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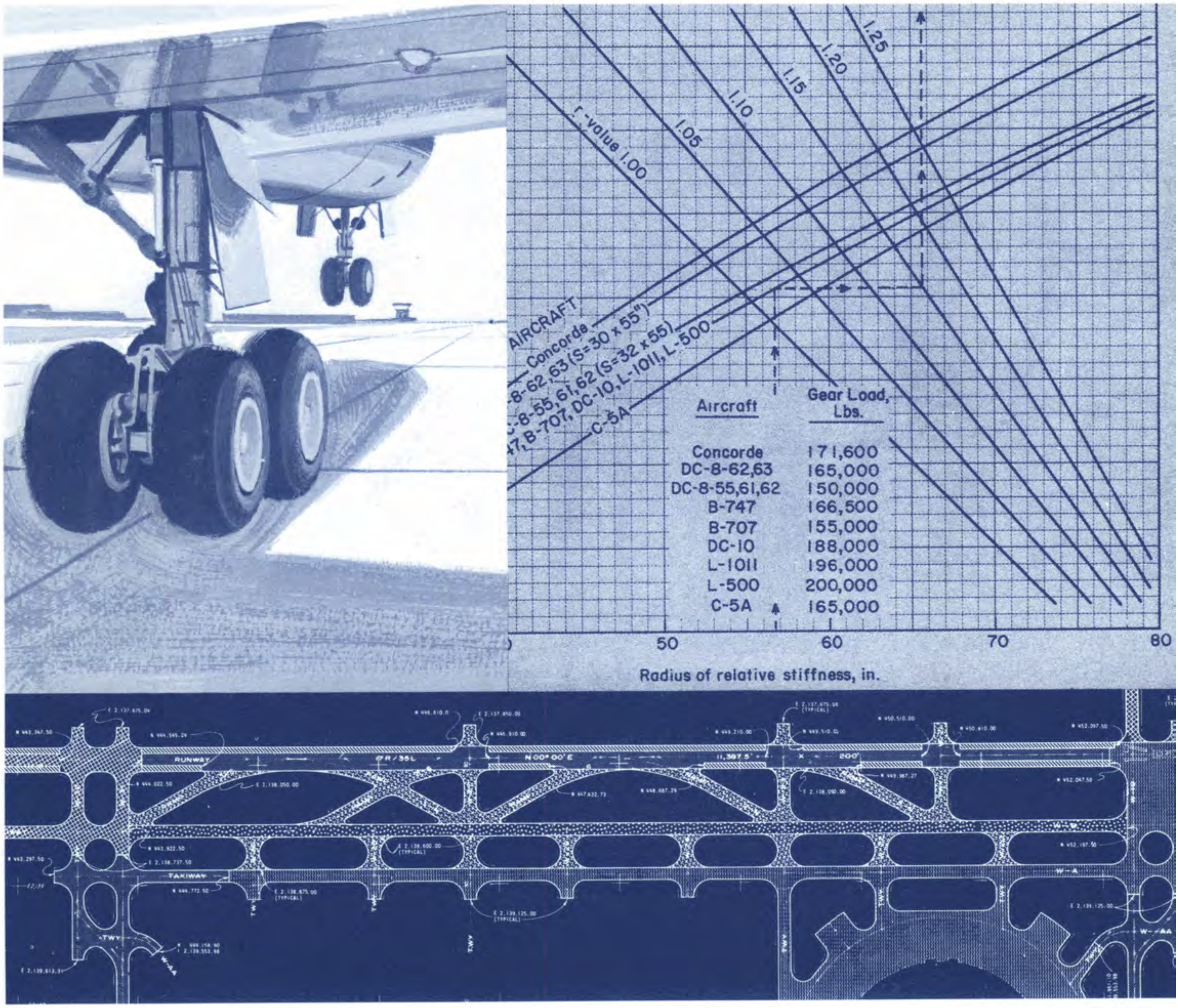
ENGINEERING BULLETIN

# Design of Concrete Airport Pavement

By Robert G. Packard




PORTLAND CEMENT ASSOCIATION

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# Design of Concrete Airport Pavement

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# INTRODUCTION

The purpose of this manual is to furnish design guides to engineers responsible for the design of concrete airport pavements. Well-established design procedures have been extended to keep pace with the rapid growth in gross weight of aircraft. Jet aircraft's high exhaust temperatures, high-velocity blast, high-pressure tires, and fuel spillage create no serious problems for concrete. However, loading conditions for new jet aircraft and future heavy, multi-geared aircraft indicate some changes in design practices to attain improved pavement performance.

The cost to the air travel industry of shutdowns for pavement maintenance or strengthening is compelling reason for airport engineers to carefully anticipate the loadings and structural demands that will be made on a pavement during its design life and to design pavements to meet those demands with a minimum of future maintenance, reconstruction, and pavement strengthening.

The design methods presented in this manual are based on knowledge from the following sources:

1. The performance of existing pavements on both civil and military airfields. This performance experience includes a wide range of pavement strengths and loadings—from thin pavements serving light civil aircraft at small general-aviation airports to thick pavements serving heavy military bombers for many years.
2. Full-scale pavement loading tests such as those conducted by the U.S. Army Corps of Engineers during and since World War II.
3. Laboratory-controlled tests of pavement sections and models, and full-scale tests made by a number of agencies including the Portland Cement Association.
4. Theoretical studies of pavement stresses and deflections by H. M. Westergaard, Gerald Pickett, Gordon K. Ray, Donald M. Burmister, and others.

Detailed procedures are given for the design of concrete pavement for runways, taxiways, and aprons, including simplified design charts for determining slab thickness for different conditions of service. (Pavement thickness design charts for specific aircraft are available as supplements to this manual and are frequently updated for new aircraft and loadings.) Recommended practices are given for the design of plain, reinforced, and continuously reinforced pavements; the design of joints and jointing arrangements; the use of tiebars and dowels; the treatment of subbases and subgrades; and the design and construction of concrete overlays.

The appendixes contain information on evaluating special loading situations, such as repeated and mixed aircraft loadings; determining pavement stiffness for heavy-duty pavements with subbases; and using the PCA computer program and influence charts to determine pavement stresses.

This manual contains no information on airport location; geometric layout of runways, taxiways, etc.; length and width of pavement; drainage; shoulders; clear zones; or terminal facilities and structures. Information on these subjects can be found in publications and directives of the Federal Aviation Administration; the U.S. Army Corps of Engineers, Department of the Army; Naval Facilities Engineering Command, Department of the Navy; the Directorate of Civil Engineering, Department of the Air Force; and in Canada, the Ministry of Transport and the Department of National Defence.

## CHAPTER 1

# SUBGRADES AND SUBBASES

Because of the rigidity of concrete pavements, loads are spread over large areas of the subgrade and pressures on the subgrade are very low. As a result, concrete pavements do not necessarily require strong support from below. However, it is important that the support be reasonably uniform with no abrupt changes in degree of support.

To design and construct a reasonably uniform subgrade and subbase, three major causes of nonuniform support must be controlled:

1. expansive soils
2. frost action
3. mud-pumping

Effective control of expansive soils and frost action is achieved through appropriate subgrade preparation techniques; prevention of mud-pumping requires a granular or stabilized subbase layer. Although a subbase also provides some control of expansive soils and frost action, the use of thick subbase layers for substantial control of these factors is no more effective than adequate subgrade preparation and usually costs more.

Pavements that will carry high volumes of heavy, channelized aircraft traffic usually require a subbase to control mud-pumping. For these heavy-duty pavements, the use of stabilized subbases is increasing because of several other important benefits in addition to the prevention of mud-pumping.

## Expansive Soils

Test methods to determine the expansive (high-volume change) capacities of soils have been developed. The simpler tests provide indexes (such as plasticity index, shrinkage limit, and bar shrinkage) for identifying the approximate volume-change potential of soils. For example, the following table shows approximate expansion-plasticity relationships:

| Plasticity index (ASTM D424) | Degree of expansion | Approximate percentage of swell (ASTM D1883) |
|---|---|---|
| 0 to 10 | Nonexpansive | 2 or less |
| 10 to 20 | Moderately expansive | 2 to 4 |
| More than 20 | Highly expansive | More than 4 |

Excessive differential shrink and swell of expansive soils in a subgrade create nonuniform support. As a result, the concrete pavement placed on such a subgrade may become distorted and warped. Several conditions can lead to nonuniform support and damage to the pavement:

1. When expansive soils are compacted in too dry a condition or are allowed to dry out prior to paving, subsequent nonuniform expansion may cause pavement roughness.
2. When expansive soils are too wet prior to paving, subsequent nonuniform shrinkage may leave the slab edges unsupported or cause an objectionable increase in pavement crown.
3. When pavements are constructed over expansive soils with widely varying moisture contents, subsequent shrink and swell may cause bumps, depressions, or waves in the pavement surface. Similar waves may occur where there are abrupt changes in volume-change capacities of subgrade soils.

Nonuniform support and pavement distortion from nonuniform shrink and swell of expansive soils are most likely to occur in arid, semiarid, or subhumid regions. Objectionable distortion can also occur in humid climates during periods of drought, during long dry periods in the summer months, or where subgrade soils are extremely expansive.

In all climatic areas, compaction of highly expansive soils when they are too dry can lead to detrimental expansion and softening of the subgrade during later rainy periods. The softening occurs more rapidly at joints and along pavement edges due to moisture infiltration. The resultant differential support may lead to pavement distress before the subgrade soils can adjust to the climatic environment and reach a more uniform and stable moisture content.

The following measures provide effective and economical control of expansive soils:

**1. Subgrade grading operations.** Selective grading, cross-hauling, and mixing of subgrade soils make it possible to have reasonably uniform conditions in the upper part of the subgrade, with gradual transitions between soils of varying volume-change properties.

**2. Compaction and moisture control.** It is critically important to compact expansive soils 1 to 3 percent wet of American Association of State Highway Officials (AASHO) T99 optimum moisture, Both research (references 1-6)* and experience show that expansive soils expand less on wetting and absorb less water when compacted at this condition. Results of compaction tests of a typical expansive clay, given in Fig. 1, show that at moisture contents several percentage points above optimum the expansion is not significant.

After pavements are built, the moisture content of most subgrades increases to about the plastic limit of the soil (ASTM D424); that is, the moisture content reached is close to or slightly above the standard optimum. If this moisture content is obtained in construction, the subsequent changes in moisture will be much less and the subgrade will retain the reasonably uniform stability needed for good pavement performance.

The Corps of Engineers and other agencies use a modified moisture-density test (AASHO T180) with a higher compaction effort that gives higher densities and lower optimum moisture contents than the usual test method (AASHO T99) The modified procedure was developed to represent higher compaction of granular subbases and base

*Reference numbers and superscript numbers in parentheses designate references listed at the back of this manual.

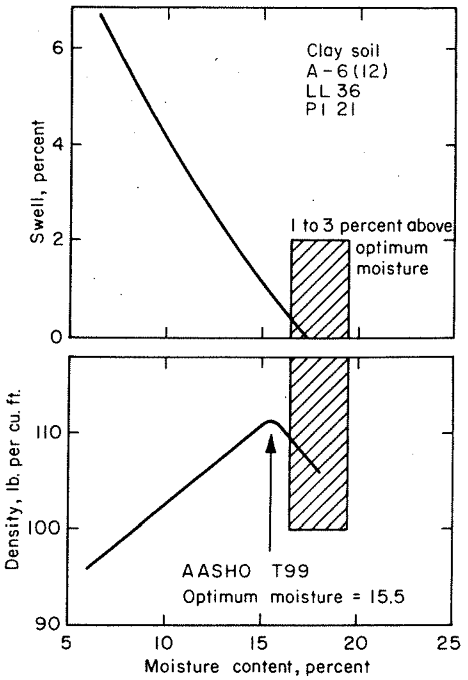



**Fig. 1. Effect of compaction moisture content on soil expansion.**

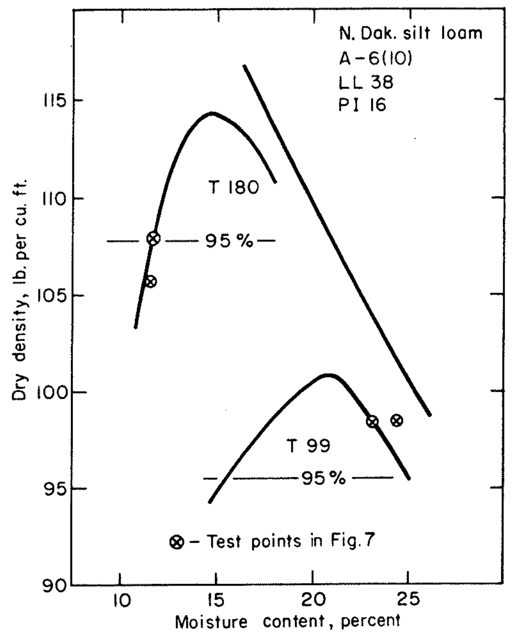
courses. It is also useful for subgrades of low plasticity. While excellent for these purposes, the higher compactive effort results in moisture contents that are much too low for expansive soils. Compaction of high-volume-change soils at these lower optimum-moisture contents results in excessive swell.

An example of the moisture, density, and expansion relationships obtained with the two compactive efforts is shown in Fig. 2. Expansion is greatly reduced when the soil is compacted wet of standard optimum (T99) compared to the high expansion obtained when the soil is compacted dry of modified optimum (T180) with the greater compactive effort. These data also show that greater strengths and lower moisture absorptions prevail, after soaking, for the soil compacted wet of standard optimum.

**3. Nonexpansive cover.** In areas with prolonged periods of dry weather, highly expansive subgrades may require a cover layer of low-volume-change soil placed full width over the subgrade. This layer minimizes changes in the moisture content of the underlying expansive soil and also has some surcharge effect. If the low-volume-change layer has low to moderate permeability, it is not only more effective but usually less costly than a permeable, granular soil. Highly permeable, open-graded subbase materials are not recommended as cover for expansive soils since they permit greater changes in subgrade moisture content.

Local experience with extremely expansive soils is the best guide for adequate depth of cover.

**4. Cement-modified subgrade.** The treatment of expansive clay soils with cement is very effective not only in reducing volume changes but in increasing the bearing strength of subgrade soils. Because of this it may be desirable and economical in some cases to modify existing soils



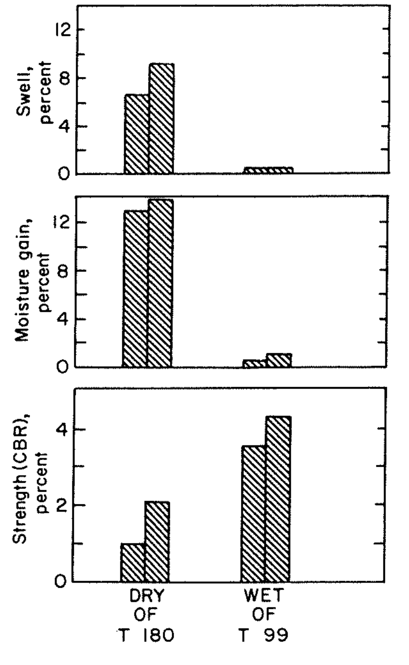




**Fig. 2. Expansion, strength, and moisture gain of soil compacted at different moisture-density conditions.**

with cement instead of importing a nonexpansive soil to cover an expansive subgrade.

For specific projects, the cement content for control of volume change and increase in strength is based on laboratory test results.* Some typical volume change and strength data for a cement-modified clay soil are shown in Fig. 3.

*For discussion of cement-modified soils and test methods, see references 7 and 8.

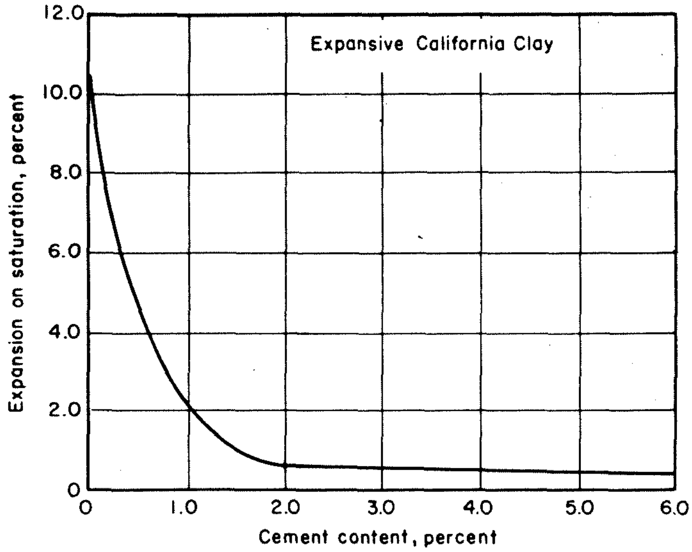



Fig. 3. Effect of cement content on soil expansion.

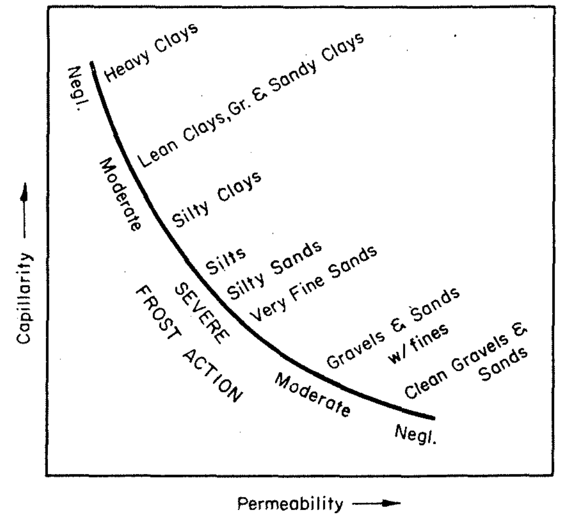



Fig. 4. Relation between frost action and hydraulic properties of soils.

## Frost Action

Uniquely for concrete pavements, field experience has shown that frost-action damage is a result of frost heave—abrupt, differential heave—rather than damage from spring subgrade thawing.

The relatively brief periods of reduced subgrade support that accompany thawing have very little effect on concrete pavements because (1) concrete reduces pressures on soft subgrades to safe limits by distributing loads over large areas; and (2) effects of fatigue during the period of reduced subgrade strength are offset by reduced fatigue during the longer period that the subgrade is frozen and offering very high support.

Further evidence that concrete pavements designed with uniform support are not influenced by spring thaw is found in the results of the AASHO Road Test.(9) The pavement performance and the equations written to relate the design variables to traffic loads show that concrete pavements, with or without a subbase, were not affected by the spring thaw periods.

Thus, subgrade softening on thaw is not a design consideration for control of frost action. It is the abrupt, differential frost heave that must be controlled.

Frost damage to concrete pavement is controlled by reducing the nonuniformity of the subgrade soil and the moisture conditions that lead to objectionable differential heaving—especially where subgrade soils change abruptly from non-frost-susceptible soils to the highly frost-susceptible silts.

Criteria and soil classifications used for identifying frost-susceptible soils usually reflect both susceptibility to heaving and to softening on thaw. Therefore, soil data must be reviewed to distinguish between data for soils susceptible to heave and data for soils susceptible to softening after thaw. As stated in *Frost Action in Roads and Airports*, Highway Research Board Special Report No. 1: "Criteria for non-frost susceptibility as they pertain to intense differential heaving need not lie identical to criteria pertinent to load-carrying capacity during the thawing period. Accordingly, the reader is asked to distinguish between adequacy of design based on needs."

The degree of frost susceptibility can be explained by the hydraulic properties of soils: (1) capillarity—the soil's ability to pull moisture by capillary forces; and (2) permeability—the soil's ability to transmit water through its voids. The relation of these hydraulic properties to frost susceptibility is visualized in Fig. 4. The worst heaving usually occurs in fine-grained soils subject to capillary action. Low-plasticity soils with a high percentage of silt-size particles are particularly susceptible to frost heave. These soils have pore sizes small enough to develop capillary potential but large enough for passage of water to the frozen zone. Coarser soils have higher rates of flow, but not the suction potential to lift enough moisture for heaving. More cohesive soils, although developing high capillarity, have low permeability and water moves too slowly for growth of the thick ice lenses that cause damage.

As in the case of expansive soils, a large degree of control of frost action is accomplished most economically by appropriate grading operations and by controlling subgrade compaction and moisture. These methods include:

1. **Selective grading and mixing.** These operations are used to replace localized areas of highly frost-susceptible soils and to iron out soil transitions to correct abrupt changes in soil types. Where soils vary widely or frequently in texture and where nonuniform conditions are not clearly defined, mixing of the soils is effective in preventing differential frost heave. With modern construction equipment, mixing of nonuniform soils to form a uniform subgrade is more economical than importing select materials from borrow pits.

