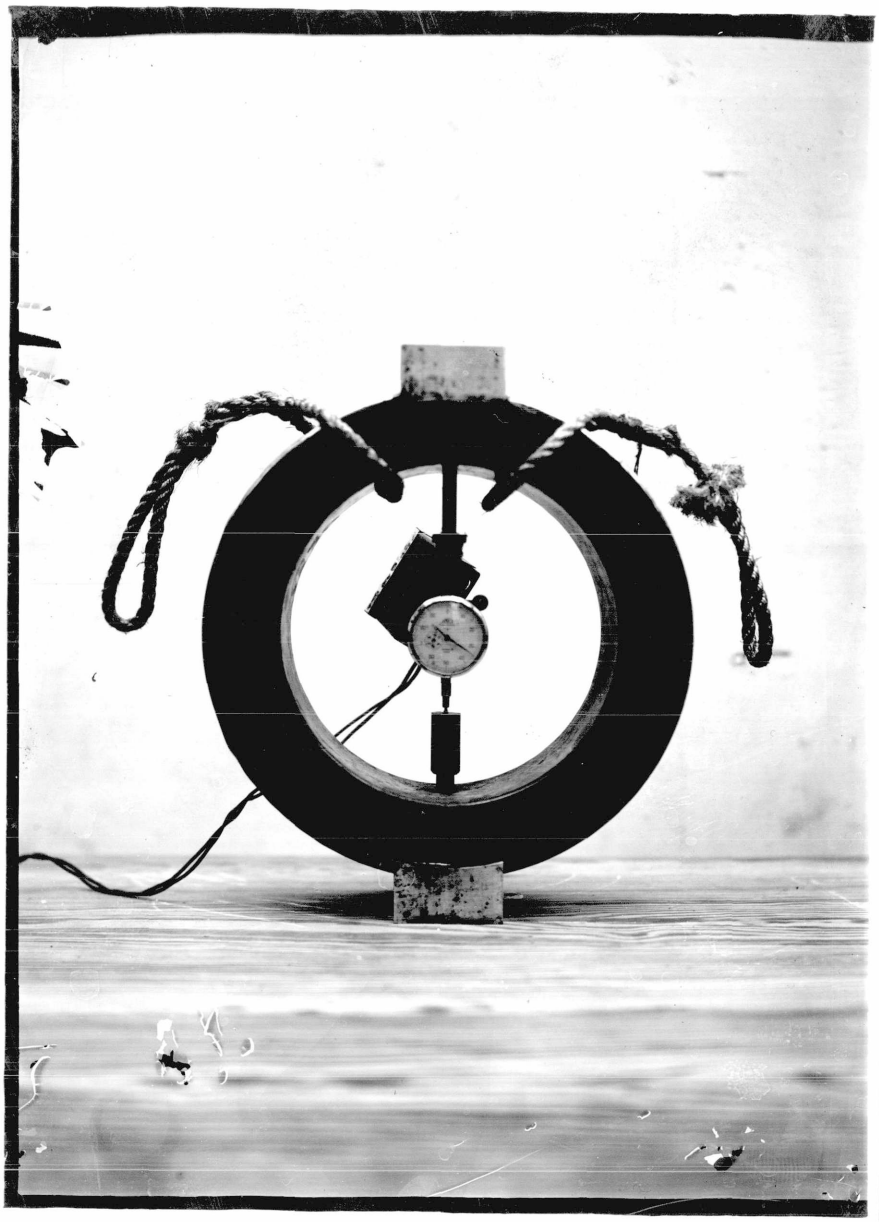
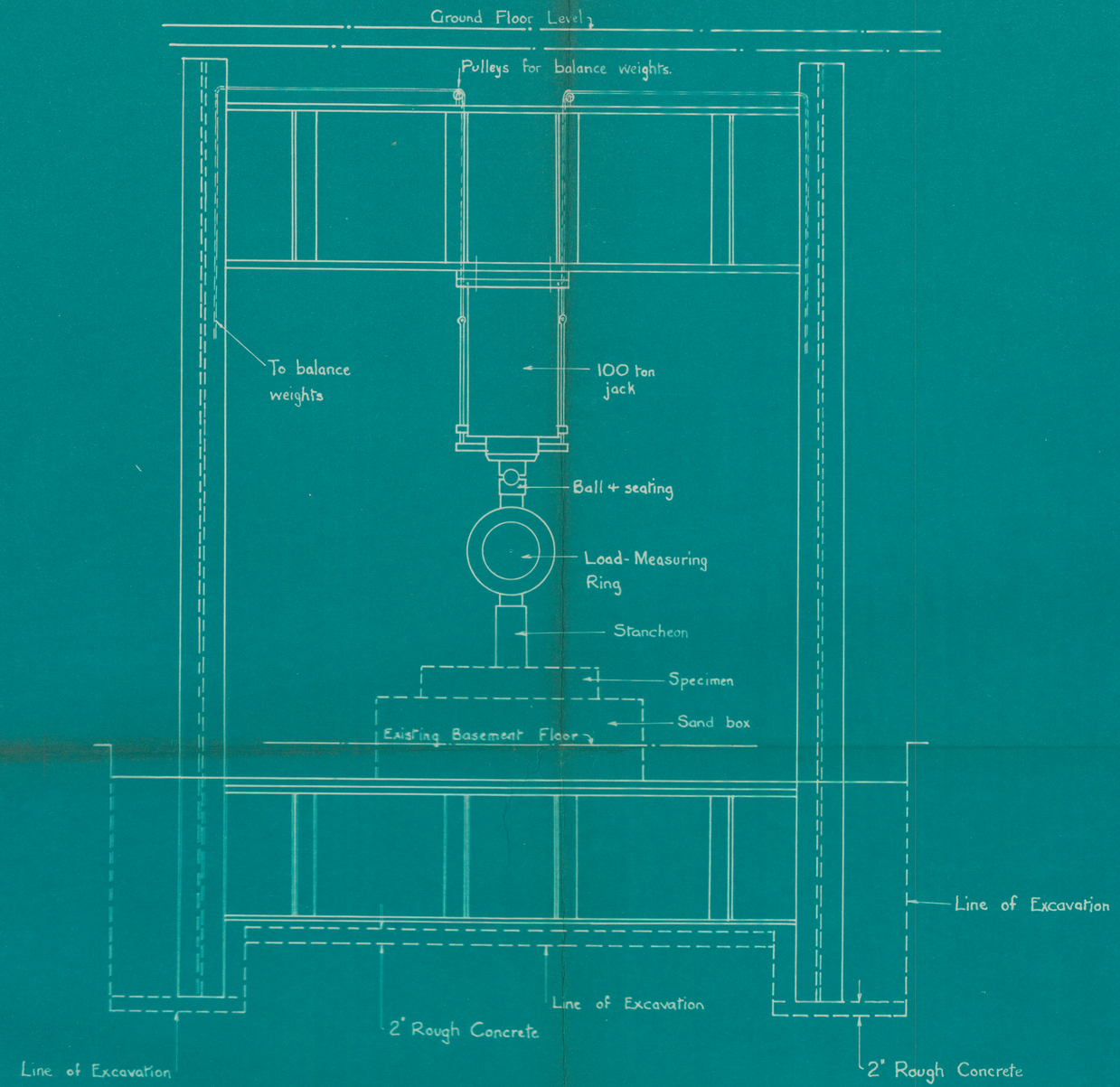
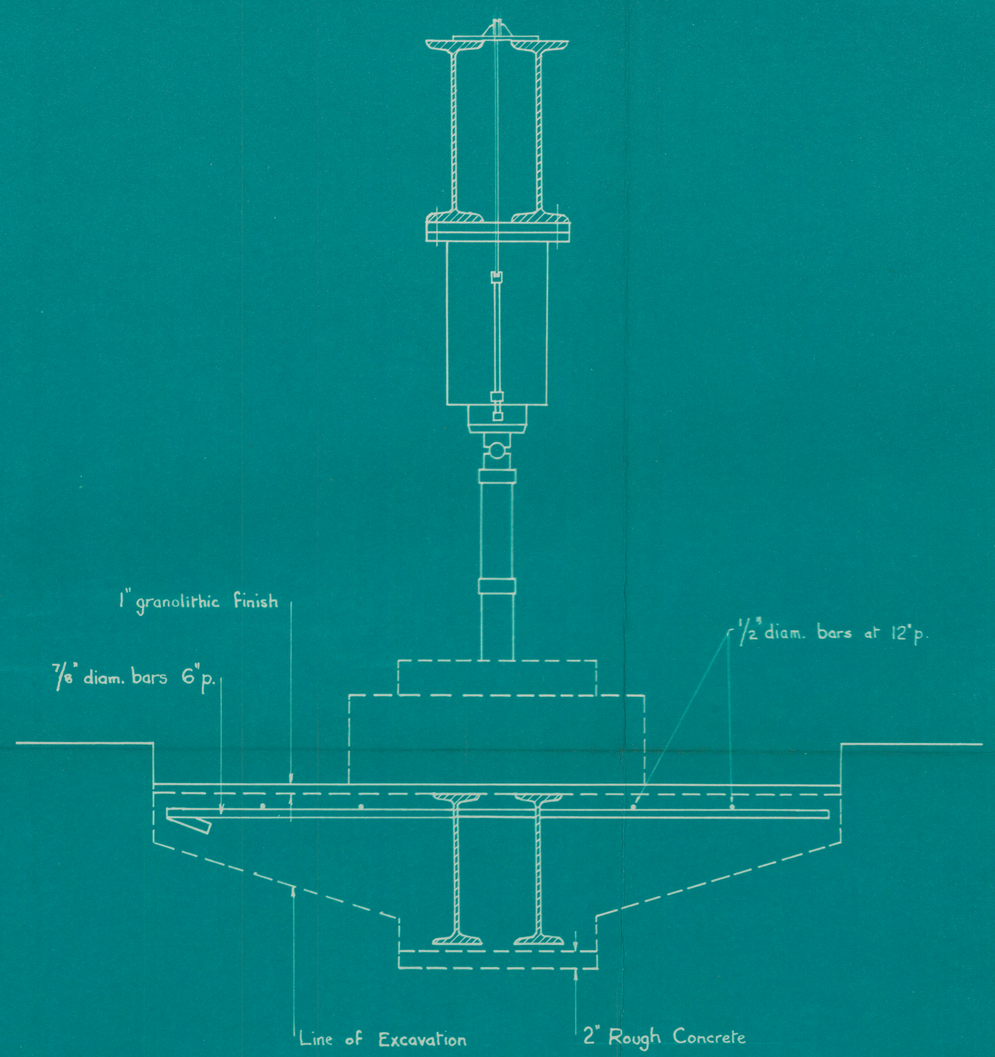


Plate 8 - Load Measuring Ring



FRONT ELEVATION

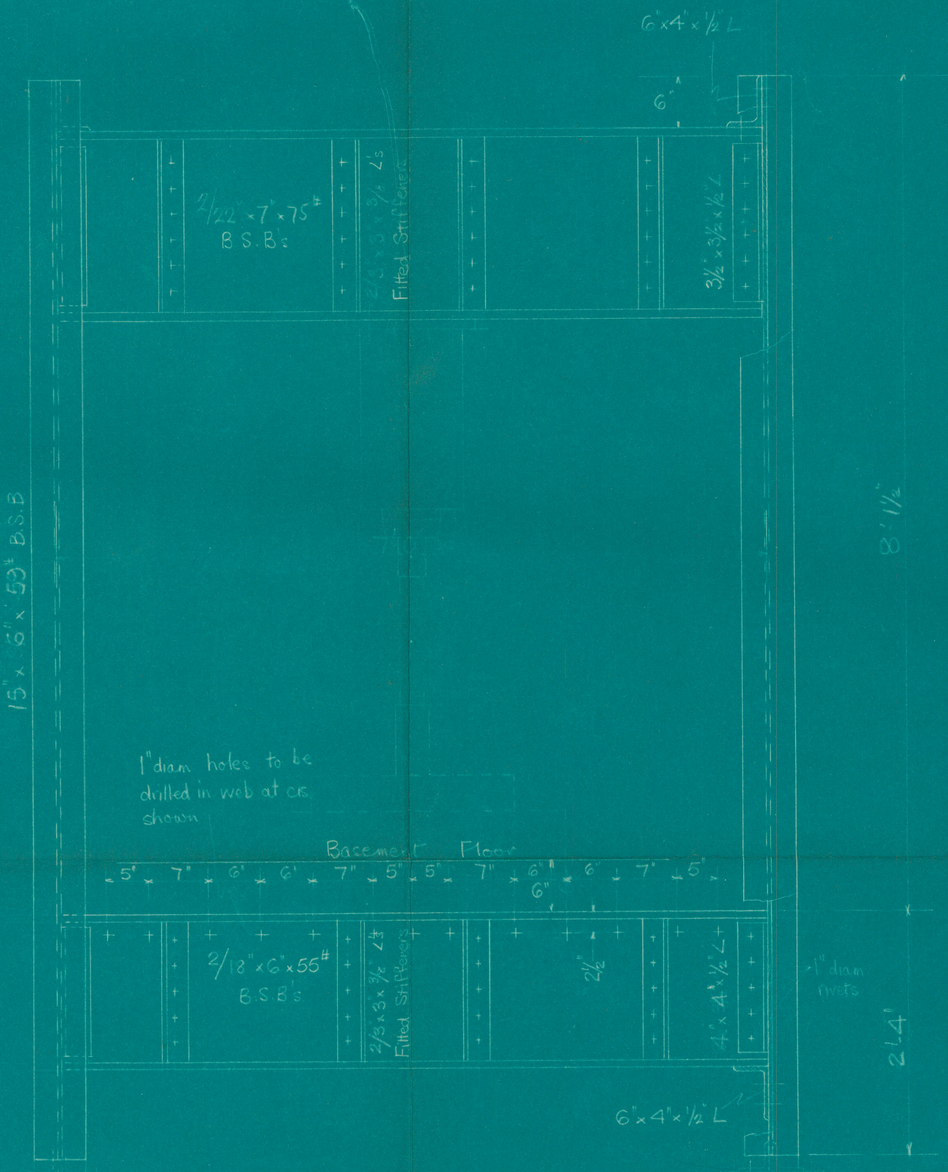


CROSS SECTION

GENERAL ARRANGEMENT OF 100 TON MACHINE

SCALE : 1" TO 1'-0"

