Basis for Rigid Pavement Design for Military Airfields

by

R. L. Hutchinson

MAY 1966

DEPARTMENT OF THE ARMY
OHIO RIVER DIVISION LABORATORIES, CORPS OF ENGINEERS
CINCINNATI, OHIO 45227

PROPERTY OF THE UNITED STATES GOVERNMENT
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BASIS OF RIGID PAVEMENT DESIGN CRITERIA
FOR MILITARY AIRFIELDS

by

R. L. Hutchinson

INTRODUCTION

The U. S. Army Corps of Engineers was assigned the responsibility for the design and construction of military airfield pavements in 1940. At this time, the then designated heavy bomber aircraft (specifically the B-17 and B-24) had gone into production and it was obvious that the wheel loadings of these aircraft would be far in excess of any vehicular or aircraft loadings existing at that time. The Chief of Engineers in turn assigned responsibilities for the development of criteria for flexible pavements to the U. S. Army Waterways Experiment Station, Vicksburg, Mississippi and for rigid and overlay pavements to the U. S. Army Engineer Division, Ohio River, Ohio River Division Laboratories, Rigid Pavement Laboratory.

A review of then available methods for the design of rigid pavements soon led to the conclusion that there was no shortage of methods in use since practically every state in the United States and many foreign countries had developed and were using design procedures that invariably differed in some particular from each other. The differences in design procedure, more often than not, could be traced to the incorporation of experience with locally available materials and empiricisms developed from performance records in the particular area of application. Because of the immediate need for criteria that could be applied universally for all conditions that would be encountered, both in this country and abroad, it was imperative that the criteria be simple, practical, and uniform. The Corps of Engineers then embarked upon a rigid pavement investigational program that would accomplish the following objectives: (a) obviate the use of untried methods; (b) insure adequately

* Now designated as the Construction Engineering Laboratory
designed pavements; (c) provide methods not subject to variation occasioned by arbitrary cost differences of local competitive materials; (d) avoid reductions in pavement thickness in order to balance cost; and (e) establish procedures that would readily lend themselves to further development through tests, investigations, and study of actual pavement behavior.

To accomplish the above objectives, a four-part investigational program was established in 1941 and consisted of: (a) theoretical studies; (b) small scale model studies; (c) full-scale accelerated test track and miscellaneous field studies; and (d) condition surveys of existing rigid airfield pavements. To present the results of the vast number of studies conducted under this investigational program would be beyond the scope of this paper; however, the results of the program have been well covered in the technical literature and have been used for the development of the published criteria which is used throughout the world by the Corps of Engineers for the design and construction of military airfield pavements. From these studies, design and construction criteria have been developed and published for both plain and reinforced rigid pavements and for rigid and non-rigid types of overlays which are used to strengthen existing rigid pavements. In addition, investigations of prestressed rigid pavements have been performed and a tentative, but unpublished, design criteria has been developed.

The investigational program, as established in 1941, continued until about 1955 with emphasis on the various facets of design and construction to maintain the criteria current with developments of aircraft and improvements in technology. By 1955, the pavement criteria for the then heaviest aircraft, the B-52, had been developed and some studies had been conducted to develop thickness requirements for a 4-wheel gear loading of 325 kips, representing a gross aircraft load of about 700 kips. It was felt, at that time, that the criteria available would be adequate for aircraft loadings in the foreseeable future and it was decided to re-orient the investigational program to place more emphasis on studies of materials and procedures for maintaining the existing pavements. Therefore, from 1956 to the present time, the major emphasis in the investigational program has been on the development of better joint sealing materials and the development or improvement of methods for
bonding concrete to concrete for repair and strengthening purposes. Some effort has been continued on theoretical studies, small scale static load model studies, and condition surveys of existing pavements to assure that the established criteria is kept up-to-date in accordance with changing aircraft operational characteristics and improving technologies in materials and construction. A minimal effort is being performed in establishing the structural benefits derived from stabilized soil layers within the pavement system.

This paper has been organized to present the various factors which influence the current design criteria with a brief explanation of how the numerical values of each was derived. No attempt has been made to present detailed results of the various investigations that have been made and how the criteria have been revised periodically to reflect these results. However, a list of pertinent references has been included which, for those interested, give a relatively comprehensive history of the development of the criteria now used for the design of rigid pavements for military airfields.

**BASIC LOADING AND STRESS CONDITIONS**

There are three basic conditions of loading on a concrete pavement; (a) interior loading; (b) corner loading; and (c) edge loading. For all three loading conditions, the load is assumed to be applied uniformly over an area equal in size and shape to the footprint of the tire on the pavement. The interior loading condition assumes the load to be applied at some distance from an edge or corner of the slab and the maximum tensile stress occurs on the bottom of the slab directly under the center of the loaded area. The corner loading condition assumes the load to be applied at the corner of the slab with the loaded area tangent to the edges of the slab. The maximum tensile stress occurs in the top of the slab at some distance back from the corner. The edge loading condition assumes the load to be applied with one edge of the loaded area tangent to the edge of the slab at some distance from the corner of the slab. The maximum tensile stress occurs at the edge, and is parallel to the edge, of the slab.
The interior loading condition was first adopted for the design of airfield pavement simply because at that time more study had gone into the analysis of stresses due to this type of loading. Equations were available which gave reasonable values of stresses and deflections for interior loading, which were being used for the design of highway pavements. However, edge loading was, even then, considered to be as, or more severe than interior loading as evidenced by the common use of thickened edge pavements. Tests by the Corps of Engineers substantiated the belief that edge loading produced greater stresses and when equations suitable for determining stress due to edge loading became available, these were readily adopted for airfield pavement design and are still used. The criteria for the design of rigid pavements for military airfields currently assume the critical loading on a concrete slab to be with a wheel of the main gear oriented tangent to a free edge or joint in the pavement.

DETERMINATION OF CRITICAL STRESSES

Three methods are used for the determination of the critical stress induced by the above described three loading conditions. These methods are: (a) by equations; (b) by influence charts; and (c) by small scale static load model tests. Each of these methods is limited to the determination of maximum stress induced by a static wheel load and do not take into consideration the effects of repetitive loading or such stress producing factors as: expansion and contraction of the concrete due to temperature changes, warping and curling due to temperature or moisture gradients in the pavement thickness, or non-uniform support of the pavement slab. The effects of such factors have been considered through the use of a "design factor" which is discussed later.

The equations for the determination of critical stress are limited to single wheel loadings for practical use and have been found to be incorrect for multiple wheel gears with large wheel spacings. The influence charts afford the simplest method for critical stress determination; however, the stresses thus determined have been found to be slightly higher than the
measured stresses obtained by model tests and full-scale slab tests. A brief description of each method for determining the critical stress for edge loading is included herein. In the development of design criteria, the critical edge stress is determined by influence charts with spot checks using measured stresses obtained by model studies. If the difference in critical edge stress by the two methods is significant, an average of the two is used for the development of criteria. For the preparation of rigid pavement design charts, the critical edge stress for static load of the aircraft gear is always determined and the stresses are then modified as necessary based upon other factors which have been determined to affect pavement performance.

Determination of Critical Stress by Equation

The three basic loading conditions were examined by Dr. H. M. Westergaard, and equations for computation of the critical stress from each loading condition was presented in papers published in 1926(1), 1933(2), 1942(3), 1945(4), and 1948(5). Westergaard's early work was concentrated on interior loading and it was not until 1946 that he published equations specifically applicable for the computation of stresses due to edge loading of large wheel loads on large contact areas. In practically all of his work, Westergaard assumed the subgrade to be represented by a "dense liquid" and called the required property of the soil for use in his equations the "modulus of subgrade reaction, k."

Westergaard's general equation for the computation of the critical stress due to edge loading is:

\[
\sigma_e = \frac{12(1+\mu)P}{\pi(3+\mu)h^2} \left[ K + 0.8659 - \frac{\mu}{4} - B_1 + \frac{1+\mu}{4} S + B_2 \frac{f}{h} \right] \ldots \ldots \ldots \quad (1)
\]

where

\[
\sigma_e = \text{maximum edge stress, psi,}
\]

\[
P = \text{wheel load, lbs.},
\]

Raised numbers in parenthesis refer to references at end of paper.
\[ h = \text{slab thickness, inches,} \]
\[ \mu = \text{Poisson's ratio of concrete,} \]
\[ y = \text{distance from edge of slab to center of gravity of the} \]
\[ \text{loaded area (contact area), inches,} \]
\[ B_1 \text{ and } B_2 = \text{dimensionless constants dependent upon } \mu, \]
\[ B_1 = 0.9627 \text{ and } B_2 = 0.4131 \text{ for } \mu = 0.20, \]
\[ l = \text{radius of relative stiffness in inches and is a measure of} \]
\[ \text{the stiffness of the slab relative to that of the subgrade,} \]
\[ l \text{ is computed from the formula:} \]
\[ l = \left[ \frac{Eh^3}{12(1-\mu^2)k} \right]^{1/4} \]
\[ k = \text{modulus of soil reaction, psi/in.,} \]
\[ E = \text{modulus of elasticity of concrete, psi,} \]
\[ K = \text{an area coefficient,} \]
\[ S = \text{an area coefficient.} \]

The tires normally used on aircraft produce an elliptical footprint and, for
\[ \text{this specific case, the above equation becomes:} \]
\[ \sigma_e = \frac{3(1+\mu)P}{\pi (3+\mu) h^2} \left[ \log \frac{Eh^3}{100k (\frac{a+b}{2})^4} + 1.84 \frac{\mu}{3} + \frac{4}{3} \mu + (1+\mu) \frac{a-b}{a+b} \right. \]
\[ \left. + 2(1-\mu) \frac{ab}{(a+b)^2} + 1.18(1+2\mu) \frac{b}{s} \right] \quad \ldots \ldots \ldots \ldots \quad (2) \]

where
\[ a = \text{semi-major axis of footprint, inches,} \]
\[ b = \text{semi-minor axis of footprint, inches,} \]
\[ K = \log \frac{2a}{a+b} + \frac{a-b}{2(a+b)}, \]
\[ S = \frac{2ab}{(a+b)^2} - \frac{a-b}{a+b} \]

The radius of relative stiffness, \( l \), is a very important parameter in
\[ \text{rigid pavement design since it relates the properties of the concrete and} \]
\[ \text{subgrade. The use of the relation will be found throughout this paper and} \]
\[ \text{will often be referred to as "} s" \text{ or "} \mu \text{ value".} \]
The above equation gives the maximum free edge stress which occurs in the bottom of the slab directly under the point where the footprint is tangent to the edge of the slab. The equation makes no allowance for load transfer across joints; however, this effect is handled by a factor in the design criteria and is discussed later.

The edge stress equation was adopted as a basis for the criteria for rigid pavements for military airfields in 1946(6), and is still used. However, as mentioned previously, the equation does not readily lend itself to the computations of edge stresses under multiple-wheel gear loadings and thus the influence chart, which is a graphic solution to the equation, is normally used.

Determination of Critical Stress by Influence Chart

In a paper published in 1950(7), Pickett and Ray presented solutions of the Westergaard equations in the form of influence charts which greatly simplified the determination of theoretical deflections and moments caused by wheel loads on pavement slabs. Eight influence charts were presented which provided solutions for four different cases; (a) interior loading assuming a liquid subgrade; (b) interior loading assuming an elastic solid subgrade; (c) edge loading assuming a liquid subgrade; and (d) one half of the radius of relative stiffness from an edge assuming a liquid subgrade. In a later paper published in 1951(8), Pickett, Raville, Jones, and McCormick presented an additional sixteen influence charts for the determination of deflection, moment, and reactive pressure under interior, near edge, and near center loadings on slabs and for liquid, elastic solid, and elastic layer subgrades.

The Corps of Engineers readily adopted the influence charts for the computation of maximum tensile stress for edge loading because of the simplification involved, especially for multiple-wheel loadings. Chart 6, represented by Figure 4 of the Pickett and Ray paper(7) is used for the determination of moments due to a load tangent to an edge or joint in the pavement assuming a liquid subgrade. The chart, included herein as Figure 1, represents a solution of the Westergaard equation for edge loading.
The following example will best illustrate the use of the influence chart. Let it be desired to determine the maximum tensile stresses for a range of pavement thicknesses due to a twin-wheel main gear loading of 100,000 pounds applied tangent to the edge of the concrete slab. Pertinent dimensions of the twin-wheel gear are: (a) the contact area of each wheel is 267 sq. in.; and (b) the center-to-center spacing of the wheels is 37.5 in. It is assumed that the modulus of elasticity of the concrete is \(4 \times 10^6\) psi, that Possion's ratio of the concrete is 0.15, that the modulus of subgrade reaction, k, is 100 psi/in., and that the footprint of the tires is elliptical.

The moments will be determined by influence chart, Figure 1, but which has been drawn to a scale of \(l\) equal to 10 inches. This produces a chart approximately 26 x 19.8 inches compared to the one shown by Figure 1. Each block on the influence chart is also enlarged by the ratio of \(l\)'s (10:2.55).

The maximum tensile stresses will be determined for the pavement thicknesses shown by Table 2. These values have been selected to yield even values for \(f\) which simplifies the construction of scaled footprints for use with the influence chart. The first step is to make scaled drawings of the contact areas of the wheels on transparent paper. The contact areas are drawn to a scale represented by the ratio of the \(l\) of the pavement to the \(l\) of the influence chart. Table 1 gives coefficients suitable for the determination of an ellipse sufficiently accurate in shape to represent the contact area.