2. **Removal of silt pockets.** Where highly frost-susceptible soils are pocketed in less-susceptible soils, they are excavated and backfilled with soils like those surrounding the pocket. Moisture and density conditions for the replacement soil should be as similar as possible to those of the adjacent soils. At the edges of the pocket, the replacement soil should be mixed with the surrounding soil to form a tapered transition zone.

3. **Compaction and moisture control.** After reasonable uniformity has been achieved through grading operations, additional uniformity is obtained by proper subgrade compaction at controlled moisture contents. The permeability of most fine-grained soils is substantially reduced when they are compacted slightly wet of AASHO T99 optimum moisture. Reducing soil permeability retards the rate of moisture flow to the frozen zone and frost heaving is reduced. Compaction at these moisture contents also makes subgrades less susceptible to nonuniform moisture changes.

4. **Drainage.** For drainable subgrade soils with high water tables or for draining permeable subbases, the installation of subgrade drains will effectively reduce the amount of water available for frost heaving. However, the benefit of subgrade drains to lower the water table in relatively impervious, fine-textured soils is questionable. Intercepting drains are useful where wet spots are found due to seepage through a permeable strata underlaid with an impervious material. Drainage and backfill details are not given here but may be found in the manuals of the agencies mentioned at the end of the introduction to this manual.

5. **Non-frost-susceptible cover.** A layer of clean gravel or sand will reduce frost heave, but such a subbase is not used for this purpose alone since less costly grading operations properly done will reduce frost heave. When a subbase layer is being used to prevent mud-pumping, it will also provide some protection against frost action. The benefit of a thick subbase layer is somewhat diminished for control of frost heave because coarse soils permit deeper frost penetration than fine-grained soils with their higher moisture contents. Thick subbase layers are more effective in preventing loss in subgrade support on thaw—and this is not a primary design consideration for concrete pavements.

Proper grade design, selective grading, and compaction control are effective and proved methods for control of frost action. These methods produce uniformity and resistance to moisture movement in the upper part of the subgrade and thus prevent differential or excessive heaving.

## Mud-Pumping

Mud-pumping is the forceful displacement of a mixture of soil and water that occurs under slab joints, cracks, and pavement edges. It is caused by the frequent deflection of slab joints from heavy wheel loads when fine-grained subgrade soils are saturated. Continued uncontrolled mud-pumping may eventually lead to the displacement of enough soil so that uniformity of support is destroyed.

Subbase studies show that three factors are necessary before mud-pumping can occur:

1. A subgrade soil that will go into suspension.
2. Free water between pavement and subgrade, or subgrade saturation.
3. Frequent passage of heavy loads.

For airport pavements, experience has shown that pumping is not a problem under conditions of light loads and moderate frequencies of operation. Studies conducted on highways have shown that pavements designed to carry not more than 100 to 200 heavy trucks per day do not require subbases to prevent damage from pumping. This can be a conservative guide for many airport pavements since airport traffic is less channelized. For roads with greater traffic volumes, it was found that no pumping occurred where pavements had a subbase material with less than 45 percent passing a No. 200 sieve and with a plasticity index of 6 or less. For the greatest volume of traffic encountered in the surveys, subbases meeting AASHO Specification M155 effectively prevented mud-pumping. Some of the requirements of this specification are: 15 percent maximum passing a No. 200 sieve, maximum plasticity index of 6, and maximum liquid limit of 25. AASHO M155 is essentially in agreement with the subbase criteria specified for airports by federal agencies.

To prevent consolidation under traffic, subbases should be compacted to a minimum of 100 percent of AASHO T99 density. For pavements that will carry large volumes of aircraft traffic, the specified density should be not less than 105 percent of AASHO T99, or 98 to 100 percent of AASHO T180.

For heavily trafficked airport pavements, a maximum subbase thickness of 6 in. is suggested for untreated granular subbases. This is sufficient to prevent pumping. When depth is more than 6 in., there is an increasing risk of poor pavement performance due to subbase consolidation under heavy volumes of traffic. Therefore, better performance and economy are obtained by building the load-carrying capacity into the concrete slab itself and using the minimum thickness of subbase that will prevent pumping and yet not consolidate.

## Cement-Treated Subbases

The performance of pavements carrying high volumes of traffic and heavy loads on multiwheeled gear has shown the benefits of stabilized subbases. Cement-treated subbases offer many benefits in addition to the prevention of mud-pumping:

1. Provide an impermeable, uniform, and strong support for the pavement.
2. Eliminate subbase consolidation.
3. Greatly improve load transfer at joints.
4. Expedite construction because the stable working base eliminates shutdowns due to adverse weather conditions.
5. Provide firm support for the slipform paver or side forms, thus contributing to the construction of smoother pavements.

In areas where acceptable subbase materials are scarce or

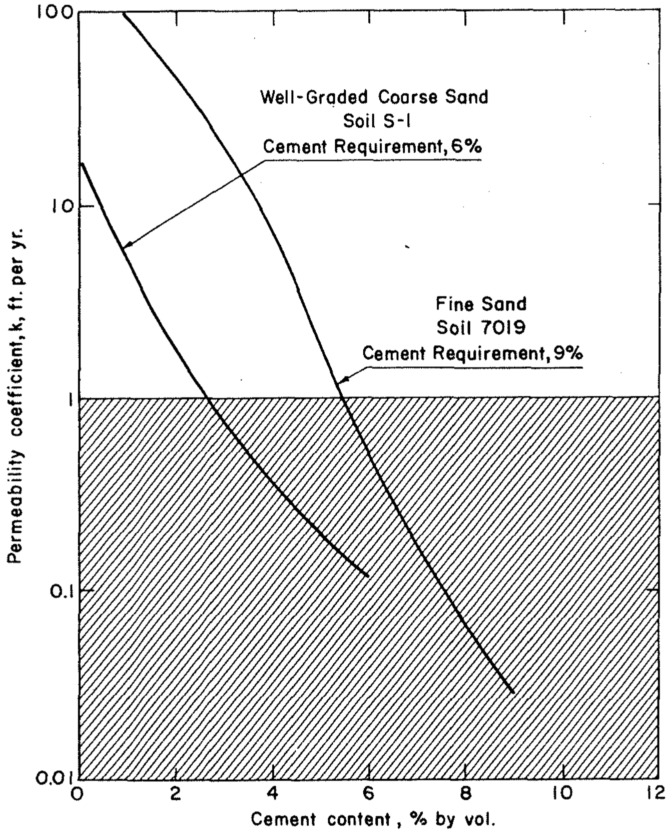



**Fig. 5. Effect of cement content on permeability for cement-treated subbase materials.**

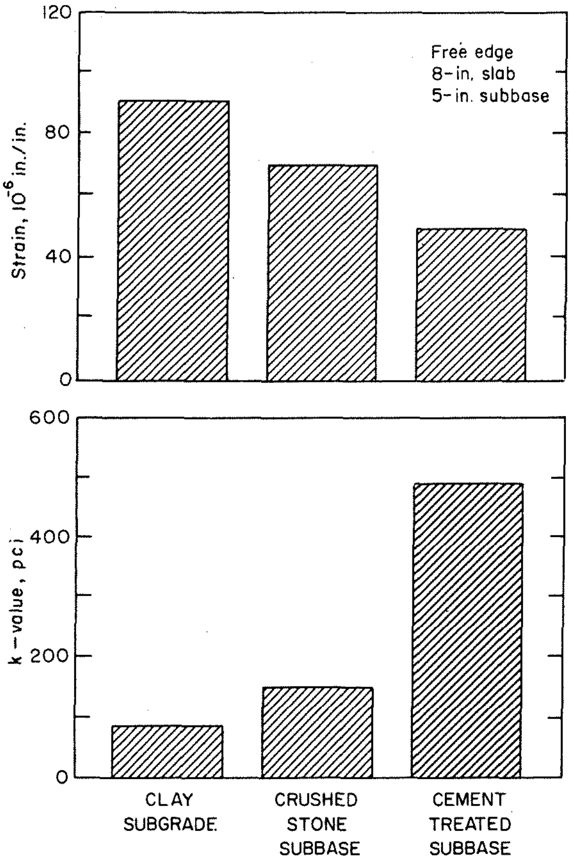



**Fig. 6. Measured strains and computed *k* values, 9-kip plate load.**

expensive, cement-treated subbases offer important economic advantages. In many instances, locally available substandard granular materials that do not meet subbase specifications can be treated with portland cement to bring them up to standard.

Cement content for cement-treated subbases for airport pavements is determined by standard laboratory wet-dry and freeze-thaw tests (ASTM D559 and D560) and PCA weight-loss criteria.[7] Other procedures that give an equivalent quality of material may be used. Details of material requirements and construction methods for quality cement-treated subbases are given in references 7, 10, and 11.

Some of the structural properties of cement-treated subbases meeting these criteria are:

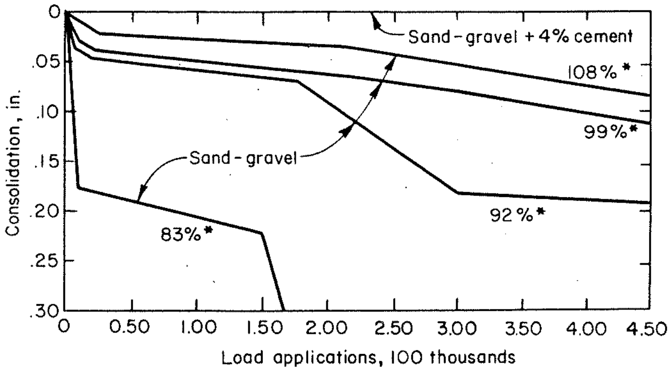
| Property | 28-day values |
|---|---|
| Compressive strength, saturated | 400-900 psi |
| Modulus of rupture | 80-180 psi |
| Modulus of elasticity (flexure) | 600,000-2,000,000 psi |

A cement-treated subbase provides a highly impermeable layer that reduces the amount of surface water reaching the subgrade and eliminates the possibility of excessive pore pressures that otherwise could develop in granular subbases. The reductions in permeability factors attained by cementation are shown in Fig. 5 for two soil-cement materials. Almost all aggregates and soils at the required cement content have extremely low coefficients of permeability[12] —less than 1 ft. per year—impervious for all practical purposes.

A desirable property of cement-treated subbases is the added high support value and stiffness that they contribute to the pavement system. It has been known for some time that plate-bearing tests made directly on cement-treated subbases produce extremely high *k* values; however, there was some question as to whether these high *k* values produce corresponding reductions of stresses in the overlying concrete slab. To determine this, full-size slabs were built at the PCA laboratories on subgrades and subbases with known *k* values as measured by plate tests.[13] Fig. 6 shows the strains measured in the slabs under a 9,000-lb. load, and the *k* values then computed from these data. The computed *k* values are in close agreement with those determined by the plate-bearing tests made directly on the subgrade and subbase, verifying that slab strains and corresponding stresses are significantly reduced.

Fig. 12 in Chapter 2 shows suggested design *k* values for different thicknesses of cement-treated subbases. The high *k* values of cement-treated subbases permit use of thinner concrete pavements for given loading conditions.

The resistance to consolidation of cement-treated subbases under repeated loads is shown in studies[14] of the effect of subbase density and cement treatment on consoli-




*Percent of standard density, AASHO T-99

**Fig. 7. Subbase consolidation under repetitive loading.**

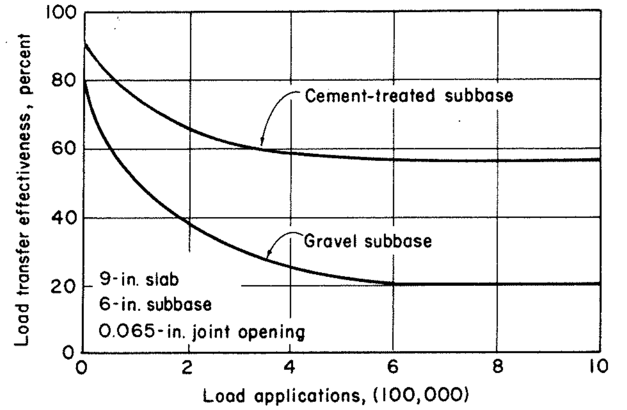



**Fig. 8. Effect of subbase type on load-transfer effectiveness.**

dation. Fig. 7 shows the effect of density on consolidation for a 6-in.-thick granular, untreated subbase. As expected, consolidation increased with decreases in the placement density of the subbase. The effect of moisture is also illustrated. The first 150,000 load repetitions were made with the subbase placed at optimum moisture. Additional water was made available for the next 150,000 load applications, and for the last 150,000 the subbase was completely saturated. The resulting consolidation increased with increasing moisture content and load applications. For a cement-treated subbase made from the same material, the same test was extended to 1,000,000 load repetitions without any measurable consolidation, as shown by top line of Fig. 7.

Another area of research on the properties of cement-treated subbases examined the effectiveness of load transfer(15) across undoweled joints, which depends on aggregate interlock. The results are shown in Fig. 8, in which "load transfer effectiveness" is the ratio of deflection of the unloaded slab to average deflection of the loaded and unloaded slabs, expressed as a percentage. As load applications were increased on the slab with untreated gravel subbase, effectiveness gradually decreased, approaching 20 percent at 1,000,000 loads. On the cement-treated subbase, the loss occurred at a much slower rate and, even after 1,000,000 loads, effectiveness remained at a level of over 50 percent. The relative performance indicates that the use of a cement-treated subbase will provide a greater degree of load transfer throughout the pavement service life.

# CHAPTER 2

## STRUCTURAL DESIGN OF AIRPORT PAVEMENT

Several major factors are involved in the structural design of concrete airport pavement:

1. Properties of the concrete
2. Supporting strength of the subgrade or subbase-subgrade combination
3. Type of aircraft and loads anticipated on the pavement and approximate frequency of operation
4. Type of pavement being designed, such as runway, taxiway, apron, hangar floors

### Properties of Concrete

#### FLEXURAL STRENGTH

Bending of concrete pavement under wheel loads produces both compressive and flexural stresses. Compressive stresses are too small, compared to compressive strength, to influence slab thickness. Ratios of flexural stress to flexural strength are much higher, often exceeding values of 0.5. As a result, flexural stresses and flexural strength of the concrete must be considered in thickness design. Flexural strength is determined by modulus of rupture (MR) tests (American Society for Testing and Materials C78, third-point loading).

Modulus of rupture tests are commonly made at 7, 14, 28, and 90 days. The 7- and 14-day test results are com-

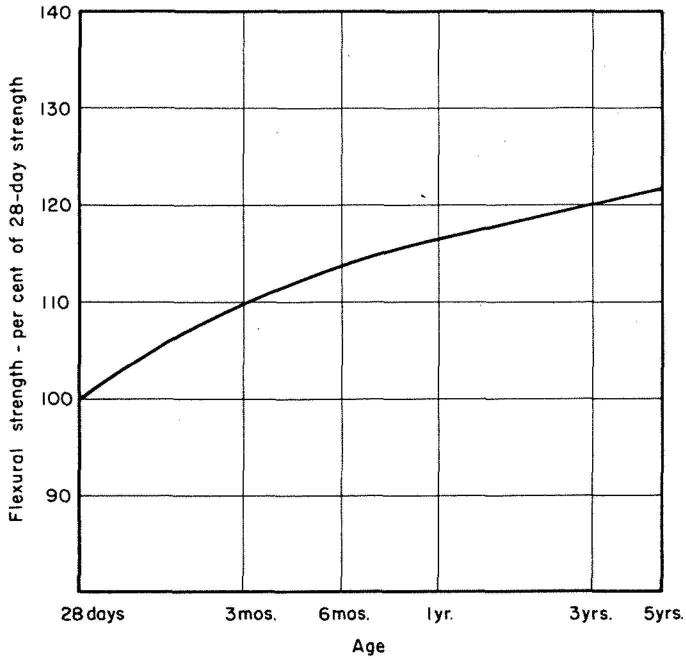



**Fig. 9. Flexural strength-age relationship.**

pared to specification requirements for job control and for determining when pavements can be put into service.

Normally for airport pavements, the 90-day test results are selected as the design strength. Condition surveys and evaluation studies show that use of a lower strength value, for example the 28-day value, usually results in an overdesigned pavement. Because of the continued strength increase that takes place during the pavement's life, the modulus of rupture soon exceeds the 28-day value. Fig. 9 shows a conservative relationship between age and flexural strength as established from several laboratory series and from test specimens cured in the field or removed from pavements in service.

The number of stress repetitions at any one spot by the full design load will be very small during the first few months after paving. For this reason, it is recommended that the modulus of rupture to be used for determining pavement thickness should be the 90-day strength, or from 110 to 114 percent of the 28-day strength if 90-day test results are not available. During this period, the design safety factor will be more than adequate to prevent a fatigue failure in the pavement.

There may be special cases where conditions, such as use of heavy construction equipment on relatively thin pavement during the first few weeks after pavement placement, indicate that a strength less than the 90-day value should be used.

The following approximate relationship between flexural and compressive strength is sometimes useful in preliminary design stages; however, the final design should be based on modulus of rupture test data:

$$MR = K\sqrt{f'_c}$$

where

$MR$ = flexural strength (modulus of rupture), psi
$K$ = a constant between 8 and 10
$f'_c$ = compressive strength, psi

**FATIGUE**

Like other structural materials, concrete is subject to the effects of fatigue. A flexural fatigue failure occurs when material ruptures under continued repetitions of loads that cause flexural stress ratios of less than unity. Both pavement performance and fatigue research on concrete indicate that, as stress ratios (ratio of flexural stress to modulus of rupture) decrease, the number of stress repetitions to failure increase. Application of concrete fatigue research and experience to pavement design is discussed in Appendix A.

Fatigue effects are reflected in the design procedure in either of two ways:

1. By selection of a conservative safety factor (see page 18) based on general knowledge of the number of load applications expected during the pavement's design life. Experience has shown this to be a valid procedure for the design of most pavements when appropriate factors are chosen to reflect future increases in the volumes, weights, and channelization of aircraft to be served.
2. When specific forecasts of traffic loads and volumes have been determined, a more detailed analysis of fatigue effects can be made, as explained in Appendix A. In this procedure, the effects of mixed traffic can be analyzed either for design of a new pavement or for evaluation of the future structural capacity of an existing pavement.

**OTHER PROPERTIES**

For each project the concrete mix should be designed to give

- adequate durability
- adequate flexural strength
- durable, skid-resistant surface

Experience indicates that concrete with a modulus of rupture from 600 to 700 psi at 28 days will usually result in a pavement at the least cost, when thickness is balanced against cost of materials. In areas where freezing conditions exist, the pavement should be built of air-entrained concrete. Mix design procedures to achieve these properties are given in the Portland Cement Association publication, *Design and Control of Concrete Mixtures.*

Variations in modulus of elasticity, $E$, and Poisson's ratio, $\mu$, have only a slight effect* on thickness design. The values used in this design procedure are $E$ = 4,000,000 psi and $\mu = 0.15$.

*Following are the approximate effects of variations in $E$ and $\mu$:
A reduction in $E$ from $4\times10^6$ to $3\times10^6$ decreases stress 5 percent.
An increase in $E$ from $4\times10^6$ to $5\times10^6$ increases stress 4 percent.
An increase in $\mu$ from 0.15 to 0.20 increases stress 4 percent.
An increase in $\mu$ from 0.15 to 0.25 increases stress 8 percent.

## Subgrade-Subbase Strength

In a design analysis assumptions are made regarding the action of the subgrade or subbase-subgrade combination. Most concrete pavement designs have been based on Westergaard's modulus of subgrade reaction, $k$, determined by load tests with a 30-in.-diameter plate. This method treats the subgrade as though it had the load-carrying properties of a dense liquid. Influence charts developed by Pickett and Ray are an extension of the Westergaard analysis and were developed for both the dense-liquid subgrade assumption and the elastic-solid subgrade assumption. The former has been used most frequently for pavement design.

The dense-liquid subgrade assumption results in computed stresses that are somewhat higher than measured stresses.* These differences are not great in most cases. The computed stress is on the conservative side and is suggested for design purposes. (When the elastic-solid assumption is used, theoretical stresses closer to measured values are obtained, provided the proper value for the subgrade modulus is used. However, there are practical difficulties in determining the elastic constants from soil test data or plate-load tests. At present, there is much experimental and analytical work in progress on these and other subgrade theories. The results of the studies may be applied in design when sufficient data are available to establish confidence by correlation with pavement performance experience.)