FIG. 20



FRONT ELEVATION

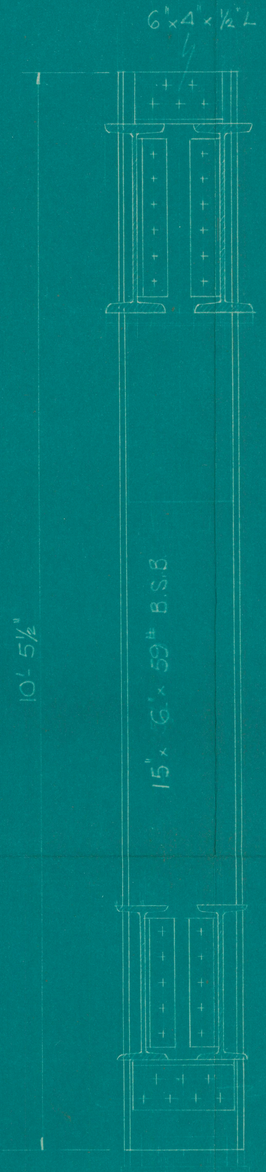
STEEL LIST

2 No. 22"×7"×75# B.S.B's 7'-0" lg. fitted with 3"×3"×⅜" Stiffeners

2 No 18"×6"×55# B.S.B's 7'-0" lg. fitted with 3"×3"×⅜" Stiffeners

2 No 15"×6"×59# B.S.B's, 10'-5½" lg.

All Rivets ⅞" diam except where otherwise shown



SIDE ELEVATION

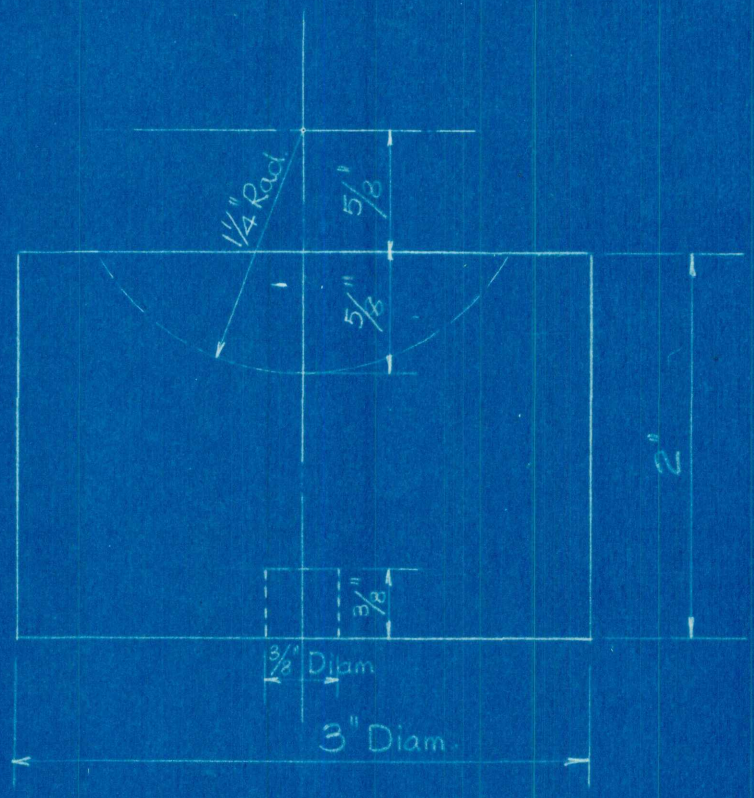
TESTING FRAME FOR CITY & GUILDS COLLEGE

SCALE :- 1" TO 1'-0"

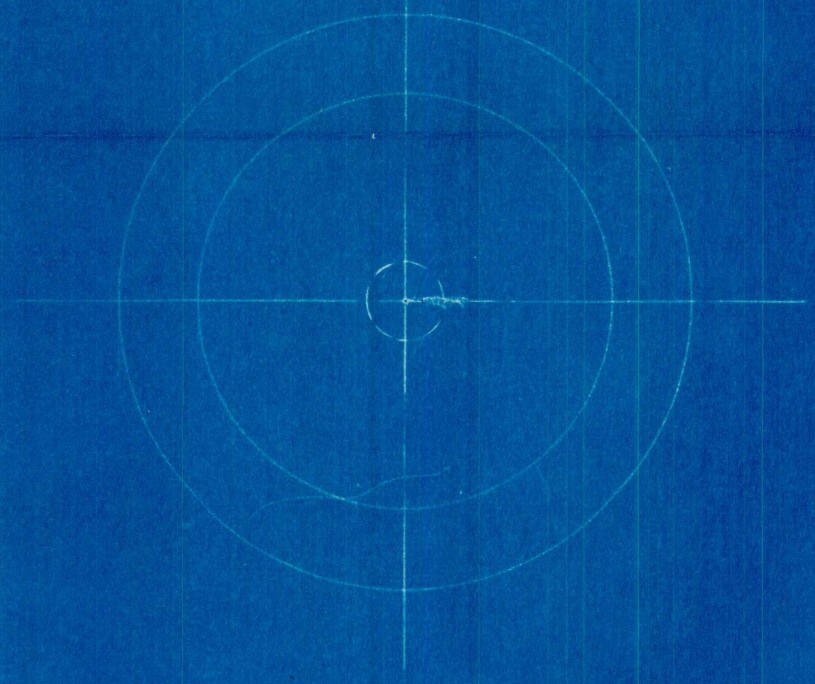
DEPT. OF CIVIL ENGINEERING

DEC. 16TH 1937

FIG. 21



ELEVATION



PLAN

SEATING FOR 2½" DIAM BALL

SCALE:- FULL SIZE

TWO REQ'D THUS OF 3½% NICKEL STEEL

CITY & GUILDS COLLEGE

FIG 22

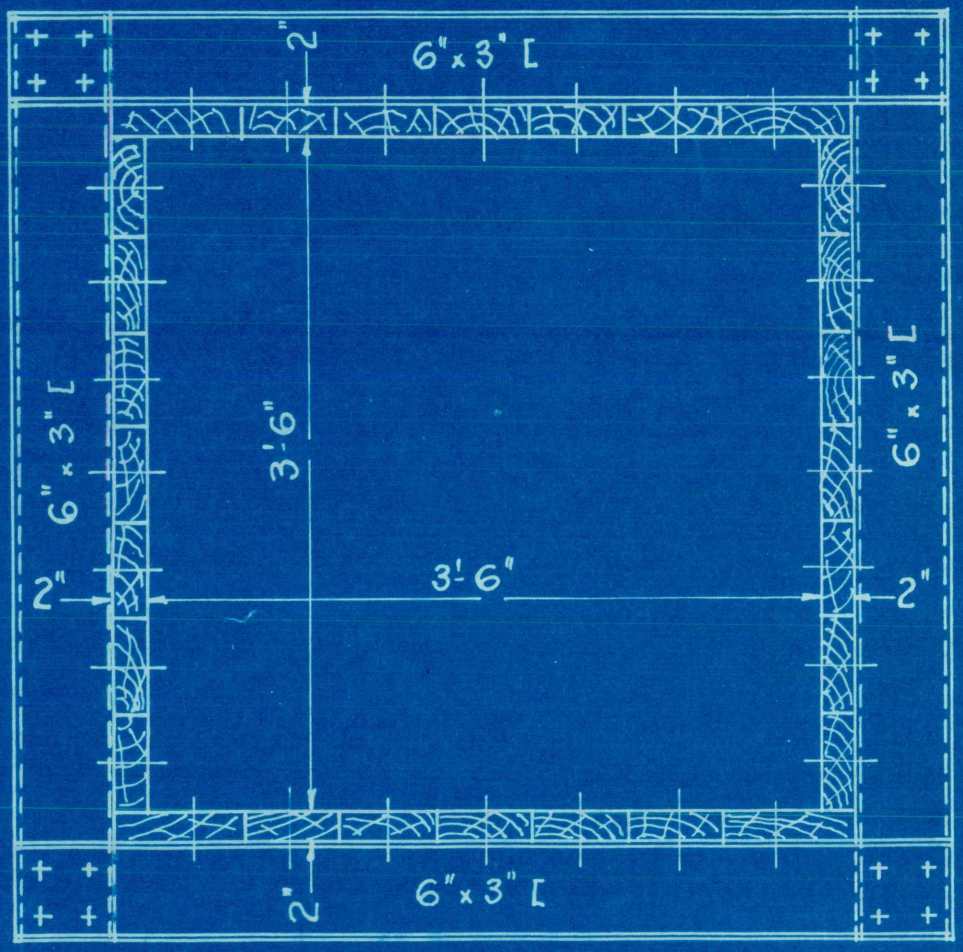
CIVIL ENG. DEPT 4/3/38

FIG. 23

SCALE : 1" TO 1'-0"

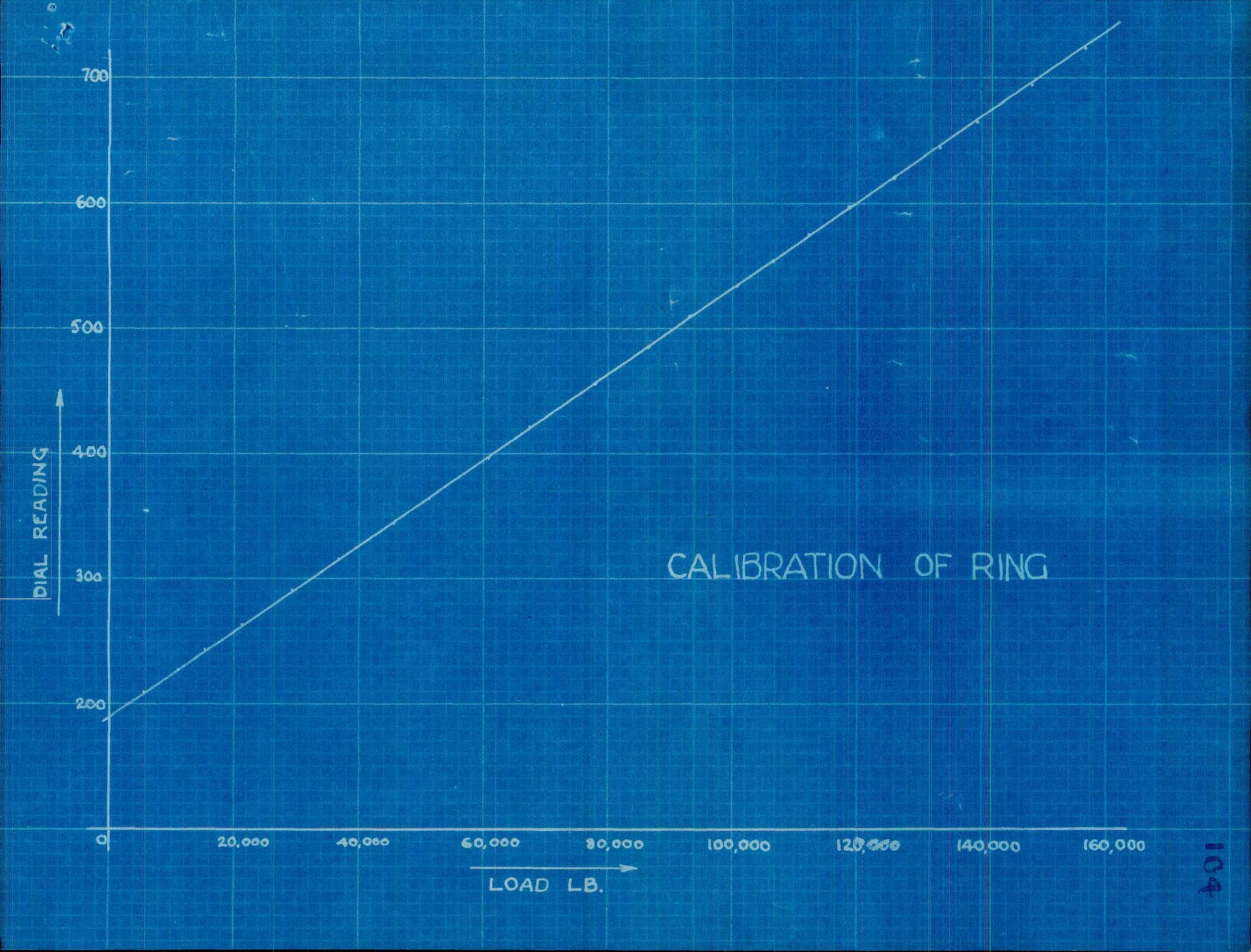


ELEVATION



PLAN

BOX FOR SAND BED


CALIBRATION OF RING
DIAL READING
700
600
500
400
300
200
0
20,000
40,000
60,000
80,000
100,000
120,000
140,000
160,000
LOAD LB.

CHAPTER VIII.

TESTS ON CONCRETE BASES.

CHAPTER VIII - TESTS ON CONCRETE BASES.

The main object of this series of tests was to determine the effect of the reinforcement on the strength and to determine this it was necessary to keep the bed on which the base rested as uniform as possible, it having been found from the previous series that the type of bed had an appreciable effect on the strength. The measurement of the central deflection of the slab enabled the compactness of the bed to be determined.

Measurement of Deflection.

During each test readings of the deflection were taken at frequent intervals: these readings are given in Tables 8 to 37 which are in Appendix II. For specimens A7, B3, B4, G2 and X2 the increase in deflection with the load is given graphically on pages 108-116 A study of these graphs gives the method for judging the consistency of the bed. Each graph is found to consist of three stages.

Stage I. The deflection during the earliest stages of the test. It was impossible to take many readings during this stage as the deflection increases rapidly. The rate of increase is about .01 in. for 500 lb. load.

Stage II. After the rapid deflection in stage I the bed has compacted and in stage II the deflection is proportional to the applied load but does not increase so rapidly as in stage I. The rate of increase is about .01 in. for 2500 lb. load.

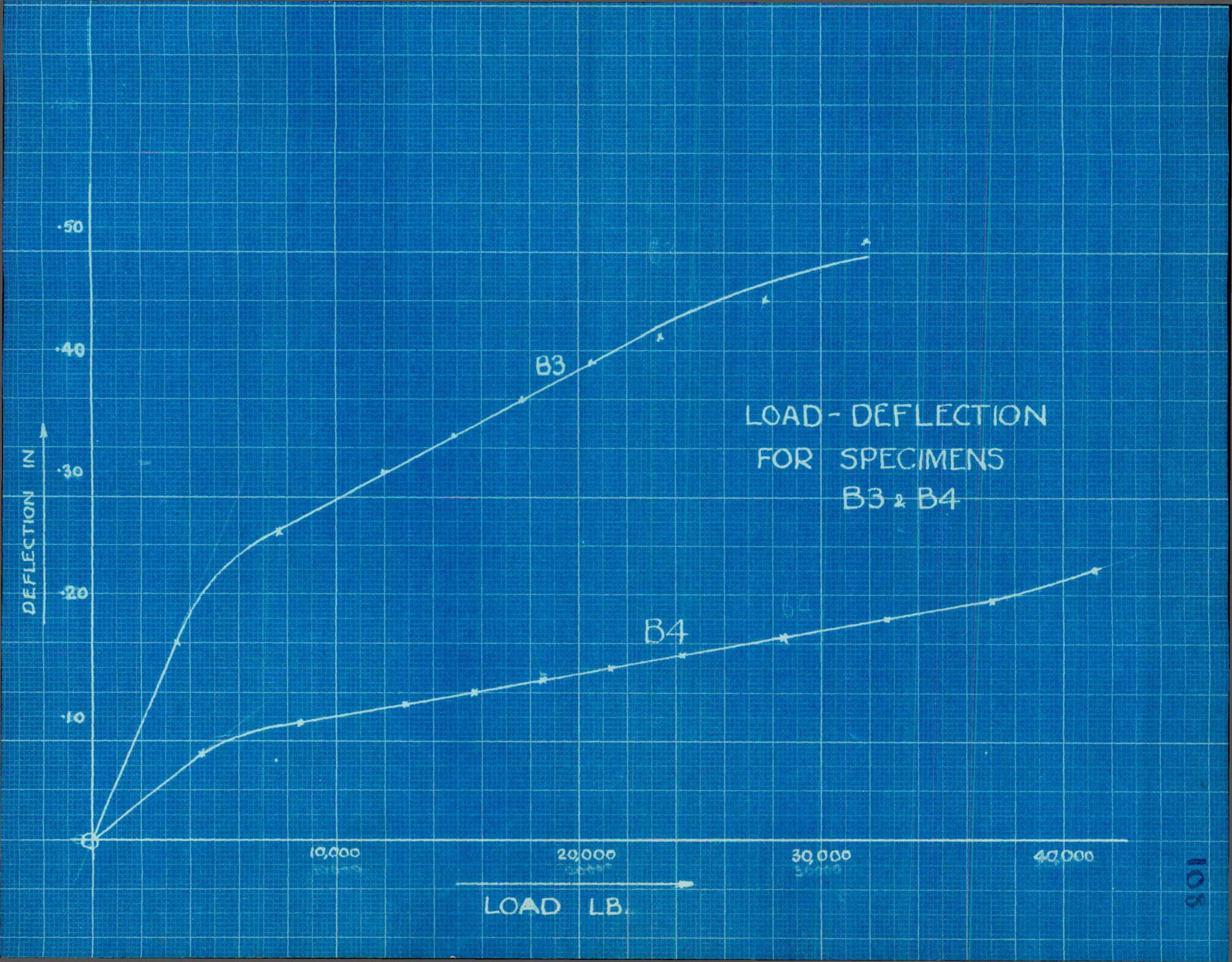
Stage III. The deflection in this stage depends on the type of base under test. In all cases the graph is not so flat as in stage II showing a rate of deflection which in some cases is as rapid as that in stage I. In the case of specimen X2 which was lightly reinforced the slope of the graph is not as great as that of B4 which was heavily reinforced.