<table>
<thead>
<tr>
<th>x</th>
<th>0</th>
<th>0.4359a</th>
<th>0.8000a</th>
<th>0.9539a</th>
<th>1.000a</th>
</tr>
</thead>
<tbody>
<tr>
<td>y</td>
<td>1.000b</td>
<td>0.9000b</td>
<td>0.6000b</td>
<td>0.3000b</td>
<td>0</td>
</tr>
</tbody>
</table>

An elliptical area having an area equal to the contact areas of the wheels (267 sq. in.) has a semi-major axis of 11.90 in. and a semi-minor
axis of 7.1\textsuperscript{1/4} in. Scaled drawings are then made of elliptical contact areas by scaling these dimensions by the ratio of the pavement $l$ to the chart $l$ (see Table 2). The semi-major and semi-minor axes of the scaled drawings for each pavement thickness and the center-to-center spacing of the contact areas are also scaled as shown by Table 2. The scaled drawings of the twin-wheel gear are then placed on the influence chart and the number of blocks, including fractional blocks, falling within the scaled contact areas are counted. The point 0 on the chart represents the point where the stress determined occurs. It will be necessary to work with several orientations of the gear to determine which produces the largest number of blocks and thus the greatest stress. For the example gear, the critical orientation is found to be when the outside end of the minor axis of one area is placed on point 0 of the chart so that the minor axis is normal to the edge of the chart representing the edge of the slab. With this orientation of each scaled drawing of the contact areas, the number of blocks falling within each contact area is shown by Table 2.

Table 2

<table>
<thead>
<tr>
<th>Development of Maximum Edge Stress, $\sigma_e$, by Use of Influence Chart</th>
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<tr>
<td>Pavement Thickness, in.</td>
</tr>
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<td>Pavement $l$, in.</td>
</tr>
<tr>
<td>Scale Ratio, $l$ pavement: $l$ chart</td>
</tr>
<tr>
<td>Scaled Semi-major axis, in.</td>
</tr>
<tr>
<td>Scaled Semi-minor axis, in.</td>
</tr>
<tr>
<td>Scaled C-C wheel spacing, in.</td>
</tr>
<tr>
<td>$N_1$, Wheel I (tangent to point 0)</td>
</tr>
<tr>
<td>$N_{II}$, Wheel II</td>
</tr>
<tr>
<td>$M_1$, in.-lbs., Wheel I</td>
</tr>
<tr>
<td>$M_{II}$, in.-lbs., Wheel II</td>
</tr>
<tr>
<td>$\sigma_1$, psi, Wheel I</td>
</tr>
<tr>
<td>$\sigma_{II}$, psi, Wheel II</td>
</tr>
<tr>
<td>$\sigma_e = \sigma_1 + \sigma_{II}$, psi</td>
</tr>
</tbody>
</table>
The moment produced by each wheel is then computed by the following formula and for the example, values are shown in Table 2:

\[ M = \frac{q l^2 N}{10,000} \]  \hspace{1cm} (3)

where

- \( M \) = moment in in.-lbs. at point 0 on the chart,
- \( q \) = load intensity, psi, on the contact area obtained by dividing the wheel load by the contact area,
- \( l \) = radius of relative stiffness of the pavement, inches,
- \( N \) = number of blocks, including fractional blocks, falling within the scaled footprint.

The edge stress produced at point 0 on the chart by each wheel is computed as follows, and for the example, values are shown by Table 2.

\[ \sigma_e = \frac{6M}{h^2} \]  \hspace{1cm} (4)

where

- \( \sigma_e \) = edge stress, psi, at point 0 on the chart,
- \( M \) = moment due to wheel load, in.-lbs.,
- \( h \) = slab thickness, inches.

The total or maximum free edge stress produced by the twin-wheel loading is then obtained by summing the edge stresses produced by each wheel load \( (\sigma_e = \sigma_{I} + \sigma_{II}) \). Values of the maximum free edge stress for the example are shown in Table 2. It is pointed out that this computation assumes the load applied at a free edge of the slab or at a joint having no provision for load transfer.

This same procedure is used to determine the maximum free edge stress for a single wheel or any combination of wheels for which the scaled drawing of the gear falls on the influence chart. For gears having several wheels spaced far apart, it may be found that some of the wheels fall within the...
portion of the chart labeled "negative blocks" which indicates negative
moments at the edge of the slab and thus these blocks are subtracted from
the number of positive blocks that fall within the footprint. If a large
portion, or all, of a footprint falls on the portion of the chart representing
negative blocks, the edge stress will be negative and will be subtracted
from the stress produced by other wheels.

Determination of Critical Stress by Model Studies

Prior to the development and publication of revised formulas for edge
loading and the influence charts, the Corps of Engineers developed a small-
scale static load model for the determination of maximum tensile stresses for
each of the three loading conditions. The model was, in essence, an analog
computer and was developed, not only to check the theory, but to permit
analysis of single and multiple-wheel gear loadings regardless of spacing and
tire size. A complete description of the model and its application to design
studies of concrete airfield pavements was published in 1955(9).

The model consists of a natural rubber subgrade which yields a constant
value for the modulus of soil reaction, $k$; the concrete is represented by a
thin Hydrostone slab which can be cast to very uniform thicknesses. The
linear horizontal dimensions of the model slab are kept sufficiently large so
that the assumption of infinite or semi-infinite extent for edge, interior,
or corner loading is satisfied. Lead cubes are uniformly distributed over
the thin Hydrostone slab to provide mass without affecting stress in the slab.
The slab thickness and the gear configuration (contact areas and wheel spac-
ings) are scaled by the following basic relationship:

$$\frac{\ell_p}{r_p} = \frac{\ell_m}{r_m} \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (5)$$

where

$\ell_p$ and $\ell_m$ = radius of relative stiffness of the pavement
and model slabs, respectively.

$r_p$ and $r_m$ = radius of circular area representing the loaded
area on the pavement and model slabs, respectively.
The Hydrostone slab is instrumented to measure strains and deflections. The scaled gear configuration is placed on the slab in the position desired for study, and load is applied through a lever arm arrangement. By applying loads in increments, the load-strain relations can be developed through the elastic range of the slab and the slab can be loaded to failure to develop the ultimate failure load and the resulting crack pattern.

The small-scale static load model has been instrumental in determining; (a) the validity of the theoretical stress determination; (b) the effects of high contact pressures and different size loaded areas; (c) the load-stress relations for single and multiple wheeled gear configurations; (d) the equivalent single wheel loadings that produce stresses equal to multiple-wheel gears; (e) the efficiency of different devices in joints to transfer load across the joint; (f) the effects of bored recesses or sawkerfs in concrete on pavement stresses; and (g) the benefits of both pre-tensioned and post-tensioned prestressed concrete pavements.

The small scale static load model tests have indicated that the theory yields slightly higher stresses as illustrated by the following table which summarizes the results of selected model studies.
Table 3

Comparison of Maximum Measured vs. Theoretical Tensile Stresses Due to Edge-Loading

<table>
<thead>
<tr>
<th>Type and Critical Orientation of Gear</th>
<th>Maximum Stress, psi</th>
<th>Percent of Total Stress Contributed by Each Wheel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Measured</td>
<td>Theoretical</td>
</tr>
<tr>
<td>Single</td>
<td>939</td>
<td>1031</td>
</tr>
<tr>
<td>Twin</td>
<td>1260</td>
<td>1409</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Twin-Tandem</td>
<td>1368</td>
<td>1627</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Twin-Twin</td>
<td>1410</td>
<td>1584</td>
</tr>
<tr>
<td></td>
<td></td>
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</table>

NOTE: The relations in the above table are for the following conditions:
1. Equal loads on all wheels, \( P_m = 6 \) lbs.
2. Equal contact areas on all wheels.
3. Radius of relative stiffness, \( h_m = 1.7626 \) in.
4. \( h_i = 0.133 \) in.; \( k_m = 65 \) psi/in.
MODULUS OF SOIL REACTION, $k$

In all of Westergaard's early work, no definite procedure was ever presented for the determination of the "coefficient of subgrade reaction, $k$" used in his equations for computation of maximum stress. This property is now commonly referred to as the "modulus of soil reaction, $k$"; and is expressed in pounds per square inch per inch or more simply in pounds per cubic inch. Westergaard recognized that this property of the subgrade would have considerable variation, even for a given set of conditions, depending upon the size of the loaded area and bending or deflection distribution of the pavement. He often referred to the "$k$-value" as an "empirical makeshift which, however, has been found in the past to give usable results".

The development of a test procedure to yield a value of $k$ for the subgrade that would satisfy the equation for stress constituted one of the early field studies by the Corps of Engineers. Through a series of loading tests on slabs (10) and plate tests on the subgrade supporting the slabs, two procedures were developed that would give reasonable values for the $k$ when used in the equations for stress. These procedures are known as the volumetric displacement method and the field plate bearing test.

**Determination of $k$ by Volumetric Displacement**

During the field test program, it was found that if the volume of the deflection basin of a slab, loaded through a uniformly loaded area, was determined and divided into the total load applied to the slab, the result was a $k$ value that when substituted into the Westergaard equations gave values of stress comparable to the values measured by means of strain gages on the loaded slabs. This procedure, while giving realistic $k$ values, was not considered feasible for design purposes since it would require the construction of large size slabs for each type and condition of foundation on which the pavements might be constructed. However, the procedure did offer possibilities when it was desired to determine the $k$ value of the soil under an existing pavement.
The procedure is relatively simple and requires only equipment to measure the slab deflection and a load heavy enough to produce measurable pavement deflections. The procedure has been used quite satisfactorily on existing pavements by using a loaded aircraft and a surveyors level and scale. The aircraft is positioned with the main gear in the center of a slab. Reference points are marked on the surface of the pavement, generally starting at the edge of the wheel and on two perpendicular lines from the wheel for a distance sufficiently large to extend outside of the deflection basin. With the loaded aircraft in position, readings are made on the scale at each reference point. The aircraft is then towed away, and after some time to permit rebound, readings are again made at each reference point. The difference in reading is the pavement deflection and thus the subgrade deflection. From the deflection measurements, the volume of the deflection basin is computed and divided into the gear load of the aircraft to obtain a k value in lbs./cu. in.

**Determination of k by Field Plate Loading Test**

From the field test program, it was determined that the load-deflection curve obtained using rigid steel plates, 30 inches or greater in diameter, would yield essentially constant k values that were comparable to the values determined by the volumetric displacement method and satisfied the values needed in the Westergaard equations for stress. Since the plate test offered a more feasible method for determining the k value, it was adopted for design purposes; and, with only minor modifications to procedure, is still used today.

The apparatus and test procedure for determining the k value by field plate loading test is described in detail in a published manual of test methods for pavements (7). The test consists of applying incremental loadings to a 30-inch diameter steel or aluminum alloy plate, of specified stiffness, and allowing each load increment to remain until essentially all settlement has occurred. A load-deflection curve is plotted and corrected by drawing a line parallel to the straight line portion of the load-deflection curve (or portion of the curve having the least curvature) through the origin. The plate
deflection at a unit loading of 10 psi is then determined from the corrected curve and divided into the 10 psi load to determine the $k$ in lbs/sq in/in. The value thus obtained is a field value, designated as $k'u'$, and is subject to corrections for bending of the plate during loading. The correction for plate bending is obtained from a prepared chart. The value after this correction is designated $k'u'$, and is representative of the modulus of soil reaction at its in situ moisture content.

Pavements are, however, designed based upon the $k$ value of the saturated subgrade since there is always the possibility that it will become saturated during the pavement life. A further correction to the $k$ value is then necessary for design. This correction is made based upon the deformations under a 10 psi loading of specimens placed in a consolidometer; one specimen at its in situ moisture content and the other specimen saturated with water after it has been placed in the consolidometer and prior to the application of the 10 psi loading. From the consolidometer tests, a ratio of the deformation at in situ moisture condition, $d$, to the deformation at a saturated condition $d_s$, is determined and multiplied by the $k'u'$ value. The resulting value is designated as $k$ and is the value used for the pavement design.

The field plate bearing test for the determination of the $k$ value has and still offers problems to design agencies because the test is cumbersome to perform and because it must be performed on the surface of the foundation representative of that on which the pavement will be placed. The $k$ value and pavement thickness must, of course, be known prior to awarding the contract. This means that there will be no prepared foundation for the pavement on which to perform the $k$ test; and, in cases where base courses are used, the source of the material will not be known. Therefore, in order to obtain a $k$ value for design, it is necessary to construct test sections which incorporate the various types of subgrade that will be encountered and the thicknesses and types of base course materials that can be used. Fortunately, the required pavement thickness is not particularly sensitive to the $k$ value; and thus, it is only necessary to establish the range rather than the absolute value.

The adequacy of the 30-inch diameter plate test for the determination of the $k$ value has been questioned many times and perhaps rightly so. The plate bearing test has been used, along with other tests, to evaluate the reaction
of the foundation materials in all static or repetitive load tests conducted by the Corps of Engineers and, in each case, the plate test always appears to yield the best value of k for analysis of the tests; however, with increases in both load and size of loaded area, larger stresses occur deeper and deeper into the subgrade, and the adequacy of the 30-inch diameter plate test to yield a realistic value of the k becomes more questionable. Also, as more use of thick base courses and stabilized soil layers are used, it is believed that another method of assessing a foundation strength value may be needed.