The $k$ value is determined by a plate-bearing test on the subgrade and on the subbase if one is used. Although the plate-load tests are preferable, the $k$ value is sometimes estimated by correlation to laboratory soil strength tests or to soil type for small projects where time and equipment for plate-loading tests are not available. An approximate correlation for this purpose is shown in Fig. 10.

If a granular or stabilized subbase is used under the pavement, there will be an increase in the $k$ value. Whenever feasible, a test section of subbase should be constructed and plate-bearing tests made. If this is not practical, an estimate of the $k$ value can be determined from Fig. 11 or Fig. 12.

Westergaard's analysis represents a two-layer system comprised of a concrete slab on a supporting foundation with reaction modulus $k$. If a subbase and possibly other layers are used, a stricter analytical description would be a three-layer or multilayer system. However, when $k$ is determined at the surface of the foundation layer(s), experience has shown that reasonable approximations of stresses and deflections are obtained by use of this $k$ value in the Westergaard analysis. This is true as long as the size of the loaded area is limited to that for single-wheel gear, dual-wheel gear, and closely spaced dual-tandem wheel gear. For heavy aircraft emerging in the 1970's and for projected future aircraft, the effective loaded areas are usually larger due to use of more widely spaced dual-tandem wheel gear and multiwheeled gear with six wheels or more. For design of heavy-duty pavements that will carry these aircraft a modification in the analytical model is appropriate. Appendix B describes a procedure based on studies of several analytical methods that gives results in reasonable agreement with stresses, deflections, and subgrade pressures measured in pavement loading tests. The mechanics of the method are intended to save the designer from the complexity of the analysis and to conveniently incorporate the modification into the scheme of the conventional design procedure.

### PLATE-BEARING TEST

Plate-bearing tests should be made with a 30-in.-diameter plate on representative soils under conditions that will approximate service conditions under the pavement. By using a system of circular plates, a large calibrated jack, and a system of anchors or very heavy loads, the subgrade is subjected to known pressures at a predetermined rate of speed. The displacement of the bearing plate on the subgrade is measured by means of calibrated gages and recorded at regular intervals of load or time. Details of equipment and procedures are given in American Society for Testing and Materials Test Designations D1195 and D1196 (see footnote on this page) and in Department of the Army Technical Manual TM-5-824-3.

### EVALUATING TEST RESULTS

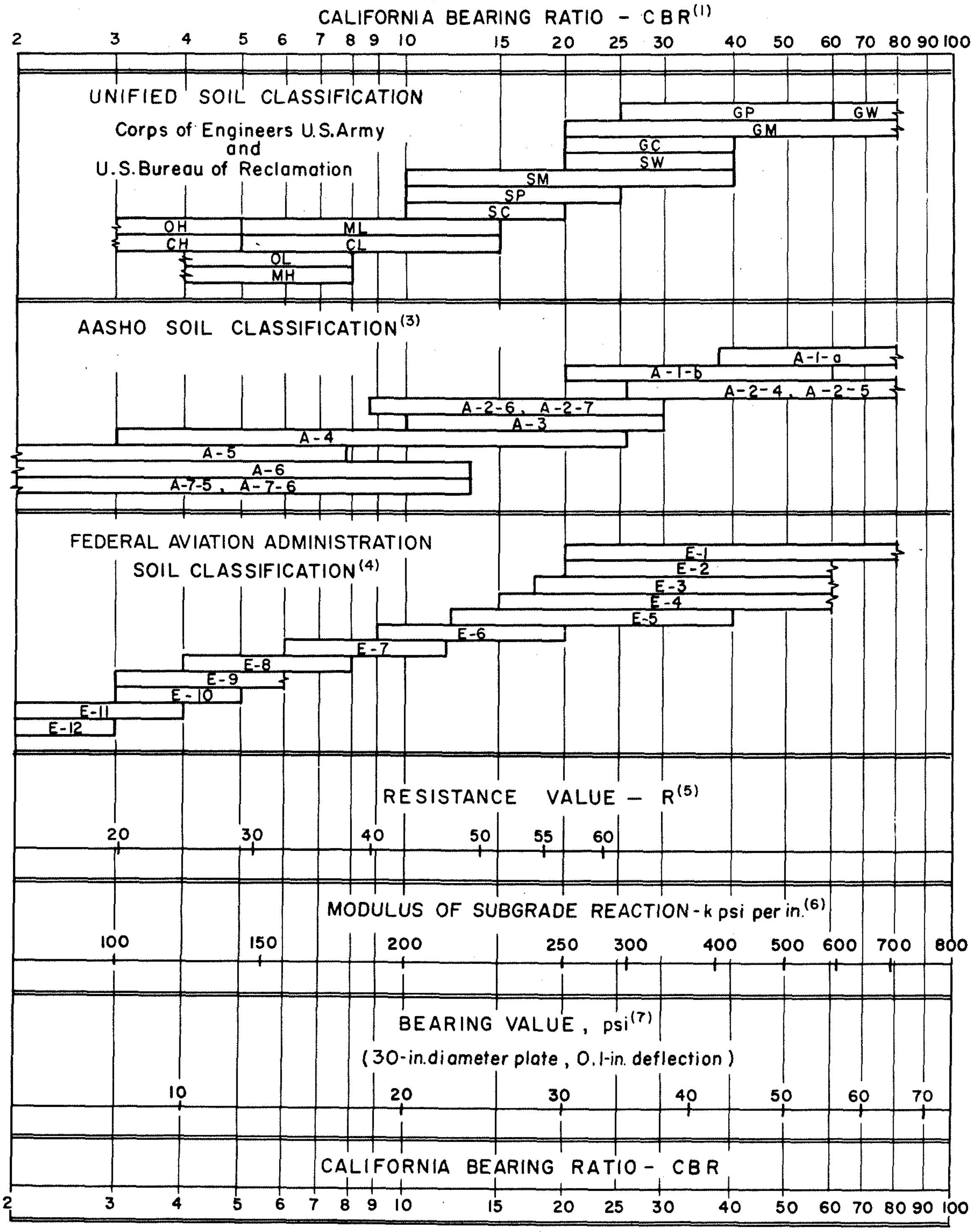
The load-deformation data obtained from plate-bearing tests can be plotted in the form of a curve. Modulus of subgrade reaction $k$ is the ratio of load in psi to displacement of the bearing plate in inches. For example, if the load-deformation curve shows that a load of 7.5 psi results in a deflection of 0.05 in., $k$ equals 7.5 divided by 0.05, or 150 lb. per cubic inch. The displacement of the bearing plate used in determining $k$ should approximate the deflection of pavement slabs under expected wheel loads. The load-deformation ratio at a displacement of 0.05 in. is generally used in determining $k$. However, the Corps of Engineers determines $k$ for the deformation obtained under a 10 psi load.

When stabilized subbases are tested, the loading equipment may not be heavy enough to obtain a deflection of 0.05 in. Even if it were, the resulting pressure on the subbase may far exceed the pressures exerted under the slab by aircraft loads and this would not represent service conditions. As a result, a maximum pressure of 10 psi is recommended for plate loading tests on stabilized subbases.

### DETERMINING $k$ BY LOADING EXISTING PAVEMENT

In evaluating the structural capacity of an existing pavement (see topic "Pavement Evaluation" in this chapter), the

*This is true when the $k_g$ value is used—gross $k$ determined from a nonrepetitive plate-load test such as ASTM D1196. In the past, most designs have been based on $k_g$ value. The elastic $k_e$ value, as determined from repetitive plate-load tests such as ASTM D1195, is a higher value, since most of the inelastic deformation is eliminated in the repetitive test. At the AASHO Road Test[9], a ratio of $k_e$ to $k_g$ of 1.77 was established for granular subbases and a clay subgrade. Use of such a $k_e$ value would reduce theoretical stresses for aircraft loads by approximately 10 percent. It is likely that the ratio is not a constant but depends on strengths of the subbase and subgrade and the relative amounts of inelastic deformations occurring during the plate-load tests. On very strong, stabilized subbases the ratio may approach unity. Although the more conservative $k_g$ value is suggested for design purposes in this manual, it is recognized that the accumulation of information on the relation of $k_e$ and $k_g$ to design and performance of pavements will be valuable.




(1) For the basic idea, see O. J. Porter, "Foundations for Flexible Pavements," Highway Research Board *Proceedings of the Twenty-second Annual Meeting*, 1942, Vol. 22, pages 100-136.
(2) "Characteristics of Soil Groups Pertaining to Roads and Airfields," Appendix B, *The Unified Soil Classification System*, U.S. Army Corps of Engineers, Technical Memorandum 3-357, 1953.
(3) "Classification of Highway Subgrade Materials," Highway Research Board *Proceedings of the Twenty-fifth Annual Meeting*, 1945, Vol. 25, pages 376-392.
(4) *Airport Paving*, U.S. Department of Commerce, Federal Aviation Agency, May 1948, pages 11-16. Estimated using values given in FAA *Design Manual for Airport Pavements*.
(5) F. N. Hveem, "A New Approach for Pavement Design," *Engineering News-Record*, Vol. 141, No. 2, July 8, 1948, pages 134-139. *R* is factor used in California Stabilometer Method of Design.
(6) See T. A. Middlebrooks and G. E. Bertram, "Soil Tests for Design of Runway Pavements," Highway Research Board *Proceedings of the Twenty-second Annual Meeting*, 1942, Vol. 22, page 152. *k* is factor used in Westergaard's analysis for design of concrete pavement.
(7) See item (6), page 184.

**Fig. 10. Approximate interrelationships of soil classifications and bearing values.**

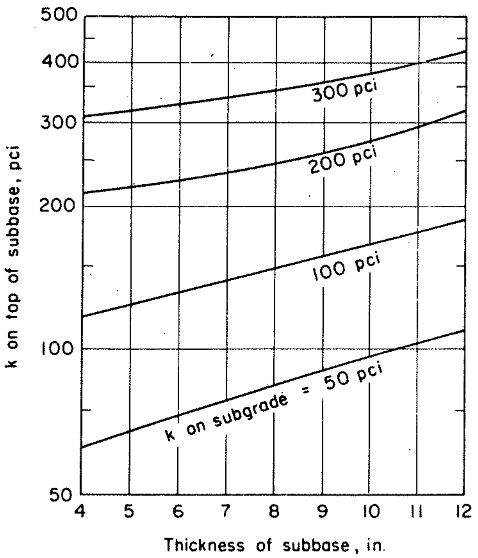



**Fig. 11. Effect of granular subbase thickness on *k* value.**

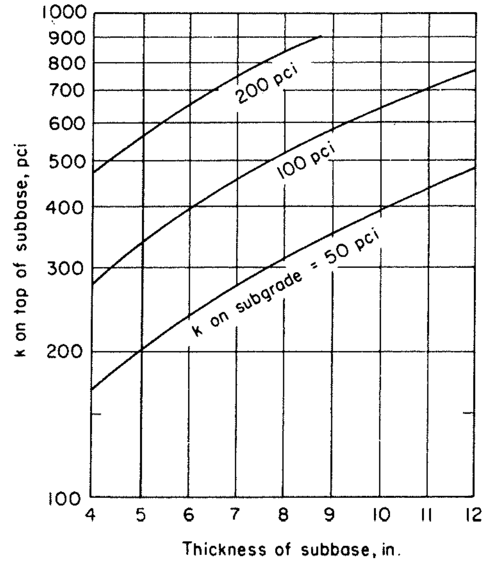



**Fig. 12. Effect of cement-treated subbase thickness on *k* value.**

*k* value can be determined by load tests on the concrete slabs. In this case the concrete pavement will distribute the load over a greater area than the area of a 30-in. plate. Therefore, a jack of greater capacity and greater loads will be required.

Both the area of slab deflected and the deflection at a sufficient number of points must be known to determine the volume of displacement under various loads. Several gages are required for this purpose. The modulus *k* is the ratio of total load in pounds to total volume of displacement in cubic inches.

Additional data concerning load tests on existing slabs are given in references 16, 17 (pages 107-108), 18, and 19.

## Load Stresses

Flexural stresses caused by aircraft loads are conveniently determined with special design charts for specific aircraft such as those shown in Figs. 13, 14, and 15. The charts, available from PCA, are prepared by computer analysis[20] or by the use of influence charts[21] for a uniform-thickness slab supported by a dense-liquid subgrade.

A brief description of the computer program and the influence charts is given in Appendixes C and D for use in analyzing special problems—for example, computing load stresses for future aircraft and other vehicles for which stress charts are not available.

The flexural stresses used in the design procedure are those at the interior of a slab, assuming that the load is applied at some distance from any free edge of a pavement slab. When the slab edges at all joints (longitudinal and transverse) are provided with adequate load transfer,* it has been found that a paved area acts as a continuous large slab. This is substantiated by the performance of existing airport pavements and by observations made on full-scale experimental slabs.

At free edges, load stresses are somewhat greater than those for the interior load condition. Because of this, the slab thickness at undoweled, butt joints—expansion joints at intersections of runways and taxiways—is increased* to compensate for the absence of load transfer and thus keep load stresses at these slab edges within safe limits. This is also true for fillets, where it is expected that aircraft may cut corners closely with a resulting channelization of traffic near the edge of the irregularly shaped slabs.

The outside edges of runways, taxiways, or aprons do not require thickening since aircraft wheels rarely, if ever, travel close to the outside edges. Where future expansion of the pavement is anticipated some means of load transfer is built into the slab edges or the edge is thickened* to provide for loads at these edges.

### DESIGN CHARTS

Design charts for most civil and military aircraft are available from the Portland Cement Association. They are not included in this manual because they are updated from time to time as loading information becomes available for modifications of existing aircraft and for new aircraft.

Careful attention to detail is required for correct use of the charts. The specific chart for data on gear load, wheel spacings, and tire contact area must be selected for the aircraft in question. Load stress is based on gear load rather than on gross weight of aircraft. Gear load at maximum aft center of gravity is usually available from the aircraft manufacturer's data. An example of data for DC-8 aircraft is

*Provisions for adequate load transfer by dowels, keyways, and aggregate interlock are discussed in Chapter 3.

*Dimensions for thickened-edge pavements are the same as shown in Fig. 20 (Chapter 3) for expansion joints.

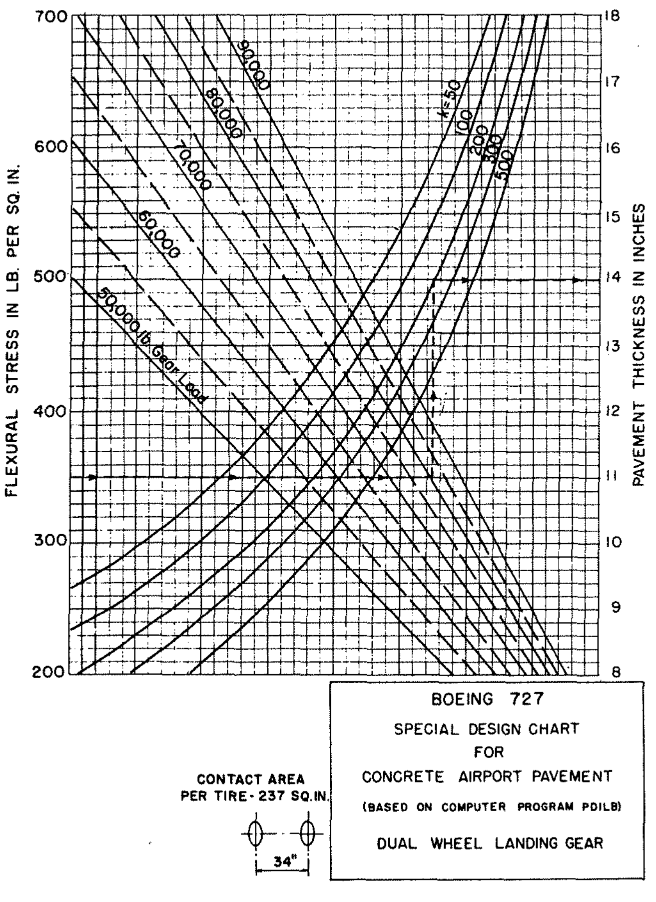



**Fig. 13. Design chart for Boeing 727.**

shown in Table 1. For most aircraft, the gear load can be estimated from the gross aircraft weight with the assumption that 93 to 95 percent of the weight is on the main gear.

Users of the charts will see a dashed-and-arrowed example line which represents the design loading reported by the aircraft manufacturers at the time of chart publication. Additional load lines, above and below the example, have also been included: those above represent future, heavier versions of the aircraft that may be developed; those below are for aircraft operated at less than the maximum design load, such as those flying into smaller airports. It is possible to interpolate between load lines or curves for subgrade-subbase *k* if intermediate values are used.

It should be noted that the sequence in use of variables is indicated by the dashed-arrowed example lines. Some design charts in previous publications follow a different sequence in use of the variables.

## Safety Factor

The safety factor (ratio of design modulus of rupture to working stress) used for airport pavement design depends on the expected frequency of traffic operations and their channelization on runways, taxiways, and aprons. In past pavement design experience, safety factors often have been selected that did not allow for the higher magnitudes of aircraft loads and the more frequent load applications to which the pavement was later subjected.

Estimating future traffic is undoubtedly one of the most important factors in airport pavement design. Data on expected future operating and load conditions can be gathered from several sources, including commercial airline forecasts, airport operating officials, and the projections of aircraft manufacturers.

Based on this information, an adequate safety factor can be selected and used to determine the allowable working stress in the design charts. (When a specific forecast is made of the mixed aircraft that will operate during the design life of the pavement, the fatigue methods outlined in Appendix A can be used for a more detailed assessment of traffic effects.) The following ranges of safety factors are recommended:

| Installation | Safety Factor |
|---|---|
| Critical areas: | |
| Aprons, taxiways, hard standings, runway ends for a distance of 1,000 ft., and hangar floors | 1.7 - 2.0 |
| Noncritical areas: | |
| Runways (central portion) and some high-speed exit taxiways | 1.4 - 1.7 |

The lower safety factors for the central portion of runways are permissible because most runway traffic consists of fast-moving loads that are partly airborne. In addition, the aircraft wheel loads are distributed transversely over a wide pavement area so that the number of stress repetitions on any one spot is quite small—much lower than on a taxiway, even on a one-runway airfield.

Where taxiways intersect runways, the runway for a short distance each way should be of the same thickness as the taxiway. Any portions of runways that will serve as taxiways should also be the same thickness as taxiways.

On airfields with a large number of operations by planes with critical loads, safety factors near the top of the suggested range should be used. On fields with only occasional operations by planes with critical loads, safety factors near the bottom of the range should be used. Those fields with a few daily operations of critical loads should use an intermediate value. Even though there may be a large number of operations by lighter aircraft, the fatigue resistance of the concrete will not be used up. A safety factor of 2.0 results in pavement adequate for full-capacity traffic operations.

For heavy-duty runways serving large volumes of traffic, designers sometimes select a keel-section design where the center section of the pavement is thicker than the outside pavement edges. Safety factors for this design are discussed in this chapter under "Keel-Section Design for Runways."

