The ultimate aims of this paper are to investigate the response of a prestressed concrete pavement slab to an applied load, and to determine the manner by which the slabs fail.

Towards these ends, the modes by which materials in general react to load has been studied; and critical strain in tension, or in shear, as applicable, has been suggested as the criterion for predicting failure.

The behaviour of concrete under the action of an external load has also been studied. It has been observed that critical strain in flexural tensile fracture could be used to predict failure in concrete pavement slabs.

The response of the subgrade to load has also been examined. It has been observed that the simple k-concept can be safely used for pavement analysis, so long as k is evaluated in a manner which best suits the particular problem.

The final step is to study the behaviour of prestressed concrete pavement slabs.

PART 5

THE LOAD CARRYING MECHANISM AND THE MODE OF FAILURE OF PRESTRESSED CONCRETE PAVEMENT SLABS UNDER CENTRAL LOADS
5. THE LOAD CARRYING MECHANISM AND THE
MODE OF FAILURE OF PRESTRESSED CONCRETE
PAVEMENT SLABS UNDER CENTRAL LOADS

5.1 The Approach

Aims

5.1.1

i) The main aims of this paper are to
provide the answers to the following questions:

a) how does a prestressed concrete
pavement slab respond to load?

b) how does a prestressed concrete
pavement slab fail?

c) how to predict, quantitatively,
the response of the slab to load?

ii) The general characteristics of prestressed
concrete pavements have been described in Part I.
Utilizing in part the relevant observations and
deductions that have been made in Parts 2, 3 and 4,
the theory that has been developed to provide the answers to the questions that have just been enumerated will be presented in this final portion. Experiments that have been carried out to investigate the validity of the theory will also be described.

The Rationale For Central Load Design

5.1.2

1) The work is confined to square slabs, loaded centrally. It is recognized that, in the case of pavement slabs, edge loads are more critical than are interior loads, with corner loads being even more critical. The point is that, as laid operationally, prestressed pavements are jointless for all practical purposes, unlike conventional concrete slabs where the joints are closely spaced and where the corners are consequently numerous. The whole of the prestressed pavement, therefore, need not be designed to carry a load that occurs only at incidental corners.

The loading of edges will occur infrequently, as most of the traffic (aircraft or vehicular) will keep away from the edges of the slab.
It is therefore uneconomical to base the design thickness of the whole slab on the need for carrying infrequent edge loads. The slab can be reinforced, or thickened, at the edges to accommodate stray edge traffic.

The rationale for designing prestressed concrete pavement slabs on the basis of central loading is therefore sound.

Analytical vs Empirical Approach

Some of the variables associated with pavement design are: the repetition of loads, the variability of the loads, tracking effects, superficial stresses due to tyre distortions, and fluctuating hygrothermal effects of the environment. In view of the nature of these variables it may not appear feasible to incorporate all these factors into a theory; in fact, it may not even be possible to do so. It may therefore seem more worthwhile to design pavements on the basis of experience, observation and tests, and on consequent empirical
rules—of—thumb, rather than on 'theory'. The Road Research Laboratory, UK, for instance, has set up criteria for the design of conventional concrete pavements based mainly on experience. The US Corps of Engineers designs on the basis of tests.

The fact remains, of course, that though it may not be possible to incorporate all the factors into an integrated, and valid, equation of state, it is usually possible to study the effects of each of these factors separately. To theorize on the effects of a single load is therefore at least a step on the path to understanding; and a validated theory provides flexibility and saves time, materials and money.

5.2 Previous Research Work

Description of The Work

5.2.1

In the immediate area of research relating to the analysis of a prestressed concrete pavement slab, the author has knowledge of original works as noted below:
i) Carlton & Ruth Behrmann - 1958: tested 0.2 in thick x 16.6 in$^2$ model gypsum cement slabs over a natural rubber 'subgrade', at prestresses upto 600 lbs/in$^2$; obtained the general pattern of cracking.

ii) Osawa - 1963: gave a theoretical quantitative analysis of pavement behaviour.

iii) Levi - as quoted by Osawa, who indicated only the pattern of cracking that Levi assumed in his analysis.

iv) Cot & Becker - as quoted by Osawa, who indicated only the pattern of cracking that Cot & Becker assumed in their analysis: as quoted by Melville, who indicated the expressions they had derived for radial and tangential moments.

v) Sale, Hutchinson & Carlton - 1961: developed an equation for moments, (including the effects of coverages) based on semi-empirical data.
vi) Christensen & Janes - 1963: tested a number of 1-in thick x 8 ft$^2$ slabs with a view to verifying Osawa's theory.

Observations
5.2.2

i) Carlton & Behrmann reported the initial development of radial cracks at the bottom of the slabs. No method of analysis was given by them. Cot & Becker, as well as Osawa, adopted this failure pattern while making their analysis.

Levi, on the other hand assumed an initial circular crack at the bottom of the slab.

The pattern of failure therefore needs to be verified.

ii) Salc, Hutchinson and Carlton's equation for moments is based on semi-empirical data. The data should be more fundamental so as to make the analysis more complete.
iii) Osawa's theory is based on the concept that the moment-curvature relation is linear up to the bottom cracking stage, after which the moment remains constant through increasing curvature. (Fig 10a). Christensen and Janes showed that this concept needed to be modified, and that the moment-curvature relation was closer to that shown in Fig 10b. They did not provide a method of analysis.

iv) The theories relating to the analysis of conventional concrete pavements, and the few that have been proposed for the analysis of prestressed concrete pavements, assume full subgrade support to the slab, or furnish semi-empirical relationships to account for the lack of support over the concerned locations.

The fact is that only a comparatively small area of subgrade under the slab reacts to the load. This is because the slab curves upwards in bending, the curvature depending in part on the relative stiffnesses of the slab and the subgrade, and on the size of the loadprint. As a result,
a significant portion of the slab is raised off the subgrade, no matter how infinitesimally. This factor will be taken into account and described more fully when the theory is set out.

The fact of such lack of subgrade support may be observed from the photographs of the tests on the model slab (Photographs Nos 127, 128, 137) and from the photographs of the tests on the experimental prestressed concrete slabs (Photographs Nos 74, 75, 76, 77).

ν) It is to be noted in passing that the search for a rational method of design has been in progress in Europe since 1946, and in the USA since 1952. Thicknesses of prestressed pavements have been assumed, rather than designed. There is therefore much scope for improvement in the methods of analysis and design.
5.3 The Basic 'Elastic-Crack' Mechanism

In Prestressed Pavement Slabs

Elementary Behaviour Of A Loaded Beam Resting On Subgrade Material (Elastic Range)

5.3.1

1) Consider first an elastic beam resting on a subgrade whose resistance to deformation is proportional to the deformation induced. In other words, the modulus of subgrade reaction is taken as being constant.

As a load is applied to the beam, the beam deflects, and the subgrade underneath deforms along with it. (See Fig 11). The stresses induced in the beam and the subgrade, (which latter are a measure of the deflection produced), serve to resist the load. The bending moment in the beam is a maximum directly under the load, and is taken as being of, say, the positive sense. The bending moment diminishes in value away from the load and on either side of it, passes through a zero value point, is
inflected to a small negative moment, and then again
dies away to zero, after which point the beam is no
longer stressed by the load.

ii) The distance from the load to the point where bending moment passes through the first zero value is dependent on the relative 'stiffnesses' of the beam and the subgrade. If this distance is small, the stress in the subgrade will be high, but the bending moment in the beam will be comparatively low. Thus, carrying the illustration to its extreme, if the 'beam' were perfectly flexible, (and applying the limitation that it does not then carry part of the load in friction-restraint tension), whatever load is applied to it is resisted wholly by the subgrade, the 'beam' then only serving to transmit the load to the subgrade, which it does by itself being in compression. The flexible 'beam' obviously then carries zero bending moment.

iii) If the distance from the load to the point of first zero moment is large, the stress in the subgrade will be low, and the bending moment in the beam will be comparatively high. A poor subgrade
or a very stiff beam will indicate this condition. Again carrying the illustration to its extreme, if the beam were perfectly rigid (i.e., capable of carrying moments without deflecting), the load on the beam would be resisted uniformly, all along its length, by the subgrade. The subgrade stress would therefore be low, and the bending moment in the beam at the point of application of the load would be very high.

iv) The point at which the bending moment diagram passes through the zero value would correspond to a point of contraflexure on the deflected beam.

Elementary Behaviour Of A Loaded Slab Resting On Subgrade Material (Elastic Range)

5.3.2

1) The description of the behaviour of the beam which has just been given can be extended to describe the action of a slab. Referring to Fig 12, it can be visualized that the planar deflection in the case of the beam will now become a dish-shaped depression in the case of a slab. Due to the double curvature now extent (resulting from torsion and twist), radial and tangential moments will be induced in the
slab, as illustrated in the figure. The distance from
the load to the point of first zero radial moment has
the same significance as the corresponding dimension
in the case of the beam; i.e., it is a measure of the
relative stiffnesses of the slab and the subgrade.

ii) The design of conventional concrete
pavements follows the lines indicated above. The slab
is not designed for working beyond the elastic range.
Cracking is not permitted, and the deflection curves
should therefore be continuous.