Various methods for determining the k value have been presented by many agencies and vary from that used by the Corps of Engineers which is based upon the so-called elastic properties of the soil. The method employed by the Corps of Engineers is based upon both the elastic and plastic properties of the soil at a unit loading approximately equal to that to which the soil will be subjected by the aircraft loading on the pavement. In the plate bearing test, by allowing the load to remain for some time, the elastic strength of the soil is exceeded and the soil deforms plastically due to settlement. Thus the k test by the Corps of Engineers method simulates a standing or slow moving aircraft load on the pavement where the load is applied for a time sufficient for the soil to deform plastically. While much of the plastic deformation of the soil can be eliminated by the requirement of high densities during construction, it has not been found to be economically feasible to eliminate all plastic deformation, which would permit designing based upon elastic properties alone. Experience indicates that plastic deformations occur in the soils to very large depths (up to 10 ft.) under thick pavements and heavy loads supported on multiple-wheel gears. Thus, compaction of the subgrade soils would have to be accomplished to these large depths; and extensive excavation and replacement would be required. Admittedly, the unit pressures at these large depths are small; however, plastic deformations take place in soils at very small unit pressures. Therefore, the Corps of Engineers has retained the plate bearing test as a method for determining the k value; since in all probability, the reactive pressures offered by the subgrade soils will be due to the combined elastic and plastic properties of the soil.
Determination of Concrete Strength

Since the critical stress in the concrete slab occurs due to bending of the slab under the applied wheel loading, the Corps of Engineers has adopted the flexural strength of the concrete as being the most representative property of the concrete for design purposes. The flexural strength is determined using a simple beam with third point loading as described in CRD-C 16-54[12]. The flexural strength of the concrete for design purposes is determined from laboratory tests of concrete mix designs made prior to construction. Samples of the aggregates and cement that will be used are submitted to the laboratory whereupon, by trial batches, the most economical mix is determined. Concrete beams, 6x6x18 inches, are cast from the design mix and flexural strength tests are performed at ages of 7, 28, and 90 days. This develops the normal strength gain with time relation as well as a strength for the determination of required thickness. From the third point loading test, the flexural strength is determined by the formula:

\[ R = \frac{PL}{bd^2} \]  

(6)

when the break occurs within the middle third of the span or by

\[ R = \frac{3Pa}{bd^2} \]  

(7)

when the break occurs outside of the middle third of the span by not more than 5% of the span length. If the fracture occurs outside of the middle third of the span by more than 5% of the span length, the results are discarded. The following notation applies to equations (6) and (7):

- \( R \) = flexural strength, psi
- \( P \) = maximum applied load, lbs.
- \( L \) = span length, in.
- \( b \) = average width of specimen, in.
- \( d \) = average depth of specimen, in.
a = distance between line of fracture and the nearest support measured along the centerline of the bottom surface of the beam, in.

For most airfield pavements, the flexural strength of the concrete at an age of 90 days is used for the determination of required thickness. However, the strength at other ages can be used if it is anticipated that the pavement will be subjected to load when the concrete is at this age. During placement, the concrete mix is controlled on the basis of 7- and 28-day tests; which, according to the strength-age relation established by the laboratory mix design, will yield the 90-day design flexural strength. Beams are also cast from the concrete mix being used for 90-day tests to assure that the design flexural strength is being attained.

Various other tests are made on the concrete, such as the static and dynamic modulus of elasticity, Poisson's ratio, freeze-thaw durability, air-entraining admixture determination, alkali reactivity tests on the aggregate, deleterious material content in the aggregate, tests of the cement for specification compliance, slump tests, etc.; however, the descriptions of these tests are beyond the scope of this paper. The test procedures are standardized and found either in American Society of Testing Materials or Concrete Research Division publications.

**TRAFFIC LOADING**

Early in the development of criteria, it was questionable whether the maximum stresses occurred due to static load or to an impact load such as an aircraft landing. A rather popular opinion at that time was that the most critical loading might develop at the point of touch-down in a landing operation. This opinion was probably influenced by bounce of the aircraft and the violent "squeal" that accompanied a rather severe landing. The effects of static and impact loadings were first studied by a series of slab tests in which some slabs were loaded statically while others were subjected to an impact loading by dropping a loaded B-26 aircraft main gear. Strain measurements in the concrete revealed that when the slabs were subjected to loads of
equal magnitude, the stress was greater due to the static load.

These tests were followed by a series of flight tests using a loaded B-26 aircraft \(^{(13)}\), in which the pilot was instructed to make landings at differing sinking speeds, and to literally fly the aircraft into the pavement to produce the most severe landing possible without damaging the aircraft. After each landing the width of the tire footprint was measured, which gave a measure of the load applied through the tire. These tests indicated that for normal landings, the load produced on the pavement ranged from 40 to 60 percent of the static load; and that under the most severe landings, the load could reach 150 to 200 percent of the static load. The pilot indicated that such severe landings would be rare; and, based upon the results of the slab impact tests, it was concluded that the static load or slow moving load represented the most critical condition, and would be used for design. Another fact which influenced this conclusion was the knowledge that the concrete tensile stress was a function of the bending of the slab due to vertical deformations in the subgrade; and considering that plastic deformations in the subgrade, which would lead to large deflections and thus high concrete stresses, are time dependent, it was felt that the load during landing would be so brief that there would be insufficient time for appreciable plastic deformations to occur.

This then led to studies of the effects of slow moving repetitive loadings on the required thickness of pavements. It was, for example, already known from cyclic loading tests on simply supported beams and from traffic tests on pavements, that a load somewhat less than the ultimate static load would fail the concrete under repeated application. To study the effects of repetitive loading, the Corps of Engineers initiated a program of accelerated traffic test tracks in 1941 which was continued until 1954. In addition to the study of the effects of repetitive loading, the test tracks were also used to evaluate such other factors as; (a) slab-size, (b) joint designs, (c) base courses, (d) temperature, moisture, and weathering effects, (e) compaction requirements, (f) steel reinforcement in the concrete, (g) rigid and non-rigid overlays of existing rigid pavements, (h) prestressed concrete pavements, (i) traffic patterns, (j) differing gear configurations and loadings, (k) construction techniques, (l) concreting materials, and (m) testing techniques.
All of these factors enter into either design criteria or construction methods.

In all, traffic testing was conducted on the existing pavements at four airfields and fourteen specially constructed test tracks. Tests have been conducted using 20, 25, 26, 37, 42, 60, and 150 kip single wheel loads with tires exhibiting differing contact areas; twin-wheel gear loads of 60 and 100 kips with tire contact areas of 267 sq. in.; and twin-tandem wheel gear loads of 150, 200, 265, and 325 kips with tire contact areas of 267 sq. in. Many of the factors used in the design criteria for rigid pavements have been developed or validated by these accelerated traffic tests, including the "design factor", which is the method for adjusting the maximum edge stress for static loading to take into account the effects of repetitive loadings or traffic volume. The "design factor" will be discussed in a later section of this paper. The test track work has also provided information for the development of design procedures for reinforced rigid pavements, rigid and non-rigid overlays to strengthen rigid pavements, and prestressed rigid pavement. These designs will also be discussed in later sections of this paper. Of equal importance, the full-scale accelerated traffic test tracks have demonstrated the adequacy of design criteria, and have instilled confidence in the procedures that are used.

COVERAGES

Coverage is a term used to define the number of maximum stress repetitions that occur in the pavement due to the aircraft operations. It dates back to practically the beginning of the development of pavement design criteria by the Corps of Engineers, and is used in both rigid and flexible pavement design. By definition, a coverage occurs when each point of the pavement surface has been subjected to one maximum stress by the operating aircraft. Maximum stress, in this definition, means the stress induced in the pavement by the aircraft wheel or wheels when the aircraft is operated at its maximum gross weight. By this definition, it is obvious that, since the aircraft load is applied to the pavement through the tires and the tire width or
widths are narrow as compared to the width of pavement, several passes or operations of the aircraft will be required to statistically subject each point on the pavement to a maximum stress. In addition, this is further complicated by the fact that many operations with the aircraft are conducted at somewhat less than maximum loads; thus these operations do not produce maximum stresses, and are ignored in the computation of coverages. This, also, further increases the total volume of traffic required to produce a coverage.

At a military airfield, the pavements are designed for operation of a specific type of aircraft, and for a specific load on that aircraft (generally the maximum gross load). It is recognized that there will be operations of other types of aircraft, both heavier and lighter than that used for design; it has been found, however, that the volume of aircraft operations heavier than the design is low (primarily because the airfield is restricted to use of aircraft not exceeding the design, except for occasional emergency use), and that aircraft lighter than the design have little effect on the life of the pavement. Therefore, in design, the effects of mixed traffic are ignored. A procedure for analyzing the effects of mixed traffic on the life of a rigid pavement has been developed and is published\(^1\); however, the results obtained are considered to be, at best, only an approximation.

The procedure for converting aircraft operations to coverages has been developed based upon studies of normal aircraft ground operational traffic patterns at several airfields. Through both visual and photographic studies, it has been found that the lateral distribution of aircraft on taxiways and runways produces a "bell" shaped distribution curve, similar to that shown by Figure 2, for each main gear. From the distribution curve, it is found that about 75 percent of the main gear paths falls within a portion of the curve where, for all practical purposes, the distribution can be assumed to be uniform. The width of pavement within this portion of the distribution curve has been designated the "traffic width". Obviously there are two such widths for tricycle geared aircraft; and studies have shown that they do not overlap; therefore, the distribution of one gear can be used for design purposes. For a
bicycle geared aircraft, however, there is only one such distribution curve, since one main gear trails the other; and for each operation, there is twice as many tire print areas on the pavement surface.

The traffic width must be determined from the traffic distribution curve. It is a function of the lateral "wander" of the aircraft and the dimension of the main gear. Since the various type aircraft have about the same amount of "wander", and the gear dimensions are not significantly different, it has been found reasonable to assume that the traffic widths are the same for all aircraft, with the exception of the heavier aircraft with steerable gears. Operations of these aircraft have tended to be more concentrated (channelized); and therefore, a narrower traffic width is used in the conversion of cycles of operations to coverages. The conversion of cycles of operation to coverages is accomplished by the following:

\[ C = D \times \frac{0.75 \times N \times w}{T \times 12} \]  \quad (8)

for tricycle geared aircraft and

\[ C = 2D \times \frac{0.75 \times N \times w}{T \times 12} \]  \quad (9)

for bicycle geared aircraft.

where

- \( C \) = coverages
- \( D \) = cycles of operations (one landing and one takeoff)
- \( N \) = number of wheels on one main gear
- \( w \) = width of tire contact area of one tire, in.
- \( T \) = traffic width in feet

From the above relation, coverage levels were established for design of pavements at military airfields, based upon the total anticipated volume of traffic, and the percent of the total traffic expected to operate at maximum gross weight. These coverage levels and applicable pavement areas are as follows:

- 200 coverages - Runway edges for B-52 aircraft
5,000 coverages - All pavements for single wheeled aircraft and light twin-wheeled aircraft. All pavements except primary taxiway and runway end pavements for heavy multiple-wheeled aircraft.

10,000 coverages - Primary taxiway and runway end pavements for B-52 aircraft.

25,000 coverages - Primary taxiway and runway end pavements for all other heavy multiple-wheeled aircraft.

TRAFFIC AREAS

The term "traffic area" is a relatively new term in pavement design, and is used only to differentiate between the various pavement facilities according to the design coverage level and severity of loading. In the design criteria, all pavement facilities are divided into four traffic areas which are defined below; however, all four traffic areas do not apply for all aircraft.

Type A Traffic Areas - These are the pavement facilities, or portions of pavement facilities, which are subjected to the greatest concentration of maximum loaded aircraft. Pavement facilities classed as Type A traffic areas are primary taxiways, aprons through taxiways, and the first 500-ft. ends of runways. These pavements are designed for 25,000 coverages for all heavy multiple-wheeled aircraft except the B-52. Because of the lower number of operations, these pavements are designed for 10,000 coverages of the B-52 aircraft. Required thicknesses for these pavements are determined from the thickness scale labeled "A" on the rigid pavement design charts.

Type B Traffic Areas - These are pavement facilities which are subjected to the normal distribution of maximum loaded aircraft. Pavement facilities classed as Type B traffic areas include the second 500-ft. ends of runways, and apron, parking, or aircraft maintenance pavements for all heavy multiple-wheel geared aircraft. For single wheel and light multiple-wheel geared
aircraft, all pavements except the runway interior (portion between the 1000-ft. ends) are Type B traffic area. These pavements are designed for 5000 coverages of the maximum loaded aircraft.

Type C Traffic Areas - These are pavement facilities which are subjected to a reduced loading of the aircraft, or where the speed of the operating aircraft results in less than maximum stresses in the pavement. Pavement facilities classed as Type C traffic areas include the runway interior, secondary taxiways, and such pavements as calibration hardstands. These pavements are designed for 5000 coverages of 75 percent of the maximum aircraft gross load.