## Design Procedure

Determination of slab thickness is made in the following steps:

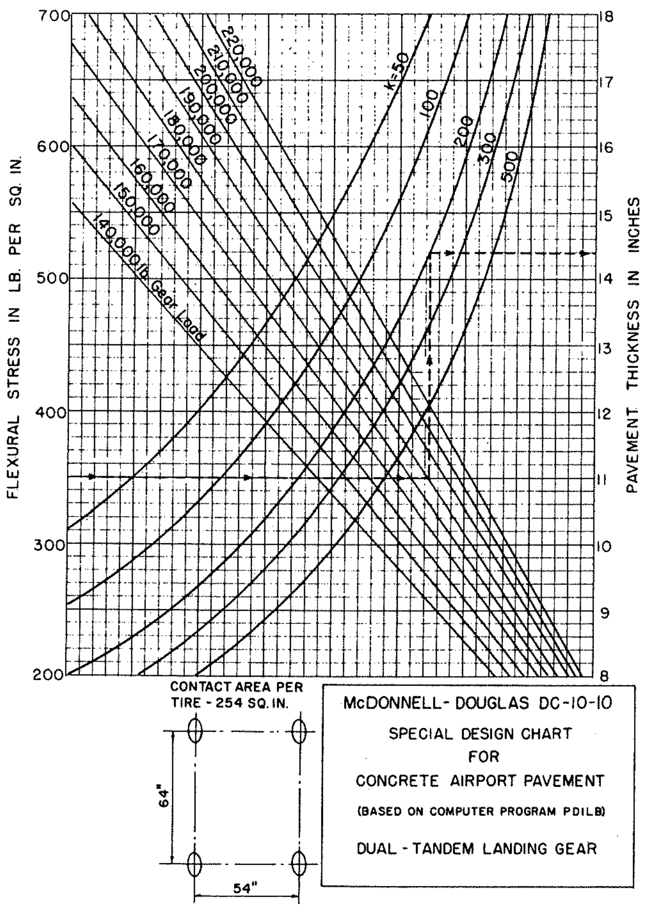



**Fig. 14. Design chart for McDonnell-Douglas DC-10-10.**

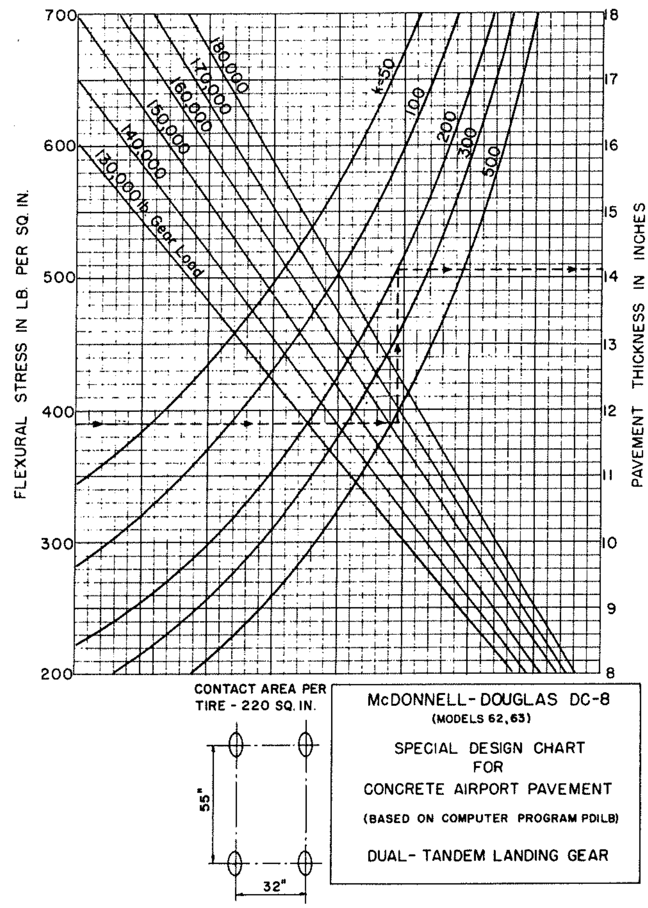



**Fig. 15. Design chart for McDonnell-Douglas DC-8 (62, 63).**

1. The $k$ value is determined by plate-loading tests or by correlation to subgrade soil test data.
2. A careful estimate of future, as well as present, operating and load conditions is made and an appropriate conservative safety factor is selected. (When a specific forecast of future traffic is made, the fatigue procedure discussed in Appendix A can be used.)
3. Working stress for a specific aircraft is determined by dividing the modulus of rupture of the concrete by the safety factor chosen.
4. From the design chart for the specific aircraft, determine the pavement thickness for the working stress determined in Step 3. Proceed horizontally from stress to gear load, vertically to $k$ value, then horizontally to thickness.
5. Repeat the process for other aircraft of critical loads, again selecting new, appropriate safety factors for the level of operations expected for these aircraft, and select a design thickness for the most critical condition.

### DESIGN EXAMPLE

The following is a simplified example of design procedure. Calculations are shown in Table 2.

Assume that a new runway and taxiway are to be designed to serve frequent operations of aircraft of which the B-727 and DC-10 produce the critical loading conditions. In addition, the runway is expected to carry occasional operations of DC-8-63's.

The existing subgrade is a pumpable sandy clay soil for which several plate-loading tests have indicated a $k$ value of 170 pci. As discussed in Chapter 1, a subbase is required to prevent the traffic from pumping the subgrade soil. Since a thick subbase layer is neither necessary nor economical, 6 in. is selected as an effective and practical thickness. Based on these data, a preliminary design $k$ value of approximately 200 pci is estimated from Fig. 11. (This is later verified by plate tests on subbase test sections constructed during the design stages of the project.)

Data on strength of concrete made with local aggregate indicate that it is reasonable to specify a 90-day design modulus of rupture of 700 psi. Appropriate safety factors are selected and listed in columns 4 and 7 of Table 2. Working stresses are computed (columns 5 and 8) and the required slab thicknesses (columns 6 and 9) are determined from published stress charts (Figs. 13, 14, and 15 for this example).

Based on these data, a slab thickness of 14.5 in. is selected for the taxiway and runway ends while 13.0 in. is re-

**Table 1. Design Data for DC-8 Aircraft** Manufacturer: McDonnell-Douglas

| Line No. | Designation | Type | DIMENSIONS: Span (ft.) | Length (ft.) | Tread (ft.) | Fore and aft wheel spacing (ft.) | LANDING-GEAR ARRANGEMENTS: Nose wheels | Main landing gear | Spacing of duals in inches: Nose wheels | Main gear | Fore and aft spacing for dual-tandems (in.) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | DC-8-52 | Comm'l | 142.4 | 150.7 | 20.8 | 57.5 | Dual | Dual | 18.5 | 30 | 55 |
| 2 | DC-8-53 | transport | | | | | | tandem | | | |
| 3 | DC-8-54 | | | | | | | | | | |
| 4 | DC-8-55 | | | | | | | | | | |
| 5 | DC-8-61 | | | 187.4 | | 77.5 | | | | | |
| 6 | DC-8-62 | | 148.4 | 157.5 | | 60.8 | | | | 32 | |
| 7 | DC-8-62 | | 148.4 | 157.5 | | 60.8 | | | | 30 | |
| 8 | DC-8-63 | | 148.4 | 187.4 | | 77.5 | | | | 32 | |
| 9 | | | | | | | | | | | |
| 10 | | | | | | | | | | | |

| Line No. | Gross weights, loaded: Total (lb.) | Nose gear (lb.) | Main gear strut load | Tire pressures: Nose wheel (psi) | Main gear wheels (psi) | Tire-contact areas: Nose wheel (sq.in.) | Main gear (sq.in.) | Remarks |
|---|---|---|---|---|---|---|---|---|
| 1 | 300,000 | 24,000 | 138,000 | 145 | 163 | 105 | 209 | |
| 2 | 315,000 | 28,600 | 143,200 | 159 | 174 | 105 | 209 | |
| 3 | 315,000 | 28,600 | 143,200 | 159 | 174 | 105 | 209 | |
| 4 | 325,000 | 29,800 | 147,600 | 168 | 182 | 105 | 209 | |
| 5 | 325,000 | 21,800 | 151,600 | 115 | 184 | 105 | 209 | |
| 6 | 350,000 | 27,400 | 161,300 | 165 | 199 | 105 | 220 | |
| 7 | 335,000 | 26,200 | 154,400 | 155 | 188 | 105 | 209 | |
| 8 | 350,000 | 25,400 | 162,300 | 140 | 201 | 105 | 220 | |
| 9 | | | | | | | | |
| 10 | | | | | | | | |

**Table 2. Example Calculations for Thickness Design** Design *k* value: 200 psi Design *MR* = 700 psi

| Aircraft | Gear load, lb. | Operations | Pavement facility — Taxiway and runway ends: Safety factor | Working stress, psi ($MR \div$ Col. 4) | Slab thickness, in. | Runway, central portion: Safety factor | Working stress, psi ($MR \div$ Col. 7) | Slab thickness, in. |
|---|---|---|---|---|---|---|---|---|
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) |
| B-727 | 80,000 | Frequent | 2.0 | 350 | 14.0 | 1.7 | 412 | 12.5 |
| DC-10 | 190,000 | Frequent | 2.0 | 350 | 14.4 | 1.7 | 412 | 12.7 |
| DC-8-63 | 165,000 | Occasional | 1.8 | 389 | 14.1 | 1.5 | 467 | 12.4 |

quired for the central portion of the runway (thicknesses increased to next half inch).

Pavement stresses induced by other, less critical aircraft in the traffic forecast are next determined from the published design charts for these specific aircraft. (For this purpose, use of the charts is in the reverse order: start with pavement thicknesses on right-hand scale and find slab stress on left-hand scale.) If stresses for these less critical aircraft are less than 350 psi, these aircraft will not fatigue the pavements (safety factor of 2.0 or greater) and the slab thicknesses established above are adequate.

## Keel-Section Design for Runways

For major airports serving large volumes of traffic, the central portion of runways may be considered a critical traffic area for which a higher safety factor is appropriate, and consequently a greater runway thickness is required than for less busy facilities.

In these areas, a keel-section design can result in substantial savings in construction effort and cost. A keel section is a thickened pavement in the center portion of a runway tapered to thinner pavement at the outside runway edge

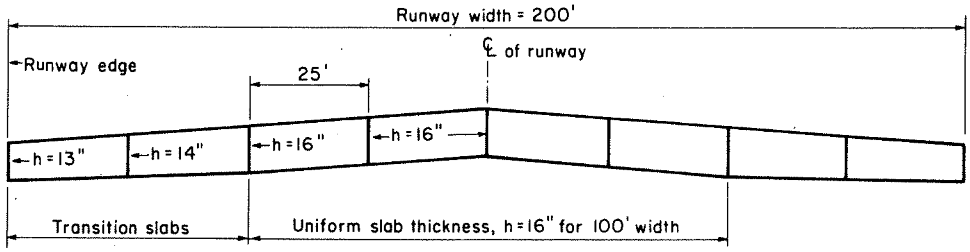



**Fig. 16. Keel-section design for runway.**

(see Fig. 16). A reduction in thickness for slabs near the outside runway edge can be justified because very few, if any, aircraft travel close to the edge, especially on the wide runways (200 ft.) often specified for major facilities.

Slab thicknesses for the runway with keel-section design can be determined with the following safety factors:

- For center portion of runway: Use a high safety factor (usually 2.0 for runways with large volume of traffic) to obtain thicker pavement down middle of runway at least 75 ft. wide. Use uniform pavement thickness for all slabs wholly or partly within this area.
- For area outside of keel section: Use an intermediate safety factor (about 1.7) to determine the lesser thickness (20 to 25 percent less than thickness of keel section) for the transition slabs between keel section and outside-edge section.
- For outside edges of runway: Use normal design procedure with lower safety factor appropriate for the infrequent number of operations to determine minimum thickness for the outer slabs of the pavement.

As shown in Fig. 16, thicknesses of the transition slabs are selected to avoid any abrupt change in grade and slab thickness and to meet minimum grade requirements on the subgrade.

Since the keel-section thickness will be greater than that required by the normal design procedure, extra strengthening at longitudinal joints is not usually required (see "Longitudinal Joints for Heavy-Duty Pavements" in Chapter 3). For example, thickness of the keel section may be about equal to thickness of the thickened-edge longitudinal joints. If so, the keel-section design is more than structurally equivalent to the alternate, strengthened-joint designs—and can be more easily constructed.

## Pavement Evaluation

It is sometimes necessary to evaluate existing airport pavements to determine what loads can be carried without overstressing the slab. The steps required are as follows:

1. Determine the *k* value for the subgrade or subbase-subgrade combination. If this information is not already available from records of previous tests, the *k* value can be determined from load tests on the existing pavement (as already explained) or by removing slab panels and making plate-bearing tests directly on the subgrade-subbase course. The results should be based on several tests, the number depending on the engineer's judgment concerning the requirements of the site.
2. Determine the modulus of rupture of the concrete by cutting beams from the pavement and making flexural tests. If this is impractical, strengths can be estimated by making a suitable age-strength increase based on strength data taken from construction records, and Fig. 9 can be used to estimate the increase in flexural strength with age. If there is any evidence of deterioration of the concrete, specimens cut from the pavement should be tested.
3. Find the thicknesses of the different installations (runways, taxiways, etc.) from construction records or from concrete cores taken with a core drill.

Use of correlative material tests is increasing for determining strengths of concrete pavement and subbase-subgrade. The tests include compressive-strength tests and tensile-splitting tests on concrete specimens; dynamic nondestructive tests on surface of the pavement; and field and laboratory soil-strength tests on subgrade and subbase materials. These data supplement and reduce the number of conventional tests required.

When the thickness and flexural strength of the concrete and the *k* value for the subgrade are known, the load-carrying capacity of the pavement can be determined from the design charts. Either an allowable load for a given safety factor or the safety factor that will be in effect under a given load can be determined.

### EVALUATION EXAMPLE

The following example shows how pavement evaluation computations are made.

Assume that an airport has 12-in. aprons, taxiways, and runway ends, and a 10-in. runway interior. The pavements are three years old and in good condition.

The problem is to determine if the airport can serve occasional operations of a DC-8-63 with a maximum take-off gross weight of 340,000 lb. (It is assumed that 5 percent of the gross weight is on the nose gear so the gear load on each of two main gears is 161,000 lb.) If the airport cannot serve these operations, what is the maximum allowable weight at which the DC-8 can operate there? Usually the landing weight of jet aircraft is no more than 75 percent of the maximum take-off weight due to fuel consumption. Thus, assume a maximum landing weight of 260,000 lb.

The subgrade-subbase reaction was determined from several load tests on the pavement. Flexural strengths were determined from tests of beams cut from various locations. Based on the test results, a *k* value of 250 pci and a modulus of rupture of 740 psi were selected as conservatively representative of the pavement strength.

Selecting a safety factor for occasional operations of 1.5 for the 10-in. portion of runway and 1.8 for the 12-in. pavements, the working stresses are 493 and 411 psi, respectively. Entering the design chart for the DC-8-63 (Fig. 15) with these working stresses and a gear load of 161,000 lb., it is found that the pavement thicknesses are inadequate for full load operations. Maximum allowable weights according to the chart are 143,000-lb. gear load (300,000-lb. gross weight) for the 10-in. runway and 147,000-lb. gear load (310,000-lb. gross weight) for the 12-in. runway ends, taxiways, and aprons.

Therefore, these pavements can safely serve occasional departures of the DC-8-63 provided the take-off weight is limited to a safe value of less than 300,000 lb. gross. Aircraft arrivals are not restricted since landing weights are less than the limits established.

## Future Pavement Design

Designers of airport pavements are faced with the need for projecting existing design procedures and experience to designs for future aircraft loadings and operational conditions.

Experience at military and civil airports shows that the performance of concrete pavements is predictable; that is, the pavements will carry the loads they are designed for and they also will have some reserve capacity to carry a limited number of heavier loads.

Current design procedures are based on the prevention of flexural failure of the concrete slab. Safe limits or working stresses have been established by considerable performance experience on both civil and military airports for a wide range of slab thicknesses and aircraft loadings—thicknesses of from 18 to 26 in. on military airfields designed for the 500,000-plus gross-weight B-52 aircraft.

This experience indicates that heavy loads and large numbers of load repetitions are conditions that have been successfully handled with present design methods. It suggests that flexural stress is a valid, critical design criterion* —that is, when flexural stresses are kept within defined limits, other possible modes of distress such as slab deflections, subgrade pressures, and shearing stresses at joints are also kept within safe, but undefined, limits.

This experience, however, is for single, dual, and dual-tandem gear aircraft, where an effective load area represented by four or less wheels affects the pavement and subgrade. For future aircraft and airport pavements, three factors increase effective size of the loaded area:

1. Increase in number of wheels so that as many as 8 or 12 wheels may be acting together to affect pavement stresses, deflections, and subgrade pressures.
2. Increase in pavement thickness—stiffer pavements spread out the zone of interaction to encompass additional wheel loads.
3. Increase in size of tire-contact areas.

As the gross weights of aircraft have increased, the number of wheels and size of tires have been increased to satisfy the allowable flexural stress in the concrete slab. With further increases in the number of wheels interacting, the question arises whether deflections and subgrade strains should be considered in addition to flexural stresses. To illustrate the problem, flexural stresses for some future heavy multiwheeled aircraft may be no greater than those for conventional aircraft, but the theoretical deflections and subgrade pressures may be two or three times greater. Thus, for heavy multiwheeled aircraft, it seems appropriate to also consider deflections and subgrade pressures and, through experience and research, establish safe working values.

As an aid to designers, deflection and subgrade pressure data will be published by PCA as loading and gear configuration information become available for heavy, multiwheeled aircraft.

*Even though the utility of the flexural design concept has been demonstrated, it is recognized that other modes of structural failure must be protected against. For example, experience has shown the definite need for a subbase under certain conditions—for silty and clayey subgrade soils, many repetitions of heavy loads will cause excessive permanent deformations or subgrade pumping if a subbase is not used. In this case, primary failure is caused by excessive subgrade displacement rather than flexural failure of the slab. Secondary failure is the result of a nonuniformly supported slab or an unsupported joint. A high degree of subgrade-subbase support provides more assurance that the pavement will behave as predicted by flexural design procedures.

## CHAPTER 3

# JOINTS AND JOINTING ARRANGEMENTS

Properly designed joints (1) control cracking due to restrained shrinkage and the combined effects of restrained warping and aircraft loads; (2) afford adequate load transfer across the joints; and (3) prevent infiltration of foreign material into the joints. Joints also divide the pavement into suitable increments for construction and accommodate slab movements at intersections with other pavements or structures.

Adequate transfer of loads across a joint must be provided to satisfy basic thickness-design principles. Depending on the type of joint, load transfer is obtained by dowels, keyways, or aggregate interlock of slabs with short joint spacings. Substantial joint support is also provided by the use of a stiff cement-treated subbase. Where no load transfer mechanism is provided, the joint edges subject to traffic are thickened to keep stresses and deflections at these free edges within safe limits.

### Longitudinal Joints

Longitudinal joints are those joints parallel to the lanes of construction. They are either construction joints (along the edges of construction lanes) or intermediate joints (sawed or insert joints) sometimes used between the construction joints.

The spacing of longitudinal joints depends on the construction equipment used, the overall width of the pavement, and the pavement thickness. In the past, equipment was best suited to paving widths of 20 to 25 ft. and this was the joint spacing commonly used for pavement thicknesses of 12 in. or more. In thinner pavements, intermediate longitudinal joints are necessary to prevent formation of irregular longitudinal cracks. The following is a guide to longitudinal joint spacing:

- For all pavements thinner than 12 in. and for pavements 12 to 15 in. thick carrying channelized traffic, longitudinal joints should not be more than 12.5 ft. apart.
- For pavements thicker than 15 in. and pavements 12 to 15 in. thick but not carrying channelized traffic, joint spacings at 12.5 ft. are not required. Convenient spacings are selected that will not exceed the contraction joint spacings suggested in this chapter under "Unreinforced Pavements."

Recent developments in paving equipment permit construction widths up to 50 ft. This allows a selection of joint spacings to satisfy specific design situations. For example, 37.5- or 50-ft.-wide construction lanes can be used with intermediate joints at 12.5, 18.75, or 25 ft., depending on pavement thickness. These intermediate longitudinal joints are also called center joints and they can be the surface-groove type, such as a saw kerf or premolded insert. Fig. 17 illustrates the common types of intermediate longitudinal joints.

Longitudinal construction joints at the edge of each construction lane can be of the keyed type (tongue and groove) to provide load transfer at that location. Details are shown in Fig. 18. The keyed joint can be extruded with a slipform paver or formed with a shaped metal strip attached to the forms to produce a groove along the edge of the slab. When adjacent slabs are placed, the new concrete will form the key portion of the joint.

Performance experience and research(22) have shown that it is important to use the keyway dimensions shown in Fig. 18 (upper right-hand corner). Larger keys reduce the strength of the joint and may result in keyway failures. The key also must be located at middepth of the slab to ensure maximum strength from the joint.

A keyway should be constructed along the outside edge of all pavement areas to provide for load transfer to future pavement expansions. A thickened outside edge can be used for the same purpose. One half of a tiebolt can be installed to permit tying the added lanes at a later date.

In narrow taxiways (75 ft. or less) all longitudinal construction and intermediate joints should be tied (held together with deformed tiebars or tiebolts) to prevent excessive opening and reduction of load transfer. In wider pavement areas, it is necessary to tie only those intermediate joints within 37.5 ft. of free edges to prevent progressive joint opening. Recommended sizes and spacings of tiebars are given in Fig. 25, Chapter 5.