It appears therefore that stages I and II are dependant on the condition of the bed and independant of the flexural rigidity of the specimen: stage III depends partly on the bed but mostly on the flexural rigidity of the specimen. The fact that stages I and II depend only on the bed gives the method for determining its consistency This method is as follows:

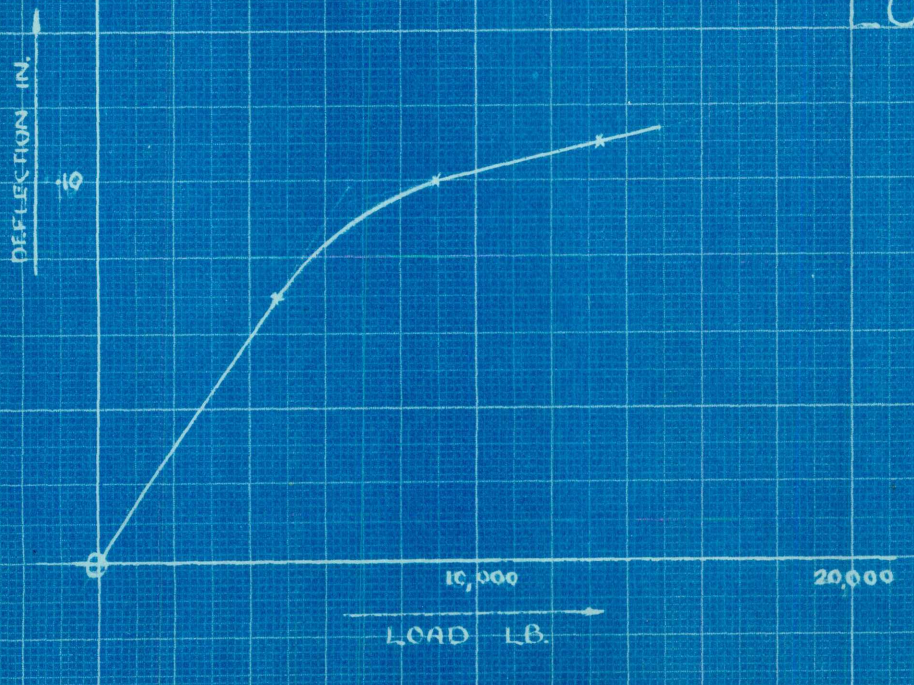
> "In all cases where the stages I and II of the load-deflection graph are similar the slabs are assumed to be on beds of equal consistencies."

Therefore those tests where the load-deflection curve was not similar in its early stages to the standard were rejected when discussing the values of the failing loads.

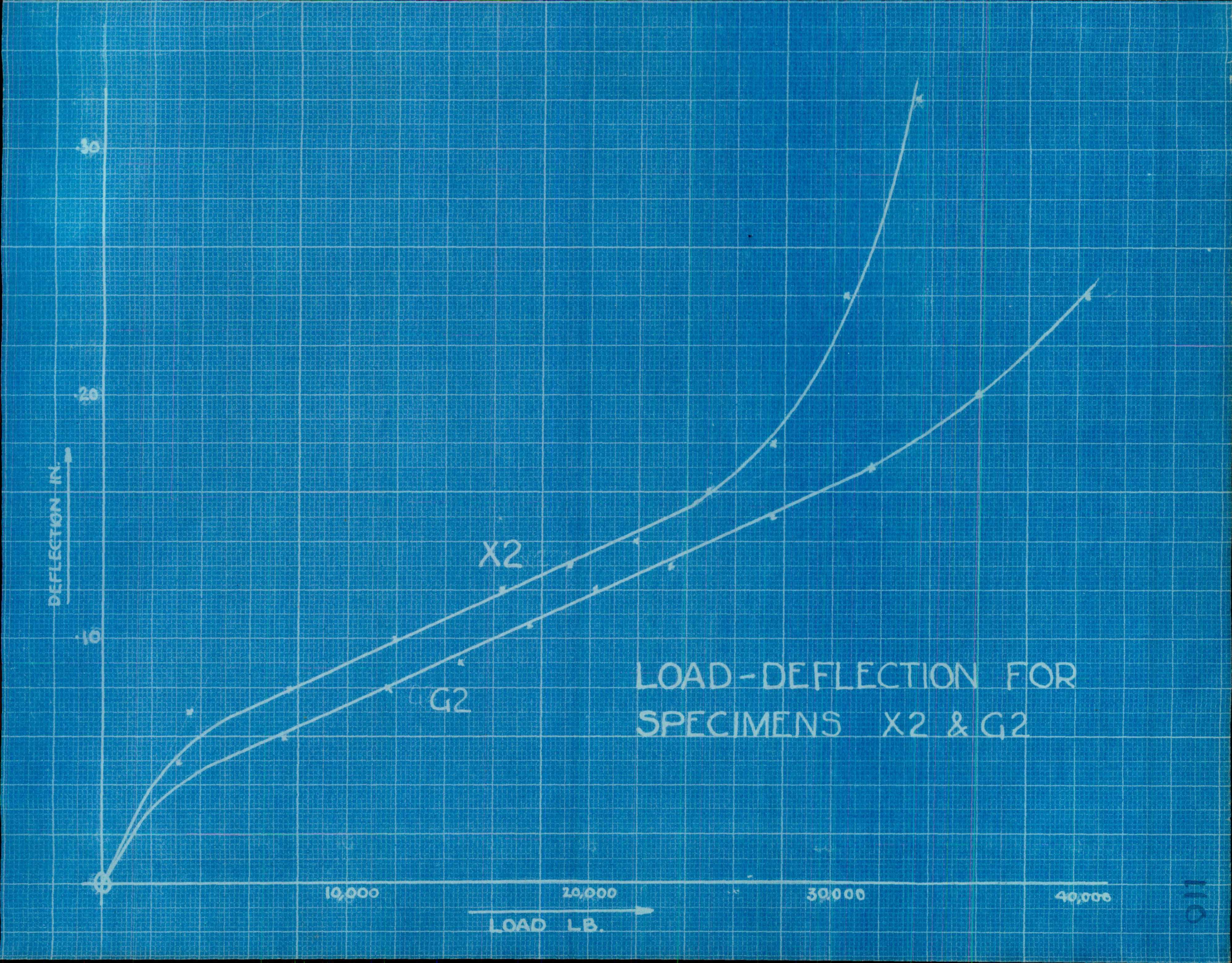
A further examination of the graphs for A7, B4, G2 and X2 shows that the early stages of all these are approximately similar and these specimens were taken as standard. On comparing B3 with B4 it is seen that stages I and II for B3 are not as flat as those for B4 showing that the bed was not so compact. As a result the failing


LOAD - DEFLECTION
FOR SPECIMENS
B3 & B4
B3
B4
DEFLECTION IN
·50
·40
·30
·20
·10
0
10,000
20,000
30,000
40,000
LOAD LB.

LOAD - DEFLECTION FOR SPECIMEN A7



A011



LOAD-DEFLECTION FOR SPECIMENS X2 & G2

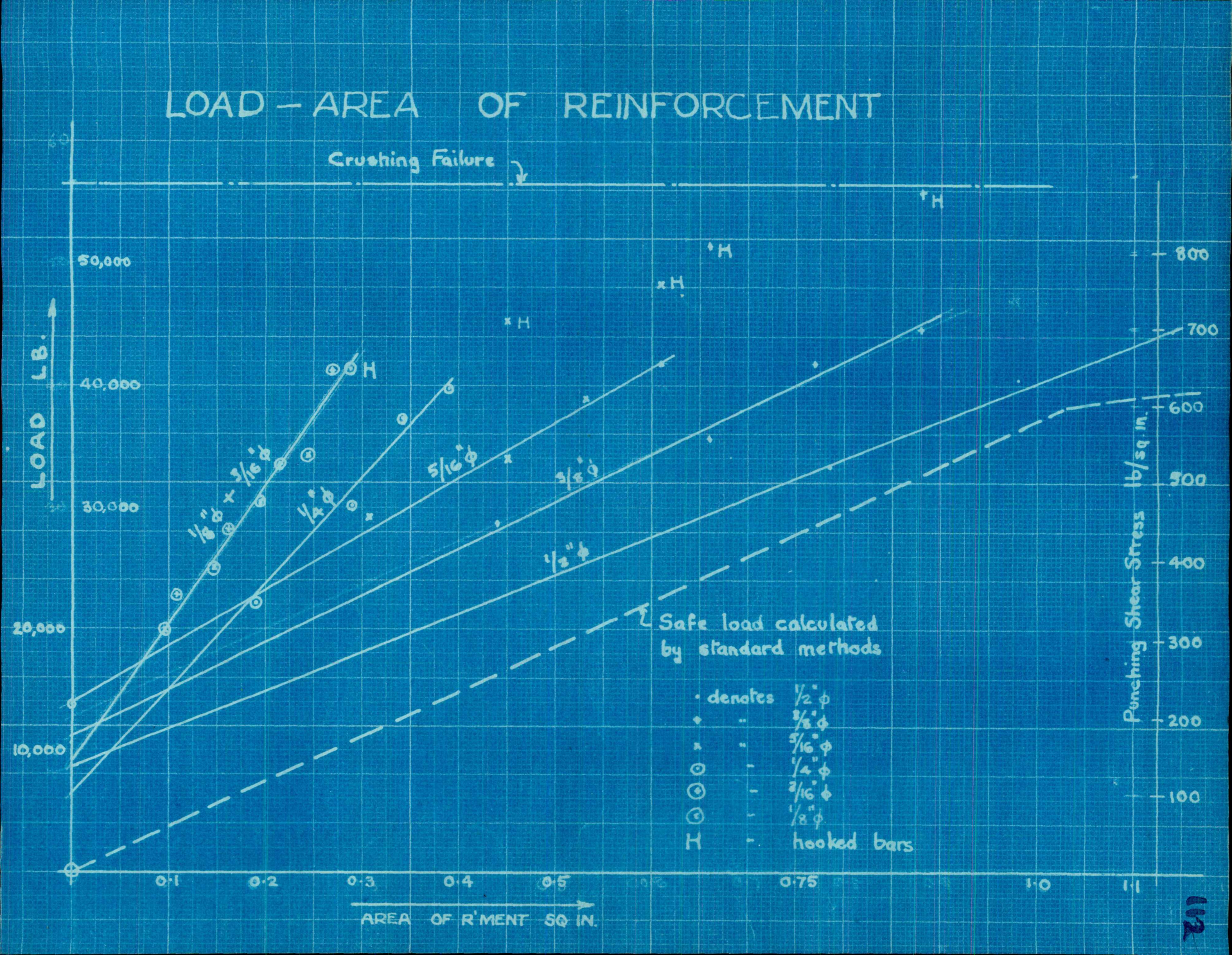
load for B3 is considerably less than that for B4 and it is therefore rejected.

Measurement of Failing Loads.

Using the assumption for the consistency of bed previously mentioned the failing loads for specimens tested on such a bed are given in Tables 6 and 7. Table 6 gives the results for straight bars and Table 7 those for hooked bars. In the tables two results are given for each specimen and also the mean result: it is seen that for any particular specimen the results are remarkably consistent for a material such as concrete, the inconsistent behaviour of which is well-known. As the tests were spread over a fair period the mean of the tests on specimens A is only given in cases where the tests were carried out at approximately the same time.

The results are also shown on the graphs on pages 112-113 These give the relationship between the strength and the area and perimeter of the reinforcement. They are of great value in forming conclusions from the tests.

In addition to the foundation slabs tests were also made for compression and tension. The results of these were as given below: the value given for the compressive strength is the average on tests made on 6 cubes whilst that given for the tensile strength is the average of tests on two specimens.