Type D Traffic Areas - These are pavements which, because of their location, are subjected to only occasional traffic at less than maximum gross load. This traffic area applies only to the design for B-52 aircraft, and consists of the outside 100-ft. width on each side of the runway, excluding areas required for access to taxiways. These pavements are designed for 200 coverages of 75 percent of the maximum aircraft gross load.

A more detailed description of the four traffic areas, along with typical layouts showing the various areas, is contained in a published manual (15). A pavement facility listed as a type traffic area may not be designed in its entirety for that type of traffic area. For example, long straight sections of primary taxiways for bicycle geared aircraft are designed with only the center 25-ft. wide paving lane as a Type A traffic area. The side pavements are tapered to the thickness required for Type B traffic areas at the taxiway edges. This can be done since practically all of the main gear traffic will be within the center paving lane.

DESIGN FACTOR

Thus far in this paper, mention has been made of the effects of repetitive loading and stresses due to temperature and moisture; however, no method for accounting for these effects in pavement design has been presented. The methods presented for determination of critical stress applied only to static loading.
It has been found to be practically impossible to isolate each of the factors which affect stresses in the pavement and pavement performance. Even if such could be done, it would be equally difficult to assess each factor and include it as a design variable which would be applicable in all areas where the criteria are used. However, even though these factors are not discussed in detail, and methods presented for incorporating each in the design method, it does not indicate that they have been ignored or not included in the criteria. This is the function of the design factor used in rigid pavement criteria which can be expressed as follows:

\[
D.\ F. = \frac{R}{\sigma_e} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (10)
\]

where

\[
D.\ F. = \text{design factor} \\
R = \text{design flexural strength of concrete, psi} \\
\sigma_e = \text{maximum free edge stress, psi}
\]

The design factor is used somewhat as a safety factor; but unlike a safety factor, a constant value is not used for all pavements nor does the factor used remain constant during the life of the pavement. In other words, a design factor of 1.3 may be used for design; but due to the effects of repetitive loading, cyclic stresses due to temperature, and moisture changes, weathering effects, etc., the design factor continually becomes lower, until at the end of the design life the factor equals 1.0; whereupon, idealistically, the pavement fails under the design loading.

Numerical values for the design factor have been determined from the results of accelerated traffic test tracks and condition surveys of existing pavements, where pavements of known strengths and thickness have been subjected to known loads and volumes of traffic. During the traffic testing, the test pavements also underwent temperature and moisture changes and weathering, so that the effects of these factors are inherent in their performance. The argument can be made, however, that because of accelerated trafficking, these pavements were not subjected to as many cycles of temperature and moisture
changes, nor would the weathering effects be as severe as for a 10-year life pavement. This is certainly a valid argument; however, one must also consider that accelerated traffic is probably more severe, since less time for relaxation of the subgrade and pavement occurs, and the concrete does not attain the same strength gain during the accelerated testing as it does during longer periods. It has been considered therefore, that these are offsetting effects and that the design factor as determined from the test track work is valid.

The design factor was developed by comparing the design flexural strength with the computed maximum free edge stress, and plotting this ratio versus the number of coverages of traffic required to produce three separate failure conditions in the test items: first crack at the surface, slab broken into about six pieces, and slab broken into about 25 to 30 pieces. These failure conditions were designated as: (a) initial crack, (b) shattered slab, and (c) complete failure. Although a plot of the design factor versus coverages for these failure conditions produced some scattering of points, the scattering was not excessive considering all of the factors which affect pavement performance and the probability of non-uniform materials properties. The points did show a definite trend toward the need of a higher design factor for higher coverage levels. A plot of design factor versus coverages for the initial crack failure is shown by Figure 3; through this relation, it is possible to design a rigid pavement for any volume of traffic up to 30,000 coverages (the maximum volume of traffic that has been applied in test track work). Design factor versus coverages for the other two conditions of failure have not been included, since pavements are designed only for the initial crack condition.

In Figure 3, it will be seen that there are two curves: one for tricycle-gearred aircraft, and one for bicycle-gearred aircraft. No impact factor had been required in rigid pavement design for tricycle-gearred aircraft; however, due to the rocking or pitching motion inherent in aircraft with the bicycle type gear arrangement, and the relatively long overhang both fore and aft of the main gears, it was found that the gear loading could exceed the static loading by as much as 15 percent. Soon after these type aircraft became
Operational, premature pavement distress was observed, and a study conducted at that time led to the fact that greater than maximum loading occurred due to the operation of these aircraft. Based upon these findings, an impact factor seemed appropriate for these type aircraft; the design factor appeared to be the best method for including this in the criteria, since it is a constant regardless of the volume of traffic. Therefore, the design factor for use in pavement design for bicycle-geared aircraft was increased sufficiently at each coverage level to provide a thickness of pavement which would be adequate for the static load plus 15 percent.

The design factor is used in design in the following manner and as illustrated in the section of this paper titled "Development of a Rigid Pavement Design Chart". The free edge stresses for the design gear load are determined using either the influence chart or model study. After the free edge stress is reduced to reflect the amount of load transferred through the joint, the resultant stress is multiplied by the design factor to determine the required flexural strength of the concrete.

The design factors currently used in design criteria for the various coverage levels are as shown in Table 4 below:

Table 4

Design Factors Used for Rigid Pavement Design

<table>
<thead>
<tr>
<th>Coverage Level</th>
<th>Design Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design</td>
<td>Tricycle Gear</td>
</tr>
<tr>
<td>200</td>
<td>N/A</td>
</tr>
<tr>
<td>5,000</td>
<td>1.30</td>
</tr>
<tr>
<td>10,000</td>
<td>N/A</td>
</tr>
<tr>
<td>25,000</td>
<td>1.54</td>
</tr>
</tbody>
</table>
An important factor in rigid pavement design, which has been mentioned but not discussed, is the load transfer that occurs from one slab to another in a rigid pavement system. Concrete pavements must of necessity be constructed in sections or slabs, due both to the limitations of construction equipment and stresses that build up in the concrete due to expansion, contraction, and temperature and moisture gradients through the thickness of the pavement. Construction equipment limits the width of the concrete (paving lanes) to a maximum of about 25 ft.; therefore, construction joints are required at the edges of each paving lane. Also, any interruption of concrete placement such as equipment breakdown or simply stopping at the end of the day requires the installation of a joint. Continuous lengths of paving lanes can be placed; however, the concrete moves due to shrinkage of the concrete material as it sets or hardens, and to the expansion and contraction caused by temperature changes. The frictional resistance between the concrete and the material on which it rests creates tensile or compressive stresses in the concrete as it moves. Due to this action, tensile stresses exceeding the tensile strength of the concrete can develop upon contraction of the concrete. Since this action would cause cracks in the paving lane, joints (designated contraction joints) are placed across the paving lane at specified intervals to control this type cracking. These joints are simply weakened planes or grooves cut in the plastic concrete during placement, and at the approximate points where cracks would occur. Therefore, as the tensile stress builds up, the concrete cracks through the weakened portion of the slab, and forms a controlled (straight) crack that is easier to maintain.

Concrete also warps or curls due to the difference between the temperature and moisture content in the concrete at the top and bottom of the slab. Without regularly spaced joints, stresses created by the warping can cause cracks to form in the pavement. Joints are, therefore, required in concrete pavements even though from the standpoint of maintenance and smoothness of operation, they are undesirable. Some joints can be eliminated through slab
reinforcement, continuous reinforcement, or prestressing the pavement; however, we must always be concerned with construction joints required because of equipment limitations.

Early in highway construction, the need to tie the various slabs, formed by joints, together to prevent separation of the slabs was evident. Since these pavements were relatively narrow, compared to the width of airfield facilities, it was found that deformed steel bars (called tie bars) would adequately afford continuity from one slab to the other. Through highway experience, it was also found that the tie bars in the joints provide some transfer of the load from the loaded slab to the adjacent unloaded slab. However, the importance of this factor was not immediately apparent since pavements were being designed for interior loading. Since it was soon evident that the edge loading was most critical, and failures of the slab originated at the edges, it became common practice in highway work to require that the edges of the slabs (at the joints) be thickened to reduce the edge stress. This practice was adopted by the Corps of Engineers in the 1943 published design criteria.

The test track work by the Corps of Engineers soon showed that the edge loading condition produced the critical tensile stress; however, this work also showed that the stress could be reduced by properly designed load transfer devices in the joints. Such devices served a dual purpose: that of lowering the edge stress by the transfer of load, and by preventing differential vertical movements of the individual slabs and thus maintaining surface smoothness. Tie bars commonly used in highway construction could not be used for airfields, since tying large widths of pavement together negated the purpose of the joints, which was to relieve stresses due to contraction of the concrete.

The study of devices that could be used in joints to provide both load transfer and slab alignment was included in the accelerated traffic test track program. At the same time, theoretical and model tests were performed to assist in this study, and to determine the amount of load that could be transferred across the joint. Numerous devices were studied, ranging from very elaborately designed and difficult to construct schemes, to the very simplest device, that provided by the aggregate interlock of a crack in the

30
pavement. From these studies, the decision was made to use three types of load transfer devices: (a) keys and keyways constructed in the joints during construction; (b) dowels, consisting of round smooth steel bars or pipe, one end of which would be bonded in the concrete and the other end left unbonded; and (c) the interlock provided by a natural crack occurring shortly after the concrete was placed. Each of these methods allows the concrete to expand and contract without restraint; however, each demonstrated that it would provide at least 25 percent load transfer and maintain slab alignment. Other devices were studied which were capable of providing greater load transfer; however, the cost of manufacture and constructing the devices in the joint exceeded the savings from the reduction in concrete thickness. In addition, the difference between the maximum stress from edge and interior loading is only about 25 percent; hence any device that reduces the edge stress by more than 25 percent then makes the interior loading condition critical.

In construction, all joints are provided with load transfer devices; therefore in design, the computed maximum free edge stress, \( \sigma_e \), is reduced by 33 percent, since the slab must carry only 75 percent of the design loading.

The concept of load transfer devices in the joint permitted the design and construction of uniform thick pavements and afforded a balanced design between edge and interior loading conditions. If for any reason, load transfer devices are not used in the joints, the edge of the slab must be thickened by 25 percent over that determined by currently published Corps of Engineers rigid pavement design criteria.

**EQUIVALENT SINGLE WHEEL LOAD**

Since the pavement stress produced by a static gear load is a function of the load size and shape of the contact area, number of wheels, spacing and arrangement of wheels, modulus of elasticity of the concrete, Poisson's ratio of the concrete, slab thickness, and modulus of soil reaction, the development of a procedure for resolving any gear configuration into an equivalent single wheel loading (ESWL)* is complex. No single equation or group of equations

* Used to denote equivalent single wheel loading hereafter in paper.
has been developed to accomplish this. Several attempts have been made to equate multiple-wheel loadings to ESWL graphically; however, these methods have, at best, been only gross approximations.

An ESWL can be developed which will produce the same free edge stress as a loading on any gear configuration through the use of the influence chart. This is, in fact, used for the development of rigid pavement design charts since it represents a work saver. The equivalent single wheel (ESW)* used for the development of design charts is one having an elliptical contact area (footprint) of 267 sq. in. This size tire was chosen simply because it is so commonly used on multiple-wheeled aircraft. It is, however, pointed out that any size tire can be used as the ESW.

Through the use of the influence charts, moments have been determined for the ESW for a range of \( l \) values and loadings where \( l \) is the radius of relative stiffness determined by formula (presented previously). These values have been used to compute moment coefficients, \( M/P \), which is simply the moment at any load divided by that load to obtain moment per pound of wheel load. A value of \( A/l^2 \) is then determined for each value of \( l \) used to determine the moment from the influence chart.

\( A \) is the contact area of the ESW (267 sq. in.). The value of \( A/l^2 \) relates the size of the loaded area to the properties of the slab and subgrade (radius of relative stiffness). The moment coefficients, \( M/P \), are then plotted versus the \( A/l^2 \) values to form the chart shown by Figure 4. From this figure, the maximum free edge stress can be computed for any load on the single wheel (ESWL) and for any pavement thickness. The modulus of elasticity of concrete, \( E \), and Poisson's ratio, \( \mu \), are assumed to be constants and equal to \( 4 \times 10^6 \) and 0.20 respectively. The free edge stress for the ESWL is determined by

\[
\sigma_e = \frac{6P}{h^2} \left( \frac{M}{P} \right) \quad \ldots \quad (11)
\]

* Used to denote equivalent single wheel hereafter in paper
where

\[ \sigma_e = \text{maximum free edge stress, psi} \]
\[ P = \text{equivalent single wheel load, lbs.} \]
\[ h = \text{slab thickness, in.} \]
\[ M/P = \text{moment coefficient determined from Figure 4 for the pavement thickness in question} \]

For the above equation, a pavement thickness, \( h \), and modulus of soil reaction, \( k \), are used to compute a value of \( \ell \), from which a value of \( A/\ell^2 \) is computed. Entering the figure with \( A/\ell^2 \), the moment coefficient is determined and used in the formula. The value of \( h \) must be the same for computation of \( \ell \) and \( \sigma_e \).