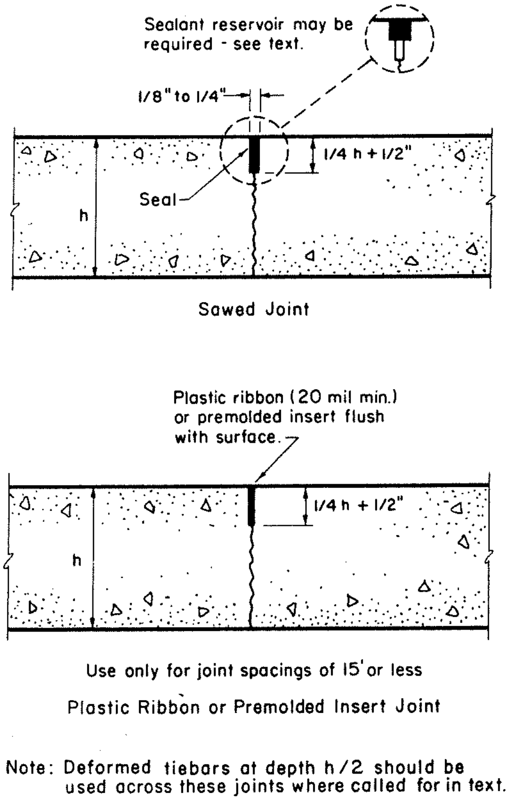
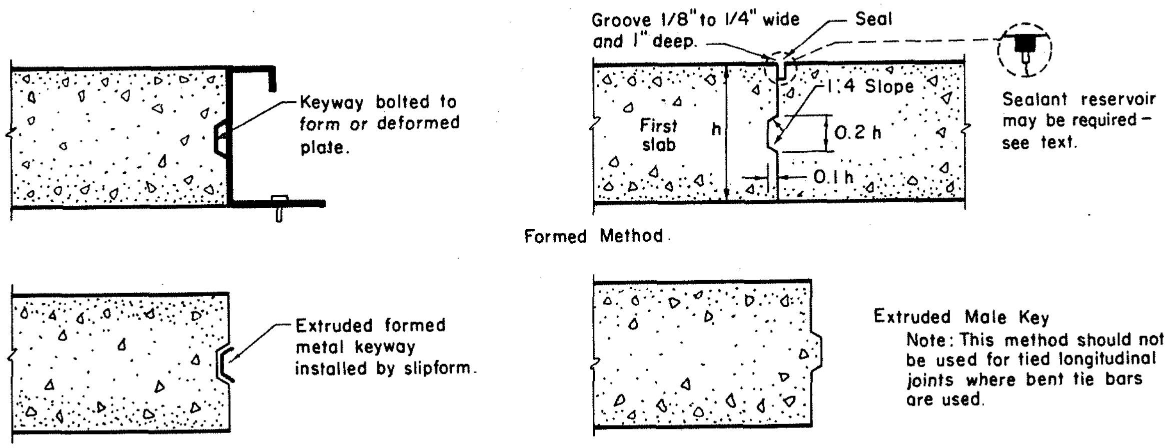


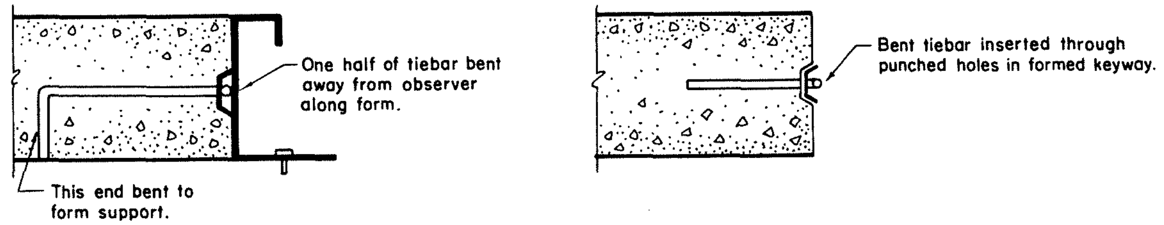

**Fig. 17. Intermediate longitudinal joints.**




Slipform Method

UNTIED KEYED JOINTS

( All dimensions, seal and sealant reservoir as shown upper right )

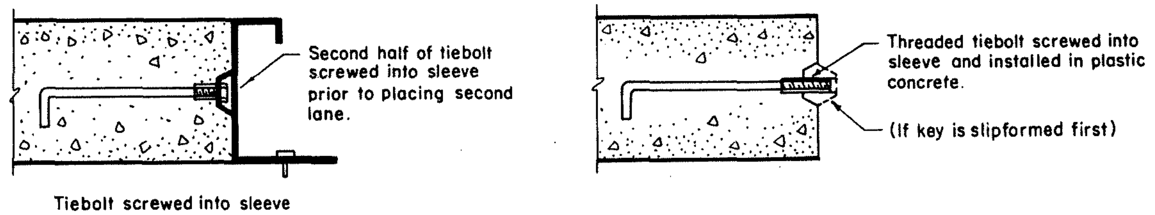



Formed Method

Slipform Method

KEYED JOINTS WITH TIEBARS

( All dimensions, seal and sealant reservoir as shown upper right )




Tiebolt screwed into sleeve attached to keyway.

Formed Method

Slipform Method

KEYED JOINTS WITH TIEBOLTS

( All dimensions, seal and sealant reservoir as shown upper right )

Notes: Tiebars or tiebolts are used only at certain locations – see text
Keyway and tiebars at depth h/2

**Fig. 18. Longitudinal construction joints.**

#### LONGITUDINAL JOINTS FOR HEAVY-DUTY PAVEMENTS

In special cases such as pavements that will carry very high volumes of heavy aircraft, the strengthening of longitudinal joints within the area of channelized traffic may be needed (for example, the 50- to 60-ft.-wide center portion of a runway). Additional joint strength can be obtained in several ways, including:

- Thickened-edge, untied keyed joints (Dimensions for thickened edges are the same as for thickened-edge expansion joints, shown in Fig. 20 in this chapter.)
- Thickened-edge butt joints
- Doweled joints, unthickened
- Conventional longitudinal joints (without thickening) supported by a stabilized subbase with $k$ value of at least 400 pci*

Runway widths of 200 ft. are now required for many heavy-duty pavements. Where conventional longitudinal joints are planned for these runways, the joints in the center portion can be tied so that they are held more tightly closed in the traffic area. Tying provides a higher degree of load transfer across the joints (whether keyed or aggregate interlock). The width of center pavement within which joints are tied depends on the expected width of traffic distribution. Based on existing information on runway traffic, a center expanse of not less than 60 ft. is suggested as the area within which all longitudinal joints are tied.

In 200-ft. runways where the joints in the center portion are tied, it is not critically important to tie the longitudinal joints outside of the center portion since loadings are infrequent there. For these heavy-duty pavements, stabilized subbases and paved shoulders are usually required, and both increase restraint to joint opening, which in turn improves load transfer at the joints. Considering also that additional joint support is provided by a stabilized subbase and that wheel loads are infrequent in the outer portion of the runway, omission of tiebars is justified in this situation.

An alternate design for attaining superior pavement and joint strength in heavy-duty runways is to thicken the pavement in the traffic area of the runway. Slab thicknesses for this keel-section type of design are discussed in Chapter 2.

Taxiway widths of 100 ft. are now often specified for airports serving large jet aircraft. Where heavy traffic is expected, it may be advisable to tie all of the longitudinal joints in these taxiways. There is, however, probably a limit to the width of pavement that can be tied together beyond which excessive shrinkage stresses will cause early pavement cracking. Experience with 75-ft. taxiways shows that this width does not exceed the limit; however, there is no performance experience for 100-ft. tied taxiways. A practical solution may be to tie the center joint in the channelized traffic area and omit the tiebars in the outside joints. As discussed for runways, since stabilized subbases and paved shoulders usually are required for heavy-duty pavements, these may provide justification for omitting tiebars in the outside joints.

*Joint support is substantially increased by use of a stabilized subbase, as discussed in Chapter 1.

### Contraction Joints

Transverse contraction joints control the formation of irregular transverse shrinkage cracks in the pavement and relieve stresses caused by restrained volume changes in the concrete.

Most contraction joints are of the surface-groove type. They may be sawed grooves or premolded inserts to control cracking and permit accurate shaping of the joint.

The ends of all airport pavements should be constructed with provision for load transfer for possible future extensions or widening.

#### UNREINFORCED PAVEMENTS

The spacing of transverse joints, and also longitudinal joints, depends on shrinkage properties of the concrete, subgrade soil conditions, climatic conditions, and slab thickness. Thus, local experience plays an important role in the selection of transverse joint spacing.

Pavement performance has shown that the allowable joint spacing for crack control increases as slab thickness increases. As a rough guide, the joint spacing (in feet) should not greatly exceed twice the slab thickness (in inches). Thus, a 15- to 20-ft. spacing is proper for an 8-in.-thick slab under average conditions, and 25- to 30-ft. spacing for a 14-in. slab. It seems likely that the relationship could be extended for thicker slabs, but this has not been firmly established by experience. Panels 25x25 ft. have given very satisfactory service for thick pavements under extremely heavy traffic conditions.

Performance has also shown that it is desirable to have panels with approximately equal transverse and longitudinal joint spacings. When slabs are long and narrow, they tend to crack under traffic into smaller slabs of nearly equal dimensions. Adequately designed slabs are not likely to develop an intermediate crack if the length-to-width ratio does not exceed 1.25.[23] Unreinforced 20-ft. by 12.5-ft. panels have given good service in some areas but this length-to-width ratio is not generally recommended unless local experience has shown it to be satisfactory.

Details of transverse contraction joints in unreinforced pavements are shown at the top of Fig. 19. Load transfer at contraction joints is provided by the aggregate interlock between the fractured faces below the groove—but only if the joints are kept quite tight by omitting expansion joints within the pavement and using a contraction joint spacing not exceeding that recommended above.

Dowels, or other load transfer devices, are not required at most transverse contraction joints with short joint spacing. Dowels, however, must be placed across contraction joints near the free ends of the pavement and near any expansion joints. Observations have shown that for a distance of about 100 ft. back from each free end and 60 ft. back from each expansion joint, the joints will gradually open to a point where the aggregate interlock may not be effective. Dowels are required in these few joints to ensure load transfer. Sizes and spacings of dowel bars are given in Table 7, Chapter 5.

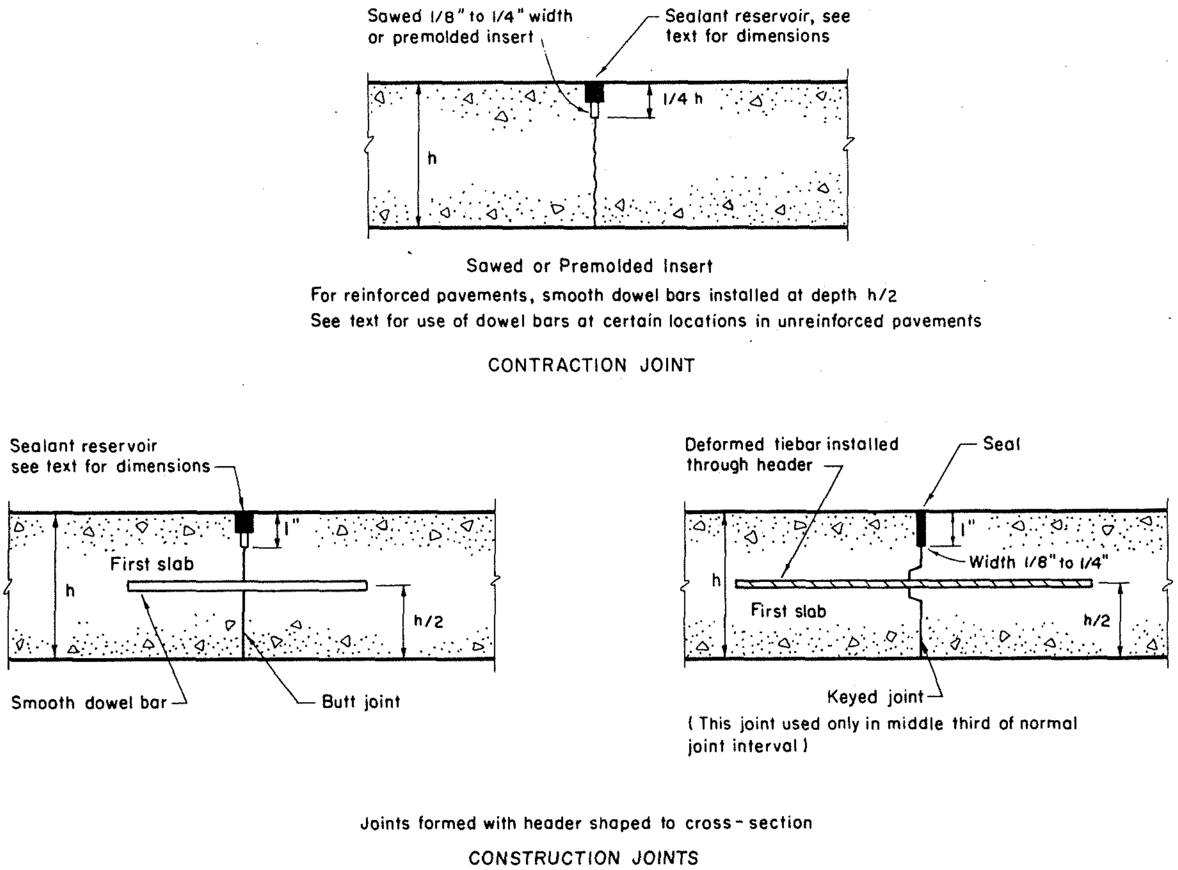



**Fig. 19. Transverse joints.**

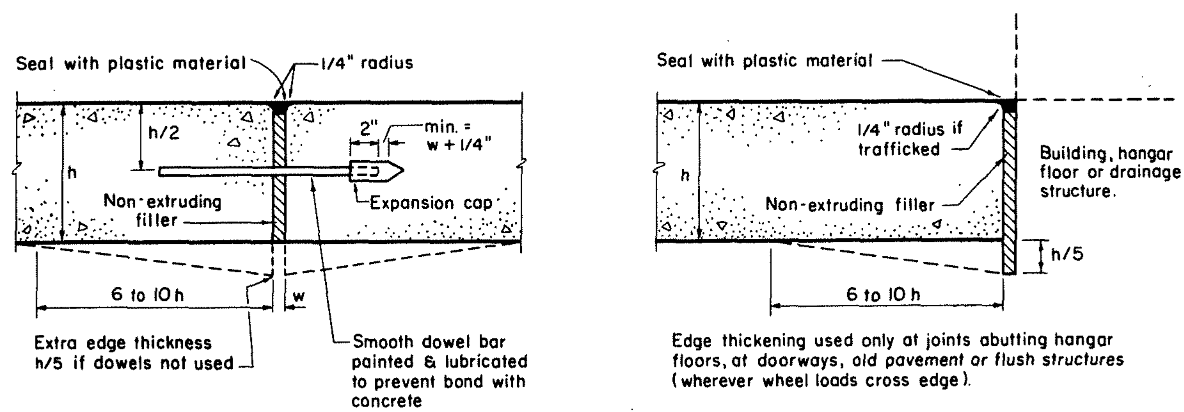
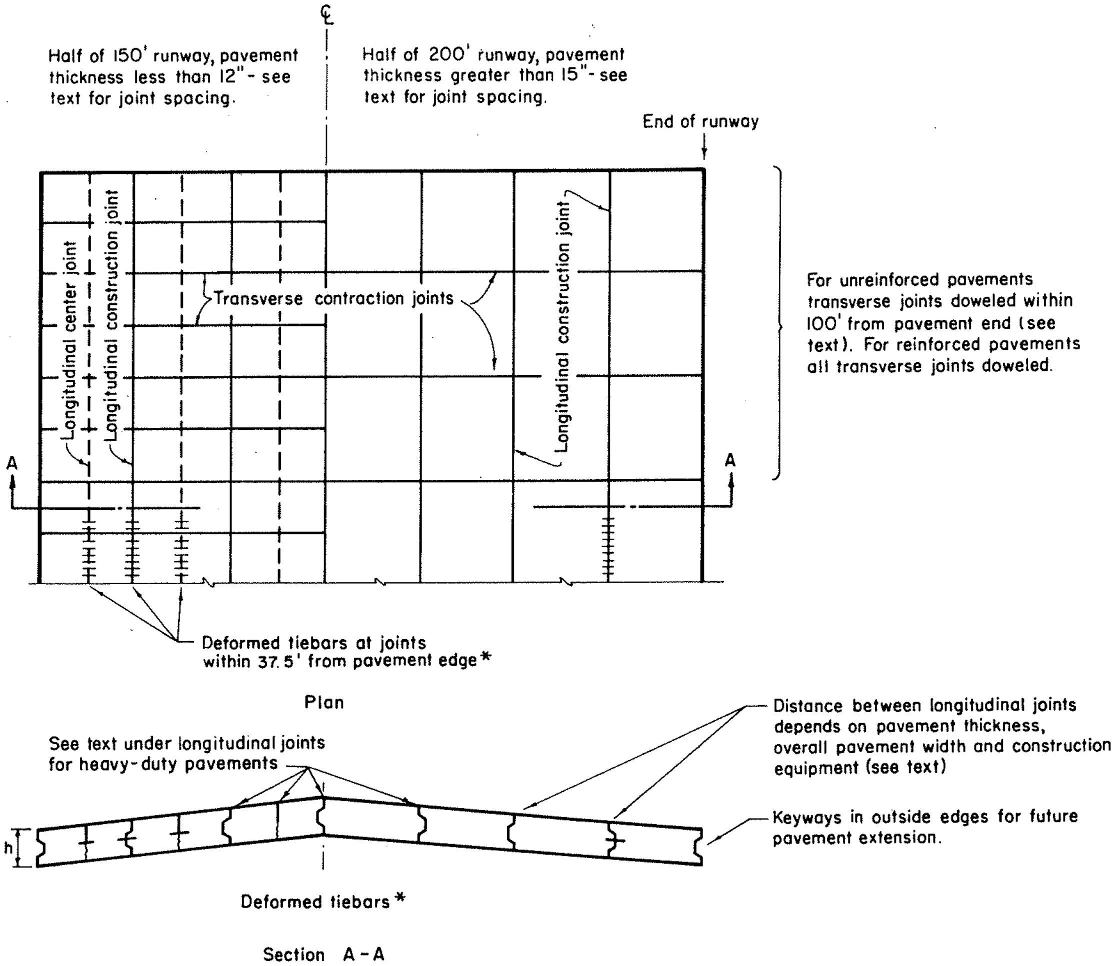



**Fig. 20. Expansion joints.**

## PAVEMENTS WITH DISTRIBUTED STEEL

Reinforced, jointed pavements (mesh-dowel pavements) have been used with joint spacings ranging from about 30 to 70 ft. For these longer joint spacings, dowels are required in all transverse joints because the joints open wider making less effective the load transfer by aggregate interlock. Design data for dowels and amount of reinforcement are in Chapter 5.

Based both on economics and performance of mesh-dowel pavements, it is desirable to limit joint spacing to about 30 or 40 ft. for airport pavements less than 12 in. thick and to about 50 ft. for thicker pavements. Importantly, the performance of all pavements depends strongly on the effectiveness of the joint seals. For longer joint spacings, joint seals may not be effective because of the greater fluctuations in joint width.




* In taxiways 75' or less in width, all longitudinal joints are provided with deformed tiebars

**Fig. 21. Jointing plan for airport pavement.**

## Transverse Construction Joints

Transverse construction joints are necessary at the end of each day's run or where paving operations are suspended for 30 minutes or more. If the construction joint occurs at or near the location of a transverse contraction joint, a butt-type joint with dowels is recommended. If the joint occurs in the middle third of the normal joint interval, a keyed joint with tiebars should be used. This is necessary to prevent an opening of the joint at this location that would cause sympathetic cracking in the adjacent lane. These construction joints should be designed in accordance with one of the details shown at the bottom of Fig. 19.

## Expansion Joints

When contraction joints are spaced as outlined above, expansion joints are not required transversely or longitudinally in airport pavements except under special conditions. The omission of expansion joints tends to hold the interior areas of the pavement in restraint and limits crack and joint opening, thus adding to the aggregate interlock effectiveness.[24]

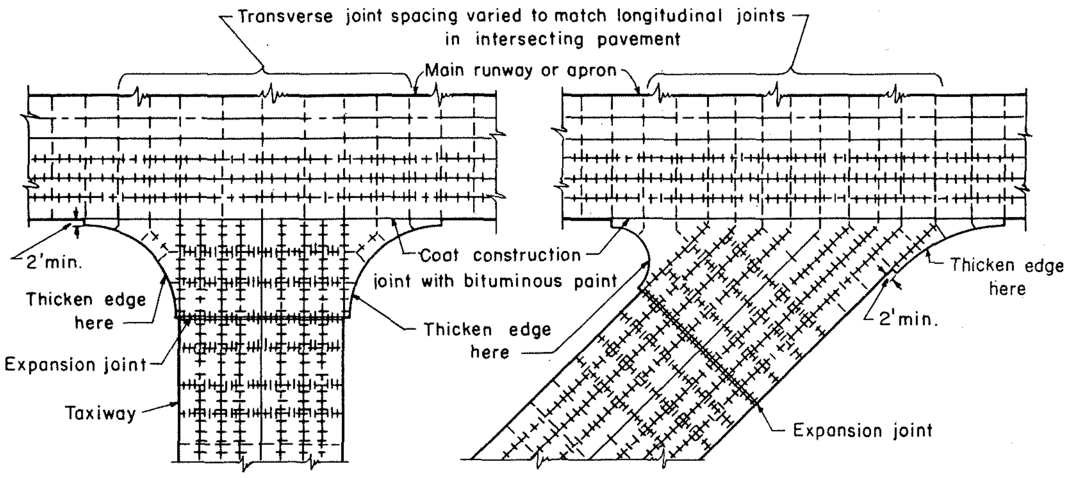
Expansion joints (¾ to 1½ in.) must be provided between concrete pavements and all buildings or other fixed airfield structures. They are sometimes required at intersections of runways, taxiways, and aprons (see "Jointing Arrangements for Runways and Intersections" following). Expansion joints used within the pavement are doweled or provided with thickened edges. Fig. 20 shows types of expansion joints to be used in the pavement and between pavements and structures.