Elementary Behaviour Of A Loaded Beam And A Slab
Resting On Subgrade Material (extra-Elastic Range)
5.3.3.

i) Suppose now that the beam of Fig 11 is
loaded beyond the load which produces maximum positive
bending moment, so that the material is stressed beyond
its elastic limit. Suppose also that the material is
ductile. The bottom of the beam under the line of
application of the load will now begin to yield in
tension, and the beam at that section will not be
able to resist a further bending moment. However, the
beam has not failed, if 'failure' implies the sense of not being able to take load. The yielded central portion will act as a plastic hinge, resulting in the two-portions of the beam on either side of the hinge functioning as what may be described as cantilevers. The deflection curve becomes angular and deeper under the load, and the negative bending moments increase. They continue to increase with increased loading till the yield point is reached once more, when the material will again yield, this time at the top.

ii) It is thus seen that even after the beam had first yielded, it would still continue to take loads much in excess of the load that caused the initial yield. Also, if loading is stopped just before the second set of plastic hinges form, and the beam is then unloaded, it could be worked through the same range again, though the deflections would now of course be greater than if the elastic range had not been exceeded.

iii) Concrete, however, is a brittle material. When its flexural strength is exceeded, it cracks rather than yields, with the result that a plastic
hinge does not form. The mechanism just described for the case of the ductile beam is therefore not operative, which is why conventional concrete pavements are not designed to be worked even to the point of first cracking.

iv) One method to overcome this could be to precompress the beam so as to counteract some or all of the eventual tension produced under load. The net result is to increase the flexural strength of the concrete. The resulting tension would be kept within the increased flexural strength, and cracks would still not be allowed to develop. By this method, the beam would be able to take more load than would an uncompressed beam. This is the normal approach in prestressed concrete structures such as buildings and bridges. When applied to the case of slabs, however, the approach becomes very uneconomical. This is because no utilization is made of the mechanism just described whereby advantage is taken of working a beam (resting on a subgrade) in the yield or extra-elastic range.
v) Suppose now that prestress is still applied to the concrete beam, but not sufficiently to prevent, totally, initial cracking at the bottom, under load. The initial cracks will therefore form, but they are now not 'Free' cracks, in the sense that the cracks induced in an un Prestressed beam are 'Free'. The cracks in this Prestressed beam are still under a restraining stress, and as such function in a similar manner as do plastic hinges in a ductile material (See Fig 13). The beam can therefore still be worked beyond the initial cracking point as in the case of the ductile beam (also see para 15.7). Once the load has passed, the cracks close under the prestress, and the beam is, for all practical purposes, monolithic again. (See Fig 13). In the conventional concrete beam, however, the cracks do not close after the load has been removed. The cracks remain open, and work up through the thickness of the concrete under repeated loading.

vi) Due to the manner in which they function, the term 'elastic cracks' will be used to identify these cracks.
vii) This behaviour of the prestressed beam can be extended to describe the action in a slab, just as was done when the working in the elastic range was described.

Thus, by applying only enough prestress to ensure the action, a concrete slab supported on a subgrade can be worked much beyond the point of bottom cracking because of the action of the elastic cracks. This is what contributes to its greatly increased capacity to carry loads, as compared with a conventional concrete slab.

viii) From what has just been described, it will be seen that the initial cracks which occur in the underside of the loaded prestressed slab do not indicate failure, as is the case with conventional slabs. On the contrary, these cracks help to redistribute the moments, which redistribution leads to a more efficient utilization of the properties of the concrete.
The Radius Of Relative Stiffness

5.3.4

To relate the stiffness of the slab to that of the subgrade, Westergaard's 'radius of relative stiffness' \( L_{\alpha} \) in length dimensions may conveniently be used.

With the subgrade being considered as being a 'liquid subgrade', \( L = \left( \frac{D}{E} \right)^{\frac{3}{2}} \), where \( D \) is the conventional flexural rigidity of the slab, (analogous to the stiffness \( EI \) of a beam) equal to \( \frac{Eb^3}{12(1-\nu^2)} \) (in force-length dimensions).

If the subgrade is considered as being an elastic subgrade, then \( L = \left( \frac{(1-\nu)D}{6} \right)^{1/3} \) (Ref: 64).

However, in practice there is not much significant error in assuming the subgrade to be a liquid subgrade, as confirmed by Mr. Clausen of the American Society of Civil Engineers in a letter to the author. The assumption also makes for considerable ease in computation, and will therefore be used in this paper.
5.4 Preliminary Observations And Deductions

The Two Concepts For Analysing Prestressed Concrete

5.4.1

There would appear to be two concepts by which prestressed concrete could be analysed:

i) that a new material is 'created', whose properties such as the modulus of elasticity in tension and the flexural strength are different from those of ordinary concrete, and greater than them by an amount equal to the amount of the prestressing force; ii) that the basic material, concrete, remains the same, and that the prestress induced in it is to be treated as an external force like any other externally applied force. The second approach would seem easier of application, and is the one used in this paper. In any case, even though a prestressing force actually does increase $E(\text{tension})$ of the material, (just as any compressive force would do) the increase is so insignificant as to be ignored.

Strain, Stress And Curvature

5.4.2

.1 The Strain Consideration

i) The basic concrete material will always crack at the same strain in tension, regardless of whether the material is prestressed or unstressed. In other words, and using the 'critical strain at failure' criterion of sub-section 2.5.5, any prestressed
concrete slab will crack at a location when the critical strain at that location reaches the same level as obtained from a failure test conducted on ordinary concrete. (E.g., from a flexure test). In flexure, curvature is a measure of the strain; and therefore, for the same thickness of slab, a prestressed slab and an ordinary slab will show the same curvature at cracking.

ii) The effect of the prestressing force is to increase the stiffness of the slab, so that the vertically applied load required to achieve the 'failure curvature' in the slab would be greater than what it would have been had there been no prestress.

### 4.2 The Stress Consideration

i) The prestress does not really increase the flexural strength (i.e., flexural stress at failure) of the concrete, which strength is a fundamental and constant property of that material. The compressive prestressing force, which is an externally applied force, serves to counteract tensile stresses brought by other externally applied forces. At cracking,
the net resultant flexural tensile stress on the concrete (and therefore the 'strength' of the concrete) will always have the same value. The prestressing force allows a greater vertical load to be applied to the slab, corresponding to the amount of the prestress. A tensile crack will therefore appear in the concrete (transverse to the direction of prestress) when the flexural stress in the longitudinal direction due to a vertically applied load becomes equal to the sum of the flexural strength of the concrete and the amount of the pre-compression.

-3 Curvature

i) Observations on curvature provide a convenient means of determining the strain which develops in a material due to bending. For all practical purposes, the simple relationship \( \frac{1}{R} = \frac{f}{E_y} \) can be taken as holding true.

ii) The greater the thickness of the slab, the less is the curvature required for inducing the same failure strain.
iii) For a given thickness of slab, the strains at all points, through a section (except at the centre of section) increase with increasing curvature.

iv) A convenient means of obtaining the curvature of a deflected slab is as follows. Refer to Fig 14. AE is the undeflected slab, and A' C' E' is the deflected profile.

The slope at B = \left[ \frac{dw}{dx} \right]_B, approximately

\[ \frac{w_0 - w_1}{x} \]

The slope at D = \left[ \frac{dw}{dx} \right]_D, approximately

\[ \frac{w_2 - w_0}{x} \]

The curvature at C = \frac{1}{R_c} = \left[ \frac{d^2w}{dx^2} \right]_c, approximately

\[ \frac{1}{x} \left[ \frac{w_2 - w_0}{x} - \frac{w_0 - w_1}{x} \right] \]

approximately = \[ \frac{w_1 - 2w_0 + w_2}{x^3} \].
But \( w_r = \frac{1}{2} ( 2w_1 - w_2 - w_3 ) \), from geometry.

\[ \frac{1}{R_c} = \frac{2w_r}{x^3}, \] numerically.

If the radius of curvature is large, when compared to the deflections, as it is in the case of slabs, this approximation can be taken as holding true.

The radius of curvature of a deflected form can therefore be obtained by noting the vertical deflection at the centre of a straight line which is allowed to 'rest' on the deflected form.

Deflections: Radius Of Subgrade Reaction; Point Of Contraflexure

5.4.3

1) On a perfectly rigid subgrade, a slab (made of some less rigid material) will 'deflect' upwards, and only a small portion of its underside will remain in contact with the subgrade.

(See Fig 15(i), and Photograph Nos, 127, 130)
11) As the subgrade becomes 'softer', the following relative responses would be observed:

a) absolute deflections in the slab (as measured from a fixed datum) increase.

b) relative deflections (as measured between adjoining points along the slab) increase.

c) the length of the underside of the slab, which is in contact with the subgrade, increases. (This length will be termed the 'radius of subgrade reaction'. It indicates the distance, measured outward from the centre of load, up to which the subgrade supports the slab).

d) the load becomes spread out over a greater length of subgrade.

e) bending moments in the slab increase.

f) the curvature of the slab increases.

g) stresses in the soil become reduced.
iii) If the slab is weightless, (See Fig 15 ii) the deflected shape of the slab, when it is loaded over a deformable subgrade, would be as shown in Fig 15(iii). The portions unsupported by the subgrade would be straighter and would become inclined to the subgrade as a geometrical consequence of the curvature of the central portion of the slab.

iv) If the slab has weight, and has not too great a length, the deflected shape would be as shown in Fig 15(iv).

v) If the slab has weight, and is long, the deflected shape would be as shown in Fig 15(v). The portion AB would be brought back towards the ground due to its weight and due to the weight of the slab beyond B. An even transition curve would form at D, and the portion ADB would remain unsupported. Bending moments would therefore be induced over the curve through D. Soil pressures in the central portion AA would be relieved slightly.
vi) The final deflected shape of the slab and the radius of subgrade reaction therefore depend upon:

a) the flexural rigidity of the slab
b) the support resistance offered by the subgrade
c) the body forces (weight) in the slab
d) the size of the slab
e) the intensity of the load
f) the size of the loadprint.

vii) The radius of subgrade reaction as defined in vi) preceding is not to be confused with the radius of relative stiffness. It would, however, seem that both these radii are interdependent.

viii) It is to be observed that the point at which the radial moment in the slab passes through a zero value, at a distance from the load which corresponds to the radius of relative stiffness, is a point of contraflexure on the deflected shape of the slab.