LOAD – AREA OF REINFORCEMENT
Crushing Failure
H
LOAD LB.
50,000
40,000
30,000
20,000
10,000
1/8" ϕ
3/16" ϕ
1/4" ϕ
5/16" ϕ
3/8" ϕ
1/2" ϕ
Safe load calculated by standard methods
· denotes 1/2" ϕ
◆ " 3/8" ϕ
x " 5/16" ϕ
⊙ - 1/4" ϕ
⊛ - 3/16" ϕ
⊛ - 1/8" ϕ
H - hooked bars
0·1
0·2
0·3
0·4
0·5
0·75
1·0
1·1
AREA OF R'MENT SQ IN.
Punching Shear Stress lb/sq in.
800
700
600
500
400
300
200
100

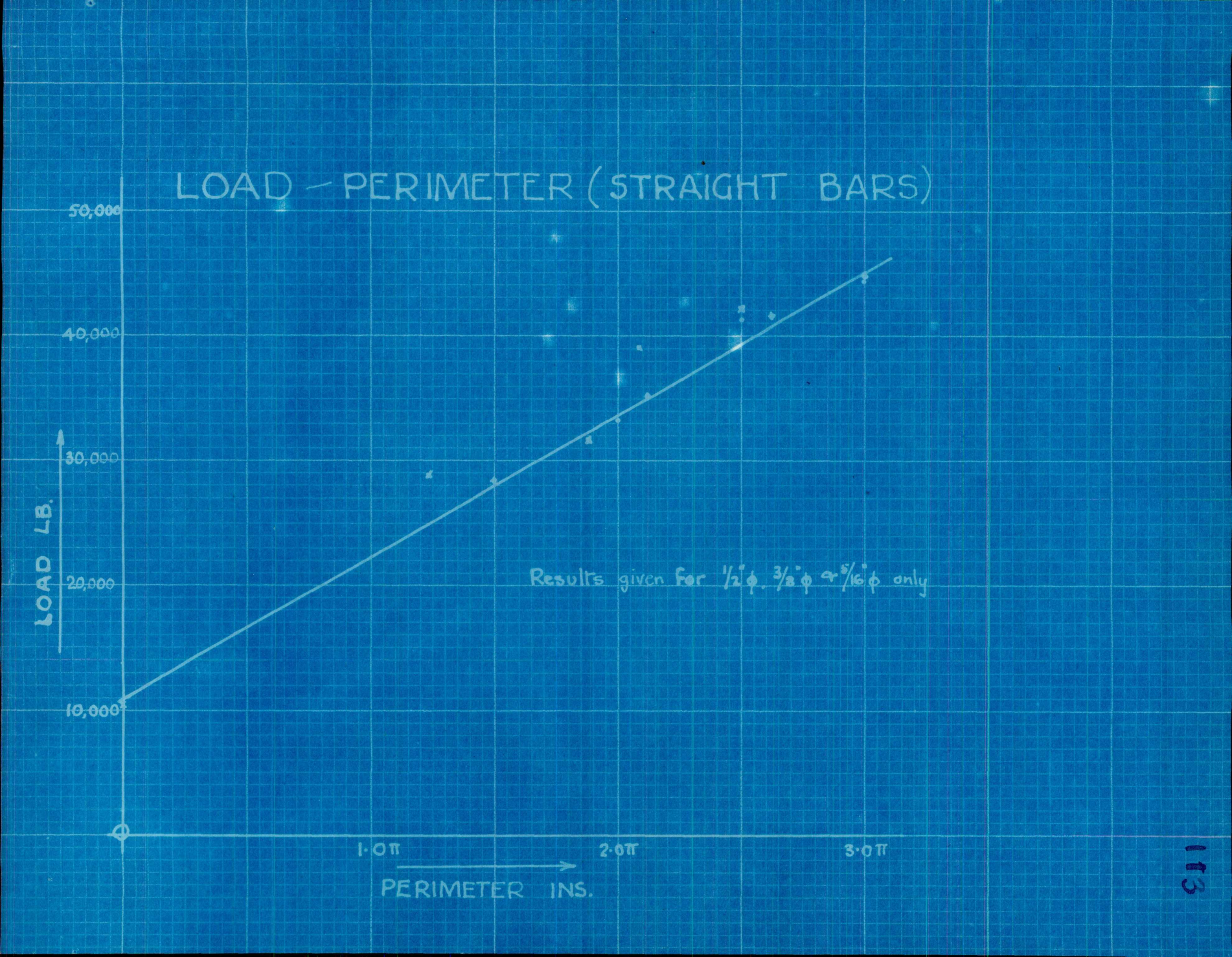

LOAD – PERIMETER (STRAIGHT BARS)
50,000
40,000
30,000
20,000
10,000
LOAD LB.
Results given for 1/2"ϕ, 3/8"ϕ & 5/16"ϕ only
1·0π
2·0π
3·0π
PERIMETER INS.

Table 6 - Tests on Slabs reinforced with Straight bars.

Type of Specimen	Failing Load Specimen No. 1	Failing Load Specimen No. 2	Average Load (lb.)
A2 & A3	10550	10550	10550
A5	12400		13870
A4 & A7	14250	14950	
B	44400	44500	44450
Q	40600	40600	40600
N	32600	33900	33300
C	44200	45100	44700
S	41900	41900	41900
D	34600	36700	35600
R	27400	30100	28750
E	41200	42500	41900
U	39800	38600	39200
F	31300	32500	31900
T	29050	29350	29200
G	38300	41300	39800
V	38200	36600	37400
H	29700	30600	30200
J	23100	22500	22800
W	41200	41500	41350
X	34500	31700	33100
Y	28200	27900	28050
Z	21800	23900	22850
	34200	-	34200
	31200	29700	30450
	25200	24800	25000
	19920	19800	19860
	27600	-	27600

Table 7 - Tests on Slabs reinforced with Hooked Bars.

Type of Specimen	Failing Load Specimen No. 1	Failing Load Specimen No. 2	Average Load (lb.)
K	56600	55000	55800
L 4 in.sq. stan.	53000	49900	51450
L 7 in.sq. stan.	70000	77000	73500
P	49900	46500	48200
O	44100	46400	45250
M	41000	41300	41150

Compressive Strength of Concrete (after 7 days).

Specimens made in October, 1938 - 3540 lb. per sq. in.
Specimens made in March, 1939 - 4100 lb. per sq. in.

Tensile Strength of Concrete (after 7 days).

Specimens made in October, 1938 - 224 lb. per sq. in.
Specimens made in March, 1939 - 287 lb. per sq. in.

Ultimate Bond Stress.

The value obtained from the "pull-out" tests on a $\frac{3}{4}$ in. diameter bar was 608 lb. per sq. in.

Examination of Failures.

The specimens were examined after test and the types of failure which occurred are shown in Plates 9 to 17. An examination of these plates shows the following features:

Plate 9. This is typical of the failure for slabs reinforced with $\frac{1}{2}$ in. and $\frac{3}{8}$ in. diameter bars (i.e. heavy bars relative to thickness of section). It shows four main cracks which run roughly along the diagonals together with cracks along lines parallel to the perimeter of the stancheon.

Plates 10, 11 and 12. These show the type of failure of a series of specimens reinforced with $\frac{1}{8}$ in. diameter bars. It is seen that the failure was caused by four main cracks which run roughly along the centre line of the slab. Comparison of the plates shows that the type of failure is independant of the number of reinforcing bars. Similar failures were noticed in the slabs which were reinforced with 3/16 in. diameter bars.

Plates 13 and 14. These show a failure where the cracks are smaller but more numerous. They consist of two series, one running along the diagonals and the other along the centre lines. It is interesting to note that both E type specimens showed a similar failure.

Plates 15, 16 and 17. These show the failure of specimens reinforced with hooked bars. The cracks in this case are more numerous and evenly distributed than in any other case. In addition to the cracks running into the edges there are also a number of cracks in the centre of the specimen. Plate 16 shows the top of type L specimen after failure; local crushing of the concrete along the edges of the stancheon is the outstanding feature.

The steel was examined after the test and in no case was it found to be stressed beyond the elastic limit.