Requirements of filler materials for expansion joints are given in current specifications of ASTM, AASHO, and federal agencies.

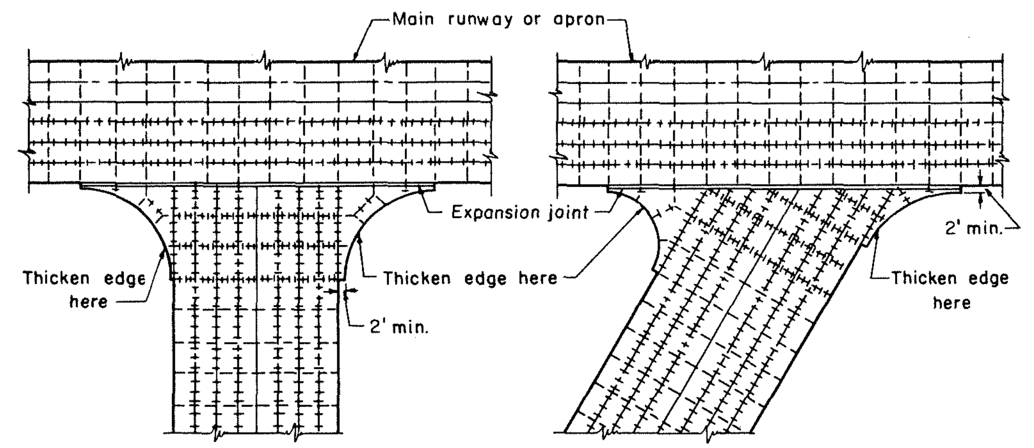
## Jointing Arrangements for Runways and Intersections

Typical jointing arrangements for concrete airport pavement are shown in Fig. 21 along with locations of various joint types and necessary tiebars and dowels.

The layout of joints at pavement intersections in airports




LAYOUTS EMPLOYING UNTIED KEYED CONSTRUCTION JOINT AT INTERSECTION




LAYOUTS EMPLOYING UNDOWELED THICKENED-EDGE EXPANSION JOINT AT INTERSECTION

Longitudinal joints tied within 37.5' of free pavement edges
Unreinforced pavements - transverse joints doweled on each side of expansion joint (reinforced pavements - all transverse joints doweled).

LEGEND

Keyed longit. construction joint.
" " " " with tiebars.
Longitudinal center joint.
" " " with tiebars.
Transverse contraction joint.
" " " with dowels.
Transverse expansion joint with dowels.
Thickened-edge " " at intersection.

Note: For conditions requiring dowels, tiebars, expansion joints and thickened edges - see text

**Fig. 22. Typical plans for jointing at intersections of runways, taxiways, and aprons.**

presents unusual problems. Because of the large, irregularly shaped areas of pavement involved and the possibility of any angle of intersection between two or more facilities, it is impossible to establish a universal joint pattern. Fig. 22 illustrates some typical intersections and shows possible jointing layouts for each.

The main body of pavement (runway or apron) will not contain any expansion joints but will be free to move at each end. The intersection of pavements (such as taxiways and runways) must be provided with an expansion joint to allow for longitudinal expansion without damage to either pavement. Since expansion joints are omitted in the main body of pavement (runways or aprons), longitudinal movement can occur along both edges. Both the expansion joint requirements and the free longitudinal movement can be provided by one of the following methods:

1. An untied, keyed construction joint with bituminous-coated edges to prevent bond (Fig. 22, top) can be used between the intersecting pavements. A doweled expansion joint is placed at the end of the fillet in the intersecting taxiway or runway perpendicular to the longitudinal centerline of the pavement in which it is located. Contraction joints on each side of the expansion joint should be doweled to provide load transfer. When a keyed joint is used between intersecting pavements, every effort should be made to match location of the longitudinal construction joint in the intersecting pavement with location of the transverse contraction joint in the main pavement. This usually can be done by changing slightly the spacing of the transverse contraction joints.
2. An undoweled, thickened-edge expansion joint (Fig. 22, bottom) can be provided between intersecting pavements. The abutting edges of both pavements are thickened 20 percent at the joint. With this arrangement, joint locations do not need to be matched because there is little chance of cracks forming opposite joints in adjacent slabs when they are separated by an expansion-joint filler. Contraction joints adjacent to the expansion joint (within 60 ft.) must be doweled to ensure load transfer.

Regardless of the method of jointing used, formation of small acute angles, less than about 60 deg., should be avoided when practical at all outside edges of fillets and curves. This can be done by bending the longitudinal and transverse joints to meet the pavement edge at right angles.

## Joint Shapes

Joint sealants are used in all joints* to keep out damaging material. They must be capable of withstanding repeated extension and compression as the pavement expands and contracts with temperature and moisture changes.

In order to maintain an effective seal, the joint width must be made large enough so that subsequent joint movement will not put undue strain on the sealant. This means that some joints must be sawed wider at the top to form a reservoir for the sealant material, as shown in Fig. 23. Sealant reservoirs are required except where joint spacings are short or where joints are tied.

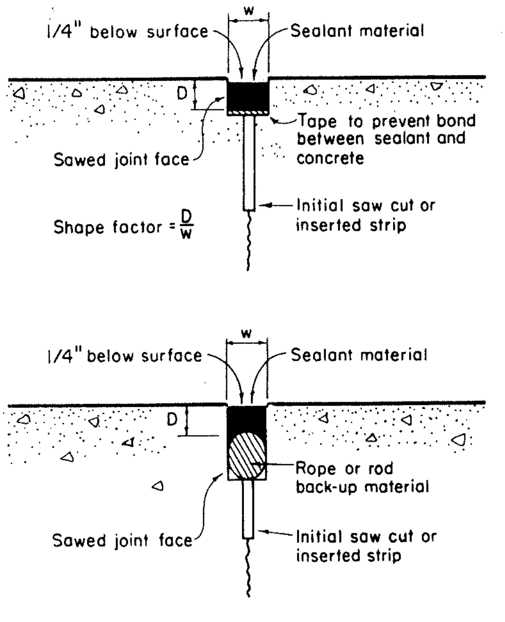



**Fig. 23. Joint sealant reservoir and shape factor.**

For poured joint sealants, the shape of the sealant reservoir has a critical effect on the sealant's capacity to withstand extension and compression. The lower the depth-to-width ratio or shape factor, the lower the strain on the sealant under a given joint movement. The required shape factor will depend on the properties of the sealant and the amount of joint movement. Joint movement is related to the joint spacing and to the maximum seasonal temperature change in the slab.*

Table 3 lists recommended depths and widths for reservoirs in transverse joints. These are for poured sealants, such as those meeting current federal and ASTM specifications. Within practical limitations of minimum joint depth, a joint shape as nearly square as possible is desired. Thus, if the joints are sawed deeper than indicated in Table 3, they should also be sawed wider to maintain or decrease the shape factor.

A stiff, self-adhering plastic strip is applied to the bottom of the sealant space to break the bond between sealant and bottom concrete surface. Frequently, a butyl of polypropylene rope is used instead in the bottom of the sealant space to break bond and prevent loss of sealant into the crack below the joint filler. When the rope is used, the reservoir must be deeper—an extra amount equal to the

*Basic information on the technology of joint sealing and sealants is given in references 25 through 29.

*Joint movement (inches) can be approximated by the expression $5 \times 10^{-6} \times T \times L$, where $T$ is seasonal temperature change (degrees F) and $L$ is slab length (inches)—see references 27 and 28.

rope diameter—so that the shapes listed in Table 3 are maintained.

For preformed, compression-type seals, recommended joint widths and seal widths are listed in Table 4. The saw cut is deep enough so that the compression seal is installed about 1/8 in. below the pavement surface. With special equipment, the seals can be installed without stretching to a compressed width of about half of their uncompressed width.

For untied longitudinal joints, the shapes listed in Tables 3 and 4 will improve sealant performance. Untied longitudinal joints must take up the movement of any adjacent tied joints, and this must be taken into consideration when using Tables 3 and 4.

**Table 3. Joint Width and Depth for Poured Sealants**

| Joint spacing, ft. | Sealant reservoir shape | |
|---|---|---|
| | Width, in. | Depth, in. |
| 20 | 1/4 | 1/2 minimum |
| 25 | 3/8 | 1/2 minimum |
| 30 | 3/8 | 1/2 minimum |
| 40 | 1/2 | 1/2 minimum |
| 50 | 5/8 | 5/8 |
| 60 | 3/4 | 3/4 |

**Table 4. Joint Width and Seal Width for Preformed Compression Seals**

| Joint spacing, ft. | Joint width, in. | Seal width, in. |
|---|---|---|
| 25 or less | 1/4 | 9/16 |
| 30 | 3/8 | 13/16 |
| 50 | 1/2 | 1 |
| 70 | 3/4 | 1-1/2 |

Adapted from Joint AASHO-ARBA Task Force 6, 1965.

## CHAPTER 4

### CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

A continuously reinforced concrete pavement is one with no transverse joints except where the pavement intersects or abuts existing pavements or structures. Due to the relatively heavy, continuous steel reinforcement in the longitudinal direction, the pavement develops transverse cracks at close intervals, averaging between 3 and 7 ft.

The design for this type of pavement must (1) provide adequate pavement thickness for the aircraft loads, and (2) provide enough longitudinal reinforcing steel so that transverse cracks are kept tightly closed and occur at the desired spacing.

Detailed information on design and construction of continuously reinforced pavements is given in references 30, 31, and 32.

#### Pavement Thickness

Recognizing a superior load transfer situation at transverse cracks compared to jointed pavements, some design specifications for highway pavements will allow a reduced thickness for continuously reinforced pavements. It is considered that any significant thickness reduction may be unconservative because of the resulting reduction in load transfer at the longitudinal joints. In addition, the increased deflections of a thinner pavement may cause excessive crack spalling, especially for pavements carrying multiwheel, heavy-gear aircraft.

Therefore, it is recommended that a thickness reduction allowance not be taken for continuously reinforced pavements and that thicknesses be determined by the methods described in Chapter 2.

## Longitudinal Steel

### AMOUNT

The amount of reinforcing steel required to control volume changes is dependent primarily on thickness of the slab, tensile strength of the concrete, and yield strength of the steel. Other factors that influence the amount of steel are contraction due to temperature drop, shrinkage due to drying, and modulus of elasticity of concrete and steel.

The controlling factor is crack width. When insufficient steel is used, cracks become too wide, permitting intrusion of solids and water. Crack-width criteria have not been firmly established, but good performance has resulted when crack spacings average between 3 and 7 ft. Since crack spacing is directly related to crack width and is more readily observed, design of continuously reinforced pavement has indirectly become a matter of determining the amount of steel needed to obtain desirable crack spacings.

Several theoretical equations have been developed for computing the amount of steel required; but in general, the amount is based on empirical data obtained from experimental pavements and pavements in service.

It is usually the practice to specify the amount of steel at 0.6 percent of the gross cross-sectional area of the pavement and a minimum yield strength of 60,000 psi. In severe climates, where freezing and thawing occur, or where unusually heavy traffic prevails, a somewhat higher percentage, such as 0.7 or 0.8 percent, should be considered.

The amount of steel should not be less than that indicated by the following formula. The formula is also used to compute minimum amount of steel based on special concretes or steels that may be selected.

$$p_s = \frac{f'_t}{f_s - nf'_t} 100 \qquad (1)$$

where

$p_s$ = percentage of steel (total cross-sectional area of steel divided by gross cross-sectional area of concrete times 100)
$f'_t$ = tensile strength of concrete in psi, assumed equal to 0.4 modulus of rupture
$f_s$ = allowable working stress of steel in psi (0.75 yield strength)
$n$ = $E_s/E_c$ (ratio of elastic modulus of steel to that of concrete)

This formula does not explicitly take into account the resistance to slab movement provided by subbase or subgrade. Such resistance is expressed by a coefficient, $C_f$, and a value of 1.5 is commonly used. (See footnote on page 34.) If there is reason to believe that the coefficient differs appreciably from 1.5, the formula should be changed to:

$$p_s = \frac{f'_t}{f_s - nf'_t} (1.3 - 0.2C_f) 100 \qquad (2)$$

(These formulas and others relating the amount of steel and crack spacing to shrinkage and temperature forces are given in references 33 through 37. They are based on the concept that shrinkage will create additional cracks in the concrete rather than yield the steel reinforcement.)

Having established the required percentage of longitudinal steel, the steel area can be computed by the formula

$$A_s = \frac{bhp_s}{100} \qquad (3)$$

where

$A_s$ = total cross-sectional area of longitudinal steel, square inches
$b$ = width of slab, inches
$h$ = slab thickness, inches
$p_s$ = specified percentage of longitudinal steel

### SIZE AND SPACING

Size and spacing of longitudinal steel members are interrelated and dependent on a number of factors. Minimum size must allow enough space between the members to permit easy placement of concrete. The clear space between members must be at least twice the top size of the aggregate being used, but in no case less than 4 in.

Maximum size is governed by percentage of steel, maximum spacing permitted, bond strength, and load transfer considerations. For good load transfer and bond strength, the spacing should not exceed 9 in. The relationship of size and spacing are as follows:

$$S_w = \frac{A_b}{hp_s} 100 \qquad (4)$$

where

$S_w$ = spacing, center to center, inches
$A_b$ = cross-sectional area of one steel bar or wire, square inches
$h$ = slab thickness, inches
$p_s$ = steel area ratio, percent

To ensure adequate bond area, maximum size is usually chosen so that the bond-area ratio, $Q$, is at least 0.03 as computed by the following formula:

$$Q = \frac{4p_s}{d_b} \frac{1}{100} \qquad (5)$$

where

$Q$ = ratio of bond area to concrete volume, in.$^2$/in.$^3$
$p_s$ = steel area ratio, percent
$d_b$ = diameter of reinforcing bar, inches

This is considered only a general guide and not a firmly established criterion.

At locations where longitudinal steel is spliced, it is important that the length of splice be adequate to resist the tensile forces caused by shrinkage of the concrete at early ages. Details of the treatment of lap splices are given in references 30, 31, and 38.

### POSITION

Since the primary function of reinforcement in continuously reinforced pavements is to hold transverse cracks tightly closed, its position vertically in the slab is not extremely critical. Practice has varied somewhat. Pavements have been built with the longitudinal steel ranging from 2½ in. below the surface to middepth of the slab. Placement at middepth results in less steel stress at cracks from wheel loads and temperature drops than other placement positions. Another approach is to place the steel above middepth because this reduces the surface width of cracks. The steel, of course, must have sufficient cover to preclude development of cracks over it and to minimize steel corrosion.

To facilitate steel placement during construction and to keep surface crack openings narrow, the recommended maximum depth of steel is middepth of the slab; the minimum depth should be one-third slab depth and also provide 2½-in. cover over the longitudinal steel.

### Transverse Steel

A relatively small amount of transverse steel is commonly, but not always,* used in continuously reinforced pavements to maintain the spacing of the longitudinal bars to which it is tied or welded. In the case of preset reinforcement, the transverse steel aids in supporting the longitudinal steel above the subbase.

The subgrade drag theory used for the design of tiebars is also used to compute the required amount of transverse steel. It is based on providing a sufficient amount of steel to hold chance longitudinal cracks tightly closed, and is expressed by the formula:

$$A = \frac{bC_f wh}{12 f_s} \qquad (6)$$

where

$A$ = area of steel per foot length of slab, square inches
$b$ = half-width of slab if not tied to adjacent slab, feet**
$C_f$ = coefficient of subgrade (or subbase) resistance to slab movement, usually taken at 1.5. (See footnote on page 34.)
$w$ = weight of concrete, pounds per cubic foot (usually assumed as 150 lb. per cubic foot)
$h$ = slab thickness, inches
$f_s$ = allowable working stress in steel, psi (usually taken at 75 percent of yield strength)

In welded, deformed wire fabric, the size of the transverse wires is related to the size of the longitudinal wires because of welding considerations. At present the maximum spacing of transverse wires is limited by manufacturing practices to 16 in.

If deformed bars are used, the transverse bars need not be spaced closer than 36 in. and not more than 60 in. apart.

The minimum size of transverse deformed wire should be W4 wire (nominal diameter of 0.225 in.) and the minimum size of transverse deformed bar should be No. 3 bar (nominal diameter of 3/8 in.).

### Transverse Joints

Some aspects of the design of transverse joints for continuously reinforced pavements need to be discussed here since they apply specifically to this pavement type. The two types of transverse joints in continuously reinforced pavements are:

1. Construction joints—placed at the end of a day's work or when paving operations are temporarily stopped.
2. Expansion joints—located at intersections with other pavements and at fixed objects.

Construction joints, because of their smooth interfaces, do not have as high a load-transfer capacity as natural cracks where aggregate interlock supplements the shear strength of the longitudinal steel. Therefore, it is necessary to strengthen construction joints. This is done by installing additional deformed bars of the same size as the longitudinal reinforcement. The extra bars should be at least 3 ft. long and installed at a reasonably uniform spacing across the pavement in sufficient number to increase the area of steel across the joint at least one-third.

Expansion joints for continuously reinforced pavements, required only at intersections and fixed structures, must accommodate large seasonal movements. Two types of expansion joints that have been successfully used are the wide-flange beam and the bridge-finger type. Joints should be designed to accommodate seasonal movements of 2 to 3 in., depending on climatic conditions.

Where pavement ends are anchored by lugs or piles built into the subgrade, experience has shown that only about 1 in. of end movement needs to be accommodated, permitting use of less costly expansion joints.

A less expensive terminal provision is to install several conventional expansion joints. This treatment, however, should be used only where experience has indicated that sufficient movement is accommodated to prevent pavement growth that will damage adjoining structures or pavement.

Details of the design of end anchorages and terminal provisions are given in references 30 and 31.

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*Transverse steel may not be required where the longitudinal reinforcement is placed in fresh concrete by a method, such as tube-feed, that will ensure its proper spacing and depth.

**If several slabs are tied (for example, slabs near the edge of a runway) the amount of transverse steel for a given slab is computed, using *b* as the distance from the farthest point in a lane to the nearest untied longitudinal joint or free edge.

## CHAPTER 5

# STEEL IN JOINTED PAVEMENTS

Steel used in jointed concrete pavements can be in the form of distributed steel, such as welded-wire fabric or bar mats distributed throughout the concrete; or in the form of deformed tiebars and smooth dowels across certain joints.*

Where the pavement is jointed to form panel lengths that will control intermediate cracking, distributed steel is not necessary.

Where joints are placed to form longer panels and some intermediate cracking can be expected, distributed steel is used. In this case, dowels are used at all transverse joints to ensure adequate load transfer since larger joint openings will result.

## Distributed Steel

The function of distributed steel in jointed pavements is to hold together the fractured faces of slabs if cracks should form. The quantity of steel used can vary from 0.05 to 0.30 percent of the cross-sectional area of pavement, depending on joint spacing, slab thickness, and other factors. Structural capacity across the cracks is achieved by the interlocking action of the rough faces of the slabs, and infiltration of foreign material into the cracks is minimized.

Distributed steel does not significantly increase flexural strength when it is used in quantities that are in the range of practical economy; therefore, pavement thicknesses for reinforced pavements are the same as for unreinforced pavements.

Experience at military airfields, where traffic volumes are generally much less than those for civil airports, indicates that a reinforced pavement will remain serviceable for some time after the initial cracking stage. But a thickness reduction allowance made on the basis of this may be unconservative for civil airports, since shutdown costs and traffic delays for routine maintenance of additional cracks could be prohibitive. In addition, increased deflections on a thinner pavement may cause excessive crack spalling under the higher volumes of traffic for civil airport pavements. Recent surveys of civil airport pavements indicate generally that unreinforced slabs with short joint spacings slightly outperform reinforced slabs with longer joint spacings.