This point is shown at CP in Fig E-Zero.
Two-way/One-way Prestressing

5.4.4

1) Prestressed slabs may be prestressed in both directions equally; or in both directions unequally; or in one direction. Where the length of the slab is very large in comparison to its breadth, the maximum tensile stresses under load will develop in the longitudinal direction, leading to transverse cracking. Prestress must therefore be applied longitudinally. In practice a certain amount of prestress is also applied transversely mainly to tie adjacent widths of slabs together. Some authorities even consider that one-way prestress in the longitudinal direction is all that is necessary, in the case of road and airfield pavements.

ii) Even though, under maximum longitudinal stress, cracks develop in the transverse direction, it is quite possible that due to some abnormal loading, cracks develop longitudinally. If there is no pre-stress in the transverse direction, these longitudinal cracks will remain open after the load has passed, progressive cracking will occur under repeated loads,
and the slab would soon suffer distress and failure.

iii) The author therefore considers that all pavement slabs should be prestressed in both directions. The quantum of transverse prestress need not be as large as that of the longitudinal prestress, in the case of slabs with a large length: width ratio. But prestress must be provided, so as to induce the elastic-crack action should longitudinal cracks develop, and so as to ensure that the slab remains a structural monad.

iv) In the experiments conducted by the author, four types of slabs were tested, the levels of prestress being as noted:

Two-way equal prestress = 250 lbs/in<sup>2</sup>

250 lbs/in<sup>2</sup>.

Two-way unequal prestress = 250 lbs/in<sup>2</sup>

125 lbs/in<sup>2</sup>.

One-way prestress = -250 lbs/in<sup>2</sup> x

0 lbs/in<sup>2</sup>.

No prestress = - 0 lbs/in<sup>2</sup> x 0 lbs/in<sup>2</sup>.

(i.e. ordinary concrete slab)

The slabs were square.
When references are made in this text to a ' (250 x 250 ) slab ', or to a ' (250 x 125 ) slab ', or to a ' (250 x 0 ) slab ', or to a ' (0 x 0 ) slab ', the connotations shall be as indicated above, and it shall be understood that the reference is to the square slabs. In square slabs the responses are obviously symmetrical about the two axes drawn through the centre, parallel to the sides.

v) a) The subgrade conditions being similar, the slab with the lesser prestress would, a priori, ultimately fail at lower loads than the slab with the higher prestress.

b) A (250 x 250) slab would initially crack at a load higher than for a (0 x 0) slab. The increase in load would correspond to the level of prestress. The curvatures at failure would, of course, be the same in both cases.

c) A (250 x 0) square slab would initially crack, at the bottom, at a load quite close to that which causes the initial bottom crack in a (0 x 0) slab.
The reasoning is as follows:

Refer to Fig 16. The square slab is prestressed to 250 lbs/in$^2$ in the $X$-direction, there being no prestress in the $Y$-direction. The slab is therefore stiffer in the $X$-direction, and will bend as shown, the curvature developing more in the less stiff $Y$-direction. It is apparent that due to the curvature, cracks, when they develop, can first develop only parallel to the $X$-axis, and will be resisted by the internal force in the transverse direction, that is by the internal force parallel to the $Y$-axis. There is no prestress parallel to the $Y$-axis, and therefore the whole of the resisting force must be supplied solely by the flexural tensile strength of the concrete; and this is the same quantum of strength which a $(0 \times 0)$ slab can provide. A square $(250 \times 0)$ slab and a square $(0 \times 0)$ slab would therefore initially crack at the same load.
5.5 The Load-Carrying Mechanism And The Mode Of Failure

Bottom Radial Cracking

5.5.1

1. Two-Way Prestressed Slabs

1) When a square, two-way prestressed concrete pavement slab is loaded centrally under a progressively increasing load, the initial response is as already described in section 5.3.

The maximum bending moment occurs under the load. Cracks begin to form when the flexural tensile strain on the underside of the slab, (at the location 0 of Fig E-Zero) becomes greater than the critical flexural tensile strain for concrete, consequent to the curvature at that location.

2) The magnitude of the tangential and radial moments at the point directly under the centre of the load are obviously the same at that point. Now, excess radial moments result in tangential cracking and excess tangential moments result in radial cracking. At the point directly below the load, the
tangential moment, when it is large enough, will produce radial cracks. The radial moments, which at that point are equal to the tangential moments, will therefore also cause failure of the concrete, this time by the formation of tangential cracks. But within the limit of the radial distance from the centre of the load tending to zero, (as it must so as to approach the dimensions of the point which lies directly below the centre of the load) tangents themselves tend to become radii. As a result, only radial cracks would develop at the bottom of the slab, directly under the load. (Fig 18)

iii) This result would also be obtained intuitively, from consideration of the symmetrical, 'orange-peeling' that is observed in confined doubly - curved shapes that are extensively strained.

iv) The tests on the experimental slabs which will be described later also clearly indicate the formation of radial cracks at the underside of the slab. (Photographs Nos 25, 48, 22, 52, 105,124).
v) These are 'elastic cracks', and the slab can continue to take load.

5.2 Ordinary Concrete Slabs

1) It is to be observed that ordinary concrete slabs will also tend to crack radially at the bottom. Due to the confining and elastic effects of the prestress being absent, however, only so many cracks would form as would be necessary eventually to turn the slab into a mechanism. Furthermore, the corners of a rectangular or square slab offer more restraint to curvature, due to the geometry of those locations, than do other portions of the slab. As a result of these two factors, the 'radial' cracks in ordinary, square slabs would tend to move outwards from the location of the central load to the middles of the sides of the slab, and would be restricted in number. (See Fig 18, and Photographs Nos 82, 83, 109. The top cracks in the photographs, which have developed in more or less straight lines running to the sides, are the original bottom 'radial' cracks.
which have worked their way up through the thickness of the slab at ultimate failure.

Ultimate failure is the next stage in the load-response process, and is described later.

These bottom cracks are of course, non-'elastic', and do not close on the removal of the load. But at this stage the slab can still take load, and in this sense the ultimate failure stage of the slab has not yet been reached.

3 One-Way Prestressed Slabs

A one-way prestressed slab would crack in agreement with the concept described in subsection 5.4.1 v). The crack, which is 'non-elastic', would develop along the underside of the load, and would extend from the position of the load to the middle portions of the respective edges, keeping parallel to the direction of prestressing. See Fig 18. Photograph Nos 56, 79, 80, 88, 78 also show such a crack in a (250 x 0) slab. The longitudinal crack that is seen is the crack which originally formed on
the underside of the slab. As seen now it has worked its way to the top at the stage of ultimate failure, (which is the next stage, to be described later). In the present instance, the slab is still capable of taking load.

**Subgrade Reaction Upto The Bottom Cracking Stage**

5.5.2

i) Upto the stage of bottom cracking, the flexural rigidity of the slab is the same at all points, for no discontinuities have occurred.

ii) The problem now is to decide what portion of the underside of the slab, if not the whole portion, is supported by the subgrade.

iii) Recourse will be made to a graph prepared (86) by Westergaard, which shows how deflections in a slab acted on by an interior load, vary with the distance from the load. (See Fig 17). From the graph it is apparent that the deflection becomes zero at a distance away from the load equal to about 4 L. If
the straight portion of the deflection curve is extended linearly, moreover, it could be deduced that most of the significant subgrade support is provided within a circle of radius of approximately 2.4 L, drawn concentric with the position of the load. In practice these figures may vary slightly; but they serve to convey the idea that only a portion of the area of the underside of a slab is reacted upon by the subgrade in support of a vertically applied interior load.

Referring to Fig. E - Zero, the point corresponding to the point where deflections become zero is shown at S, and the corresponding distance $L_b$ will be defined as the 'radius of subgrade reaction at which the slab deflection is zero'.

The point $S_b$ is the point up to which most of the significant subgrade reaction is exercised, up to the stage of bottom cracking. The corresponding radius, $L_b$, is the 'radius of significant subgrade reaction up to the stage of bottom cracking'.

iv) The response to load of the slab sub-grade structure, up to the bottom cracking, occurrence of is shown diagrammatically in Fig E- Zero, Stages 1,2,3.
Top Circumferential Cracking

5.5.3

1. Two-Way Prestressed Slabs

1) Immediately at and after the formation of the bottom cracks, the flexural rigidity of the two-way prestressed slab changes in certain areas. The slab remains a structural monad, due to the pre-stressing force which is still operative; but that portion under the load and some distance away from it becomes more flexible, due to the radial cracking. Deflections in this portion will thus increase.

2) As the load becomes progressively greater, negative radial moments increase, causing the slab to curve convex upwards some distance away from and all around, the load. The radial cracks in the bottom of the slab also extend outward.

3) Cracks will begin to appear at the top of the slab, corresponding to the points of maximum curvature, when the flexural tensile strain at those locations becomes greater than the critical flexural tensile strain for the concrete. These cracks will
obviously be circumferential cracks, caused as they are by radial moments. (Fig 18).

The radial distance from the loadprint at which the circumferential cracks appear will be designated as $L_e$.

iv) The radial cracks in the underside of the slab would be deepest under the load. They become progressively less deep as the crack extends outwards, and the concrete in this portion assumes the shape of wedges. See Photograph Nos 105, 124. (This effect would also be obtained intuitively from consideration of the manner in which 'peeling' and splitting occur in some materials which are curved sharply in bending (...... eg ...... in a moderately dry stick).