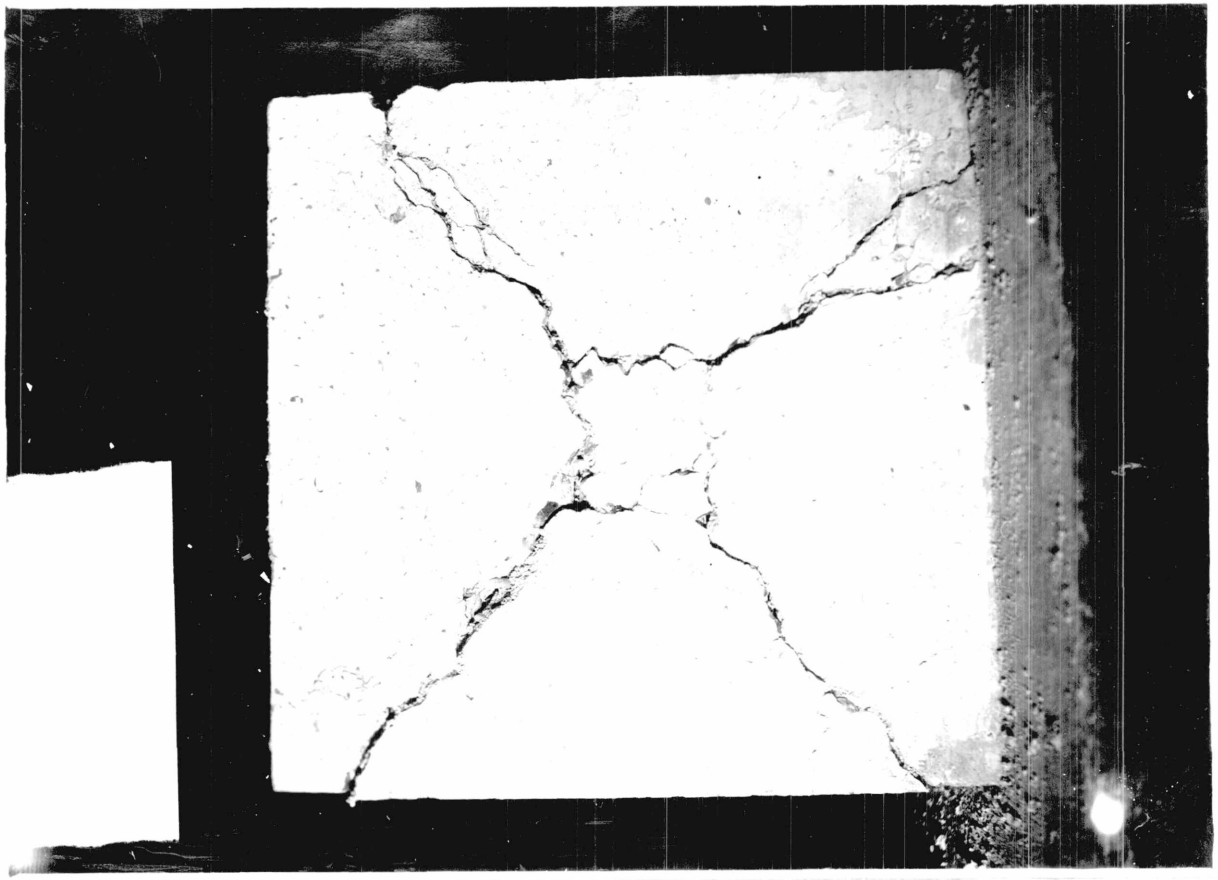


Plate 9 - Failure of Type B Specimen



Plate 10 - Failure of Type α Specimen

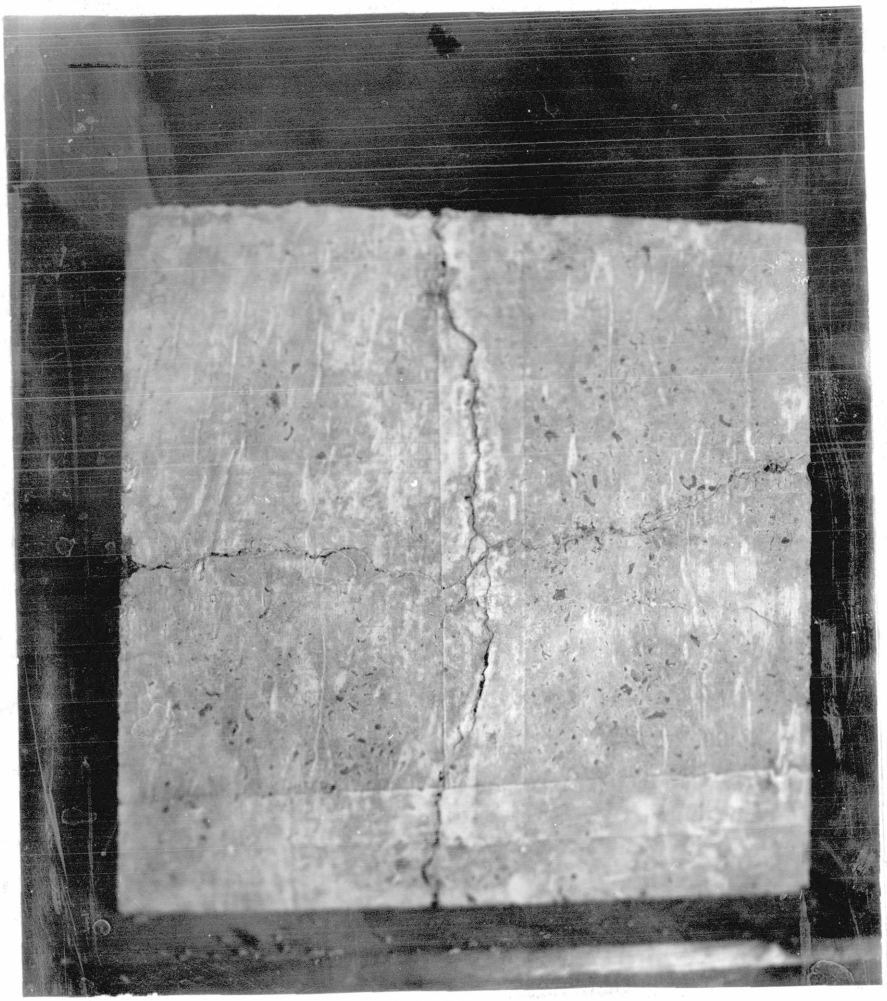


Plate 11 – Failure of Type β Specimen



Plate 12 - Failure of Type 8 Specimen

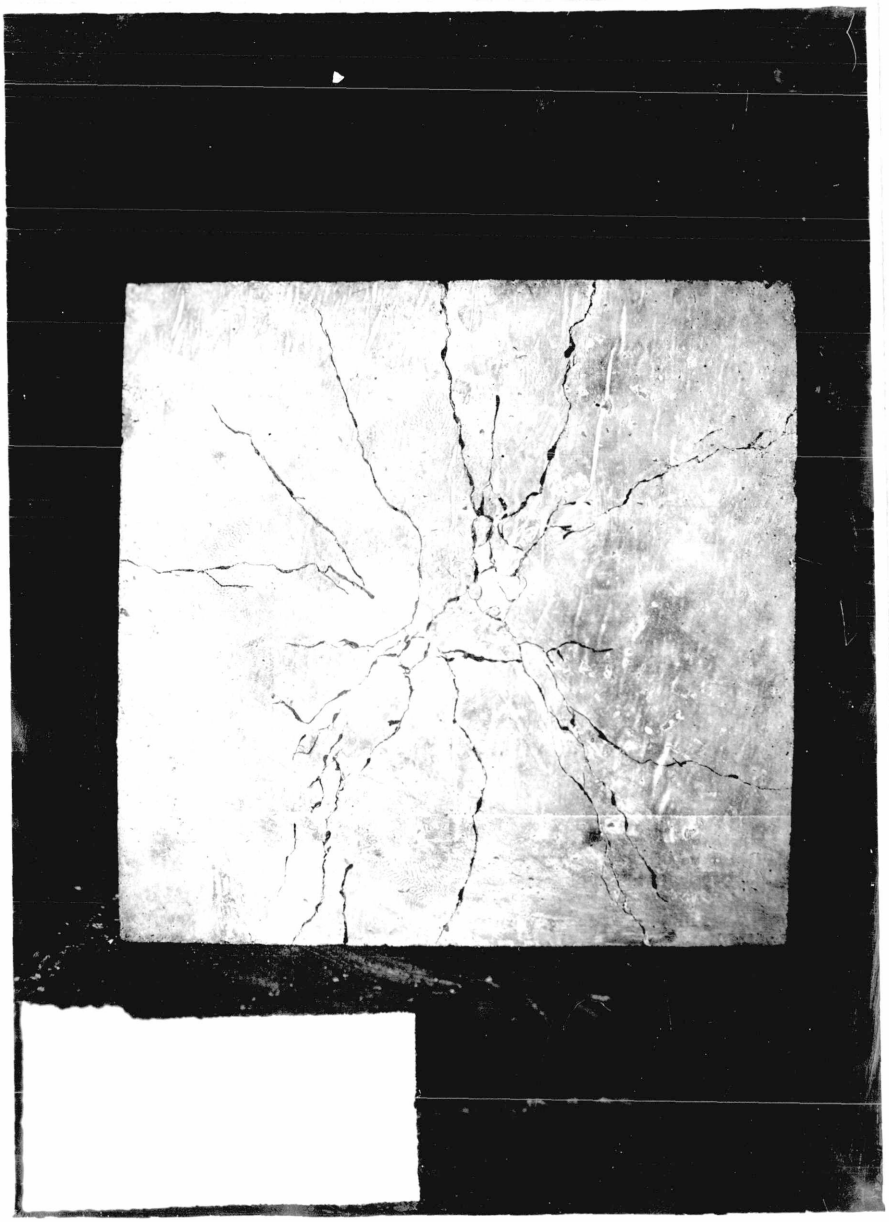


Plate 13 - Failure of Type E Specimen

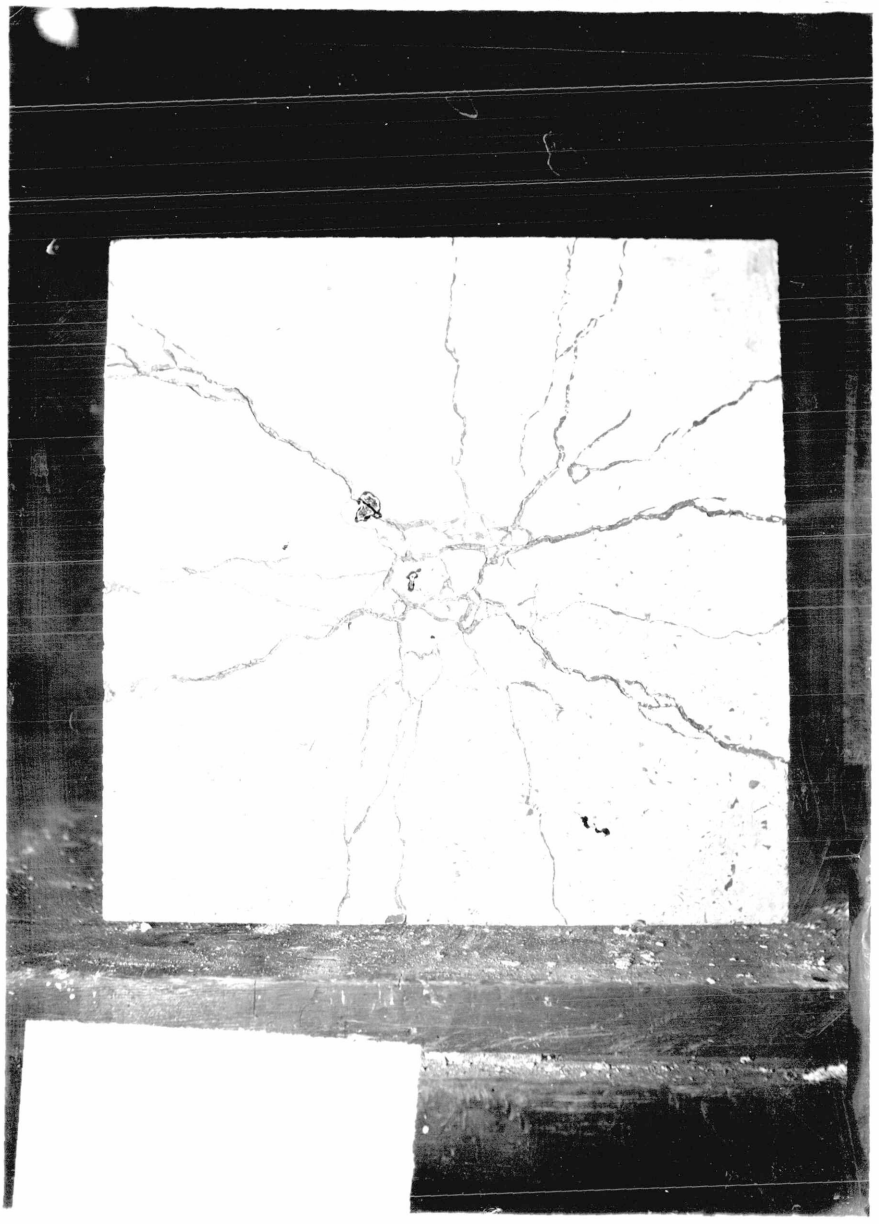


Plate 14 - Failure of Type E Specimen



Plate 15 - Failure of Type L Specimen



Plate 16 - Failure of Type L Specimen (top view)



Plate 17 - Failure of Type K Specimen

Calculation of Theoretical Failing Loads.

In this part of the chapter the various methods for calculating the failing loads are given. Comment and comparison between them and the actual loads is reserved for the following chapter.

1. Failing due to crushing under stancheon.

   (a) Early stages of work c = 3540 lb. per sq. in.
   Failing load = 16 x 3540 = 56600 lb.

   (b) Later stages of work c = 4100 lb. per sq. in.
   Failing load = 16 x 4100 = 65600 lb.

2. Failure due to flexure.

The bending moment at failing load, W, is given by $\frac{W}{4}(x - \frac{4}{3})$ where x is the distance of the centre of gravity of the pressure quadrant from the centre of the stancheon.

For parabolic distribution of pressure x = .3B (where B = width of base) and in the case in point x = 7.2ins To check up x use is made of the unreinforced specimens series A.

Specimens A1 and A3.

Tensile strength = 224 lb. per sq. in.

Resistance moment = $(224 \times 24 \times 4^2 \times \frac{1}{6})$ in. lb.

Failing load = 10550 lb.

Giving x = 6.64 ins.

Specimens A4, A5 and A7.

Tensile strength = 287 lb. per sq. in.

Resistance Moment = $(287 \times 24 \times 4^2 \times \frac{1}{6})$ in. lb.

Average Failing load = 13870 lb.

Giving x = 6.63 in.

These two values check up very well and the value of x taken for the remaining calculations is 6.64 in.

It is stated on page 29 that the bending moment may be expressed by the formula

$$M = \frac{P}{8\beta}$$

where $\beta = \sqrt[4]{\frac{K}{EI.}}$

To test the validity of the formula it can be applied to the specimens A1 and A3, taking K the subgrade modulus as 100 lb. per cu. in. and E for concrete = 2.66 x10^6 lb. per sq. in.

This gives $\beta = \sqrt[4]{\frac{10^2 \times 6}{24 \times 64 \times 2.66 \times 10^6}}$

$$= \frac{1}{51.2}$$

Hence $M = \frac{10550 \times 51.2}{8}$

$$= 67500 \text{ in. lb.}$$

A value which is hopelessly out of proportion to the actual value showing that the mathematical analysis given on pages 26-29 is not applicable to column bases of the type under test.

The calculation of the failing loads due to flexure in the reinforced specimens is made difficult because experiment showed that at failure the steel had not reached its yield point. Calculation by the straight-line no-tension

theory is therefore impossible. This theory is used to calculate the safe loads and the comparison between them and the failing loads gives the true factor of safety. The safe loads are calculated on the following assumptions:

1. Modular ratio = $\frac{40,000}{\text{Cube strength}}$ = 11.3

2. Safe stress in concrete = $\frac{\text{Cube strength}}{3}$ = 1150 lb. per sq. in.

3. The pressure under the base is parabolic.

The depth to the centre of the reinforcement varies with the bar diameter but little error is involved in taking an average value of 3.19 ins.

The calculations are somewhat elementary and they are not given in detail. A table of values for a number of different reinforcement areas is given in the table below

Area of Reinforcement sq. ins.	0.1	0.2	0.4	0.6	0.8	1.0	1.2
Safe Load lb.	3710	7400	14820	22200	29600	37000	39400

3. Failure due to shear.

It has already been pointed out that failure due to shear may develop in two ways, due to plain shear and punching shear.

On page 73 certain assumptions are given with regard to the calculation of the failing loads due to excessive plain shear. On applying the method based on

these assumptions to the problem under consideration the following values are obtained for the failing load, taking the ultimate shear strength of the concrete as $\frac{1}{10}$ of cube strength, i.e. 350 lb. per sq. in. and the yield point stress in the steel as 40,000 lb. per sq. inch.

Area of Reinforcement	None	0.1	0.2	0.4	0.6	0.8	1.0
Failing Load (lb.)	59500	61500	64800	70100	75400	80700	86100

The area of reinforcement is given in square inches each way

If it is assumed (1) that the punching shear is independant of the reinforcement, (2) that the ultimate punching shear stress is twice the plain shear and, (3) that the ground pressure is uniform, then the ultimate load taken by the specimens in punching shear is 44800 lb.

4. Failure due to excessive bond stress.

This can occur due to either excessive shear force or excessive bending moment. In the first case the relationship between the bond stress and shear is given from the formula,

$$S_b = \frac{S}{ao}$$ (see standard text books)

In applying this formula to a column base the following assumptions will be made.

1. That S = upward shear force along planes YY and Y_1Y_1 (see Figure 2).

2. That o = total perimeter of bars cut by planes XX.

3. That the ground pressure is uniform.

4. That in the case under consideration S_b = 608 lb. per sq. in.

The calculation of the failing load is then quite simple and values for different perimeters of bars are given in the table below:

Perimeter of bars. ins.	0.5π	1.0π	1.5π	2.0π	2.5π	3π
Ultimate Load lb.	4050	8100	12150	16200	20300	24300

The relationship between the bond stress and the bending moment is given from the following formula:

$M = S_b \times o \times l \times a$ (see standard text books)

In applying this formula to the case under consideration the following assumptions will be made -

1. That $M = \frac{W}{4}(6.64 - 1.33)$.

2. That l = length from centre of specimen to end of reinforcement, i.e. $10\frac{1}{2}$ ins.

3. That o = total perimeter each way of mesh.

The ultimate load calculated for various bar diameters is given in the table below:

Perimeter of bars. ins.	0.5π	1.0π	1.5π	2.0π	2.5π	3π
Ultimate load lb.	24100	48200	72300	96400	120500	144600

Comparison between loads carried by 7 in. sq. and 4 in. sq. stancheons.

A comparison between the loads carried by these two stancheons is of assistance in determining the cause of failure.

If failure is due to the applied bending moment then the increase in load carried under the larger stancheon is due to the decrease in lever arm from (6.64 in. - 1.33 in.) to (6.64 in. - 2.33 in.) i.e. from 5.31 in. to 4.31 in. which corresponds to a 20% increase in load.

If failure is due to shear along the 45° plane previously mentioned then the increase is due to the increase in area of this plane and it can be shown that such an increase should be of the order of 70 per cent.

If failure is due to punching shear then the increase is due to the increase in the perimeter of the stancheon and it should be of the order of 75 per cent.

On comparing the results given in Table 7, it is seen that the 7 in. x 7 in. stancheon increases the load carried from 51,450 to 73,500 lb., an increase of about 40 per cent. This seems to show that the failure is due to flexure and not shear. The reason for the increase being greater than the estimated one of 20 per cent is probably due to the fact that under the 4 in. sq. stancheon the concrete is stressed almost to failure consequently it is not able to take the full flexural load.

It is interesting to apply the table given on page 31 of the stresses calculated by Westergaards methods to this case. Taking a subgrade modulus of 100 lb. per cu. in. the maximum stress for a 10,000 lb. load 4 ins. wide is 788 lb. per sq. in.: for a 7 in. wide load it is 575 lb. per sq. in. (approximately) thus giving an increase of 37 per cent due to use of a wider load. This theoretical increase agrees very closely with that obtained from the experiment. As Westergaards figures are based on strength in flexure it gives added support to show that the failure of the base was due to this.

CHAPTER IX.

DISCUSSION OF RESULTS OF TESTS ON CONCRETE BASES.

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The discussion of the results of the tests carried out by examining the graphs shown on pages 112-113 which give the relationship between the failing loads and the area and perimeter of reinforcement.

It is seen that the load-area relationship consists of a family of straight lines. Two interesting features are to be noted.

1. That the slope of each line increases as the bar diameter decreases and for the smallest diameter bars (3/16 in. and $\frac{1}{8}$ in.) the line has the same slope.
2. That each line gives approximately the same value of load with no reinforcement.

The load-perimeter graph shows one interesting point, namely, that for the larger diameter bars the load is directly proportional to the perimeter, the values for both $\frac{3}{8}$ in. and $\frac{1}{2}$ in. diameter bars lying on the same straight line. This line intersects the axis of load at the same value as the lines in the load-area graphs. It is important to note that this value of failing load with no reinforcement is almost the same as that taken by the unreinforced specimens.

The graph showing the relationship between load and area of reinforcement for hooked bars shows the

following interesting points:

1. The increase in strength due to the use of a hooked bar instead of a straight bar.

2. The $\frac{1}{4}$ in. diameter hooked bar lies on the same line as the 3/16 in. and $\frac{1}{8}$ in. diameter straight bars.

The tests on the 7 in. square stancheon show that failure of the base is due to flexure and not to shear. This fact is also proved by an examination of the specimens after failure, the cracks always being of the type associated with tension failures.

The points previously mentioned show the cause of the tension failures. The load-perimeter graph shows that for the large diameter bars the failure is due to lack of anchorage. It is seen on plate 9 that the failure took place by cracks along the diagonals.

The load-area graph shows that for small diameter or concrete bars the failure is due to the steel being overstressed. The failure in this case being by cracks along the centre lines (plate 12).

The failure of 5/16 in. and $\frac{1}{4}$ in. diameter bars as shown on plate 13 and from the graphs seems to be due to a combination of both lack of anchorage and over-stressing.

Failure due to bond.

The failing load of the $\frac{1}{2}$ in. and $\frac{3}{8}$ in. bars can be expressed from the following formula

$P = P_o + K_1 d$

where P = failing load

P_o = failing load of unreinforced specimen.

K_1 = a constant independant of bar diameter

d = diameter of bar

The value of K obtained from the experiment = 3620

From theoretical considerations it is obvious that

$K_1 = S \times o \times l \times a$.

where S = bond stress.

o = perimeter of bars.

l = anchorage length.

a = lever arm depth.

In assessing the value for l it must be remembered that failure took place along a diagonal therefore the value taken for l must only be one quarter of the total length of bar.

On substituting for o, l and a in the expression for K_1 the value obtained for S_b is only 256 per sq. in. This value is considerably less than that obtained from the "pull-out" tests but it must be remembered that 256 lb. per sq. in. is the value of the average bond stress and that the maximum value is much greater than this. Hajnal-Konji (18) has shown that the bond stress near a cracked section is very much higher than the average value. These tests show very conclusively that bars used in column bases must have adequate grip length.

The load-area graphs show that for both $\frac{1}{2}$ in. and $\frac{3}{8}$ in. diameter bars the load is proportional to the area and that for no reinforcement the load taken is approximately equal to that taken by the unreinforced specimens. Therefore the relationship between load and area may be expressed as follows:

$$P = P_0 + K_2 A$$

where K_2 = a constant depending on bar diameter and

A = area of reinforcement.

The relationship between K_2 and K_1 is as follows:

$$K_2 = K_1 \times \frac{4A}{d.}$$

The fact that K_2 depends on the bar diameter explains the family of straight lines on the "load-area" graph. It is of interest to compare the values of K_2 obtained from the graph with those obtained from the formula given above when k_1 is taken as 3620 lb. per inch.

Bar Diameter	K_2 (actual)	K_2 (calculated)
$\frac{1}{2}$	32000	29000
$\frac{3}{8}$	40000	38600
5/16	49200	46500
$\frac{1}{4}$	81000	58000

This table shows good agreement for the large diameter bars again showing that their failing load depends on the bond. For the $\frac{1}{4}$ in. diameter bars the actual load greatly exceeds the theoretical (when calculated on bond resistance) but is not as great as that which could be carried if the bar were properly anchored.