Since the steel is intended to keep cracks tightly closed, it must have sufficient strength to hold the two slabs together during contraction of the concrete. Maximum tension in the steel members across a crack is computed as equal to the force required to overcome the resistance between pavement and subgrade developed over a distance from the crack to the nearest free joint or edge. This force

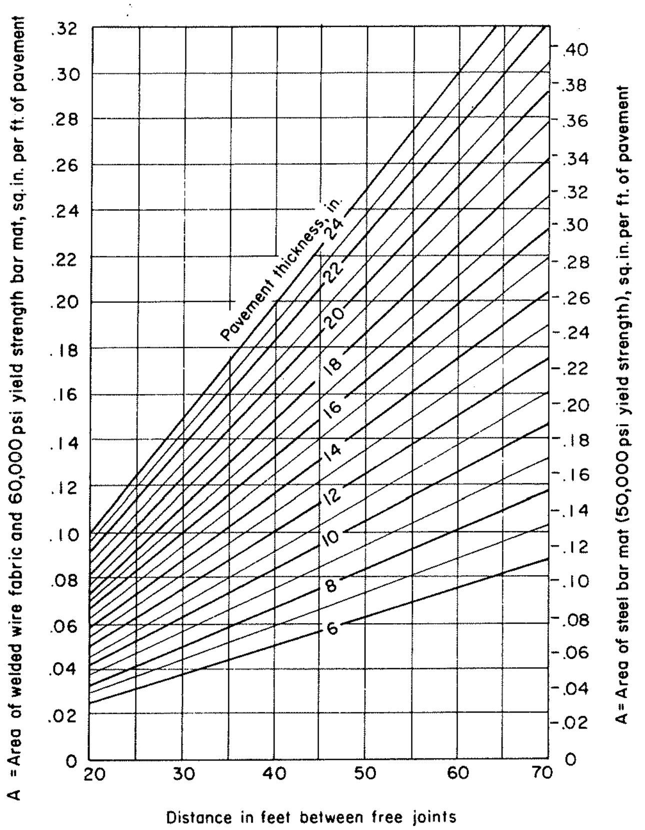


**Fig. 24. Design chart for distributed steel.**

is greatest when the crack occurs at the middle of a slab. For practical reasons, steel of the same weight is usually used throughout the length of the slabs.

Factors that must be considered in the design of distributed steel include the weight of the concrete slab, the coefficient of subgrade resistance, and the tensile strength of the steel to be used. The amount of steel required per foot width of slab is given in Fig. 24, as computed by the following formula:

$$A = \frac{LC_f wh}{24f_s} \tag{7}$$

in which

$A$ = area of steel required per foot of width of slab, square inches

$L$ = distance between free (untied) joints, feet

$C_f$ = coefficient of subgrade (or subbase) resistance to slab movement

$w$ = weight of concrete, pounds per cubic foot (150 lb. per cubic foot for normal-weight concrete)

$h$ = slab thickness, inches

$f_s$ = allowable working stress in the steel, psi

When this formula is used to calculate longitudinal steel, $L$ will be the distance between transverse joints. For transverse steel, $L$ is the distance to the nearest free (untied)

*Joint locations requiring tiebars or dowels are discussed in Chapter 3.

longitudinal joint or pavement edge. A value for $C_f$ of 1.5 is most commonly used for design.*

The allowable working stress in the steel, $f_s$, depends on the type of steel used and it should provide a small factor of safety. However, safety factors need not be as high as those used for building and bridge design. Fig. 24 shows steel areas for working stresses of 45,000 psi (welded-wire fabric and 60,000 psi yield-strength bar mats) and 35,000 psi (50,000 psi yield-strength bar mats).

Where the spacing between free longitudinal joints is sufficiently close to control intermediate cracking, transverse steel does not have to be as heavy as that required by Formula 7, but only a sufficient amount to serve as spacers for the longitudinal steel.

#### SELECTION OF STEEL SIZE AND SPACING

If welded-wire fabric is to be used, a style can be selected from manufacturers' tables. The tables give diameter and spacing of wire in both longitudinal and transverse directions as well as weight per 100 sq.ft. and per 1 sq.yd. for each style. Table 5 gives several styles of welded-wire fabric suitable for concrete pavement.

If a bar mat is to be used, the size and spacing can be determined by using the data in Table 6. The area of steel required per foot, as determined from Formula 7 or Fig. 24, is divided by the area of the bar to obtain the number of bars required per foot. Dividing 12 by the number of bars per foot gives the maximum spacing of bars in inches.

#### INSTALLATION OF DISTRIBUTED STEEL

Since distributed steel is not intended to act in flexure, its position within the slab is not critical, except that it should be adequately protected from corrosion with a minimum concrete cover of 2 in. The steel can be placed at middepth of the slab or slightly higher—up to one-third the slab thickness below the top surface.

Plans usually call for distributed steel to be discontinued at transverse and longitudinal joints; there should be a gap of at least 6 in. between sheets to ensure that the joint can function properly.

When welded-wire fabric sheets or bar mats are lapped, the amount of splice should allow longitudinal wires to lap by a distance not less than 30 times the diameter of the wire. In some cases a 1-ft. lap is specified for both welded-wire fabric and bar mats. Transverse lap should be at least 6 in. and not less than 20 times the diameter of the transverse wire.

*The resistance coefficient, $C_f$, is sometimes referred to as the coefficient of friction between the slab and subbase (or subgrade). The situation is more complex than pure sliding friction since shearing forces in the subbase or subgrade and warped slabs may be involved in the resistance. For subgrades and granular subbases, coefficients of resistance range from 1 to 2 depending on type of material and moisture conditions. Coefficients for stabilized subbases are likely to be slightly greater. Research indicates that the coefficient also varies with respect to slab length and thickness. While these variations may be taken into account (see references 31, 37, 39, and 40), use of a coefficient other than 1.5 does not seem justified by pavement performance at this time.

### Design of Dowels

Dowels are installed across joints in concrete pavements to act as load-transfer devices that permit the joint to open and close. Their function is to distribute part of the load to the adjacent slab, thus reducing deflection and stress at the joint. Patented or proprietary load-transfer devices are available and may be used instead of dowels. Some have merit if properly installed at correct spacings. This discussion is confined to the design and installation of dowels made of smooth, round steel bar or pipe. The location of joints where dowels are used is discussed in Chapter 3.

Several methods of theoretical analysis have been proposed for the design of dowels. Most of them will result in dowel sizes and spacings that give satisfactory service. Condition surveys of existing pavements and extensive tests on full-scale slabs have shown no clear cases of dowel failure where the pavement slab itself is adequate for the loads carried.

The dowel size should be in correct proportion to the load for which the pavement is designed. Since the pavement thickness is also in proportion to the load, dowel design may be related to pavement thickness.

Table 7 lists suggested dowel sizes and spacings. It is based on highway pavement studies and airport pavement experience. Dowels are installed at middepth of the slab.

### Design of Tiebars

Tiebars or tiebolts are deformed steel bars. They are used across joints in concrete pavement where it is necessary to hold the faces of the slab in firm contact. The location of joints where tiebars are used is discussed in Chapter 3. Tiebars by themselves are not designed to act as load-transfer devices. Load transfer across a joint having tiebars or bolts is provided by aggregate interlock or a keyway.

Tiebars are designed to overcome the resistance of subgrade or subbase to horizontal slab movement when the pavement is contracting. This resistance is developed over the distance between the tied joint and the nearest free edge. The required cross-sectional area of tiebar per foot length of joint is obtained by the formula:

$$A = \frac{bC_f wh}{12f_s} \tag{8}$$

in which

$A$ = cross-sectional area of steel required per foot length of joint, square inches

$b$ = distance between joint and the nearest untied joint or free edge, feet

$C_f$ = coefficient of subgrade (or subbase) resistance to slab movement, usually taken at 1.5 (See footnote on this page.)

$w$ = weight of concrete, pounds per cubic foot (150 lb. per cubic foot for normal-weight concrete)

$h$ = slab thickness, inches

$f_s$ = allowable working stress in steel, psi (usually taken as 2/3 of the yield strength)

**Table 5. Styles of Welded-Wire Fabric for Concrete Pavement**

| Style* | Weight of fabric based on net width of 60 in. | | Spacing of wires, in. | | Steel wire, gage No.* | | Cross-sectional area, sq.in./ft. of width | |
|---|---|---|---|---|---|---|---|---|
| | lb./100 sq.ft. | lb./1 sq.yd. | Longitudinal | Transverse | Longitudinal | Transverse | Longitudinal | Transverse |
| 316-610 | 45 | 4.05 | 3 | 16 | 6 | 10 | .116 | .011 |
| 412-67 | 40 | 3.60 | 4 | 12 | 6 | 7 | .087 | .025 |
| 412-4/04 | 146 | 13.14 | 4 | 12 | 4/0 | 4 | .365 | .040 |
| 412-5/03 | 175 | 15.75 | 4 | 12 | 5/0 | 3 | .437 | .047 |
| 412-6/02 | 201 | 18.09 | 4 | 12 | 6/0 | 2 | .502 | .054 |
| 412-7/01 | 227 | 20.43 | 4 | 12 | 7/0 | 1 | .566 | .063 |
| 66-88 | 30 | 2.70 | 6 | 6 | 8 | 8 | .041 | .041 |
| 66-77 | 36 | 3.24 | 6 | 6 | 7 | 7 | .049 | .049 |
| 66-66 | 42 | 3.78 | 6 | 6 | 6 | 6 | .058 | .058 |
| 66-55 | 49 | 4.41 | 6 | 6 | 5 | 5 | .067 | .067 |
| 66-46 | 50 | 4.50 | 6 | 6 | 4 | 6 | .080 | .058 |
| 66-44 | 58 | 5.22 | 6 | 6 | 4 | 4 | .080 | .080 |
| 66-33 | 68 | 6.12 | 6 | 6 | 3 | 3 | .093 | .093 |
| 66-22 | 78 | 7.02 | 6 | 6 | 2 | 2 | .108 | .108 |
| 66-11 | 91 | 8.19 | 6 | 6 | 1 | 1 | .126 | .126 |
| 66-00 | 107 | 9.63 | 6 | 6 | 0 | 0 | .148 | .148 |
| 68-11 | 80 | 7.20 | 6 | 8 | 1 | 1 | .126 | .094 |
| 612-77 | 27 | 2.43 | 6 | 12 | 7 | 7 | .049 | .025 |
| 612-66 | 32 | 2.88 | 6 | 12 | 6 | 6 | .058 | .029 |
| 612-55 | 37 | 3.33 | 6 | 12 | 5 | 5 | .067 | .034 |
| 612-46 | 40 | 3.60 | 6 | 12 | 4 | 6 | .080 | .029 |
| 612-44 | 44 | 3.96 | 6 | 12 | 4 | 4 | .080 | .040 |
| 612-43 | 46 | 4.14 | 6 | 12 | 4 | 3 | .080 | .047 |
| 612-36 | 45 | 4.05 | 6 | 12 | 3 | 6 | .093 | .029 |
| 612-34 | 49 | 4.41 | 6 | 12 | 3 | 4 | .093 | .040 |
| 612-33 | 51 | 4.59 | 6 | 12 | 3 | 3 | .093 | .047 |
| 612-25 | 52 | 4.68 | 6 | 12 | 2 | 5 | .108 | .034 |
| 612-24 | 54 | 4.86 | 6 | 12 | 2 | 4 | .108 | .040 |
| 612-22 | 59 | 5.31 | 6 | 12 | 2 | 2 | .108 | .054 |
| 612-17 | 56 | 5.04 | 6 | 12 | 1 | 7 | .126 | .025 |
| 612-11 | 69 | 6.21 | 6 | 12 | 1 | 1 | .126 | .063 |
| 612-06 | 65 | 5.85 | 6 | 12 | 0 | 6 | .148 | .029 |
| 612-00 | 81 | 7.29 | 6 | 12 | 0 | 0 | .148 | .074 |
| 612-2/04 | 78 | 7.02 | 6 | 12 | 2/0 | 4 | .172 | .040 |
| 612-2/03 | 81 | 7.29 | 6 | 12 | 2/0 | 3 | .172 | .047 |
| 612-2/0 1/4 | 82 | 7.38 | 6 | 12 | 2/0 | 1/4 | .172 | .049 |
| 612-21/64 3 | 80 | 7.20 | 6 | 12 | 21/64 | 3 | .169 | .047 |
| 612-21/64 1/4 | 80 | 7.20 | 6 | 12 | 21/64 | 1/4 | .169 | .049 |
| 612-3/04 | 91 | 8.19 | 6 | 12 | 3/0 | 4 | .206 | .040 |
| 612-4/04 | 105 | 9.45 | 6 | 12 | 4/0 | 4 | .244 | .040 |
| 612-5/03 | 125 | 11.25 | 6 | 12 | 5/0 | 3 | .291 | .047 |
| 612-6/02 | 144 | 12.96 | 6 | 12 | 6/0 | 2 | .335 | .054 |
| 612-7/01 | 163 | 14.67 | 6 | 12 | 7/0 | 1 | .377 | .063 |
| 88-33 | 51 | 4.59 | 8 | 8 | 3 | 3 | .070 | .070 |

*A new method of designating wire size was adopted by ASTM in 1970. For example, style 612-24 above is designated as 6x12—W5.4xW4, where the W-number wire size refers to the cross-sectional area in hundredths of a square inch. For a short period of time, welded-wire fabric may be specified by using either the former steel wire gages or by the new W numbers.

**Table 6. ASTM Standard Reinforcing Bars***

| | | Nominal dimensions – round sections | | |
|---|---|---|---|---|
| Bar size designation | Weight, lb./ft. | Diameter, in. | Cross-sectional area, sq.in. | Perimeter, in. |
| #3 | .376 | .375 | .11 | 1.178 |
| #4 | .668 | .500 | .20 | 1.571 |
| #5 | 1.043 | .625 | .31 | 1.963 |
| #6 | 1.502 | .750 | .44 | 2.356 |
| #7 | 2.044 | .875 | .60 | 2.749 |
| #8 | 2.670 | 1.000 | .79 | 3.142 |
| #9 | 3.400 | 1.128 | 1.00 | 3.544 |
| #10 | 4.303 | 1.270 | 1.27 | 3.990 |
| #11 | 5.313 | 1.410 | 1.56 | 4.430 |

The three 1968 ASTM Bar Specifications are:

A615—Billet steel deformed bars
- Grade 40 - Sizes #3-#11; #14 and #18
- Grade 60 - Sizes #3-#11; #14 and #18
- Grade 75 - Sizes #11, #14 and #18

A616—Rail steel deformed bars
- Grade 50 - Sizes #3-#11
- Grade 60 - Sizes #3-#11

A617—Axle steel deformed bars
- Grade 40 - Sizes #3-#11
- Grade 60 - Sizes #3-#11

*Courtesy of Concrete Reinforcing Steel Institute.

**Table 7. Dowel Size and Spacing**

| Slab depth, in. | Dowel diameter, in. | Total dowel length,* in. | Dowel spacing, in. c to c |
|---|---|---|---|
| 5-6 | 3/4 | 16 | 12 |
| 7-8 | 1 | 18 | 12 |
| 9-11 | 1-1/4 | 18 | 12 |
| 12-16 | 1-1/2 | 20 | 15 |
| 17-20 | 1-3/4 | 22 | 18 |
| 21-25 | 2 | 24 | 18 |

*Allowance made for joint openings and minor errors in positioning of dowels.

Tiebars should be long enough so that anchorage on each side of the joint will develop the allowable working strength of the tiebar. In addition, an allowance of about 3 in. should be made for inaccurate centering of the tiebar. Expressed as a formula, this becomes

$$L_t = 1/2 \frac{f_s \times d_b}{350^*} + 3 \qquad (9)$$

where

$L_t$ = length of tiebar, inches

$f_s$ = allowable working stress in steel, psi (same as used in Formula 8)

$d_b$ = diameter of tiebar, inches

Recommended tiebar dimensions and spacings are given in Fig. 25. It is worthwhile to standardize the length and spacing of tiebars to simplify construction procedures and reduce overall pavement costs.

The tiebar dimensions shown in Fig. 25 satisfy formulas 8 and 9 when the following factors are used:

$C_f$ = 1.5, $w$ = 150 lb. per cubic foot, and $f_s$ = 25,000 psi.**

*The maximum working stress for the bond in deformed bars is generally taken as 0.10 of the compressive strength of the concrete, up to a maximum of 350 psi. It is permissible to use this maximum value in the design of tiebars because paving concrete should have a compressive strength in excess of 3,500 psi.

**A working stress of 25,000 psi is used for steels with yield strengths of 33,000 and 40,000 psi, which are normally specified if tiebars are to be bent and later straightened.

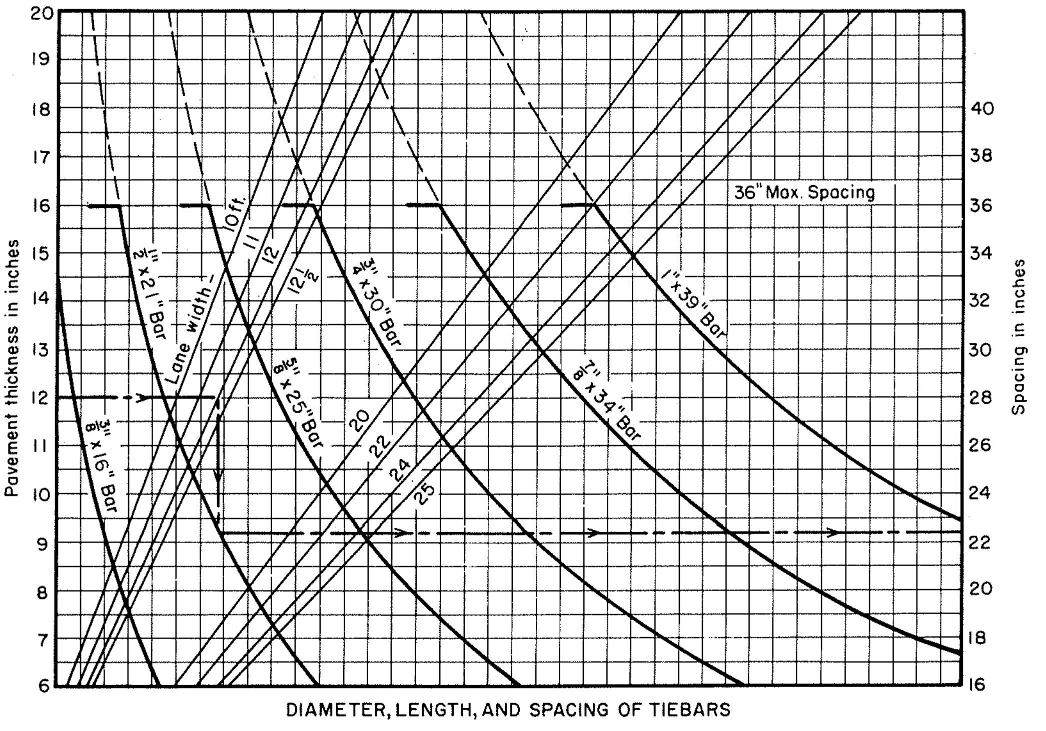


**Fig. 25. Design chart for tiebars.**

## CHAPTER 6

### CONCRETE OVERLAYS

A pavement overlay is required when the existing pavement is no longer serviceable or when the pavement must be strengthened to carry greater loads than it was designed for. In such cases it is both practical and economical to strengthen the pavement with a concrete overlay.

The design of the three basic types of overlays (partially bonded, unbonded, and bonded) for existing concrete pavements is discussed here, as well as concrete overlays for flexible pavement. General information on the design of overlays is given in references 41 through 44.

#### Flexural Strength

As explained in Chapter 2, thickness design for new airport pavement is based on 90-day flexural strength value. For overlay design, the 90-day strength value is often used to determine the thickness described in this chapter. This practice is conservative since the design of a concrete overlay should involve consideration of the flexural strengths of both overlay and base slabs. Since the base slab may be several years old, the strength gain during this period should result in flexural strengths well above the 90-day design value (see Fig. 9).

The strength values of the two layers can be considered in design by methods proposed by the U.S. Army Corps of Engineers.[42,43] For methods of determining the flexural strength of the existing slab, see "Pavement Evaluation" in Chapter 2.