It is also probable that due to the geometry of the deflected slab, the radial cracks would not extend beyond the point of contraflexure $C_{D}$.

The radius of the area within which radial cracks occur will be designated as $L_r$. 
Furthermore, the depth of the radial crack under the position of the load would probably extend only up to the centre of the section of the slab, as this is the thickness that is in flexural tension.

Within experimental limits, these concepts have been shown to be fairly valid from the results of the tests on the experimental slabs.

v) It is observed that though cracks have appeared both in the bottom and in the top of the slab, a concrete slab which has been prestressed both ways has not yet 'failed', if failure is taken to mean the inability to carry load while remaining a structural monad.

Materials fail when the strain becomes so excessive that the lines of interparticle forces of bonding become broken. In the present case of the prestressed concrete slab, the 'lines' of prestressing force have not yet become broken, even though the concrete itself may have cracked. The structural integrity of the slab is therefore still maintained. The cracks are still elastic, and will close on removal of the load.
2 Ordinary Concrete Slabs

1) An ordinary concrete pavement slab (the O×O slab) will also tend to crack at the top in the same fashion as does a two-way prestressed slab. In this case however, the confining effects of the prestress are absent. The 'circumferential' crack at the top is therefore irregular, (in much the same way that compressing a loaf of bread while cutting it will give a cleaner cut than if the loaf were kept uncompressed). The shape of the crack is also affected by the geometry of the slab and by the presence of the cracks already extant in the underside of the slab. The top crack therefore tends to take the form of an irregular quadrilateral. (See Fig.18) Photograph Nos 82, 83 indicate the shape of the top crack, (which, in the photographs, is the central closed crack). The other straight cracks extending from the corners of the 'quadrilateral' to the middle of the edges of the slab are the bottom 'radial' cracks which have worked upwards at failure. See sub-section 5.5.1 52)
Unlike the case of the two-way prestressed slab, the cracks are non-elastic and do not close on the removal of the load. The 0 x 0 slab must now be deemed to have failed. It has shattered into a number of separate pieces. There is no line of bonding force passing completely through the slab. Its structural integrity has been lost, and it is no longer a structural monad.

The load which causes cracking at the top therefore signifies the ultimate load that a 0 x 0 slab can carry.

In practice, however, unpresetressed slabs are not worked even up to the point of bottom cracking. This is because the cracks are non-elastic, and would work their way to the top under the action of repetitive loads of even moderate intensity.

.3 One-Way Prestressed Slabs

A one-way prestressed slab (250 x 0 slab) would tend to crack at the top in the same
fashion as does a 0 x 0 slab. (See Fig 16, and Photograph Nos 58, 59). As in that case also, the formation of the top crack would signify ultimate failure, since no lines of force exist across the cracked and separated portions.

In practice, a square one-way prestressed slab should not be worked to the point of initial cracking at the bottom.

Subgrade Reaction Upto To The Top Cracking Stage

5.5.4

1) Due to the increased flexibility of the central portion of the slab, most of the significant part of the subgrade reaction would occur within a radius less than the radius \( L_b \) of the previous stage.

In Fig E-Zero, the point \( S_z \) represents the point upto which the significant part of the subgrade reaction would occur in the range of loading between the bottom cracking stage and top cracking stage. \( L_t \) is the corresponding radius of significant subgrade reaction. It would appear that just as \( L_b \) is related to \( L_s \), so would \( L_t \) be related to \( L_s \).
ii) It would appear that the point S at which the deflection in the slab becomes zero should move inwards, due to the increased flexibility of the central portion. However, the tests that were conducted on the experimental slabs showed little or negligible regression of the point S; and it is probable that though such regression does occur, it is so small, due to the integrity of the slab being maintained though even the cracked portion, that for all practical purposes the regression of S can be ignored.

iii) The response to load of the slab-subgrade structure, between the stages of bottom and top cracking, is shown diagrammatically in Fig E - Zero, Stage 4.

Punching Shear Failure

5.5.5

1) The (0x0) slab and the (250x0) slab having already reached their respective ultimate load
capacities when top cracking occurred, they will not be considered further.

ii) In the case of the two-way prestressed slab, the flexible rigidity of the central portion becomes further reduced immediately at and after the formation of the top cracks. Deflections will increase; but, due to the prestressing force which is still extant, the slab remains a structural monad.

iii) In fact, it is difficult to conceive of the structural integrity of the slab being disrupted till the prestressing force is itself destroyed. In practice, this would entail the snapping or relaxation of the steel tendons, or the outward displacement of the abutments, whichever is relevant.

iv) With increasing deflections, a stage is reached when the subgrade directly beneath the loadprint deforms so much as to offer little resistance to the load. The soil around this deformed portion now supports the slab, while the prestress aids in 'confining' the concrete; Conditions conducive to the occurrence
of punching shear are now set up. The shearing resistance of the concrete is probably increased from that of normal concrete due to the precompression mobilising internal friction. When this increased shearing resistance is overcome, the load punches through the slab. (See Fig 16, and Photograph Nos 47, 71, 72, 46)

v) The portion of the slab that is being punched out meets the radially cracked portion after it has descended about half-way through the thickness of the slab. The unsupported radial wedges therefore also break away, so that the final failure is not in the form of a cylindrical hole, but rather as shown in Fig E - Zero, Stage 6. (See also Photograph Nos 22, 52, 105, 124, which show the central punched portion, from the top of the slab, and the radial wedges from the bottom of the slab).

vi) The ultimate failure of the slab may now be deemed to have occurred, as far as its capacity to carry a load over the punched location is concerned.
It is to be observed, however, that because the major element of the prestressing lines of force has not as yet been broken, the integrity of the slab is still maintained. It is still a structural monad, and can still take load over other locations (See Photograph No 140).

This fact is of little practical consequence, of course, in the sense that though a vehicle can move over the rest of the slab, its wheels cannot be allowed to move over the locations of the punched holes.

vii) It constitutes sound practice if the aspect of punching shear is kept in mind at all stages of loading, especially if the slab is particularly thin.

Subgrade Reaction Upto The Punching Shear Failure Stage

5.5.6

i) Due to the increased flexibility of the central portion of the slab, most of the significant part of the subgrade reaction would now occur within a radius less than the radius \( L_1 \) of the previous stage.
Let $S_p$ represent the point upto which most of the significant subgrade reaction takes place. $L_p$ is then the 'radius of significant subgrade reaction between the stages of top cracking and punching failure'.

ii) Now, the concrete in the central portion may be conceived of as being hinged at the radial elastic-cracks under the slab at $Q$, and also at the location of the top, circumferential elastic-cracks, at a distance $L_c$ from the loadprint. It is therefore apparent that this portion of the slab is quite flexible and will deform most under load. It therefore would appear reasonable to assume that the point $S_p$, besides indicating the point upto which the significant part of the subgrade reaction would occur in the range of loading between the top cracking stage and the punching shear stage, also represents the location of the top cracks. In which cases, $L_c = L_p$. 
iii) The response to load of the slab-subgrade structure, between the stages of top cracking and punching shear, is shown diagrammatically in Fig 2-Zero, States 5 and 6.

5.6 Computing Deflections And Curvatures
By Theoretical Analysis

Approach
5.6.1

1) It follows from what has been observed previously in this paper, that a convenient means of predicting whether or not the slab would crack at given locations under the action of particular loads would be to evaluate the strain at those locations and to compare these value with the value of the critical strain for concrete as determined from tests.

ii) If the curvature of the loaded slab could be determined, then the corresponding strain could also be determined from the basic equation for bending, the thickness of the slab being known.
iii) a) The curvature in a particular direction at a given point can be determined if the deflections at that point, and at two adjoining points one on each side of it, were known by using the equation given in sub-section 5.4.2.

b) The curvature could also be determined from resolving the basic differential equation relating deflection to coordinate distance.

iv) The problem could therefore be reduced to evaluating deflections at various points in the slab, the slab being under the action of the following forces:

a) the vertical, externally applied load.

b) the vertical, externally induced subgrade reaction.

c) the precompression.

v) The assumptions that will be used are:

that the concrete is linearly elastic and isotropic to mechanical properties; that the concrete within the
regions where discontinuities may otherwise occur is itself homogeneous; that the vertically applied external load, and the subgrade reaction, remain normal to the slab; that the subgrade reaction at a point is proportional to the deflection at that point; that no friction operates on the underside of the slab; that the slab is thin enough for the theory of thin plates to be applicable.

The Equation Of State For Prestressed Slabs

5.6.2

1) a) The pavement slab may be treated as a thin plate, in which case, and ignoring for the moment the force of pre-compression, the usual plate equation will apply: i.e., the biharmonic equation

\[
\frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} = \frac{q}{D}
\]

Must be satisfied at all points.

(w is the deflection; D is the flexural rigidity of the slab; q is the resultant vertical external force at a point).
b) This equation may conveniently be solved by the process of relaxation, especially since the mechanical aspect of the relaxation method makes the method particularly suited to being applied to a computer.

ii) a) The equations of equilibrium for a plate are also obtained in the form:

\[
\frac{d^2 M_x}{d x^2} + \frac{d^2 M_y}{d y^2} - 2 \frac{d^2 M_{xy}}{d x \, d y} = - q \quad (1)
\]

where:

\[
M_x = - D \left( \frac{\partial^2 w}{\partial x^2} + u \frac{\partial^2 w}{\partial x \partial y} \right)
\]

\[
M_y = - D \left( \frac{\partial^2 w}{\partial y^2} + u \frac{\partial^2 w}{\partial x \partial y} \right)
\]

\[
M_{xy} = - M_{yx} = D (1 - u) \frac{\partial^2 w}{\partial x \partial y} \quad (2)
\]

b) The effects of the forces of precompression will now be taken into account.
c) The prestress in a slab is usually quoted in terms of lbs/in$^2$ of compression induced in the concrete, taken along each of the two X and Y axes. This may be converted into the 'force/unit length acting on each of the edges, parallel to the X and Y axes'. These forces act normal to the edges, and will be designated by $N_x$ and $N_y$ respectively.