The New Code (19) recommends that the grip length for bars be 59d in the case of straight bars and 45d in the case of hooked bars. The length required for the various bars used in the tests is given in the table below:

Bar Diameter	$\frac{1}{2}$in.	$\frac{3}{8}$in.	5/16in.	$\frac{1}{4}$in.	3/16in.	$\frac{1}{8}$in.
Grip Length without hooks	2ft.5$\frac{1}{2}$in.	1ft.10$\frac{1}{4}$in.	1ft.6$\frac{1}{2}$in.	1ft.2$\frac{3}{4}$in.	11in.	7$\frac{3}{8}$in.
Grip Length with hooks	-	1ft. 5in.	1ft.2in.	11$\frac{1}{4}$ in.	-	-

The grip length available was 10$\frac{1}{2}$ in. thus only the 3/16 in. and $\frac{1}{8}$ in. diameter straight bars and the $\frac{1}{4}$ in. diameter hooked bars were adequately anchored, and it is noticed that bases reinforced with these bars all showed the same type of failure.

Failure due to excessive tension in steel.

The "load-area" graph shows that for bars which are properly anchored the relationship between load and

area is independant of the bar diameter and can be expressed as follows:

$$P = P_o + K_3 A$$

where P_o = load taken by unreinforced specimen.

K_3 = a constant independant of bar diameter.

A = area of reinforcement.

The value of K_3 obtained from the experiments was 113,000 lb. per sq. in.

It has already been shown that the bending moment due to a load $P = 5.33 \times \frac{P}{4}$. From theoretical considerations (on the straight line no tension theory) it can be shown that

$$K_3 = \frac{4}{5.33} \times t \times a$$

where t = stress in steel.

a = depth of lever arm.

The value of the steel stress at failure to correspond with the value of K_3 obtained from experiment is 53,000 lb. per sq. in. in a figure which greatly exceeds the yield point stress.

It is a well-known fact that whilst the straight-line no-tension gives fairly reliable results within the working range it is not to be depended on at failing loads. This is true in the case under consideration for it is certain that the steel in the bases was never stressed beyond the yield point.

Dr. Hajnal - Konyi (18) has given a formula for calculating the load at failure which has been developed on the assumption that failure is always due to crushing of the concrete. In his paper he gives a series of curves showing the relationship between $\frac{M}{bd\sigma_y}$ and p (the percentage of reinforcement) for varying values of $\frac{\sigma_y}{\sigma_p}$ where

M = moment at failure.

b = width and d = depth of section.
σ_y = yield point stress in steel
σ_p = prism strength of concrete.
= $\frac{3}{4}$ x cube strength.

In the case under consideration

σ_y = 40,000 lb. per sq. in.

σ_p = $\frac{3}{4}$ x 4100 = 3080 lb. per sq. in.

Therefore $\frac{\sigma_y}{\sigma_p}$ = 13.

Taking p = 0.5 per cent, $\frac{M}{bd\sigma_y}$ = 0.006.

Therefore M = 3.31^2 x 24 x 40000 x 0.006

= 63,100 in.lb.

But M = $\frac{5.33}{4}$ W

Therefore Theoretical failing load = 47500 lb.

A percentage of 0.5 per cent corresponds to an area of 0.396 sq. ins. From the graph the actual failing load for this area = 52,500 lb., a value which shows fairly good agreement with Hajnal - Konyi.

Load Factor.

The value of the safe loads calculated on page 129 are given on the load-area graph. It is seen that in the case where bars are properly anchored the load factor (more commonly called the factor of safety) is approximately 4, the value which is aimed at in design.

Most concrete structures show a much lower value than this. Numerous tests on columns show a load factor varying between 2.5 and 2.8.

The values for the safe load given on page were obtained by assuming a parabolic pressure distribution. If the practical design method assuming uniform pressure had been used the safe load would have been lower and an even greater value obtained for the factor of safety.

Punching Shear Stresses.

It is seen that there is no apparent relationship between the punching shear stress and the failing load. Some specimens failed when the punching shear stress was as low as 200 lb. per sq. in. whilst for others values as high as 700 lb. per sq. in. were obtained. These results show conclusively that there is no object in limiting the punching shear to twice the permissible shear.

Ordinary Shear Stresses.

The values given on page 130 for the ultimate strength in shear are all greater than the actual loads

carried, therefore failure due to shear was not to be expected. It has already been mentioned that this was so and that all the bases failed by flexure.

CHAPTER X.

CONCLUSIONS FROM THE TESTS AND SUGGESTIONS FOR FURTHER INVESTIGATION.

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CONCLUSIONS FROM THE TESTS AND SUGGESTIONS FOR FURTHER INVESTIGATION.

It is intended in this chapter to summarise the results of the work and to suggest some lines on which further research could be carried out.

Conclusions from the tests.

1. The strength of a given base depends on the soil on which it is bedded, the strength being greatest on well compacted sands and least on wet clays.

2. When a column base fails in shear such failure occurs along planes at 45° to the direction of loading. Thus Clause 810 of the New Code can be applied to isolated column bases and is the only shear criterion that should be applied.

3. Failure of a column base never occurs by punching shear. There is therefore no justification for limiting the punching shear to any particular value as in the L.C.C. Regulations.

4. Failure of a thin section column base is generally due to flexure and not shear.

5. It is most important to use bars which are properly anchored. The grip length suggested by the New Code is adequate in column bases.

6. When failure is due to flexure column bases designed on the elastic no tension theory have a load factor (factor of safety) of 4.

Suggestions for further research.

It is realised that the work described in this thesis contributes only a little knowledge towards the problem of the isolated column base. It is suggested that tests carried out on the following lines would provide further useful information.

1. A series of tests on clay beds. These tests could be carried out in two ways,

   (1) by testing a number of different bases on the same clay bed, or,

   (2) by testing the same base on a number of different beds.

The first series of tests would be very difficult to carry out as it is almost an impossibility to get a clay which remains constant under a series of tests. The second series could be carried out by making laboratory tests of the different clays used and comparing these with the strength of a base bedded on them.

2. A series of tests on pyramidal shaped bases. This shape is the one most commonly used in practice and for this reason alone information on its behaviour would be helpful. If, however, the New Code Regulations 810 applies to column base design the shearing stresses will be higher in a pyramidal base than one of uniform thickness and this gives a further reason for such tests.

3. A series of tests on bases which are heavily reinforced against flexure with bars of adequate grip length. Such bases would probably fail in shear and would

some useful information. The Author realises the inadequacy of the present series of tests to provide this information.

4. When failure due to shear has been investigated a series of tests might be carried out to test the efficiency of different types of reinforcement to resist the shear forces. Such reinforcement could consist of either (a) bars in the top of the slab, or (b) bars bent up across the 45 degrees plane of fracture.

APPENDIX.

THE DESIGN OF THE TEST MACHINE.

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THE DESIGN OF THE TEST MACHINE.

It is proposed in the appendix to give the structural calculations for the test machine described on pages 92-95 and as seen in Plate 7 and Figures 20 - 23.

Data.

Capacity of machine = 100 tons.

Safety stress in steel in tension or compression = 8 tons per sq. in.

Safe stress in steel in shear = 6 tons per sq. in.

Top Beam.

Span = 7 ft.: point load = 100 tons.

$M = 100 \times 7 \times 3 = 2100$ in. tons.

$Z = \frac{2100}{8} = 263$ in^3 units.

Use 2 - 22 in. x 7 in. x 75 lb. B.S.B. giving total Z of $2 \times 152.4 = 304.8$ in^3 units.

Rivets at ends.

Load taken by $\frac{7}{8}$ in. rivet in single shear = 3.61 tons

No. of rivets required for shear between beam and stancheon. $= \frac{50}{3.61} = 14$

No. provided = 17

At the ends of the beam the group of rivets have to resist bending in addition to shear.

Bending moment on group

= 50 x 2 = 100 in. tons in bending.

Let P = load taken by rivet furthest away from centre (see Figure 21).

Then resistance of group

$$= 4P\ (7\tfrac{1}{2} + \frac{4\tfrac{1}{2}}{7\tfrac{1}{2}} \times 4\tfrac{1}{2} + \frac{1\tfrac{1}{2}}{7\tfrac{1}{2}} \times 1\tfrac{1}{2})$$

$$= 4P(7.5 + 2.7 + 0.3)$$

$$= 42P \text{ in. tons.}$$

Therefore $P = \frac{100}{42} = 2.37$ tons.

Load taken by group in shear = 50 tons less amount taken by 2 rivets in direct tension

$$= 50 - 2 \times .6 \times 8 = 40.4 \text{ tons.}$$

Therefore Load per rivet in shear $= \frac{40.4}{12} = 3.35$ tons.

Therefore Resultant maximum load on rivet

$$= \sqrt{3.35^2 + 2.37^2} = 4.0 \text{ tons.}$$

Load taken by $\frac{7}{8}$ in. rivet in single shear = 3.61 tons.

Hence there is a slight deficiency but as the two rivets in direct tension will also take a certain amount of bending the joint can be justly assumed safe.

Stancheons.

These are in direct tension and carry a load of 50 tons each.

Area required $= \frac{50}{8} = 6.25$ sq. ins.

Use 1 - 15 in. x 6 in. x 59 lb. B.S.B. giving 17.35. sq.ins.

This heavy section being taken because of the thick web which ensures that bearing load of rivets is greater than single shear load.

Bottom Beam.

Assume that the 100 ton load is spread over the central 2 ft. width of the beam.

Then in the centre of the span

$M = 50(3.5 - 1) \times 12 = 1500$ in. tons.

Required $Z = 187.5$ in ^{3} units.

Use 2 - 18 in. x 6 in. x 55 lb. B.S.B. giving total Z of

$2 \times 93.5 = 187.0$ in ^{3} units.

Rivetted Joint at end.

No. of rivets required for shear between beam and stancheon = 14 as before.

No provided = 17.

Of these 7 are in the 6 in. x 4 in. angle bracket and these take $7 \times 3.61 = 25$ tons.

Therefore the group of rivets in the end of the joist take 25 tons.

Therefore shear load per rivet = 2.5 tons.

Bending Moment on group = 100 in. tons as before.

Let P = load taken by rivet furthest away from centre, then resistance of group

$$= 4P(6 + \frac{1}{2} .3) = 30P.$$

Therefore $P = \frac{100}{30} = 3.33$ tons.

Therefore Resultant maximum load per rivet

$$= \sqrt{2.5^2 + 3.33^2} = 4.16 \text{ tons.}$$

Using 1 in. diameter rivets load taken in single shear = 4.71 tons.

Therefore Joint is safe when 1 in. diameter rivets are used.

Ball and Socket for Centralising Load (see Figure 22).

This is made of chromium steel which can take a safe load of 20 tons per sq. in.

Steel ball = $2\frac{1}{2}$ in. diameter.

Contact area required = $\frac{100}{20}$ = 5. sq. ins.

Surface area given by design

= $2\pi \times 1.25 \times \frac{5}{8}$ = 4.9 sq. ins.

Balance Weights.

The weight required to overcome the frictional resistance of the ram and to pull it back into place after test had to be found experimentally as the co-efficient of friction was not known. It was found that two weights each 250 lb. were required. The ultimate load taken by the wire carrying them was 2000 lb. giving a factor of safety of 8.

Plates for Pulleys carrying Balance Weights.

Span = 9 in. Load = 250 lb.

$M = \frac{9}{4} \times 250 = 546$ in. lb.

$Z = \frac{564}{8 \times 2240} = .0315$ in^3 units.

With 2 in. x $\frac{1}{2}$ in. plate

$Z = 2 \times \frac{1}{4} \times \frac{1}{6} = .084$ in ^{3} units.

Therefore 2 in. x $\frac{1}{2}$ in. plate safe.

Reinforced Concrete Floor.

To design the floor it is assumed that in the worst case the 100 ton load is spread over an area of 7 ft. x 6 ft. 6 ins.

Load per sq. ft. = $\frac{100}{7 \times 6.5} = 2.2$ tons.

Cantilever bending moment on reinforced concrete foundation = $2.2 \times 2240 \times 3^2 \times 6$

= 266,000 in. lb. per ft. width.

Area of steel required = $\frac{266,000}{18000 \times .86 \times 15.}$

= 1.14 sq. ins.

Use 2 - $\frac{7}{8}$ in. diameter bars giving 1.20 sq. ins.

Box carrying sand.

The internal dimensions of the box are 3 ft. 6 ins. sq. It is assumed in design that the 100 ton load is spread over the whole surface giving a surcharge of $\frac{100}{3.5^2} = 8.30$ tons per sq. ft. The sand is assumed to have an angle of repose of 30 degrees, therefore the horizontal pressure exerted by this surcharge

$= 8.30 \times \frac{1 - \sin 30^\circ}{1 + \sin 30^\circ} = \frac{8.30}{3} = 2.75$ tons per sq. ft.

Span of channels at side = 4 ft.0 ins. (say).

Assuming channels partly encastre at ends.

$M = 2.75 \times 4^2 \times 1.0 = 44$ in. tons.

Therefore required $Z = \frac{44}{8} = 5.5$ in ^{3} units.

Use 6 in. x 3 in. x 12.41 lb. Channel having $Z = 7.09$ in. ^{3} units.

Shear at ends $= 2.75 \times 3.5 \times \frac{1}{2} = 4.82$ tons.

Load carried by $\frac{5}{8}$ in. rivet = 1.84 tons.

Use 4 - $\frac{5}{8}$ in. rivets.

TABLE 8 TESTS on TYPE A SPECIMENS.

REINFORCEMENT :- UNREINFORCED

Specimen No: A1		Specimen No: A2	
Date Made	21 - 9 - 38	Date Made	3 - 10 - 38
Date Tested	19 -10 - 38	Date Tested	10 - 10 - 38

Dial Reading	Load lb.	Deflection (in)	Dial Reading	Load lb	Deflection (in)
189	0	00	188	-	.00
200	3130	.08	200	3400	.04
215	7410	.12	215	7600	.08
230	11710	.135	225	10550	failed
231	12000	failed			

Remarks: 28 days test

Remarks: Bed satisfactory

Specimen No: A3		Specimen No: A4	
Date Made	14 - 10 - 38	Date Made	18 - 1 - 39
Date Tested	21 - 10 - 38	Date Tested	25 - 1 - 39

Dial Reading	Load lb	Deflection (in)	Dial Reading	Load lb	Deflection (in)
189	-	.00	198.5	-	.00
200	3130	.07	215	4700	.06
215	7410	.10	230	8980	.07
226	10550	failed	240	11830	.08
			248.5	14250	failed

Remarks: Bed satisfactory

Remarks: Fresh Cement; Bed satisfactory

TABLE 8 (contd) TESTS on TYPE A SPECIMENS.

REINFORCEMENT :- UNREINFORCED

Specimen No: A5			Specimen No: A7		
Date Made	23 - 1 - 39		Date Made	21 - 3 - 39	
Date Tested	30 - 1 - 39		Date Tested	28 - 3 - 39	
Dial Reading	Load lb	Deflection (in)	Dial Reading	Load lb	Deflection (ins)
197.5	-	.00	133.5	-	.00
215	4990	.07	150	4700	.07
230	9270	.10	165	8980	.10
240	12000	.11	180	13260	.11
241.5	12400	failed	186	14950	failed
Remarks: Fresh cement Bed satisfactory			Remarks: Fresh Cement & aggregate Bed satisfactory		

Specimen No: A6		
Date Made	14 - 2 - 39	
Date Tested	21 - 2 - 39	
Dial Reading	Load lb	Deflection (ins)
145	-	.00
155	3420	.09
170	7700	.12
185	11960	.14
185.5	12100	failed

Remarks : Dirty aggregate
Bed satisfactory

TABLE 9 TESTS on TYPE B Specimens

REINFORCEMENT - 6 - $\frac{1}{2}$" diam. bars each way.