With the relationship shown by Figure 4, all factors needed are known, and determination of the maximum free edge stress becomes a computational process. Therefore, it is only necessary to determine an ESWL for the gear load and configuration in question; the free edge stress can then be computed for any gear configuration at any loading. This is accomplished again through the use of the influence chart to determine an equivalent single wheel load factor, \( F_{\text{ESWL}} \), which can be divided into the load on the gear in question to obtain the ESWL. The \( F_{\text{ESWL}} \) is determined by the ratio of the blocks within the footprint of the ESW to the blocks within the footprint of the gear in question and is expressed by the following equation:

\[
F_{\text{ESWL}} = \frac{W N_E}{N_I + N_{II} + \ldots N_n} \quad \ldots \ldots \ldots \ldots \quad (12)
\]

where

\[ F_{\text{ESWL}} = \text{the equivalent single wheel load factor} \]
\[ W = \text{number of wheels on gear in question} \]
\[ N_E = \text{number of blocks under the ESW from the influence chart} \]
\[ N_I + N_{II} \ldots N_n = \text{number of blocks under each wheel of the gear in question} \]
\[ n = \text{number of wheels on gear} \]
It is necessary to determine a value of $F_{ESWL}$ for $l$ values of 20, 40, 60, 80, 100, 120, and 140 inches using the influence chart in the manner described in the section of this paper titled "Determination of Maximum Stress by Influence Chart". The values of $F_{ESWL}$ are then plotted versus the $l$ value used to determine the factor to produce a curve similar to those shown by Figure 5. This figure shows $F_{ESWL}$ vs. $l$ relations for several different gear configurations. By entering this figure with a $l$ value, and projecting vertically to a line representing a specific gear configuration, then horizontally, an $F_{ESWL}$ value is obtained which when divided into the load on the specific gear configuration will give an ESWL. Using the same value of $l$ and Figure 4, the free edge stress for the ESWL can be computed, and will be the same as the free edge stress for the gear in question.

It can be seen that there is no such thing as one ESWL that is representative of another gear for all conditions of pavement thickness and modulus of soil reaction. Instead, it is necessary to determine a separate ESWL for each combination of $h$ and $k$ if it is desired that the ESWL produce the same free edge stress as the gear load and configuration that it represents.

**FAILURE CONCEPT**

Until the early 1950's the Corps of Engineers' design criteria for rigid pavements was based upon the concept that the first crack occurring in a concrete slab due to traffic constituted failure of that slab irregardless of the strength of the foundation materials. Idealistically, all slabs would crack at exactly the same time, if all physical properties of the concrete and foundation were the same, and the traffic and other factors that cause stress in the concrete were the same. However, obviously, these ideal conditions never exist; therefore, failure of a pavement facility was considered to occur when approximately 50 percent of the slabs within the traffic area had developed the first crack. This meant that while some slabs would still be intact, a few slabs would probably show multiple cracking due to weak concrete or subgrade, concentrated traffic, or other factors which can produce stress in the slab.
This concept of failure is still used for pavements constructed on foundations having $k$ values less than 300 lbs/sq in/in. Condition surveys conducted during the 1940's indicated that pavements constructed on high strength foundations continued to satisfactorily carry the design traffic for long periods after the slabs had cracked; and the cracked slabs did not displace or create undue maintenance problems. This same performance was observed in the accelerated traffic test track studies, when the test pavements were constructed on foundations having high $k$ values. However, both the condition surveys and test track work indicated that for subgrades having $k$ values less than 200 lbs/sq in/in, the slabs tend to develop multiple cracking and differential displacement soon after the initial crack occurred.

Based upon these observations, the decision was reached that more than initial cracking could be tolerated in pavements constructed on high strength foundations without either endangering the aircraft operations or creating an undue maintenance problem. Therefore, when criteria was revised and published in 1954, the pavement thickness requirements for $k$ values of 300 lbs/sq in/in and higher were reduced by the following percentages which allows some cracking to occur in these pavements during their design life:

<table>
<thead>
<tr>
<th>$k$ Value</th>
<th>Reduction in Thickness, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>300</td>
<td>4.6</td>
</tr>
<tr>
<td>400</td>
<td>10.6</td>
</tr>
<tr>
<td>500</td>
<td>19.2</td>
</tr>
</tbody>
</table>

The above reduction in thickness was accomplished by shifting the 300, 400, and 500 $k$ lines on the rigid pavement design charts a sufficient amount to effect the reduction in required pavement thickness. No change was made in the failure concept for pavements constructed on foundations exhibiting
k values of 200 lbs/sq in/in or less, and failure is still based upon the initial crack concept.

**FROST CONSIDERATIONS**

Studies and experience have shown that frost penetration (freezing and thawing) into certain types of soils under rigid pavements can cause detrimental effects unless adequate protection is provided. Materials which are adversely affected by frost penetration are termed "frost-susceptible" and have been determined to be any soil containing 3 percent or more of grains finer than 0.02 mm in diameter by weight. The detrimental effects of frost action and such soils are manifested by excessive heaving, and often non-uniform heaving, of pavements during the winter months, and by a loss of subgrade strength (k value) during the periods when the frozen soils are thawing. Non-uniform heaving of these soils can be so severe that it can create intolerable roughness; and often, slabs are cracked. The loss of subgrade strength can result in overloading of the pavements during the thawing period. A complete description of the detrimental effects of frost action is contained in the Pavement Design Manual for Frost Conditions\(^{(16)}\).

Several procedures are available for the protection and design of rigid pavements in areas where frost action will occur. These procedures are briefly described below and are presented in detail in the manual for Pavement Design for Frost Conditions\(^{(16)}\).

1. If the subgrade soils are uniform in nature, so that non-uniform heaving will not be a problem, the thickness design can be based upon a reduced value of the modulus of soil reaction, \(k_r\), that will occur during the frost melting period. The \(k_r\) value will depend upon the type of material present, and is determined from a chart presented in the above mentioned manual.

2. If the frost action will produce non-uniform heaving which will be detrimental, the frost-susceptible material is removed to the depth of frost penetration and replaced with a material known to be non-frost-susceptible. Procedures are presented in the above mentioned manual for determining the anticipated depth of frost penetration from air temperature.
measurements. When the procedure is used, a k-value at the surface of the non-frost-susceptible material must be determined and used for the pavement thickness design.

(3) For the extreme northern climates, where frost penetration to large depths occurs, a procedure for the determination of the required thickness of non-frost-susceptible material to reduce non-uniform heaving to an acceptable amount is given in the above mentioned manual. In this procedure, some frost penetration into frost-susceptible materials is permitted; however, the thickness of non-frost-susceptible material over these materials is sufficiently great to prevent detrimental effects to the pavement because of non-uniform heaving. The k-value for design must be determined by test on the surface of the non-frost susceptible materials.

**DRAINAGE CONSIDERATIONS**

In the design of rigid pavements, the strength of the foundation, which is represented by the modulus of soil reaction, k, is determined based upon the assumption that the material will reach 100 percent saturation. Therefore, drainage to reduce the saturation of the foundation materials is not required. However, drainage is required to prevent the accumulation of free water under the pavements. Water gains access to the foundation materials both through the pavement joints and from movement horizontally through the foundation materials. Free water beneath a pavement can result in undue weakening of the foundation materials, and pumping of fine grained subgrade materials through the pavement joints as loads pass over the pavements.

In order to prevent pumping in fine grained subgrade soils, base or filter courses are required beneath rigid pavements. When base or filter courses are used, drainage along the sides of the paved areas and through the interior of large paved areas must be provided to assure that free water will not accumulate in the base course. Drainage is also required to prevent sources of water from outside of the paved areas from flowing under the paved areas, either subsurface or surface flows. Surface water drainage must also be provided to prevent the accumulation of water on the paved areas. This is
either accomplished by providing slopes to the paved areas, so that water runs off, or by providing catch basins in large paved areas and underground drains to carry off the surface water.

Criteria for the provision of drainage and design of the drainage systems is described in detail in published manuals (17)(18).

DEVELOPMENT OF RIGID PAVEMENT DESIGN CHART

The following description of the development of a rigid pavement design chart is presented to demonstrate the use of the various factors affecting required thickness of pavement that have been discussed herein. The chart developed as an example is for a tricycle geared aircraft having a twin-wheel main gear; each wheel having a tire footprint (contact) area of 267 sq. in., and the two wheels spaced 37.5 in., center to center. The gross weight of the aircraft, for pavement design purposes, is 222,222 pounds; this weight is distributed so that each main gear carries 100,000 pounds, and the nose gear carries 22,222 pounds. The main gear load is distributed equally on the two wheels, making a wheel loading of 50,000 pounds. Values of the modulus of elasticity, \( E \), and Poisson's ratio, \( \mu \), of the concrete are assumed to be \( 4 \times 10^6 \) psi and 0.20, respectively.

To develop the maximum edge stress for different values of pavement thickness and modulus of soil reaction for the twin-wheel loading, use is made of the influence chart and the concept of equivalent single wheel load (ESWL). The procedure for the use of the influence chart for determining edge stress and the method for determining the ESWL has been described in other parts of this paper.

The step by step procedure for the development of a chart is as follows:

1. Select radius of relative stiffness values, \( t \), of 20, 40, 60, 80, 100, 120, and 140 inches. Using the procedures described in the section of this paper titled "Determination of Critical Stresses by Influence Charts", construct scaled drawings of the twin wheel gear, and from the influence chart count the number of blocks under the contact area of each wheel. The number of blocks will be designated as \( N_I \) for the wheel tangent to the edge, and \( N_{II} \)
for the other wheel. These values for the selected $t$ values are summarized in Table 6 below:

### Table 6

**Determination of Equivalent Single Wheel Load Factor for Twin-Wheel, 267 sq. in., 37.5" C-C Gear**

<table>
<thead>
<tr>
<th>Radius of Relative Stiffness, $t$, in.</th>
<th>20</th>
<th>40</th>
<th>60</th>
<th>80</th>
<th>100</th>
<th>120</th>
<th>140</th>
</tr>
</thead>
<tbody>
<tr>
<td>Semi-Major Axis, 11.90 in.</td>
<td>5.95</td>
<td>2.98</td>
<td>1.98</td>
<td>1.48</td>
<td>1.19</td>
<td>0.99</td>
<td>0.85</td>
</tr>
<tr>
<td>Semi-Minor Axis, 7.14 in.</td>
<td>3.57</td>
<td>1.79</td>
<td>1.19</td>
<td>0.89</td>
<td>0.71</td>
<td>0.59</td>
<td>0.51</td>
</tr>
<tr>
<td>Spacing, C-C, 37.5 in.</td>
<td>18.75</td>
<td>9.38</td>
<td>6.25</td>
<td>4.69</td>
<td>3.75</td>
<td>3.12</td>
<td>2.68</td>
</tr>
<tr>
<td>$N_I$, Wheel I</td>
<td>2000</td>
<td>611.8</td>
<td>340.0</td>
<td>210.5</td>
<td>141.4</td>
<td>104.7</td>
<td>88.0</td>
</tr>
<tr>
<td>$N_{II}$, Wheel II</td>
<td>213</td>
<td>111.1</td>
<td>81.1</td>
<td>63.5</td>
<td>51.7</td>
<td>45.0</td>
<td></td>
</tr>
<tr>
<td>$N_I + N_{II}$</td>
<td>2213</td>
<td>759.1</td>
<td>451.1</td>
<td>291.6</td>
<td>204.9</td>
<td>156.4</td>
<td>133.0</td>
</tr>
<tr>
<td>$N_E$ (267 sq. in.)</td>
<td>2000</td>
<td>611.8</td>
<td>340.0</td>
<td>210.5</td>
<td>141.4</td>
<td>104.7</td>
<td>88.0</td>
</tr>
<tr>
<td>$F_{ESWL}$ (2 N_E / N_I + N_{II})</td>
<td>1.81</td>
<td>1.61</td>
<td>1.51</td>
<td>1.44</td>
<td>1.38</td>
<td>1.34</td>
<td>1.32</td>
</tr>
</tbody>
</table>

(2) Using the same procedures, determine the number of blocks falling within the footprint (267 sq. in. elliptical shape) of the ESW when it is oriented tangent to the edge with the minor axis perpendicular to the edge, for each of the above $t$ values. The number of these blocks will be designated $N_E$ and are also shown by Table 6. It is pointed out that the ESW contact area is always assumed to be 267 sq. in., therefore, the number of blocks needs to be determined only once, regardless of the type gear for which the design chart is being prepared.

(3) An equivalent single wheel load factor $F_{ESWL}$, can now be found from the number of blocks determined from the influence chart by use of the following formula which is an adoption of the general expression given in the section of this paper on ESWL.
\[ F_{ESWL} = \frac{2 N_E}{N_I + N_{II}} \quad \ldots \ldots \ldots \ldots \quad (13) \]

where

- \( F_{ESWL} \) = a factor for expressing the load on the gear under consideration in terms of equivalent single wheel load,
- \( N_E \) = number of blocks within the ESW footprint determined from influence chart,
- \( N_I \) and \( N_{II} \) = number of blocks within the footprint of each wheel of the twin wheel gear.

A value of \( F_{ESWL} \) is determined for each value of \( l \) as shown in Table 6.

(4) The values of \( F_{ESWL} \) are plotted versus the corresponding values of \( l \) to produce a curve similar to those shown by Figure 5. The curve labeled "twin wheels spaced 37.5 in. C-C, 267 sq. in." of this figure represents a plot of the values for this example. Similar plots on this figure for other gear configurations are shown for illustrative purposes.