Thus, if the prestress in a 6 ft x 6 ft x 1\(\frac{1}{2}\) ins thick slab is 250 lbs/in$^2$ parallel to the X axis, and 125 lbs/ins$^2$ parallel to the Y axis, the transformed values of precompression would be:

$$N_x = \frac{250 \times (6 \times 12) \times 1\frac{1}{2}}{(6 \times 12)} = 375 \text{ lbs/in}^2$$

$$N_y = \frac{125 \times (6 \times 12) \times 1\frac{1}{2}}{(6 \times 12)} = 167.5 \text{ lbs/in}^2$$

d) Considering the equilibrium of a square parallel element cut from the plate, it is seen that
\[
\begin{aligned}
\frac{d N_x}{dx} &= 0 \\
\frac{d N_y}{dy} &= 0 
\end{aligned}
\] (3)

The projection of the \( N_x \) and \( N_y \) forces on the vertical \( z \) axis must account for the small angle between \( N_x \) and \( N_y \) consequent to the bending of the plate. The projections will thus become:

For the \( N_x \) force:

\[
-N_x \, dy \, \frac{d w}{dx} + N_x \left( \frac{d N_x}{dx} \right) \left( \frac{d w}{dx} + \frac{d^2 w}{dx^2} \right) dy
\]

which, after ignoring small quantities higher than the second order, reduces to

\[
N_x \frac{d^2 w}{dx^2} \, dy \, dx + \frac{d N_x}{dx} \frac{dw}{dx} \, dx \, dy \quad (4)
\]
Similarly, the projection of the $N_y$ force on the $Z$ axis gives
\[
N_y \frac{d^2 w}{c y^2} \, dx \, dy + \frac{d N_y}{c y} \frac{d w}{c y} \, dx \, dy \tag{5}
\]

The vertical load on the element is $q \, dx \, dy \tag{6}$

The expressions (4), (5) and (6) are now algebraically summed, and the equation of equilibrium (1) is then converted, by using equations (3), to the following form:
\[
\frac{d^4 w}{c x^4} - 2 \frac{d^2 M_{xy}}{c x dy} + \frac{d^2 M_y}{c y^3} = - \left( q + N_x \frac{d^2 w}{c x^2} + N_y \frac{d^2 w}{c y^2} \right) \tag{7}
\]

The substitution of $M_x$, $M_y$ and $M_{xy}$ by the right hand terms of equation (2) then gives
\[
\frac{d^4 w}{c x^4} + 2 \frac{d^4 w}{c x^2 dy^2} + \frac{d^4 w}{c y^4} = \frac{1}{B} \left( q + N_x \frac{d^2 w}{c x^2} + N_y \frac{d^2 w}{c y^2} \right) \tag{8}
\]
ii) Equation (8) will be considered to be the equation of state for prestressed pavement slabs, relating deflections to loads.

Application Of The Prestressed Slab Equation To The Various Stages Of Loading.

5.6.3

It has been shown in section 5.5 that the flexural rigidities of portions of the slab change through the various stages of loading. It has also been shown that the radius of significant subgrade reaction will also vary, but that the radius \( L_g \) at which deflections become zero can, for all practical purposes, be taken as being constant.

The resultant vertical load \( q \) at any point is made up of the algebraic sum of: the weight of the slab at that point; the intensity of subgrade reaction at that point; the externally applied vertical load at that point. \( q \) will therefore also vary over the area of the slab.
The separate stages of loading and cracking are shown in Fig E-Zero. The plan of the slab, indicating the circular zones of subgrade reaction and of cracking limits, is also shown.

Therefore, from considerations of the different flexural rigidities, and of the changes in intensity of loading, the prestressed slab equation must be separately applied at each stage of loading.

1. Upto The Stage Of Bottom Cracking

(Fig E-Zero, Stages 1 to 3)

1) Upto the occurrence of bottom radial cracking, the flexural rigidity of the slab is the same throughout, and is equal to \( \frac{Eh^3}{12(1-w)} \) where \( h \) is the whole thickness of the slab.

2) The intensity of resultant vertical load, \( q \), will vary thus:

   a) From the edge to the point \( S \),

   \( (\text{where } w = 0) \):

   \( q = \text{weight of the slab/unit area} \).
b) From $S$ to $F_2$, the edge of the loadprint:

\[ q = \text{weight intensity of the slab} - \]
\[ \text{the intensity of subgrade reaction}. \]

c) From $F_2$ to $F_2$, ie., over the loadprint:

\[ q = \text{weight intensity of the slab} - \]
\[ \text{the intensity of the subgrade reaction} + \text{the intensity of the applied load}. \]

The subgrade reaction at any point is itself given by $kw$.

52 Between The Stages Of Bottom And Top Cracking

(Fig E - Zero, Stage 4)

1) a) The flexural rigidity of the slab outside the limit of radial cracking, i.e., beyond the circle of radius $L_x$, (see sub-section 5.5.3 iv) remains at

\[ \frac{Eh^3}{12(1-\nu^2)} \]
b) Within the circle of radius \( L_r \), the cracks have a depth of about \( \frac{h}{2} \) at the point 0, and a depth equal to zero at the point \( CF_b \). (See subsection 5.5.3 iv) On the average, therefore, the thickness of the crack is \( \frac{h}{4} \).

The flexural rigidity in the circle of \( \text{radius } L_r \) may therefore be taken to be

\[
\frac{E (0.75 h)^3}{12(1-\mu^2)}
\]

ii) The intensity of resultant vertical load, \( q \), will vary as in the case for bottom cracking, \( w \) being taken equal to zero at the location \( s \), as before.

1.3 Between The Stages of Top Cracking And Punching Shear Failure

(Fig E - Zero, Stage 5; applicable only to two-way prestressed slabs).

i) a) The flexural rigidity of the slab outside the limit of circumferential cracking, \( \text{i.e.,} \)
b) Within the circle of radius \( L_0 \), the radial cracks have a depth of about \( \frac{h}{2} \) at the point \( O \); the circumferential cracks would have a depth of about \( \frac{h}{2} \) at the point \( S_p \), the cracking extending up to half the thickness of the section in the tensile zone.

For practical purposes therefore, the flexural rigidity in the circle of radius \( L_0 \) may be assumed as being approximately

\[
\frac{E (0.5h)^3}{12(1-\mu^2)}
\]

ii) The intensity of resultant vertical load \( q \) will vary as in the case of bottom cracking, \( \omega \) being taken equal to zero at the location \( S_p \), as before.
4 Relaxation Solution

1) The nature of the conditions imposed on the resolution of the equation would probably preclude any attempts at 'exact' mathematical solutions. Within limits, and as has been previously discussed, this is of no practical consequence. 'Approximation methods' for obtaining the solution are therefore acceptable.

11) The relaxation method is particularly suited in this respect. Depending upon the conditions that are to be satisfied at the various boundaries in the net, the prestressed slab equation is set up, with the usual established finite difference approximations being substituted for the derivatives. The mechanical portion of the relaxation technique can then be carried out manually, or on computers.

111) Marcus used 4x4 and 8x8 meshes while analysing ordinary square slabs, using the fundamental plate equation, and reported a maximum error of 1% in deflections and 4½% in central moments as compared
with 'exact' solutions. For composite slabs, he used a 6 x 6 mesh successfully.

In the present instance, of solving for prestressed slabs, the mesh should be adjusted for the particular problem, so that the loadprint, areas of changing flexural rigidity, and the area within the radius \( \text{L} \) are accommodated between whole squares. The circular areas will also need to be idealized into squares, either that the squares circumscribe the circle, or that the area of the idealized squared portion is made equal to the area of the actual circle.

5.7 Experimental Work

Aim
5.7.1

The aim of the experimental work was expressed as:

"To experimentally verify the concepts that had been deduced concerning the load-carrying mechanism, the criterion for cracking, and the pattern of failure of prestressed concrete pavement slabs, and to verify the applicability of the derived prestressed slab equation."
Scope And Plan
5.7.2

5.1 Prototype Slab

1) a) A prototype slab was first tested, so as to examine the mechanical aspects of the task. This slab was 12 ft long x 5 ft wide x 2 in thick. It was cast on the laboratory floor, ducts being formed in the concrete by keeping the prestressing steel wires in position, longitudinally and transversely, at the time of pouring. (The wires were spaced at 1-ft centres) The wires were tapped to and fro while the concrete hardened. This prevented any tendency to bond and enabled the wires to be used to form their own ducts.

b) When, after 28 days, the slab was moved slightly over the floor so as to make room to position the prestressing jacks, it was found that the slab had already cracked. (Photograph 44 shows the slab under the reaction frame before the load test. The painted-over cracks are visible)
c) In spite of the cracks the wires were then tensioned to give a precompression of 200 lbs/ in\(^2\) in both directions, whereupon it was found that the cracks had closed up, and that the slab could be lifted as a whole.

d) The slab was then manhandled from the laboratory to the test-bed. It was laid on a sand subgrade and loaded against a reaction frame, over a 4-in diameter loadprint. No deflection readings were taken, as only the mechanics of the casting, stressing, handling and loading operations were being worked out. The slab eventually failed by punching shear.

e) The lessons learnt from the test on this prototype slab were:

- The slab was too big. In the absence of lifting plant, manhandling a 12 ft x 5 ft x 2-in slab through a narrow laboratory door and over a distance of 50 yards to the test-bed was a major problem, especially if the slab were not to crack in transit.
The number of wires should have been even along both edges, so as to leave a clear space in the centre of the slab, free of intersecting wires, where the load could be applied.