Specimen No:	B1	Specimen No:	B2
Date Made	26 - 9 - 38	Date Made	28 - 10 - 38
Date Tested	3 -10 - 38	Date Tested	4 - 11 - 38

Dial Reading 6666666	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
188	-	.00	187.5	-	.00
225	4840	. -	200	3570	.07
250	11960	.24	215	7830	.10
275	19080	.28	230	12110	.12
300	26200	.32	240	14760	.13
325	33320	.38	250	17610	.145
333	41400	failed	260	20460	.16
			270	23310	.17
			285	27580	.175
			300	31860	.19
			315	36130	.21
			330	40410	.23
			343	44400	failed

Remarks : Poor result; Bed not sufficiently Compacted

Remarks: Bed satisfactory

TABLE 9 (contd) TESTS ON TYPE B SPECIMENS

REINFORCEMENT - 6 - ½" DIAM. BARS EACH WAY.

Specimen No:	B3	Specimen No:	B4
Date Made	7 - 12 - 38	Date Made	31 - 1 - 39
Date Tested	14 - 12 - 38	Date Tested	7 - 2 - 39

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
188	-	.00	145	-	.00
200	3420	.16	160	4280	.07
215	7700	.25	175	8550	.095
230	11980	.30	190	12830	.11
240	14830	.33	200	15680	.12
250	17680	.36	210	18530	.13
260	20530	.39	220	21380	.14
270	23380	.41	230	24230	.15
285	27660	.44	245	28500	.165
300	31900	.49	260	32780	.18
302	32500	failed	275	37050	.195
			290	41330	.22
			301	44500	failed

Remarks: Poor result; bad not sufficiently compacted

Remarks: Bed Satisfactory

TABLE 10 TESTS ON TYPE C SPECIMENS.

REINFORCEMENT - 8 - $\frac{3}{8}$" DIAM. BARS EACH WAY

Specimen No:	C1	Specimen No:	C2
Date Made	10 - 10 - 38	Date Made	7 - 11 - 38
Date Tested	17 - 10 - 38	Date Tested	14 - 11 - 38

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
189	-	.00	189	-	.00
200	3130	.10	200	3130	.06
215	7400	.11	215	7400	.09
230	11780	.135	230	11780	.12
245	16050	.15	240	14630	.14
260	20330	.17	250	17480	.15
275	24600	.19	260	20330	.16
290	28880	.20	270	23180	.18
305	33150	.22	285	27460	.20
320	37430	.25	300	31730	.21
335	41700	.27	315	36000	.24
344	44200	failed	330	40280	.26
			345	44900	.31
			347	45100	failed

Remarks: Bed satisfactory

Remarks: Bed satisfactory

TABLE 11 TESTS ON TYPE D SPECIMENS

REINFORCEMENT)- 6 - $\frac{3}{8}$" DIAM. BARS EACH WAY

Specimen No:	D1	Specimen No:	D2
Date Made	5 - 10 - 38	Date Made	8 - 11 - 38
Date Tested	12 - 10 - 38	Date Tested	15 - 11 - 38

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
187.5	-	.00	189	-	.00
200	3570	.06	200	3130	.08
225	10690	.11	215	7400	.14
250	17810	.14	230	11780	.19
260	20460	.15	240	14630	.22
275	24730	.17	250	17480	.24
285	27580	.18	260	20330	.27
300	31860	.20	270	23180	.29
309	34600	failed	285	27460	.32
			300	31730	.35
			315	36000	.41
			317.5	36700	failed

Remarks: Bed satisfactory

Remarks: Bed satisfactory

TABLE 12 TESTS ON TYPE E SPECIMENS

REINFORCEMENT - 8 - 5/16"- DIAM. BARS EACH WAY.

Specimen No:	E1	Specimen No:	E 2
Date Made	18 - 10 - 38	Date Made	11 - 11 - 38
Date Tested	25 - 10 - 38	Date Tested	18 - 11 - 38

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
189	-	.00	190	-	.00
200	3130	.06	200	2850	.06
215	7400	.085	215	7120	.09
230	11780	.10	230	11400	.11
245	16050	.13	240	14250	.12
260	20330	.15	250	17100	.135
275	24610	.17	260	19950	.15
290	28890	.19	270	22800	.16
305	33160	.20	285	27070	.18
320	37440	.24	300	31350	.20
333.5	41200	failed	315	35620	.22
			330	39900	.24
			339	42500	failed

Remarks : Bed satisfactory

Remarks: Bed satisfactory

TABLE 13 TESTS ON TYPE F SPECIMENS

REINFORCEMENT - 6 - 5/16" DIAM. BARS EACH WAY.

Specimen No: F1		Specimen No: F2	
Date Made	7 - 10 - 38	Date Made	15 - 11 - 38
Date Tested	14 - 10 - 38	Date Tested	22 - 11 - 38

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
188	-	.00	190	-	.00
200	3420	.04	200	2850	.05
215	7700	.045	215	7120	.07
230	11980	.055	230	11400	.09
245	16260	.08	240	14250	.10
260	20540	.10	250	17100	.11
275	24810	.125	260	19950	.12
290	29090	.145	270	22800	.135
298	31300	failed	285	27070	.16
			300	31350	.19
			304	32500	failed

Remarks : Bed satisfactory

Remarks: Bed satisfactory

TABLE 14 TESTS ON TYPE G SPECIMENS

REINFORCEMENT - 8 - $\frac{1}{4}$" DIAM. BARS EACH WAY.

Specimen No:	G1	Specimen No:	G2
Date Made	26 - 10 - 38	Date Made	16 - 11 - 38
Date Tested	2 - 11 - 38	Date Tested	23 - 11 - 38

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
189	-	.00	189	-	.00
200	3130	.07	200	3130	.05
215	7400	.10	215	7400	.06
230	11780	.12	230	11780	.08
240	14630	.14	240	14630	.09
250	17480	.15	250	17480	.105
260	20330	.17	260	20330	.12
270	23180	.18	270	23180	.13
285	27460	.21	285	27460	.15
300	31730	.23	300	31730	.17
315	36000	.26	315	36000	.20
323	38300	failed	330	40280	.24
			334	41300	failed

Remarks: Bed fairly satisfactory Result on low side due excessive deflections.

Remarks: Bed satisfactory

TABLE 15 TESTS ON TYPE H SPECIMENS

REINFORCEMENT - 6 - $\frac{1}{2}$" DIAM. BARS EACH WAY.

Specimen No:	H1	Specimen No:	H2
Date Made	25 - 10 - 38	Date Made	18 - 11 - 38
Date Tested	1 - 11 - 38	Date Tested	25 - 11 - 38

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
189	-	.00	190	-	.00
200	3130	.16	200	2850	.07
215	7400	.20	215	7120	.09
230	11780	.23	230	11400	.12
240	14630	.25	240	14250	.13
250	17480	.275	250	17100	.15
260	20330	.31	260	19950	.17
270	23180	.35	270	22800	.19
285	27460	.42	285	27070	.23
293	29700	failed	297.5	30600	failed

Remarks : Bed fairly satisfactory low result due to excessive deflections.

Remarks: Bed satisfactory

TABLE 16 TESTS ON TYPE J SPECIMENS

REINFORCEMENT - 4 - $\frac{1}{4}$" DIAM. BARS EACH WAY

Specimen No.	J1	Specimen No:	J2
Date Made	21 - 10 - 38	Date Made	22 - 11 - 38
Date Tested	28 - 10 - 38	Date Tested	29 - 11 - 38

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
189	-	.00	190	-	.00
200	3130	.06	200	2850	.05
215	7400	.08	215	7120	.07
230	11780	.10	230	11400	.09
240	14630	.13	240	14250	.10
250	17480	.15	250	17100	.13
260	20330	.19	260	19950	.16
270	23100	failed	269	22500	failed

Remarks: Bed satisfactory

Remarks: Bed satisfactory

TABLE 17 TESTS ON TYPE K SPECIMENS

REINFORCEMENT - 8 - $\frac{3}{8}$" DIAM. BARS WITH HOOKS each way

Specimen No:	K1	Specimen No:	K2
Date Made	11 - 10 - 38	Date Made	1 - 11 - 38
Date Tested	18 - 10 - 38	Date Tested	8 - 11 - 38

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
189	-	.00	189.5	-	.00
200	3130	.06	200	3000	.075
215	7400	.085	215	7260	.09
230	11780	.11	230	11540	.11
245	16050	.13	240	14390	.12
260	20330	.15	250	17240	.135
275	24610	.17	260	20090	.145
290	28890	.18	270	22940	.16
305	33160	.20	285	27210	.17
320	37440	.22	300	31490	.18
335	41600	.24	315	35760	.20
350	45900	.27	330	40040	.22
365	50200	.30	345	44300	.245
380	54500	.34	360	48600	.275
388	56600	failed	375	52900	.32
			382.5	55000	failed

Remarks : Bed satisfactory

Remarks: Bed satisfactory

TABLE 18 TESTS ON TYPE L SPECIMENS

REINFORCEMENT - 6 - $\frac{3}{8}$" DIAM. BARS WITH HOOKS EACH WAY.

Specimen : No:	L1	Specimen No:	L2
Date Made	12 - 10 - 38	Date Made	4 - 11 - 38
Date Tested	19 - 10 - 38	Date Tested	11 - 11 - 38

Dial Reading	Load	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
188	-	.00	189	-	.00
200	3420	.06	200	3130	.06
215	7700	.08	215	7400	.10
230	11980	.105	230	11780	.12
245	16260	.125	240	14630	.14
260	20540	.14	250	17480	.15
275	24810	.16	260	20330	.165
290	29090	.18	270	23180	.18
305	33400	.20	285	27460	.20
320	37600	.22	300	31730	.215
335	41900	.24	315	36000	.24
350	46200	.26	330	40280	.26
365	51400	.31	345	44900	.30
374	53000	failed	360	49180	.39
			364	49900	failed

Remarks : Bed satisfactory

Remarks : Bed satisfactory

TABLE 18 (contd) TESTS ON TYPE L SPECIMENS

REINFORCEMENT - 6 - $\frac{3}{8}$" Diam. Bars with Hooks each way.

Specimen No:	L3	Specimen No:	L4
Date Made	13 - 12 - 38	Date Made	16 - 3 - 39
Date Tested	20 - 12 - 38	Date Tested	23 - 3 - 39

Dial Reading ϕϕϕ	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
189	-	.00	132.5	-	.00
200	3136	.05	150	4990	.09
215	7400	.07	165	9250	.12
230	11780	.09	180	13530	.13
240	14630	.10	190	16380	.14
250	17480	.11	200	19230	.15
260	20330	.12	210	22080	.16
270	23180	.14	220	24930	.17
285	27460	.15	230	27780	.18
300	31730	.16	240	30650	.19
315	36000	.18	250	33450	.20
330	40280	.20	265	37800	.21
345	44900	.22	280	42100	.22
360	49180	.24	295	46300	.23
375	53000	.26	310	50600	.24
390	57200	.29	325	54900	.26
405	61500	.33	340	59000	.28
420	65800	.38	355	63400	.31
434	70000	failed	370	67600	.33
			385	72000	.38
			402.5	77000	failed
Remarks:	Bed satisfactory Stancheon 7"x7"		Remarks:	Bed satisfactory Stancheon 7"x7"	

TABLE 19 TESTS ON TYPE M SPECIMENS

REINFORCEMENT - 6 - $\frac{1}{4}$" DIAM. BARS WITH HOOKS EACH WAY

Specimen No:	M1	Specimen No:	M2
Date Made	23 - 11 - 38	Date Made	6 - 12 - 38
Date Tested	30 - 11 - 38	Date Tested	13 - 12 - 38

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
188	-	.00	190	-	.00
200	3420	.06	200	2850	.05
215	7700	.10	215	7120	.08
230	11980	.12	230	11400	.10
240	14830	.135	240	14250	.11
250	17680	.155	250	17100	.12
260	20530	.17	260	19950	.14
270	23380	.185	270	22800	.155
285	27660	.21	285	27070	.175
300	31900	.23	300	31350	.195
315	36200	.28	315	35620	.23
330	40500	.--	330	39900	.28
332	41000	failed	335	41300	failed

Remarks: Bed satisfactory

Remarks: Bed satisfactory

TABLE 20 TESTS ON TYPE N SPECIMENS

REINFORCEMENT - 4 - $\frac{1}{2}$" DIAM. BARS EACH WAY.

Specimen No:	N1	Specimen No:	N2
Date Made	25 - 11 - 38	Date Made	29 -11 -38
Date Tested	2 - 12 - 38	Date Tested	6 -12 -38

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
190	-	.00	189	-	.00
200	2850	.06	200	3130	.05
215	7120	.08	215	7400	.07
230	11400	.10	230	11780	.095
240	14250	.12	240	14630	.11
250	17100	.13	250	17480	.12
260	19950	.145	260	20330	.135
270	22800	.16	270	23180	.145
285	27070	.17	285	27460	.16
300	31350	.19	300	31730	.18
304	32600	failed	308	33900	failed

Remarks: Bed satisfactory

Remarks: Bed satisfactory

TABLE 21 TESTS ON TYPE O SPECIMENS

REINFORCEMENT - 6 - $^{5}/_{16}$" DIAM. BARS WITH HOOKS EACH WAY

Specimen No:	01	Specimen No:	02
Date Made	30 - 11 - 38	Date Made	9 - 12 - 38
Date Tested	7 - 12 - 38	Date Tested	16 - 12 - 38

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
190	-	.00	189	-	.00
200	2850	.05	200	3130	.08
215	7120	.07	215	7400	.13
230	11400	.08	230	11780	.17
240	14250	.09	240	14630	.20
250	17100	.105	250	17480	.23
260	19950	.12	260	20330	.25
270	22800	.13	270	23180	.28
285	27070	.15	285	27460	.32
300	31350	.16	300	31730	.36
315	35620	.19	315	36000	.37
330	39900	.22	323	38200	failed
339.5	44100	failed			

Remarks: Bed satisfactory

Remarks: Poor result
Bed not sufficiently consolidated.

TABLE 21 (contd) TEST ON TYPE O SPECIMENS

REINFORCEMENT - 6 - 5/16" DIAM. BARS WITH HOOKS EACH WAY.

Specimen No:	03	Specimen No:
Date Made	15 - 3 - 39	Date Made
Date Tested	22 - 3 - 39	Date Tested

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
132.5	-	.00			
150	4990	.06			
165	9250	.08			
180	13530	.10			
190	16380	.115			
200	19230	.125			
210	22080	.135			
220	24930	.15			
230	27780	.16			
240	30650	.18			
250	33450	.20			
265	37800	.23			
280	42100	.28			
295	46400	failed			

Remarks: Bed satisfactory

TABLE 22 TESTS ON TYPE P SPECIMENS

REINFORCEMENT - 8 - 5/16" DIAM. BARS WITH HOOKS EACH WAY.