(5) Knowing the total gear loading, the curves on Figure 5 permit the computation of the ESWL which would produce the same edge stress as the gear under consideration for any value of \( l \) simply by dividing the gear load by the factor, \( F_{ESWL} \).

(6) The moment coefficients, \( M/P \) (moment per pound of wheel load), are then determined for the equivalent single wheel for each \( l \) value. This is accomplished by determining the moment, \( M \), for a range of loadings using the \( N_E \) values and the moment formula:

\[ M = \frac{q \ell^2 N_E}{10,000} \quad \ldots \ldots \ldots \ldots \quad (14) \]

where \( q \) is the wheel load divided by the contact area. The moment coefficient \( M/P \) is then obtained by dividing the moment by the wheel load. The \( M/P \) values are plotted versus values of \( A/\ell^2 \) as shown in Figure 4, where \( A \) is the contact area. This relation need only be determined once, since it is for the ESW which remains constant.

(7) Using the curve for twin wheels spaced 37.5 in. C-C, 267 sq. in. on Figure 5, and the relations expressed by Figure 4, the edge stress
produced by the twin wheel gear of this example can then be computed for any
desired combination of thickness, \( h \), and modulus of soil reaction, \( k \), using
the formula:

\[
\sigma_e = \frac{6P}{F_{ESWL}} \left[ \frac{M}{P} \right] h^2
\]  

where

- \( \sigma_e \) = maximum free edge stress, psi,
- \( F_{ESWL} \) = factor determined from Figure 5 for
  a specific \( t \) value,
- \( P \) = total load on the twin-wheel gear, lbs.,
- \( M/P \) = moment coefficient determined from Figure 4
  for the same \( t \) value used to determine \( F_{ESWL} \),
- \( h \) = pavement thickness, in., corresponding to
  the \( t \) and \( k \) values used for the determination
  of \( F_{ESWL} \) and \( M/P \)

(8) To construct a rigid pavement design chart, it is necessary to
select several values of \( h \) for each \( k \) value and determine the free edge stress
for each combination of \( h \) and \( k \). It is normal practice to select \( k \) values of
25, 50, 100, 200, 300, 400, and 500 lbs/sq in/in. Some ten or twelve separate
thickness values are selected within the range of thickness that will yield
maximum edge stress values that will correspond to the normal range of concrete
flexural strength values. Experience in constructing design charts will soon
indicate a reasonable range of thickness values to select. It is seen that
approximately 70 to 80 computations of edge stress will be required to provide
sufficient points for the construction of a design chart.

(9) Table 7 summarizes the results of a few such computations made
to produce the twin-wheel design chart shown by Figure 6. Although Table 7
is not complete, sufficient data have been included to illustrate the pro-
cedure.
Table 7

Summarization of Free Edge Stress Determinations and Required Flexural Strengths to Develop a Rigid Pavement Design Chart for a Tricycle-Gear Aircraft Having a 100,000-pound Twin-Wheel Load, 267 sq. in. Tire Contact Area and 37.5 in. C-C Spacing of Wheels

<table>
<thead>
<tr>
<th>Slab Thick. h, in.</th>
<th>Mod.of Soil React., k, pci</th>
<th>Soil Mod. of Adjust. for High k Values, in.</th>
<th>Rad.of Rel. Stiff. l, in.</th>
<th>A ( \frac{\pi^2}{l^2} )</th>
<th>Moment Coeff. for ESWL, M/P</th>
<th>F ESWL, psi</th>
<th>ESWL</th>
<th>( \sigma_{e} ), psi</th>
<th>Required Flexural Strength, R, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>25</td>
<td>--</td>
<td>86.85</td>
<td>0.0354</td>
<td>0.541</td>
<td>1.400</td>
<td>71,429</td>
<td>905</td>
<td>679 (1)</td>
</tr>
<tr>
<td>19</td>
<td>25</td>
<td>--</td>
<td>98.79</td>
<td>0.0274</td>
<td>0.570</td>
<td>1.380</td>
<td>72,464</td>
<td>687</td>
<td>515 (2)</td>
</tr>
<tr>
<td>22</td>
<td>25</td>
<td>--</td>
<td>110.28</td>
<td>0.0220</td>
<td>0.596</td>
<td>1.365</td>
<td>73,260</td>
<td>542</td>
<td>406 (3)</td>
</tr>
<tr>
<td>14</td>
<td>100</td>
<td>--</td>
<td>55.56</td>
<td>0.0865</td>
<td>0.437</td>
<td>1.530</td>
<td>65,359</td>
<td>873</td>
<td>655 (4)</td>
</tr>
<tr>
<td>17</td>
<td>100</td>
<td>--</td>
<td>64.27</td>
<td>0.0646</td>
<td>0.471</td>
<td>1.485</td>
<td>67,340</td>
<td>658</td>
<td>494</td>
</tr>
<tr>
<td>20</td>
<td>100</td>
<td>--</td>
<td>72.60</td>
<td>0.0507</td>
<td>0.498</td>
<td>1.450</td>
<td>68,966</td>
<td>516</td>
<td>387</td>
</tr>
<tr>
<td>12</td>
<td>300</td>
<td>11.45</td>
<td>37.61</td>
<td>0.1888</td>
<td>0.352</td>
<td>1.640</td>
<td>60,976</td>
<td>893</td>
<td>670</td>
</tr>
<tr>
<td>15</td>
<td>300</td>
<td>14.31</td>
<td>44.46</td>
<td>0.1351</td>
<td>0.389</td>
<td>1.590</td>
<td>62,893</td>
<td>652</td>
<td>489</td>
</tr>
<tr>
<td>18</td>
<td>300</td>
<td>17.17</td>
<td>50.97</td>
<td>0.1028</td>
<td>0.417</td>
<td>1.560</td>
<td>64,103</td>
<td>495</td>
<td>372</td>
</tr>
<tr>
<td>11</td>
<td>500</td>
<td>8.89</td>
<td>31.01</td>
<td>0.2777</td>
<td>0.311</td>
<td>1.700</td>
<td>58,824</td>
<td>907</td>
<td>680 (3)</td>
</tr>
<tr>
<td>14</td>
<td>500</td>
<td>11.31</td>
<td>37.15</td>
<td>0.1935</td>
<td>0.349</td>
<td>1.615</td>
<td>60,790</td>
<td>649</td>
<td>487 (4)</td>
</tr>
<tr>
<td>17</td>
<td>500</td>
<td>13.74</td>
<td>42.98</td>
<td>0.1445</td>
<td>0.382</td>
<td>1.605</td>
<td>62,305</td>
<td>493</td>
<td>370</td>
</tr>
</tbody>
</table>

(1) 1 Coverage, 100% Load
(2) 25,000 Coverages, 100% Load
(3) 5,000 Coverages, 100% Load
(4) 5,000 Coverages, 75% Load
Columns 1 and 2 of Table 7 list values of pavement thickness, \( h \), and moduli of soil reaction, \( k \), selected for computational purposes. Column 3 shows the thickness adjustment made for high \( k \) values which was discussed in the section of this paper titled "Failure Concept". The unadjustable thicknesses (column 1) are used for computation of stress and required flexural strength; however, when preparing the chart, the required flexural strength is plotted versus the adjusted thickness (column 3). Thus, when the chart is used for design, the thicknesses determined for the high \( k \) values will permit some cracking to occur during the design life of the pavements. Column 4 is the radius of relative stiffness, \( \ell \), of the pavement computed from the equation:

\[
\ell = \left[ \frac{Eh^3}{12(1-\mu^2)k} \right]^{\frac{1}{4}}
\]  

in which \( E \) and \( \mu \) are assumed to be \( 4 \times 10^6 \) psi and 0.20, respectively.

Column 5 is a computed value obtained by dividing the contact area of the ESW (267 sq. in.) by the square of the \( \ell \) value. Column 6 is the moment coefficient, \( M/P \), determined from Figure 4 for each value of \( A/\ell^2 \). Column 7 is the equivalent single wheel load factor, \( F_{ESWL} \), for each \( \ell \) value determined from the curve on Figure 5 labeled "twin wheels spaced 37.5 in. C-C., 267 sq. in."

Column 8 is the equivalent single wheel load, ESWL, determined by dividing the twin-wheel gear loading by the \( F_{ESWL} \) factor \( (100,000/F_{ESWL}) \). Column 9 is the free edge stress computed from the equation:

\[
\sigma_e = \frac{6 \text{ ESWL}}{h^2} \left[ \frac{M}{P} \right]
\]  

Columns 10 through 13 represent the concrete flexural strength, \( R \), required for the stated loading conditions, assuming that all joints contain load transfer devices capable of transferring at least 25 percent of the applied gear load to the adjoining concrete slab. The required flexural strengths are determined by the following expression:

\[
R = \sigma_e \times 0.75 \times \text{D.F.} \times \% L
\]
where

\[ R = \text{required concrete flexural strength, psi} \]
\[ \sigma_e = \text{free edge stress, psi} \]
\[ 0.75 = \text{load transfer factor which is a constant} \]
\[ \text{D.F.} = \text{design factor for design coverage level from Figure 2.} \]
\[ \% L = \text{percent maximum gear load used for pavement design.} \]

Column 11 then represents the required flexural strength for Type A traffic area pavements; column 12 represents the required flexural strength for Type B traffic area pavements, and column 13 represents the required flexural strength for Type C traffic area pavements for the 100,000 pound tricycle type, twin-wheel main geared aircraft, and for each of the combinations of pavement thickness and modulus of soil reaction selected.

(10) The values shown in columns 1, 2, 3, 11, 12, and 13 are then used to construct a design chart for the tricycle gear 100,000-pound twin wheel loading. It has been found convenient to construct the chart in the form shown by Figure 6. Scales of flexural strength and thickness are established at each side of the chart which will encompass the values shown in columns 11, 12, and 13 and column 1, respectively. Next, a load line representing the 100,000-pound twin-wheel gear is positioned on the chart. Experience in constructing charts will serve as a guide for establishing the location and slope of this line. Using the values of flexural strength shown in column 11, 12, or 13 (values shown in column 12 are normally used) and the values shown in columns 1 and 3 for thickness, lines representing each k value are constructed. This is accomplished by making a horizontal projection on the chart at the value of R, and another horizontal projection at the value of h which corresponds to the R value. The projection associated with the h value is continued until it intersects the load line. Through this intersection, a vertical projection is made until it intersects the horizontal projection of R. This point of intersection represents the k value (column 2) that corresponds to the selected h and R values. All points representing a constant value of k are thus positioned, and a curve through the points produces a k
line like one of those shown in Figure 6. This process is continued until all k lines have been positioned on the chart. It is to be remembered that the adjusted thickness values shown in column 3 are used to position the 300 and 500 k lines.

(11) The above described process produces a chart from which the thickness required for the operation of a 100,000-pound twin wheel load for 5000 coverages is obtained (if the R values in column 12 were used). The thickness scale thus produced is normally labeled B to denote that this scale is to be used for the design of pavements in the Type B traffic areas. Thickness scales must then be established for the Type A and Type C traffic area pavements. This is accomplished by entering the chart with the R values in columns 11 and 13, projecting horizontally to the k line representing the value in column 2, then projecting vertically. From the intersection with the load line a horizontal projection is made to a vertical scale at the opposite side of the chart. At this intersection, the thickness value (column 1) is plotted. Through a series of such projections, thickness scales A (for R values in column 11) and C (for R values in column 13) are constructed. The chart shown by Figure 6, now is applicable for the determination of required thicknesses for Types A, B, or C traffic area pavements for a 100,000-pound twin wheel gear load.

(12) If it is desired to include load lines other than 100,000 pounds on the design chart, it will be necessary to determine required R values as shown in columns 11, 12, and 13 for other twin wheel gear loadings. However, all values in columns 1 through 7 of Table 7 will remain constant. New ESWL values (column 8) will be computed and the stresses and required R values (columns 9 through 13) will be computed in the manner already described. The new values will then be used to construct additional load lines on the chart.

METHODS FOR REINFORCED RIGID PAVEMENT DESIGN

A method for the design of reinforced rigid pavements has been developed and published\(^{(19)}\) for use by Corps of Engineers agencies at military airfields.
The method has been developed from full-scale accelerated traffic test tracks, and is entirely empirical. The method has no theoretical basis except through its relationship to plain rigid pavement design. A reduction in the required thickness of concrete is allowed through the use of nominal percentages of steel reinforcement. The design method is based upon the concept that a concrete slab will continue to carry traffic satisfactorily after it has cracked, providing the cracks are held tightly together to provide good load transfer across the cracks and the portions of the cracked slabs do not move differential to one another, thus causing raveling and spalling along the edges of the slab. It is to be remembered that plain concrete pavement design is based upon the concept that the initial crack constitutes failure of the slab. However, it has long been observed that even plain concrete slabs continue to satisfactorily carry load for some time after they have cracked. It has also been observed that continued deterioration of the cracked slabs is accelerated by the working of the cracks due both to traffic and temperature changes. Therefore, it seems logical to assume that reinforcement to keep the cracks that develop tightly closed will result in an even longer life. In the design method, the reinforcement is used only to maintain integrity of the slabs after they crack, which permits the pavement to carry additional traffic. The reinforcement does not deter cracking, nor is the reinforcement depended upon to carry the applied load. Thus it might be considered that the steel is provided to minimize opening of the cracks due to temperature changes, and to provide good slab alignment and load transfer across the cracks.