- The slab should have been square in plan, so that the effects of loading would be symmetrical, thus making observation and analysis easier.
- The arrangements for pouring the slab, stressing the wires, and applying the load were eminently satisfactory.

f) It was therefore decided:

- to make the slabs 6 ft long x 6 ft wide and 1 1/8 in. in thickness.
- to keep the spacing of the wires at 1 ft, since, with the new dimensions, 6 wires could be positioned along each edge.
- to keep all other arrangements as they were for the prototype test.

3 Plan

The plan of the task was then outlined thus:
1) A set of four slabs, with varying degrees of prestress, would be tested on each of three types of subgrades.

ii) The slabs in each set would be stressed thus:

   a) 250 lbs/in\(^2\) (along the X axis) \times 250 lbs/in\(^2\) (along the Y axis).

   b) 250 lbs/in\(^2\) (along the X axis) \times 125 lbs/in\(^2\) (along the Y axis).

   c) 250 lbs/in\(^2\) (along the X axis) \times 0 lbs/in\(^2\) (along the Y axis).

   d) 0 lbs/in\(^2\) (along the X axis) \times 0 lbs/in\(^2\) (along the Y axis).

The stressing would be done in the laboratory so as to obviate subgrade friction and hygrothermal effects. Slabs would be cast, stressed and tested such that the age of each slab was the same (28 days) at the time of testing.
iii) The subgrades would be coarse aggregate, sand, and black-cotton soil respectively.

(These subgrades were chosen as they are representative C, S and G soils)

The subgrades would not be compacted, so as to give the anticipated flexibility of the pre-stressed slabs full play by setting up conditions that would enhance deflections. The subgrades would be brought to the same condition before each fresh test.

iv) The slabs would be loaded in the centre over a 4-in. diameter loadpoint. The diameter was chosen so as to be appreciably larger than the slab thickness, and of a much smaller dimension than the slab length. Deflections would be observed at stated points along the X and Y axes, for each increment of load. The incidence of cracks would be noted. The pattern of cracking at failure would be observed. A photographic record of the testing would be maintained.

v) Deflection profiles for each slab would be drawn. Load-deflection curves would also be drawn,
and from these graphs the loads at each stage of cracking would be evaluated.

vi) The deflections under the loadprint and at any other convenient points would be noted from the graphs, corresponding to the load at each stage of cracking. These values for the deflections would be compared with the calculated values obtained by applying the derived prestressed slab equation.

vii) The curvature of each slab at the critical sections, corresponding to each stage of cracking, would be obtained from the deflection profile graphs, using the relationship derived in sub-section 5.4.2. This curvature would be compared with the critical curvature for the thickness of the slab, corresponding to the critical strain in flexural tension as obtained from tests on representative specimens of the concrete.

viii) Control tests would be performed on the:

a) Fine aggregate for the concrete
b) coarse aggregate for the concrete

c) cement

d) hardened concrete from works test specimens (cubes, cylinders, beams)

e) hardened concrete cored from the slab

f) the material from the subgrade.

Apparatus Used

5.7.3

1 For Testing Concrete, Aggregates, Cement

i) Olsen Model L 400,000 lbs Universal Testing Machine.

ii) Soil-test Inc. 'Hot-It' Concrete Core cutting Machine.

iii) Cawkwell Ultrasonic Concrete Test Apparatus.

iv) Other items of cement and concrete testing apparatus as per ISS 269 and ISS 516.
v) Items of aggregate testing apparatus as per ISS 2366, including aggregate impact and crushing apparatus, and elongation/flakiness index test apparatus.

.2 For Prestressing The Concrete

1) Gifford-Udall 25-inch extension pre-stressing jack and accessories.

ii) 7 mm. high tensile steel wire, of 280,000 lbs/in² ultimate tensile strength.

.3 For Loading The Slab And Measuring Deflections

i) Reaction frame, consisting of a steel table, loaded with sandbags.

ii) 10 - ton hydraulic jack.

iii) 10 - ton proving ring.

iv) Dial gauges reading to 0.0005 inch.

v) Datum bars and gauge holders.

vi) Hardened steel disc, giving a loadprint of 4-in diameter.
For Evaluating Subgrade Characteristics

i) Modified AASHO Compaction Apparatus.

ii) Plate-beaming apparatus.

iii) Tanifuji Impact-Type Beaming apparatus.

iv) CBR apparatus.

v) Liquid Limit device.

Test Procedure & Results : Fine Aggregate For Concrete

5.7.4

i) The fine aggregate to be used in the concrete was tested for the properties noted below according to the methods specified in IS 2386 :

a) Grading.

b) Clay, Silt and Fine Dust Content.

c) Specific Gravity.

d) Bulk Density.

e) Organic Impurities.
ii) a) The results of the tests are given in Table T-3.

b) The sand was found suitable.

Test Procedure & Results: Coarse Aggregate For Concrete

i) The coarse aggregate to be used in the concrete was tested for the properties noted below according to the methods specified in ISS 2386:

a) Grading.
b) Specific Gravity.
c) Bulk Density.
d) Aggregate Crushing Value.
e)Aggregate Impact Value.
f) Flakiness Index.
g) Elongation Index.

ii) The results of the tests are given in Table T-4. The coarse aggregate was found suitable.
Test Procedure & Results : Cement

5.7.6

1) The cement was tested for the properties noted below according to the methods specified in ISS 269:

   a) Setting times.
   b) Soundness.
   c) Fineness.
   d) Briquette Tensile Strength.
   e) 2.78-in Cube Strength in Compression.

ii) a) The results of the tests are given in Table T - 5.

   b) The cement showed a strength development significantly below the minimum specified. However, no other cement of the requisite quality was available, and the cement therefore had to be used.

Test Procedure & Results : Concrete

5.7.7

1) a) A mix of 1 : 1½ : 3 (cement, fine
aggregate, coarse aggregate) by weight was used. Design curves indicated that, with a water-cement ratio of 0.65, the 28-day, 6-in cube crushing strength should have been about 5000 lbs/in$^2$.

The apparently high value for the water-cement ratio was taken because of the dry condition of the aggregates: the slump finally achieved was 1 - 1½ ins.

ii) The following tests were conducted:

a) Slump and Kelly Ball Tests on the fresh concrete.

b) Laboratory 6-in cube test, on the design mix.

c) Works 6-in cube test.

d) Works flexural test.

e) Works 6-in diam x 1½-in disc test.

f) Modulus of Elasticity In Compression (From ultrasonic test relationships).
g) Job 6-in diam x 1\(\frac{1}{2}\)-in disk test (from cores drilled from the slab immediately after the load-deflection tests.)

( A 'works' test is a test conducted on a specimen moulded from the concrete that is being poured into the slab at the works site.

A 'job' test is a test conducted on a specimen extracted from the finished, hardened job. Unlike a 'works' test, a 'job' test therefore evaluates the characteristic of the concrete as it actually exists in the finished construction).

iii) a) The results of the tests are given in Table 7-6.

b) The concrete developed a strength much below what the design curves indicated. This was presumably due to the poor quality of the cement.

c) The actual strength of the concrete as poured and hardened in a job is almost always significantly less than the strengths that are obtained from works specimen tests, due to different conditions
of compacting, curing, boundaries, environment, etc.

It was therefore thought desirable to obtain a rationally estimated value for the actual strength of the concrete in the job, rather than to rely on the results of works cube tests.

The estimated job strengths were therefore worked out by comparing the results of 'works 6-in disc tests' and of 'job 6-in core tests', which was the only means whereby the strengths of the two concretes could be compared. The works specimen test results were then scaled down proportionately to obtain the estimated strengths of the concrete as it existed in the slab.

d) The critical tensile strain at failure in flexure was then evaluated on the basis of the estimated job flexural strength, by applying the equation of simple bending. The critical curvature to which a 1\(\frac{1}{2}\)-in slab would have to be bent so as to produce this critical strain was calculated.
Test Procedure & Results: Coarse Aggregate Subgrade

5.7.6

i) The coarse aggregate subgrade comprised aggregates between 20 mm. and 4.75 mm in particle size. The size of the aggregates obviously precluded the CBR test being performed.

ii) The Japanese Impact - Type apparatus was used. (See Photograph No 24) The K-value obtained by this test was about 10% higher than the value obtained from the conventional plate-bearing test.

iii) The plate-bearing test was performed over a 12-in diameter plate. (See Photograph No 85) The k-value that was to be used in the prestressed slab deflection calculations was obtained as an approximation from the results of the plate-bearing test, averaged out over a range of plate deflections up to 0.75 in. The figure 0.75 in. was chosen as it was anticipated that deflections in the loaded prestressed slabs would not go beyond this value. Because the progressive load in the slab tests would be applied only once, the plate-bearing test was conducted for only one application of load.
iv) The results are given in Table T-7.

Test Procedure & Results : Sand Subgrade

6.7.9

1) The tests that were performed on the sand subgrade were :

   a) Grading
   b) Classification
   c) Natural Moisture Content
   d) Modified AASHO Compaction : OMC

and maximum dry density.

   e) Laboratory CBR, on unsoaked samples, compacted at OMC to maximum dry density.

   f) The Japanese Impact - Type test. The test was attempted, but the soil mass was not dense enough for any useful readings to be obtained.

   g) The in-situ CBR test was attempted. (See Photograph No 69) The sand was so loose that very large penetrations were obtained for small loads. The results could not be used.
h) Plate-bearing Test. The in-situ sand in the test-bed was left loose, as it would have been for the actual load tests on the prestressed slabs. The observations made regarding the plate-bearing test on the coarse-aggregate subgrade apply in this instance also.

ii) The results of the tests are given in Table T-8.