Specimen No:	P1	Specimen No:	P2
Date Made	2 - 12 - 38	Date Made	12 - 12 - 38
Date Tested	9 - 12 - 38	Date Tested	19 - 12 - 38

Dial Reading ¢¢¢¢¢¢	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
188	-	.00	188	-	.00
200	3420	.05	200	3420	.07
215	7700	.07	215	7700	.10
230	11980	.09	230	11980	.12
240	14830	.10	240	14830	.13
250	17680	.11	250	17680	.15
260	20530	.12	260	20530	.16
270	23380	.14	270	23380	.17
285	27660	.15	285	27660	.19
300	31900	.17	300	31900	.21
315	36200	.18	315	36200	.23
330	40500	.21	330	40500	.265
345	44800	.25	345	44800	.--
360	49000	.32	357	46500	failed
363	49900	failed			

Remarks: Bed satisfactory

Remarks: Bed fairly satisfactory

TABLE 24 TESTS ON TYPE Q SPECIMENS

REINFORCEMENT - 5 - ½" DIAM. BARS EACH WAY

Specimen No:	Q1	Specimen No:	Q2
Date Made	3 - 1 - 39	Date Made	11 - 1 - 39
Date Tested	10 - 1 - 39	Date Tested	18 - 1 - 39

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
189	-	.00	188.5	-	.00
200	3130	.09	200	3270	.06
215	7400	.11	215	7540	.08
230	11780	.13	230	11920	.09
240	14630	.15	240	14770	.10
250	17480	.16	250	17620	.115
260	20330	.18	260	20470	.125
270	23180	.19	270	23320	.135
285	27460	.21	285	27600	.15
300	31730	.24	300	31870	.165
312	35100	failed	315	36140	.185
			322	40600	failed

Remarks: Bed fairly satisfactory: Result on low side due to excessive deflection.

Remarks: Bed satisfactory

TABLE 23 (contd) TESTS ON TYPE Q SPECIMENS

REINFORCEMENT - 5 - $\frac{1}{2}$" DIAM. BARS EACH WAY

Specimen No:	Q3	Specimen No:
Date Made	1 - 2 - 39	Date Made
Date Tested	8 - 2 - 39	Date Tested

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
145$\frac{1}{2}$5	-	.00			
160	4130	.08			
175	8400	.11			
190	12680	.125			
200	15530	.14			
210	18380	.15			
220	21230	.16			
230	24080	.17			
245	28350	.185			
260	32600	.20			
275	36900	.22			
288	40600	failed			

Remarks: Bed satisfactory

TABLE 24 TESTS ON TYPE R SPECIMENS

REINFORCEMENT - 4 - $\frac{3}{8}$" DIAM. BARS EACH WAY

Specimen No:	R1	Specimen No:	R2
Date Made	4 - 1 - 39	Date Made	17 - 1 - 39
Date Tested	11 - 1 - 39	Date Tested	24 - 1 - 39

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
188	-	.00	198	-	.00
200	3420	.05	215	4840	.08
215	7700	.07	230	9120	.10
230	11980	.09	240	11970	.11
240	14830	.10	250	14820	.12
250	17680	.11	260	17670	.13
260	21530	.13	270	20520	.14
270	23380	.14	285	24790	.16
284	27400	failed	300	29070	.18
			303.5	30100	failed

Remarks: Bed satisfactory

Remarks: Bed satisfactory

TABLE 25 TESTS ON TYPE S SPECIMENS

REINFORCEMENT - 7- $\frac{3}{8}$" DIAM. BARS EACH WAY.

Specimen No: S1			Specimen No: S2		
Date Made	5 - 1 - 39		Date Made	16 - 1 - 39	
Date Tested	12 - 1 - 39		Date Tested	23 - 1 - 39	
Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
189	-	.00	201	-	.00
200	3130	.07	215	3990	.07
215	7400	.09	230	8260	.10
230	11780	.11	240	11110	.11
240	14630	.12	250	13960	.12
250	17480	.135	260	16810	.135
260	20330	.145	270	19660	.14
270	23180	.155	285	23930	.16
285	27460	.17	300	28200	.175
300	31730	.19	315	32500	.19
315	36000	.21	330	36800	.21
330	40200	.23	345	41100	.24
336	41900	failed	347	41900	failed
Remarks : Bed satisfactory			Remarks: Bed satisfactory		

TABLE 26 TESTS ON TYPE T SPECIMENS

REINFORCEMENT - 4 - 5/16" DIAM. BARS EACH WAY.

Specimen No: T1			Specimen No: T2		
Date Made 9 - 1 - 39			Date Made 25 - 1 - 39		
Date Tested 16 - 1 - 39			Date Tested 1 - 2 - 39		
Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
188.5	-	.00	198.5	-	.00
200	3270	.07	215	4700	.07
215	7540	.09	230	8980	.09
230	11920	.11	240	11830	.10
240	14770	.125	250	14680	.11
250	17620	.14	260	17530	.125
260	20470	.15	270	20380	.135
270	23320	.17	285	24660	.16
285	27600	.21	300	29000	.21
290.5	29050	failed	301.5	29350	failed
Remarks: Bed satisfactory			Remarks: Bed satisfactory		

TABLE 27 TESTS ON TYPE U SPECIMENS

REINFORCEMENT - 7 - 5/16" DIAM. BARS EACH WAY.

Specimen No: U1		Specimen No: U2	
Date Made	6 - 1 - 39	Date Made	24 - 1 - 39
Date Tested	13 - 1-39	Date Tested	31 - 1 - 39

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflecti (ins)
189	-	.00	198.5	-	.00
200	3130	.06	215	4700	.07
215	7400	.09	230	8980	.09
230	11780	.10	240	11830	.105
240	14630	.12	250	14680	.12
250	17480	.13	260	17530	.13
260	20330	.145	270	20380	.14
270	23180	.16	285	24660	.16
285	27460	.18	300	29000	.17
300	31730	.19	315	33200	.19
315	36000	.24	330	37500	.24
328	39800	failed	334	38600	failed

Remarks : Bed satisfactory

Remarks: Bed satisfactory

TABLE 28 TESTS ON TYPE V SPECIMENS

REINFORCEMENT - 7 - $\frac{1}{4}$" DIAM. BARS EACH WAY.

Specimen No: V1			Specimen No: V2		
Date Made	10 - 1 - 39		Date Made	30 - 1 - 39	
Date Tested	17 - 1 - 39		Date Tested	6 - 2 - 39	
Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
188	-	.00	146.5	-	.00
200	3420	.06	165	5280	.09
215	7700	.08	180	9550	.11
230	11980	.10	190	12400	.12
240	14830	.12	200	15250	.13
250	17680	.13	210	18100	.14
260	20530	.14	220	20950	.155
270	23380	.15	230	23800	.165
285	27660	.17	245	28070	.185
300	31900	.19	260	32350	.21
315	36250	.23	275	36600	failed
322	38200	failed			
Remarks : Bed satisfactory			Remarks : Bed satisfactory		

TABLE 29 TESTS ON TYPE W SPECIMENS

REINFORCEMENT - 10 - 3/16" DIAM. BARS EACH WAY

Specimen No: W1		Specimen No: W2	
Date Made	13 - 2 - 39	Date Made	2 - 3 - 39
Date Tested	20 - 2 - 39	Date Tested	9 - 3 - 39

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
141	-	.00	138.5	-	.00
160	5420	.08	155	4700	.07
175	9690	.11	170	8980	.09
190	13960	.12	185	13250	.10
200	16810	.13	200	17530	.12
210	19660	.14	210	20380	.13
220	22510	.155	220	23230	.14
230	25360	.17	230	26080	.15
240	28210	.185	240	28930	.16
250	31060	.22	250	31780	.18
260	33910	.28	260	34630	.20
270	36800	failed	275	39910	.26
			283	41200	failed

Remarks: Bed satisfactory but low result due to poor aggregate.

Remarks: Bed satisfactory

TABLE 29 (contd) TESTS ON TYPE W SPECIMENS.

REINFORCEMENT - 10 - 3/16" DIAM. BARS EACH WAY.

Specimen No:	W3	Specimen No:	
Date Made	22 - 3 - 39	Date Made	
Date Tested	29 - 3 - 39	Date Tested	

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
133.5	-	.00			
150	4700	.07			
165	8980	.09			
180	13250	.11			
190	16100	.12			
200	18950	.13			
210	21800	.14			
220	24650	.16			
230	27500	.17			
240	30350	.185			
250	33200	.205			
265	37480	.26			
279	41500	failed			

Remarks: Bed satisfactory

TABLE 30 TESTS ON TYPE X SPECIMENS.

REINFORCEMENT - 8 - 3/16" DIAM. BARS EACH WAY

Specimen No: X1			Specimen No: X2		
Date Made 8 - 2 - 39			Date Made 1 - 3 - 39		
Date Tested 15 - 2 - 39			Date Tested 8 - 3 9 39		
Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflectn (ins)
143	-	.00	143	-	.00
160	4840	.10	155	3420	.07
175	9120	.13	170	7700	.08
190	13390	.15	185	11980	.10
200	16240	.16	200	16260	.12
210	19090	.18	210	19110	.13
220	21940	.195	220	21960	.14
230	24790	.22	230	24810	.16
240	27640	.28	240	27660	.18
246	29400	failed	250	30510	.24
			260	33360	.32
			"864	34500	failed
Remarks: Bed satisfactory but low result due to poor aggregate.			Remarks: Bed satisfactory		

TABLE 30 (contd) TESTS ON TYPE X SPECIMENS

REINFORCEMENT - 8 - 3/16" DIAM. BARS EACH WAY.

Specimen No: X3			Specimen No:		
Date Made	14 - 3 - 39		Date Made		
Date Tested	21 - 3 - 39		Date Tested		
Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
133	-	.00			
150	4840	.07			
165	9120	.09			
180	13390	.11			
190	16240	.12			
200	19090	.13			
210	21940	.145			
220	24790	.16			
230	27640	.18			
240	30490	.27			
244	31700	failed			

Remarks: Bed satisfactory

TABLE 31 TESTS ON TYPE Y SPECIMENS.

REINFORCEMENT - 6 - 3/16" DIAM. BARS EACH WAY.

Specimen No: Y1		Specimen No: Y2	
Date Made	7 - 2 - 39	Date Made	28 - 2 - 39
Date Tested	14 - 2 - 39	Date Tested	7 - 3 - 39

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflectn (ins)
140	-	.00	142	-	.00
160	5700	.06	155	3710	.08
175	9980	.08	170	7980	.10
190	14250	.10	185	12260	.12
200	17100	.11	200	16530	.14
210	19950	.12	210	19380	.155
220	22800	.14	220	22230	.19
230	25650	.18	230	25080	.26
239	28200	failed	240	27900	failed

Remarks: Bed satisfactory

Remarks: Bed satisfactory

TABLE 32 TESTS ON TYPE Z SPECIMENS

REINFORCEMENT - 4 - 3/16" DIAM. BARS EACH WAY.

Specimen No: Z1			Specimen No: Z2		
Date Made 6 - 2 - 39			Date Made 22 - 2 - 39		
Date Tested 13 - 2 - 39			Date Tested 29 - 2 - 39		
Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflectn (ins)
145.5	-	.00	142	-	.00
160	4130	.07	155	3710	.06
175	8400	.10	170	7980	.08
190	12680	.11	185	12260	.095
200	15530	.13	200	16530	.14
210	18380	.16	210	19380	.18
220	21230	.22	220	22230	.24
222	21800	failed	226	23900	failed
Remarks: Bed satisfactory			Remarks: Bed satisfactory		

TABLE 33 TESTS ON TYPE ∝ SPECIMENS

REINFORCEMENT - 20 - $\frac{1}{8}$" DIAM. BARS EACH WAY.

Specimen No:	∝ 1	Specimen No:	
Date Made	20 - 3 - 39	Date Made	
Date Tested	27 - 3 - 39	Date Tested	

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflectn (ins)
137	-	.00			
150	3710	.06			
165	7980	.08			
180	12260	.10			
190	15110	.11			
200	17960	.12			
210	20810	.13			
220	23660	.14			
230	26510	.16			
240	29360	.20			
250	32210	.24			
256	34200	failed			

Remarks: Bed satisfactory

TABLE 34 TESTS ON TYPE β SPECIMENS

REINFORCEMENT - 16 - $\frac{1}{8}$" DIAM. BARS EACH WAY

Specimen No: β 1		Specimen No: β 2	
Date Made	21 - 2 - 39	Date Made	9 - 3 - 39
Date Tested	28 - 2 - 39	Date Tested	16 - 3 - 39

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
141.5	-	.00	133	-	.00
155	3850	.08	150	4840	.06
170	8120	.10	165	9120	.085
185	12390	.12	180	13390	.105
200	16670	.135	190	16240	.115
210	19520	.145	200	19090	.13
220	22370	.18	210	21940	.16
230	25220	.24	220	24790	.21
240	28170	.29	230	27640	.28
251	31200	failed	237	29700	failed

Remarks: Bed satisfactory

Remarks: Bed satisfactory

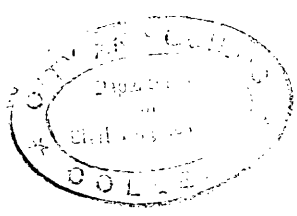


TABLE 35 TESTS ON TYPE Y SPECIMENS

REINFORCEMENT - 12 -$\frac{1}{8}$" DIAM BARS EACH WAY.

Specimen No:	Y 1	Specimen No:	Y 2
Date Made	20 - 2 - 39	Date Made	8 - 3 - 39
Date Tested	27 - 2 - 39	Date Tested	15 - 3 - 39

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflection (ins)
141.5	-	.00	132.5	-	.00
155	3850	.06	150	4990	.07
170	8120	.08	165	8270	.10
185	12390	.09	180	12550	.115
200	16670	.11	190	15400	.125
210	19520	.12	200	18250	.16
220	22370	.18	210	21100	.24
230	25200	failed	219.5	24800	failed

Remarks : Bed satisfactory

Remarks: Bed satisfactory

TABLE 37 TESTS ON TYPE △ SPECIMENS

REINFORCEMENT - 8 - $\frac{1}{8}$" DIAM. BARS EACH WAY.

Specimen No: △ 1		Specimen No. △ 2	
Date Made	15 - 2 - 39	Date Made	6 - 3 - 39
Date Tested	22 - 2 - 39	Date Tested	13 - 3 - 39

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflectn (ins)
142	-	.00	133	-	.00
155	3710	.08	150	4850	.08
170	7980	.10	165	8130	.11
185	12240	.15	180	12410	.13
200	16530	.20	190	15260	.14
210	19380	.32	200	18110	.26
212	19920	failed	202.5	19800	failed

Remarks: Bed satisfactory

Remarks: Bed satisfactory

TABLE 37 TESTS ON TYPE ϵ SPECIMENS

REINFORCEMENT :-- 1 Layer of No.22 gauge expanded metal each way.

Specimen No. ϵ 1		Specimen NO:	
Date Made	27 - 3 - 39	Date Made	
Date Tested	3 - 4 - 39	Date Tested	

Dial Reading	Load lb	Deflection (ins)	Dial Reading	Load lb	Deflectn (ins)
131	-	.00			
150	5420	.06			
165	9690	.08			
180	13960	.10			
190	16810	.11			
200	19660	.12			
210	22510	.14			
220	25360	.17			
228	27600	failed			

Remarks : Bed satisfactory