As mentioned, the design method, which in essence simply consists of a nominal reduction in required thickness, was developed from test track work. Test items with varying percentages of steel reinforcement were constructed and trafficked to a failure condition, defined to be when the cracks began to ravel and spall; this is considered to produce an operational hazard and a severe maintenance problem. The performance of these items was then compared to plain concrete test items trafficked under similar conditions in order to determine the thickness of plain concrete required to provide equal performance. The difference represented the reduction in concrete thickness that could be allowed for reinforcement.
During the Lockbourne test track program (1944-1948), some fourteen reinforced concrete test sections, ranging in thickness from 8 to 16 inches, and percent reinforcement, ranging from 0.13 to 1.84 were subjected to traffic with single wheel loadings of 60,000 and 150,000 pounds \((20)(21)\). The conclusions reached from these tests were that the steel reinforcement prolonged pavement life after the initial cracking occurred, and thus, structurally was better than plain concrete test items of equal thickness. It was also concluded, however, that the savings in concrete thickness would not pay for the steel, and thus, reinforced rigid pavements for structural benefits were not utilized.

Reinforced concrete pavement test sections were again studied in the Sharonville channelized traffic test tracks in 1955-1956 \((22)\). Eleven reinforced concrete test items, ranging in thickness from 8 to 14 inches were included in these tracks. Percent steel reinforcement ranging from 0.167 to 0.366 was included with some items reinforced both top and bottom; others were reinforced in the middle only. These test items were trafficked with a 100,000-pound twin wheel loading.

Analyzing the results of the reinforced concrete test items led to the development of the relationship shown by Figure 7, which shows the increase in effective plain concrete versus the percent steel for equal performance of reinforced and non-reinforced concrete test items. The results of the earlier Lockbourne results were also used for the development of this relation. From this relationship, the nomograph shown by Figure 8 was developed for reinforced concrete design.

The use of the nomograph for design is as follows:

1. Using the design flexural strength of the concrete, \(R\), and the modulus of soil reaction, \(k\), the required thickness of plain concrete pavement, \(h_d\), is determined for the design gear loading and coverage level from an appropriate rigid pavement design chart.

2. With the value of \(h_d\), the nomograph is used to determine either a thickness of reinforced rigid pavement, \(h_r\), for a preselected percent of steel reinforcement, \(S\), or to determine the required percentage of steel, \(S\), for a preselected value of reinforced rigid pavement thickness, \(h_r\). The
combination of S and \( h_r \), determined from the nomograph using \( h \), will yield a reinforced rigid pavement thickness that will sustain the design gear loading for the same number of coverages as the plain concrete thickness determined from the rigid pavement design chart. The percent steel, \( S \), determined from the chart or selected, must be provided in both the longitudinal and transverse directions.

Thus, it is seen that some reduction in required plain rigid pavement is afforded through the use of steel reinforcement. Since the steel is not provided to carry tensile stresses developed from applied wheel loads, the steel is placed slightly above the mid-depth of the slab; specifically at a depth of \( \frac{h_r}{4} + 1" \).

Thus, the steel, itself, is subjected to stresses primarily developing due to the horizontal movement of the slab resulting from contraction and expansion of the concrete. This then allows the use of longer slabs, and thus a reduction in the number of transverse joints required, since the steel is available to control transverse cracking that might occur due to the sliding resistance between the slab and subgrade as the concrete contracts. The length of the slab has been found to be a function of the slab thickness, sliding resistance, and amount and strength of the reinforcing steel; this is expressed as follows:

\[
L = \frac{3}{\sqrt{0.00047 \ h_r \ (f_s S)^2}}
\]  

(19)

where

\( L \) = maximum allowable length of slab, ft.
\( h_r \) = thickness of reinforced pavement, in.
\( f_s \) = yield strength of reinforcing steel, psi
\( S \) = percent of steel used = \( \frac{A_s}{A_p} \times 100 \)
\( A_s \) = cross sectional area of steel, sq. in. per foot of pavement length or width.
\( A_p \) = cross sectional area of pavement, sq. in. per foot of pavement length or width.
The solution of the above equation, using two values of $f_s$, has also been included on the nomograph, Figure 8.

In airfield pavements, equal percents of steel are required in both the longitudinal and transverse directions in the concrete. This is required since the direction of maximum stress in the concrete depends upon such things as gear configuration, orientation of gear with respect to joints, and the patterns of applied traffic.

Certain limitations have been established for the design of reinforced rigid pavements. These are:

1. No reduction in required thickness of plain rigid pavement will be allowed when percentages of reinforcing steel less than 0.05 are used.

2. No further reduction in required thickness of plain rigid pavement shall be allowed over that indicated for percentages of steel of 0.5 regardless of the percent used.

3. The maximum length of reinforced concrete slab allowed is 100 ft., regardless of percent steel reinforcement used.

4. The minimum thickness of reinforced rigid pavement shall be 6 inches.

Some of the above limitations have been based upon experience; others are based upon the lack of information regarding the performance of reinforced rigid pavements.

The question of economics in the use of reinforced rigid pavement is still an important consideration in their use for airfield pavements. Normally, it has been the experience that plain rigid pavements are most economical; except for special situations, such as grade problems, where the reduction in required pavement thickness of a few inches will result in the saving of a considerable amount of other work. The Corps of Engineers requires the use of reinforcement in special situations. Such instances present themselves when odd-shape (non-rectangular) slabs are used, when blockouts within a slab are required, and when the joints in a new pavement abutting an old pavement cannot be matched. Experience has shown that in such instances, cracking of individual slabs is likely to occur; unless controlled, they can progress on into other paving slabs. Therefore, a minimum amount
of steel reinforcement, generally 0.05 percent, is required in these isolated slabs to reduce the working of any cracks that might develop.

DESIGN OF OVERLAYS FOR STRENGTHENING EXISTING RIGID PAVEMENTS

The design methods for various types of overlays that can be applied to increase the load carrying capacity of an existing rigid pavement have likewise been developed empirically through the accelerated traffic test track program. The detailed design procedures are published in a manual\(^{(19)}\), and the development of the procedures have been described in a paper published in an ASCE Journal\(^{(23)}\).

There are two general types of overlay used for strengthening an existing rigid pavement: rigid overlay and non-rigid overlay. Rigid overlays consist either of plain, rigid pavement or reinforced rigid pavement cast on the existing rigid pavement. Non-rigid overlays consist either of all-bituminous concrete or a combination of bituminous concrete surfacings on a granular base course material constructed on the existing rigid pavement. The use of these general types of overlays for strengthening purposes is dependent upon first costs and operational requirements. Normally, there are no restrictions, insofar as operational requirements are concerned, on the use of rigid overlays. The use of non-rigid overlays is subject to the same restrictions as the use of flexible pavements. Non-rigid overlays, in most cases, are not used in areas subject to excessive fuel spillage or extreme heat and blast, unless a special surfacing is provided.

The required thicknesses of both general types of overlay for strengthening purposes have been developed by constructing full-scale test sections of various thicknesses, subjecting the sections to accelerated traffic, and comparing their performance to plain concrete test items subjected to the same loadings. Thus, in each case, the required thickness of overlay is related to the thickness of plain concrete required for the design loading. A large number of test items were studied during the Lockbourne and Sharonville test track programs, the results of which are contained in the
references. Overlay test items have been subjected to single wheel, twin wheel and twin-tandem wheel loadings.

In the design of rigid overlays, there are three distinct methods available, depending upon the degree of bond developed between the overlay and existing pavement. A deliberate and concentrated effort may be exerted to produce a complete bond between the new and old pavements, in which case, a minimum thickness of overlay is required. In this case, the overlay and old pavement act as a single thickness pavement. Secondly, a partial bond may be obtained between the new and existing pavement, which is the condition most often used for strengthening purposes. Thirdly, a deliberate effort is made to prevent bond between the new and existing pavement; a condition seldom used, and one which requires the greatest thickness of overlay. These various overlay conditions have been designated bonded, partially bonded, and non-bonded. Their use and method of design are as follows:

(1) Bonded overlays currently are used only for resurfacing an existing pavement when large increases in load carrying capacity are not required. The bond between the new and existing pavement can be developed by either of two methods: use of a sand-cement grout mix, or use of a chemical adhesive, such as an epoxy resin. In either case, the success of the bonding lies with the preparation of the surface of the existing pavement. It is imperative that the surface be clean, free of foreign materials, and good sound concrete. To assure this, the surface of the existing pavement is abraded to remove about 1/8 to 1/4 inch of the existing weather-weakened concrete surface. Acid etching of the abraded surface to rid it of dust and other contaminants may be required; however, flushing of the abraded surface with high pressure water jet is required. The bonding medium, sand-cement grout or chemical adhesive, is then applied and the overlay immediately constructed. The sand-cement grout method of bonding has been used for several bonded overlays, and has been found to be satisfactory when properly used. Only limited use has been made of the chemical adhesive bonding, primarily because of its expense and lack of experience in its use. For either bonding method, the required thickness of overlay is determined from the formula:
where

\[ h_o = h_d - h_e \]  \hspace{1cm} (20) \]

\[ h_o = \text{thickness of overlay pavement, in.} \]

\[ h_d = \text{thickness of plain rigid pavement, in., determined from the appropriate rigid pavement design chart for the design loading using the k and R of the existing subgrade and concrete, respectively.} \]

\[ h_e = \text{thickness of existing pavement, in.} \]

As stated previously, the bonded overlay is used only for resurfacing purposes. Two reasons for this exist: first, a method for providing load transfer devices in the joints of the overlay which will be compatible with the load transfer in the existing pavements has not been developed and proven; and secondly, when large thicknesses of overlay are required (6 in. or more), it is generally found to be more economical to use the partially bonded overlay method.

(2) Partially bonded rigid overlay is by far the most widely used type of rigid overlay for strengthening existing rigid pavements. In this case, no special effort is made to create or destroy bond between the two pavements. Some cleaning of the existing surface is required to rid it of greases, oils, paints, and debris, whereupon, the overlay is then constructed directly on the existing surface. The required thickness is determined from the following formula:

\[ h_o = \frac{1.4}{h_d - C h_e} \]  \hspace{1cm} (21) \]

where

\[ h_o \text{ and } h_e \text{ are as defined above, } h_d \text{ is the required thickness of plain concrete determined from the rigid pavement design chart using the flexural strength of the overlay pavement, and } C \text{ is a coefficient depending upon the structural condition of the base pavement. The numerical value of } C \text{ is based upon a visual inspection of the existing pavement and is determined as follows:} \]

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C = 1.00 when the existing slabs are in good condition, with little or no structural cracking.

C = 0.75 when the slabs show initial cracking caused by applied loads, but little or no multiple cracking.

C = 0.50 when a large number of slabs show multiple cracking, but the majority of slabs are intact and contain only initial cracking.

C = 0.35 when the majority of slabs show multiple cracking.

The above formula assumes the flexural strength of the concrete used for the overlay will be approximately equal to that of the base pavement. When such is not the case, and the flexural strengths differ by more than 100 psi, the following modified formula will be used to determine the required thickness of the overlay:

\[ h_o^{1.4} = h_d^{1.4} - C \left[ \frac{h_d}{h_{db}} h_e^{1.4} \right] \ldots \ldots \ldots \ldots (22) \]

where

\[ h_{db} = \text{the required thickness of plain concrete obtained from the design chart using the flexural strength of the base pavement.} \]

(3) Nonbonded rigid overlays are used when the existing rigid pavement is badly deteriorated and broken, or when very thick overlays are used to strengthen thin existing pavements. In such cases, a bond-breaking material isolating the slabs to prevent the base pavements from adversely affecting the performance of the overlay is used. Nonbonded overlays are also used when it is impossible or impractical to make the joints in the overlay coincide with those in the base pavement. The required thickness of the overlay is determined by the following formula:

\[ h_o^2 = h_d^2 - C h_e^2 \ldots \ldots \ldots \ldots (23) \]

When the flexural strength of the overlay and existing pavements differ by more than 100 psi, the formula is modified as follows:

\[ h_o^2 = h_d^2 - C \left[ \frac{h_d}{h_{db}} h_e^2 \right] \ldots \ldots \ldots \ldots (24) \]

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Partially bonded and nonbonded rigid overlays have been used extensively over the years to strengthen existing rigid pavements, and have proven to be satisfactory. Slight modifications have been made to the formula for determining the required thickness of partially bonded overlay based upon performance observed during the condition survey program, and as additional test track information was developed.