Test Procedure & Results: Black-Cotton Soil Subgrade

5.7.10

1) The tests that were performed on the black-cotton soil subgrade were:

   a) Natural Moisture Content.

   b) Liquid Limit.

   c) Plastic Limit.

   d) Classification.

   e) Modified AASHO Compaction: OMC and maximum dry density.

   f) Laboratory CBR, on unsoaked samples, compacted at OMC to maximum dry density.
g) The Japanese Impact - Type test was attempted. The soil mass was not dense enough for any useful readings to be obtained.

h) The in-situ CBR test was attempted. The soil was so loose that very large penetrations were obtained for small loads. The results could not be used.

i) Plate-bearing Test. The in-situ black-cotton soil in the test bed was left uncompacted, as it would have been for the actual load tests on the pre-stressed slabs. The observations made regarding the plate-bearing tests on the coarse-aggregate subgrade apply in this instance also.

ii) The results of the tests are given in Table T-9.

Procedure For Casting And Stressing The Slabs

5.7.11

i) a) Each slab was cast within a wooden formwork, 1½ in high, the concrete being poured over
the smooth PCC floor of the laboratory, which was used to form the underside of the slab. Oilpaper was used to break the bond between the slab and the floor, and to provide a friction-reducing base during the prestressing operation.

b) The wooden edge-formwork had holes drilled parallel to the length, and spaced 1 ft apart. These holes were meant to hold and position the prestressing wires that were run straight through the slab. The holes were therefore drilled just off-centre of the height of the forms such that the wires from adjacent sides of the slab would be just clear of each other where they crossed at right angles in the concrete, the points of contact lying in a horizontal plane at the mid-height of the slab.

c) The concrete was poured till the height of pour was just over 0.75 in, after rodding and vibrating. The 7 mm. prestressing wires were then passed through the holes in the formwork. A square mesh of separated wires was thus formed at mid-height, the ends of the wires being kept protruding from the forms to allow sufficient length of grip for subsequent prestressing.
d) The rest of the concrete was then poured and compacted so as to cover the wires and to make up the whole height of the slab. (1\(\frac{1}{2}\) ins) The wires were tapped very gently so as to form a smooth deck around each wire, without the concrete being disturbed. After the final set of the cement, the wires were gently tapped to and fro, periodically, so as to inhibit any tendency to the formation of bond.

e) Even though only two slabs in each set of four would be prestressed both ways, it was decided to form ducts in all the slabs, so that the cross-sections of all the slabs would be the identical.

The loose tendons which were not to be stressed were removed from their ducts before the rest of the wires were stressed.

f) Small cylindrical steel markers were cast flush with the top of the concrete slab, to act as reference marks for deflection readings. For the sake of ease of reference, coordinate rectangular axes were considered to have been drawn on the top of each slab, the origin lying at the centre of the slab, the axes
running parallel to the sides. (Fig E - Zero). Five markers were placed along the X axis, at distances of 4\(\frac{1}{2}\), 6\(\frac{1}{2}\), 8\(\frac{1}{2}\), 10\(\frac{1}{2}\) and 14\(\frac{1}{2}\) ins. from the centre, and designated X\(_1\), X\(_2\), X\(_3\), X\(_4\) and X\(_5\) respectively. Five markers were similarly placed along the Y axis and designated Y\(_1\), Y\(_2\), Y\(_3\), Y\(_4\) and Y\(_5\) respectively. (In Fig E - Zero, only X\(_1\) and Y\(_1\) are shown). The load-print being 4 in. in diameter, the point on the slab along the axes just clear of the load print, that is, the points just over 2 in away from the centre, were designated as P\(_1\) and P\(_2\) respectively along the X and Y axes.

g) To make referencing easier, it was decided to vary the prestress in the slabs only in the direction parallel to the Y axis (except, of course, in the 0X 0 slab).

h) The slabs were numbered serially as they were cast. Quite a few slabs cracked in handling and were discarded. The twelve slabs that were finally used carried the same serial numbers as were given to them at the time of casting, to make identification uncomplicated.
11) a) The slabs were stressed using the Gifford–Udall system of prestressing. To obtain a prestress of 250 lbs/in\(^2\) parallel to an axis, each of the 6 wires in the corresponding edge of the slab was stressed to \(\frac{250 \times (6 \times 12) \times 1.5}{6}\) 4500 lbs. To obtain a prestress of 125 lbs/in\(^2\) parallel to an axis, each of the 6 wires in the corresponding edge of the slab was stressed to 2250 lbs. The force in each wire was checked after anchoring by attaching a grooved steel to the stressing end of the jack, and tensioning the wire again. The moment the anchorage left the concrete, as indicated by thin tissue inserts, the gauge reading was noted. In this way it was possible to bring the tolerance of the load on each wire to \(\pm 50\) lbs.

b) Before stressing it was ensured that the wire was absolutely free in its self-formed duct. The frictional resistance to prestressing was therefore negligible.
c) The particulars of the slabs are:

Dimensions in plan = 6 ft x 6 ft for all slabs.

Thickness = 1 1/4 ins.

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<th>Serial No</th>
<th>Prestress (parallel to)</th>
<th>X-axis (lbs/in²)</th>
<th>Y-axis (lbs/in²)</th>
<th>Subgrade over which to be laid</th>
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<td>0</td>
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Procedure For Testing The Slabs

5.7.12

1) The set-up for the load-deflection tests is shown in Photographs Nos 35, 27, 32.

ii) The slabs were loaded centrally over the 4-in diameter steel cylindrical loadprint, by means of the reaction frame and hydraulic jack described previously. Values of reaction loads were obtained from readings on a proving ring. The weight of the jack/proving ring / load print assembly was added to the reaction load to obtain the actual externally applied load on the slab.

iii) Dial gauges, rigidly fixed to the datum bars, were previously set over the reference points.

iv) Deflections were noted at each of the points \( x_1 \) to \( x_5 \), \( y_1 \) to \( y_5 \), \( p_1 \) and \( p_2 \) at each increment of load. The lifting of the corners and edges was also observed.

v) The incidence of cracks was noted.
vi) When the slab failed (such that the portion of the slab under and around the loadprint broke off from the rest of the slab and sank deep into the subgrade) observations were made on the pattern of failure as visible from the top. The separate portions of the slab (in the case of the 250 x 0 slabs and 0 x 0 slabs) were then carefully removed and observations were made as to the state of the portions remaining on the subgrade, and of the underside of the other portions. In the case of the 250 x 250, and 250 x 125 slabs, the load assembly was first removed; the sound portion of the slab was lifted bodily off the subgrade; the condition of the punched out portion, and of the underside of the rest of the slab, was then observed.
Observations And Results

5.7.13

i) The deflection profiles of each slab were drawn, relating deflections to the distances along the two axes, for the various stages of loading. These are given in the 12 graphs, Fig E - 13 to Fig E - 24. The deflection of the point 0 was estimated from the preceding portions of the curve.

ii) Load-deflection curves were drawn for each slab. The graphs connect the deflection to the load, for each reference point at which deflections were noted. These curves are given in the 12 graphs, Fig. E - 25 to Fig E - 36. The plotted points were fairly linear between distinguishable zones, and when fitted into straight lines, indicated points of abrupt changes of direction which were very consistent over the set of 6 reference points. These were averaged out, and the common point was taken to be the point at which deflections increased suddenly due to the formation of a major crack at that point, which itself indicated a change in the manner in which the slab was responding to the load.
Thus, the first abrupt change of direction indicated the formation of the bottom cracks; the second abrupt change of direction indicated the formation of the top cracks. (In the case of the (250 x 0) slabs, and the (0 x 0) slabs, ultimate failure also occurred at this second stage). The loads causing bottom cracks, top cracks and punching failure are thus obtained from these load-deflection curves.

In the case of the (250 x 250) and the (0 x 0) slabs, the deflections for the X stations and P<sub>1</sub> are almost identical to those for the Y stations and P<sub>2</sub> respectively, and are therefore not shown. In the case of the (250 x 125) and the (250 x 0) slabs, the deflections along the Y axis are critical, and only these are shown.

iii) The patterns of failure of each slab are given in Fig E - 1 to Fig E - 12, which are drawn to scale. It is to be noted that for the (250 x 0) and the (0 x 0) slabs, the bold lines show the cracking pattern as observed at the top of the
slab when it had failed. The straight cracks extending from the corners of the irregular, quadrilateral shaped central cracked portion to the mid-portions of the corresponding edges of the slab, though shown in bold lines, were originally bottom cracks that had worked their way to the top. They are therefore also shown by dashed lines.

iv) a) The critical deflections occur under and immediately around the loadprint. It was therefore decided, while comparing the observed and calculated values of the deflections, to make this comparison at the locations of these critical deflections. Comparisons were therefore made at the points $0$, $p_2$ and $y_1$.

b) The deflections were calculated at the stage of bottom cracking and at the stage of top cracking, for all the slabs. The deflections were also calculated at the stage of the top punching shear, in the case of the two-way prestressed slabs. Calculations were based on the derived prestressed slab equation.
c) The intensity of external applied load to be used in the equation was that obtaining at the respective stage of cracking.

d) The critical curvature in a slab, up to the point of bottom cracking, occurs under the location 0 of the load. The critical curvature between the stages of bottom and top cracking occurs at the location $S_p$. The actual curvatures at these locations were obtained by applying the curvature equation of sub-section 5.4.2 to the actual deflections obtaining at these locations respectively. Comparisons were then made with the critical curvature for the slab as obtained from the previous flexural tests.

e) The results of the relevant observations and calculations are noted in Tables E-37.1 to E-37.4.

f) Table E-37.1 records the preliminary data.

Table E-37.2 gives the calculated and observed values at the stage of bottom cracking.
Table E - 37.3 gives the calculated and observed values at the stage of top cracking.

Table E - 37.4 gives the calculated and observed values at the stage of punching failure.

v) Observations made on the lifting of the corners and edges indicated that, for a given prestress, the lifts increased with decreasing $k$ - values of the subgrade; and that, for a given subgrade, the amount of corner lift decreased with increasing prestress, the load being constant.

The edges of the (250 x 250) slabs curved concave upwards, and quite evenly, as did the edges of the (0 x 0) slabs. It was difficult to detect significant differences in the curvatures of the adjacent edges of the (250 x 125) slabs. In the case of the (250 x 0) slabs, however, the slabs curved concave upwards in a pronounced manner when viewed parallel to the direction of the prestress.
The curvature in the other direction was difficult to detect.

Discussion of Results

5.7.14

1. **Bottom Cracking: Two-way Prestressed Slabs**

1) In the case of the (250 x 250) slabs and the (250 x 125) slabs the observed curvatures under the position of the load at bottom cracking corresponded fairly satisfactorily with the critical curvature for the slab.

11) The pattern of underside cracking was clearly radial. (See Photographs Nos 25, 48, 22, 52.) The depth of the cracks under the load generally extended from the bottom of the slab to a little over half the thickness of the section. The radius of the area within which radial cracking occurred varied from about 7 3/4 ins. for the slabs on the coarse aggregate subgrade, to about 10 ins. for the slabs laid on sand, to about 11 ins. for the slabs laid on the black-cotton soil. These radii extend up to the point CPb, and correspond well with
the respective radii of relative stiffness. The
depth of the radial cracks tapered out to zero at
the outer limits of the cracking zone. The area of
underside radial cracking thus comprised a number
of wedge-sectioned sectors. (See Photographs, Nos
105, 124).

iii) The observed and calculated deflections
at the points 0, P₂ and Y₁, corresponding to the load
at which the both cracks were deduced to have
occurred (from the load-deflection curves), were in
fairly good agreement.

2 Bottom Cracking : One-Way Prestress Slab

1) The observed curvatures under the
position of the load, looking transverse to the
direction of the critical Y-axis, corresponded
fairly satisfactorily with the critical curvature
for the slab.

ii) The main bottom crack started from under
the position of the load, and moved out towards the
edges of the slab, keeping approximately parallel to
the direction of prestress (See Photograph Nos 56,
79, 80, 88, 79.)
iii) The observed and calculated deflections at the points 0, P₂ and Y₁, corresponding to the load at which the bottom cracks were deduced to have occurred (from the load-deflection curves), were not in good agreement. This was probably because the method assumed for depicting the different zones of flexural rigidity, prior to applying the prestressed slab equation, was not realistic. The fact that cracking occurs only along one axis complicates the problem by introducing an unsymmetry. More work needs to be done on this aspect.

.3 Bottom Cracking : (0 x 0) Slab

1) The observed curvatures under the position of the load corresponded fairly satisfactorily with the critical curvature for the slab.

ii) The cracks that initially developed on the underside of the slab extended from the position of the load parallel to the axes. It is assumed that the extent of the initial cracking was of the same order as in the two-way prestressed slabs. (See Photographs Nos 82, 83, 109).
iiii) The observed and calculated deflections at the points 0, P2 and Y4, corresponding to the load at which the bottom cracks were deduced to have occurred (from the load-deflection curves), were in fairly good agreement.

4 Top Cracking : Two-Way Prestressed Slabs

1) The load causing the top crack was obtained from the load-deflection curves. By following the curve corresponding to this load in the graph of deflection profiles, the point of maximum positive curvature was obtained (producing maximum tensile strain in the radial direction at the top of the slab). This point was taken as the location where top circumferential cracking occurred. The curvature at this point was in satisfactory agreement with the critical curvature for the slab. It was very difficult visually to detect these top cracks in the slab, as they closed tight after the load was released. (The radial, underside cracks would also have closed tight, of course, except that the load, in punching through the slab at failure, punched through at the portion of the radial cracks. The whole of the underside, radial cracked portion thus came cleanly
away. Careful loading of the central portion of the slab with sandbags, after the main tests were over, enabled the top cracks to open again, though very slightly.

ii) The radius of the circumferential top cracks varied from about 6.5 ins. to 11 2/5 ins. to 12 2/5 ins., when the slabs were over the coarse aggregate subgrade, sand subgrade, and black-cotton soil subgrade respectively.

iii) The observed and calculated deflections at the points 0, P2 and Y1, corresponding to the loads at which the top cracks were deduced to have occurred (from the load-deflection curves), were in fairly good agreement.

.5 Top Cracking: One-Way Prestressed Slab

1) The load which produced top cracking was the ultimate failure load. The point of maximum curvature along the corresponding load curve in the graphs of deflection profiles was obtained, and was taken to be the location where the top crack occurred. The maximum curvature corresponded fairly satisfactory with the critical curvature for the slab.
ii) The shape of the top crack was roughly that of an elongated, jagged, irregular quadrilateral, with the longer diagonal lying along the centre of the slab, parallel to the direction of prestress. The bottom crack apparently became extended and worked itself up at failure. It finally projected from the ends of the longer diagonal towards the middle of the corresponding edges of the slab.

The slab was completely broken into a number of separate pieces, corresponding to the cracks. (See Photographs Nos 30, 58, 59, 61)

iii) The observed and calculated deflections were not in good agreement, probably for the reason given in sub-section (5.7.14 .2)

.6 Top Cracking : 0 x 0 Slab

i) The load which produced the top cracks was the ultimate failure load. The point of maximum curvature along the corresponding load curve in the graphs of deflection profits was obtained, and was taken to be the location where the top crack occurred. The maximum curvature corresponded fairly satisfactorily with the critical curvature for the slab.
ii) The shape of the top crack was roughly that of jagged, quadrilateral, with the diagonals lying along the directions of the prestress. The bottom cracks apparently became extended and worked themselves up at failure. They finally projected from the ends of the diagonals to the middle of the corresponding edges of the slab.

The slab was completely broken into a number of separate pieces, corresponding to the cracks. (See Photographs Nos 82, 83, 64, 65, 66).

iii) The observed and calculated deflections were in moderately fair agreement.

.7 Punching Shear Failure : Two-Way Prestressed Slabs

i) The two-way prestressed slab continued to take load after the stage of top cracking, till the loadprint suddenly punched cleanly through the slab. (See Photographs Nos 47, 71, 72, 46). The failure was primarily a shear failure, aggravated by the flexural cracks that were extant in the slab. The hole in the top of the slab corresponded neatly to the shape of the footprint. As the shear developed
through the thickness of the slab, it carried the radially cracked wedge-shaped sectors along, so that this central, conical shaped portion became separated from the rest of the slab. The conical portion was deposited on the depressed subgrade (See Photographs Nos 22, 52). The rest of the slab, now relieved of the load, sprang back into its original horizontal plane, its structural integrity preserved, except for the central 'hole'. (See Fig E - Zero, Stage 6; Photograph No 140).

ii) Critical curvatures, which produce flexural tensile cracks, had no meaning in this context.

iii) The observed and calculated deflections were in moderately fair agreement.

6 Load Capacity

1) The two-way prestressed slabs showed a large capacity for taking load between the stages of initial, bottom radial cracking and of top circumferential cracking. They showed an even greater capacity for taking load between the stages of top cracking and of ultimate failure in punching shear.
Even so, in practice they should not be worked much beyond the stage of bottom cracking, so as to provide a reservoir of over-load carrying capacity.

ii) One-way prestressed slabs showed a restricted capacity for taking loads up to the stage of initial bottom cracking, and between the stages of bottom cracking and of top cracking, when they failed.

These slabs should not be worked much that bottom cracks appear transverse to the direction in which there is no prestress.

iii) (0 x 0) slabs similarly showed a restricted capacity for taking loads up to the stage of initial bottom cracking, and between the stages of bottom cracking and of top cracking, when they failed.

These slabs should not be worked up to the stage of bottom cracking.

iv) For a given subgrade, the load-capacity of the slab increased with the prestress.
v) For a given subgrade, the (250 x 0) slab and the (0 x 0) slab cracked initially at loads that were approximately the same.

vi) On the average, the loads that caused top cracking were about 3.9 times as large as the loads that caused the initial cracks at the bottom of the slabs.

vii) On the average, the ultimate failure loads in the two-way prestressed slabs were about 8 times as large as the loads that caused the initial bottom cracks.

9 Radii Of Cracking; Radii Of Subgrade Reaction

i) The radius, $L_T$, of the area within which radial cracks occurred, was generally equal to about the radius of relative stiffness, $L$.

ii) The radius of circumferential cracking, $L_C$, was generally equal to about 1.1 $L$ to 1.2 $L$. 
iii) The radius $L_b$ was generally equal to about 2.2 $L$ to 2.4 $L$.

iv) The radius $L_e$ was generally equal to about 1.8 $L$ to 2.0 $L$.

v) The radius $L_p$ was generally equal to about 1.1 $L$ to 1.2 $L$.

vi) The radius $L_g$ was generally equal to about 3.5 $L$.

vi) For a given subgrade, and for a given load, the two-way prestressed slabs deflected less than did the other slabs. The deflections at cracking were, of course, greater in the case of the prestressed slabs, as the load required to produce cracking was much greater for these slabs.