Although there are two types of non-rigid overlay (all-bituminous and a combination of bituminous surface and base course designated as "flexible") used to strengthen rigid pavements, the method for determining required thickness is the same. The design method, developed from performance data from accelerated test track work\(^{(24)}\), is based upon a concept of a deficiency in the existing rigid pavement thickness to carry the design loading; it assumes that a controlled degree of cracking will take place in the existing rigid pavement. The deficiency in thickness is the difference between the thickness of the existing rigid pavement and the thickness of a rigid pavement, having the same flexural strength as the base pavement that would be required if constructed on the same subgrade as the existing rigid pavement. The cracking of the existing pavement during the life of the overlay is controlled by a factor that has been developed from the observed cracking that takes place as traffic is applied to a rigid pavement. This relation was developed from the test track work in which the rate of cracking versus applied traffic was observed. The required thickness of non-rigid overlay is determined by the formula:

\[
t = 2.5 (F h_d - C h_e) \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (25)
\]

where

\[
t = \text{required total thickness of non-rigid overlay, in.}
\]

\[
F = \text{factor relating controlled degree of cracking in base pavement and is dependent upon the modulus of soil reaction, } k. \text{ Values of } F \text{ are determined from Figure 7B.}
\]

\[
h_d = \text{required thickness, in., of plain rigid pavement determined from appropriate rigid pavement design chart using design loading, } R \text{ of existing concrete, and existing } k \text{ value.}
\]
\( h_e \) = thickness of existing rigid pavement, in.

\( C \) = condition factor based upon structural condition of existing rigid pavement; equals 1.0 when slabs contain only nominal initial cracking and 0.75 when slabs exhibit multiple cracking.

The total required thickness of non-rigid overlay, \( t \), can be made up either of all-bituminous concrete or a combination of a bituminous wearing and binder courses with high quality base course material. The thicknesses of bituminous surface courses and quality of bituminous and base course materials must meet the same requirements as those established for flexible pavements \(^{25}\). The required thickness of bituminous surface courses depends upon the design loading, traffic volume, and quality of base course material; however, it ranges between 3 and 7 inches. Considering that it is difficult to construct a high quality base course less than about 4 inches in thickness, the determination of whether to use an all-bituminous or flexible overlay then lies with the total thickness of non-rigid overlay required. If the thickness is sufficiently great to allow the incorporation of a \( \frac{1}{4} \)-inch or greater base course, the flexible overlay is normally used. Otherwise, the all-bituminous overlay is used.

Non-rigid overlays, like rigid overlays, have been used extensively for strengthening existing rigid pavements, and the performance has been found to be satisfactory. However, as stated previously, non-rigid overlays are limited to areas where fuel spillage or heat and blast are not excessive, since both will deteriorate the surface unless special surfacing materials such as tar-rubber or epoxy-asphalt are used.

Other overlay conditions, such as rigid overlay of flexible pavements, rigid overlay of composite pavement, flexible overlay of flexible pavements, and rigid and flexible inlay sections are described and discussed in the paper \(^{23}\) mentioned earlier; design methods are published in engineering manuals \(^{19}(25)\). Since these are special conditions and are not used extensively, they have not been discussed herein.
DEVELOPMENT OF DESIGN METHOD FOR PRESTRESSED CONCRETE PAVEMENTS

Although not published as official design criteria by the Corps of Engineers, sufficient work has been performed to develop a tentative design method for prestressed concrete. This design method along with a discussion of its development was published in a technical society paper in 1961 (26). Other reports related to work involving prestressed concrete pavements are listed in references at the end of this paper.

A prestressed concrete pavement is one in which a significantly high compressive stress is applied to the concrete during construction, and which is assumed to remain throughout the life of the pavement. There are various methods for applying the compressive stress; however, these can be generalized by two methods, termed "pre-tensioning" and "post-tensioning". Prestressing by pre-tensioning is accomplished by using prestressing tendons within the concrete, which are tensioned before the concrete is placed to a sufficiently high level to produce the desired compressive stress in the concrete. After the concrete has been placed and has gained sufficient strength, the tension on the prestressing tendons is released and the stress is transferred to the concrete through either end anchors on the tendons, or bond between the concrete and the tendons, or both. Prestressing by post-tensioning is accomplished by encasing the prestressing tendons in conduit or other bond breaking material. The concrete is placed, and after it has gained sufficient strength, the tendons are tensioned using the concrete for a reaction which in turn produces the compressive stress in the concrete.

After the desired compressive stress in the concrete is reached, the tendons are anchored at the ends. When tendons encased in conduit are used, the space between the tendons and conduit is filled with a cement grout after the tendons are tensioned. There are, of course, other ways of prestressing the concrete, many of which have been used by other countries; any method which will produce and retain the compressive force in the concrete pavement is satisfactory.

If the structural benefits derived from prestressing were limited merely
to increasing the stress range for elastic behavior of the rigid pavement, it is doubtful that the benefits derived would ever be sufficient to warrant the extra cost of construction. Fortunately, prestressing permits the structural behavior of such pavements to be analyzed completely different from that of plain concrete pavements. For plain rigid pavements, the analysis is based upon the development of a tensile stress at the bottom of the pavement, due to bending moment equal to the strength of the concrete. While this same action occurs for prestressed pavement, the bending moment must be sufficiently large to produce a tensile stress exceeding the flexural strength of the concrete plus the compressive stress in the concrete before the crack forms in the bottom of the pavement. However, even this condition does not constitute failure of a prestressed concrete pavement; since this initial crack serves as a momentary or partial plastic hinge under passage of the load, which, due to the prestress, closes up after the load passes. In addition, due to the high compressive force in the concrete, the crack does not progress to the top of the slab. The formation of the hinge action produces a re-distribution of the bending moment in the slab, such that the negative radial moments are increased substantially. Through small-scale static load model studies and full-scale accelerated traffic test tracks, it has been found that ultimate failure of a prestressed concrete pavement occurs only after the negative radial moments become large enough to produce tensile cracking in the surface of the slab, whereupon subsequent traffic breaks up the pavement rapidly. Based upon these tests, failure of a prestressed pavement has been defined as the occurrence of the secondary tensile cracking in the upper surface of the slab, due to the negative radial moments.

Based upon this definition, and using the results of model studies and full-scale accelerated traffic test results, the following design formula for prestressed concrete pavements has been developed.

\[
R_p = \frac{6P(DF)[C(M/P) - M_r/P_o]}{F_{ESWL}h^2} - R + F \quad \ldots \ldots \ldots \ldots \ldots (26)
\]
where

\[ R = \text{required compressive force, psi, in concrete by prestressing.} \]

\[ P = \text{gear loading in pounds} \]

\[ \text{DF} = \text{design factor for repetitive loading.} \]

\[ C = \text{radial moment correction factor} \]

\[ \frac{M}{P} = \text{bending moment coefficient for rigid pavement for interior loading of ESW.} \]

\[ \frac{M}{P} = \text{radial moment coefficient for prestressed concrete for interior loading of ESW.} \]

\[ R = \text{concrete flexural strength of concrete, psi} \]

\[ F = \text{maximum subgrade restraint stress, psi} \]

\[ F_{\text{ESWL}} = \text{factor used to express gear load in terms of equivalent single wheel load} \]

\[ h = \text{thickness of prestressed concrete pavement, in.} \]

In design, both pavement thickness and magnitude of prestress must be determined; since both are variables in the above equation, it is necessary to assume one and compute the required value of the other. Normally, the thickness is assumed, and the magnitude of prestress computed. This procedure is repeated until an acceptable level of prestress is determined. Both the longitudinal and transverse prestress levels are obtained from the formula; the only difference being based on the value of \( F \), which is dependent upon the length or width of the concrete slab. A design example is shown in the referenced 1961 paper\(^{(26)}\); and, because of the number of charts required to present an example, will not be included in this paper. It can be seen that relations for design factor versus coverages, a chart for radial moment correction factor, moment coefficient charts for interior loading of plain and prestressed concrete, and a chart for determination of the equivalent single load factor must be available to use the formula. These charts have been developed and presented in the referenced paper\(^{(26)}\).

At the beginning of this paper, it was mentioned that although methods for the design of prestressed concrete pavements had been developed, they have not been issued as criteria for design because of the lack of information.
regarding the effects of repetitive loading. These effects are represented in the formula by the design factor. Two full-scale accelerated test tracks were constructed which included a total of 16 different test sections. Variables studied in these test sections included levels of longitudinal and transverse prestress, including longitudinal prestress with transverse steel reinforcement only, and various designs and treatments of joints between the concrete paving lanes. Both test tracks were 9 inches thick; therefore, thickness was not a variable. However, two types of subgrade were used giving some information on the effects of k. Of the 16 test sections, 11 were trafficked to failure under twin-tandem gear loadings of 200,000 and 240,000 pounds. Failures occurred at coverage levels between 500 and 2250. Therefore, it can be seen that the information needed to develop the required design factor versus coverage relationship is extremely limited, and not considered sufficient for publishing a criteria which must encompass the design coverage levels of from 200 to 25,000 used in today's criteria.

It is pointed out, however, that from the results of these studies, an experimental section of taxiway pavement was designed for the B-52 aircraft (265,000 pound twin-twin wheel gear loading) and constructed at Biggs Air Force Base, Texas. The taxiway section is 1500 feet long, made up of three 500-ft. long prestressed concrete slabs, and 75 feet wide, made up of three 25-ft. wide paving lanes, prestressed transversely as a unit. The pavement thickness is 9 inches, and the longitudinal and transverse prestressing 350 and 175 psi, respectively. Complete construction details of this pavement are reported in a technical report (27). Construction of the taxiway section was completed in 1959, and since that time (6 years), the pavement has been subjected to the B-52 and other aircraft traffic occurring at the base. Periodic inspections of this pavement have revealed no indications of distress insofar as the prestressed concrete pavement is concerned. Maintenance of the joints at the ends of the 1500-ft. section and between the 500-ft. sections has been a constant problem due to the excessive movements that occur due to contraction and expansion of the concrete. The problem has been one of trying to find a material which will perform in the wide joints (1 1/2"±), and accommodate daily and seasonal movements of 1"±.
During the design of the taxiway section, a mechanical joint was considered; however, due to the extremely high cost of such a joint, the decision was made to utilize more conventional designs of grooves and pourable or pre-formed materials. Although the problem of joint maintenance exists, the fact remains that the prestressed concrete taxiway section has performed satisfactorily for 6 years with no signs of distress.
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\[ M = \frac{qH^3}{24EI} \]

\[ q = \text{Intensity of loading} \]

\[ N = \text{Number of positive blocks minus number of negative blocks} \]

(Shaded blocks count only as fractions)

\[ \frac{1}{(R)^k} \]

\[ D = \text{Flexural rigidity of pavement} \]

\[ k = \text{Rigidity of subgrade (density of liquid)} \]

*Influence Chart for the Moment at Edge (Point 0) of a Concrete Slab Due to a Load in the Vicinity of the Edge*

(Subgrade assumed to be a dense liquid, Poisson's ratio for pavement = 0.15)

**Figure 1**
TYPICAL TRAFFIC DISTRIBUTION CURVES FOR
TWIN & TWIN-TANDEM WHEEL GEAR AIRCRAFT

TYPICAL TRAFFIC DISTRIBUTION CURVES FOR
SINGLE & LT. TWIN WHEEL GEAR AIRCRAFT

FIGURE 2
BICYCLE GEAR AIRCRAFT

TRICYCLE GEAR AIRCRAFT

NOTE: DESIGN FACTOR = \( \frac{R}{V_e} \)

WHERE:
- \( R \) = CONCRETE FLEXURAL STRENGTH, PSI
- \( V_e \) = EDGE STRESS, PSI

DESIGN FACTOR VS COVERAGE
Moment per pound of wheel load

SINGLE WHEEL: 267 SQ. IN CONTACT AREA

FREE EDGE LOADING

\[ \sigma_e = \frac{6P}{h^2} \left[ \frac{M}{P} \right] \]

- \( P \): Wheel load, pounds
- \( A \): Contact area, sq. in.
- \( h \): Slab thickness, in.
- \( k \): Radius of relative stiffness, in.

\[ k = \frac{E_b^3}{2(1-\mu^2)h^4} \]

- \( E \): Modulus of elasticity of concrete assumed equal to 4 x 10^6 psi
- \( \mu \): Poisson's ratio of concrete assumed equal to 0.20.

FIGURE 4
Factor to relate total load on multiple wheel gear to load on single wheel producing equal maximum stress.
DESIGN CURVES FOR CONCRETE AIRFIELD PAVEMENTS
TWIN WHEELS SPACED 37\textdegree\hspace{1pt}C.C.
(267 SQ IN. CONTACT AREA EACH WHEEL)

(C-121, C-97, KC-97, B-29, B-50)
A. EFFECTS OF STEEL REINFORCEMENT

B. FACTOR FOR NONRIGID OVERLAY DESIGN
FIGURE 8

REINFORCED RIGID PAVEMENT DESIGN

h_r, INCHES
30
29
28
27
26
25
24
23
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6

h_d or h_o

A_s

L, FT

0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.19
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.10
0.09
0.08
0.075
0.07
0.06
0.05

INCHES

SO IN./FT

0.50
0.40
0.35
0.30
0.25
0.20
0.19
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.10
0.09
0.08
0.075
0.07
0.06
0.05

f_y = YIELD STRENGTH OF REINFORCING STEEL IN PSI
S = PERCENT OF REINFORCING STEEL

A_s = CROSS SECTIONAL AREA OF STEEL IN SQ IN. PER FOOT OF PAVEMENT
L = MAX ALLOWABLE LENGTH OF REINFORCED PAVEMENT SLAB

h_r = THICKNESS OF REINFORCED PAVEMENT
h_d = THICKNESS OF NONREINFORCED PAVEMENT
h_o = THICKNESS OF NONREINFORCED OVERLAY PAVEMENT

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