## Condition of Existing Pavement

The condition of the existing pavement is an important factor in selection of the type of concrete overlay. In the overlay design charts and formulas given here, a coefficient $C$ is used to express the structural condition* of the pavement as follows:

- $C = 1.0$ when existing pavement is in good overall structural condition
- $C = 0.75$ when existing pavement has initial joint and corner cracks, but no progressive structural distress or recent cracking
- $C = 0.35$ when existing pavement is generally badly cracked or structurally shattered

Careful consideration must be given to assignment of the $C$ value with regard to the way that structural defects will influence the performance of the restructured pavement. Cracking may or may not represent a failed condition. For example, cracking due to warping stresses is not progressive and not structurally detrimental since load transfer is provided by aggregate interlock.

Cracking due to nonuniform foundation support (subgrade pumping, subbase consolidation, subgrade settlement) may not be detrimental if the condition has reached equilibrium and the slabs have cracked and settled so that uniform support is again provided.

Progressive structural defects–cracks or joints where load transfer has been lost, rocking slabs, or progressive foundation settlement–are conditions that will seriously affect performance of the overlay and they must be carefully evaluated.

Thus, the selection of type and design of overlay, and the preliminary repair work, should be based on a thorough knowledge of the pavement condition and the causes of structural defects.

## Partially Bonded Overlays

Experience both with actual pavements in service and full-scale test pavements has shown that the use of a separation course** between the existing slab and the overlay slab leads to greater deflections and more breaking in the overlay slab than where separation courses are not used. As a result, pavements with an overlay slab placed directly over the existing slab are stronger when no separation course is used. The direct overlay with no separation course is termed a partially bonded overlay.

Based on studies of overlay pavements, the U.S. Army Corps of Engineers[44] uses this formula for design of overlays placed directly on the existing pavement:

*$C$ values apply to structural condition only and should not be influenced by surface defects.

**"Separation course" as used here refers to any material between the two slabs that will break the bond (bituminous coating, plastic sheet, granular layer, or asphaltic concrete layer).

$$h_r = \sqrt[1.4]{h^{1.4} - Ch_e^{1.4}} \qquad (10)$$

where

$h_r$ = thickness of overlay, inches
$h$ = required thickness of a hypothetical single slab constructed directly on sugrade or subbase, inches
$C$ = coefficient indicating structural condition of existing pavement
$h_e$ = thickness of existing slab, inches*

The formula recognizes that friction between the two slabs or the development of some degree of bond provides somewhat greater capacity than when a separation course is used. As a result, thinner overlays are obtained than would be obtained for unbonded overlays.

The required thicknesses for concrete overlay pavements can be taken directly from the curves in Fig. 26. Where thicknesses are required for other values of $C$, they can be interpolated from these curves.

Partially bonded overlays are not used when the existing pavement is in extremely poor condition unless the structural defects can be repaired so that the $C$ value is significantly better than 0.35.

## Unbonded Overlays

A separation course between the slabs is needed if the existing slab has an irregular surface, or is in poor condition, or when the grade line is to be raised appreciably. This is called an unbonded overlay and the thickness is determined by the following formula (or Fig. 27):

$$h_r = \sqrt{h^2 - Ch_e^2} \qquad (11)$$

(Notations are the same as for Formula 10.)

Formula 11 recognizes that there is not as much interaction between the slabs in the form of friction or bond as with partially bonded overlays and accordingly results in greater thicknesses.

In some cases where the grade line is to be raised and a thick layer of material is necessary between slabs, it may be more economical to determine a new $k$ on top of the layer by plate-bearing tests and to design a new full-depth slab. This may result in a thinner slab than indicated by Formula 11, particularly if the separating layer consists of well-graded and well-compacted granular material or stabilized material.

## Bonded Overlays

A bonded overlay is a resurfacing placed on a carefully

*As explained, flexural strength of existing pavement is often substantially greater than 90-day design strength of overlay. This should be considered in the design by using, in formulas 10 and 11 and Figs. 26 and 27, a modified value of $h_e$ equal to $(h/h_{db} \times h_e)$, where $h$ is based on 90-day design flexural strength and $h_{db}$ is based on flexural strength of existing pavement. This modification is based on results[42] of pavement performance and full-scale test track studies.

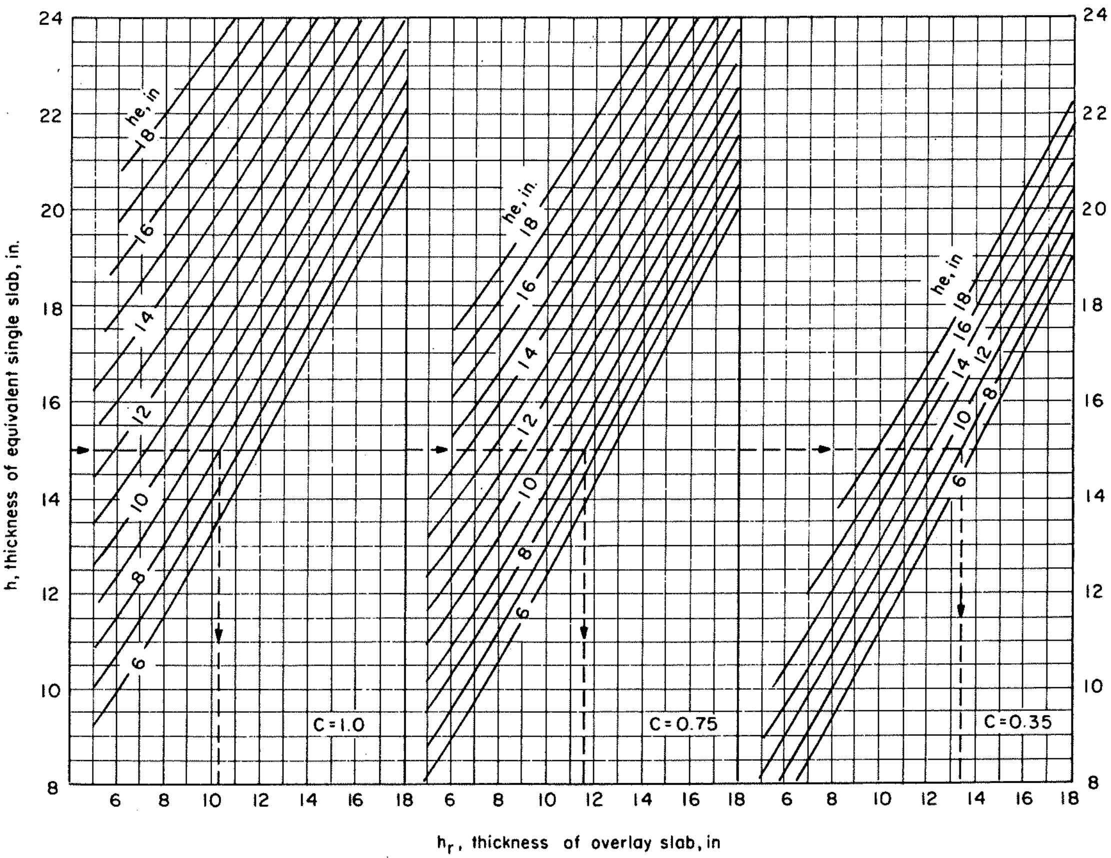


**Fig. 26. Design chart for partially bonded overlays, based on the formula:** $h_r = \sqrt[1.4]{h^{1.4} - Ch_e^{1.4}}$

cleaned pavement surface that has been prepared with a bonding agent made of a sand-cement grout or epoxy mixture.[45,46] Bonded overlays have been used on large areas of airfield pavements both to correct surface defects and to increase the structural capacity of pavements.

Design thickness for bonded overlays is based on the flexural strength of the existing pavement because the original slab and the resurfacing act together as a monolithic slab. When used on a structurally sound slab to increase the load-carrying capacity, the overlay and base slab should have a combined thickness equal to a single slab of adequate design for the planned loading. In this case,

$$h_r = h - h_e \tag{12}$$

(Notations are the same as for Formula 10.)

Of the three overlay types, the bonded overlay will be the thinnest section because of the monolithic action. The economy of the thinner overlay, however, is somewhat offset by the extra cost of surface preparation and grouting. Bonded overlays are recommended for use only where the existing slabs are in good structural condition or where structural defects have been repaired.

## Slab Replacement or Repair

For a pavement with a few localized areas of structural defects, the $C$ value can sometimes be raised appreciably by a limited program of repair, patching, or slab replacement. The increased $C$ value results in a thinner overlay for the entire pavement, which may more than offset the costs of localized repairs.

## Joint Location

Joints and random cracks in the base pavement will be reflected in bonded and partially bonded overlays unless preventative measures are taken. To prevent joint reflection, the usual practice is to match joint locations in the overlay with those in the existing slab.

Many old concrete pavements were built with expansion joints at regular intervals. In partially bonded overlays, expansion joints usually can be omitted in the overlay and contraction joints placed over the expansion joint location.

When a bonded overlay is used, joints in the overlay must match precisely the location of joints in the base pave-

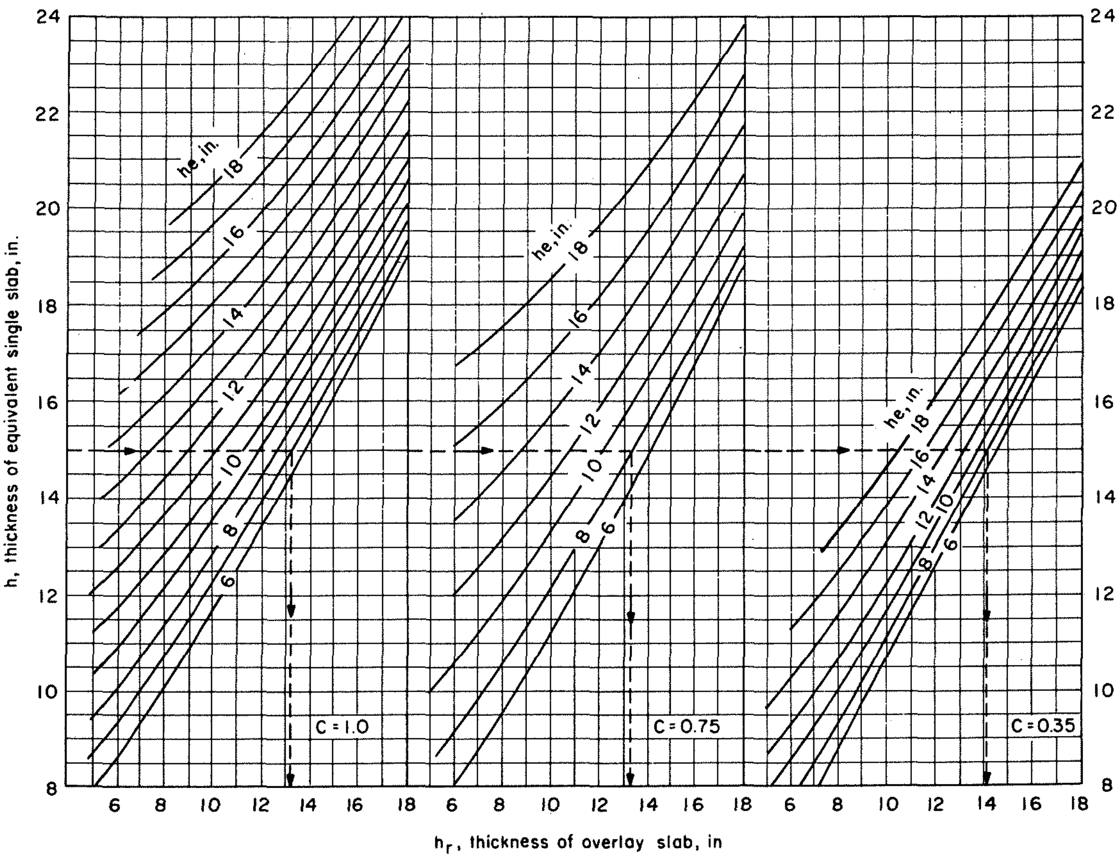


**Fig. 27. Design chart for unbonded overlays, based on the formula:** $h_r = \sqrt{h^2 - Ch_e^2}$

ment, and the same type and width of joint must be used (expansion or contraction). Where existing joints are not tightly closed, space must be provided through the full depth of bonded overlay at least equal to the width of joint opening in the old pavement.

When a separated or unbonded overlay is used and the separation course is of substantial thickness or when a continuously reinforced overlay is used, it is not necessary to match transverse joints in the overlay to those in the existing concrete, either in location or type. This is one of the advantages of the separated overlay. This type of overlay is required frequently when the joint pattern of the existing pavement is irregular or incorrect and it is not desirable to repeat it in the overlay.

In unbonded overlays with thin separation courses and in partially bonded overlays that are not continuously reinforced, contraction joints can be placed directly over existing expansion joints, contraction joints, and construction joints. If these contraction joints do not result in slab lengths short enough to control cracking, additional intermediate contraction joints should be placed to form equal slab lengths that are short enough to control cracking (see Chapter 3). Load transfer at transverse joints is provided by aggregate interlock except near the ends of pavements where dowels should be used. A plain (unreinforced) overlay with short joint spacings can be placed on a reinforced slab with long joint spacings provided that the intermediate cracks in the existing pavement are tightly closed and in good condition. In this case, dowels are used in the overlay pavement only at locations matching the existing doweled joints.

For all overlay types, longitudinal joints in the resurfacing should match the joints in the base. The longitudinal construction joints should be provided with a keyed joint* for load transfer to the adjacent slab. Tiebar requirements at longitudinal joints in concrete resurfacing are the same as for full-depth pavement.

## Tiebars, Dowels, and Distributed Steel

The use of tiebars, dowels, and reinforcing steel in overlay slabs is the same as for single-slab pavements.**

If they exist in the base slab at the proper location and are functioning adequately, tiebars, dowels, and distributed

*For thin, bonded overlays, keyways and other load-transfer devices are usually omitted.

**Locations where tiebars, dowels, and distributed steel are needed and design requirements are discussed in Chapters 3 and 5.

steel in all types of overlays are designed based on the overlay slab thickness. Otherwise, the design of tiebars and distributed steel in bonded and partially bonded overlays is based on the combined thickness of old and new slabs; and dowel design is based on the thickness of an equivalent slab ($h$ in Formulas 10 and 11).

Reinforcement serves the same purpose in concrete overlays as it does in regular pavement. It is not required when joint spacings are short, but it is required in longer joint spacing to keep cracks from opening enough to present a maintenance problem. If the old pavement is extensively cracked, use of distributed steel or continuous reinforcement may be the most dependable method of minimizing uncontrolled cracking in unbonded or partially bonded overlays. (Bonded overlays, as mentioned, are for use only where the existing slabs are in good structural condition or where structural defects have been repaired.)

### Continuously Reinforced Overlays

Because they are less susceptible to reflective cracking, continuously reinforced overlays offer an advantage over other overlays. For continuously reinforced overlays, a separation course is normally used over the existing pavement and the overlay thickness is determined as described for unbonded overlays. The amount of reinforcing steel is based on the overlay thickness requirements. Other design details are the same as for regular continuously reinforced concrete pavements.

A few partially bonded (no separation course), continuously reinforced overlays have been constructed. Partially bonded overlays should be used only if the existing pavement is in fairly good condition with short joint spacings so that joint movements will not greatly affect the overlay. A thinner overlay can be used because it is partially bonded, but additional steel may be required to prevent excessive crack opening.

### Separation Courses

Some success has been experienced in preventing reflective cracking by use of a separation course between base slab and overlay. Not sufficient data are available, however, to establish the minimum thickness needed for the separation course to be completely effective. Indications are that any type of bond breaker will reduce the amount of reflective cracking. As discussed previously, use of a bond breaker or separation course requires a thicker overlay.

### Concrete Overlays for Flexible Pavement

Concrete overlays have been used on flexible pavements for several years. They have performed well and demonstrate the feasibility of this type of construction.[47]

Design for a concrete overlay on flexible pavement is the same as design for a concrete pavement on grade. The modulus of subgrade reaction, $k$, is determined by plate-bearing tests made on the surface of the flexible pavement. Several agencies specify that no $k$ value greater than 500 lb. per cubic inch be used in designing rigid overlays for flexible pavements. The limitation, however, appears to be arbitrary and more development work is needed to fully realize the advantages of this composite design.

## APPENDIX A

### FATIGUE CONCEPTS APPLIED TO TRAFFIC ANALYSIS

The purpose of this appendix is to provide engineers with a quantitative method for evaluating the effect of repeated aircraft operations on airport pavements. It applies to the design and evaluation of pavements at airports serving large volumes of heavy, multigeared aircraft of different types. When specific data on mixed aircraft traffic are available or forecast, this procedure can be used instead of using safety factors as described in Chapter 2. Its applications are:

- design for specific volumes of mixed traffic
- evaluation of future traffic capacity of existing pavements or of an existing pavement's capacity to carry a limited number of overloads
- evaluation of the fatigue effects of future aircraft with complex gear arrangements
- more precise definition of the comparative thicknesses of runways, taxiways, and other pavement areas depending on operational characteristics

Use of this quantitative method introduces three additional design parameters:

1. Traffic widths for taxiways, runways, and ramps
2. Variability of concrete strength
3. Downgrading of service life where a good subbase support is not provided

## Coverages and Fatigue

The procedure described here was developed from a study and correlation[48] of two methods that reflect pavement design and performance experience at both civil and military airports. The first is the coverage method developed by the U.S. Army Corps of Engineers as part of their pavement design methodology for both rigid and flexible pavements.[49-53] It is based on pavement performance at military airfields and full-scale test track studies. The second is the fatigue method[54] used for highway pavement design. Based on concrete fatigue research, this method was applied early in the development of the Portland Cement Association's highway pavement design procedures. In a general way, the fatigue concept is also inherently part of airport design methodologies of the Federal Aviation Administration and the Portland Cement Association.

### COVERAGES

The effect of the lateral distribution of traffic on runways and taxiways is taken into account in the Corps of Engineers' design procedure. That procedure uses the term "pass-coverage ratio" to refer to a conversion of the number of traffic operations to the number of design load repetitions; that is, a coverage occurs when each point of the pavement within the traffic lane has been subjected to a maximum stress by the operating aircraft. The following equation* relates coverages to the number of operations (passes) for a specific aircraft:

$$C = D \times \frac{0.75Nw}{12T} \qquad \text{(A1)}$$

where

$C$ = coverages
$D$ = number of operations at full load**
$N$ = number of wheels on one main gear
$w$ = width of contact area of one tire, inches
$T$ = traffic width, feet

Fig. A1[49] shows the relationship between slab thicknesses and allowable coverages, developed by the Corps of Engineers.

The coverage curves reflect the following increases in the required pavement thickness for more than 5,000 coverages:[52]

| Coverages | Increase in pavement thickness, percent |
|---|---|
| 10,000 | 5 |
| 15,000 | 8 |
| 20,000 | 10 |
| 30,000 | 12 |

*This equation is specified by the Corps of Engineers for tricycle-geared aircraft and applies to aircraft with single, dual, or dual-tandem gear configurations.

***Corps of Engineers defines D as cycles of operation where one cycle is one landing and one takeoff. Since landing weight is usually significantly less than takeoff weight, D can be considered to be the number of takeoffs or, more generally, the number of full-load passes.*

**Traffic Width.** $T$ in Formula A1 expresses the traffic width within which the aircraft wanders—the transverse distribution of the traffic on runways and taxiways. Distribution curves are approximately bellshaped as shown in Fig. A2.[51]

Traffic width is considered as the width within which 75 percent of the main gear paths fall, and for practical purposes the Corps of Engineers assumes that the distribution is uniform within the traffic width. Traffic widths of 7.5 ft. for taxiways and 37.5 ft. for runways have been indicated by these studies.

### FATIGUE

Concrete, like other construction materials, is subject to the effects of fatigue. A fatigue failure occurs when a material ruptures under continued repetitions of loads that cause stress ratios of less than unity. Since the critical stresses in concrete are flexural, fatigue due to flexural stress is used for thickness design; and the stress ratios are the ratios of flexural stress to modulus of rupture.

Flexural fatigue research on concrete has shown that, as stress ratios decrease, the number of stress repetitions to failure increases. It has also shown:

1. When the stress ratio is not more than about 0.55, concrete will withstand virtually unlimited stress repetitions without loss in load-carrying capacity. Hence, concrete has a flexural fatigue endurance limit at a stress ratio of approximately 0.55.
2. Repetitions of loads with stress ratios below the endurance limit increase concrete's ability to carry loads with stress ratios above the endurance limit, that is, concrete's fatigue resistance is improved.
3. Rest periods also increase the flexural fatigue resistance of concrete.

For thickness design purposes, the stress ratio for the endurance limit of concrete is reduced from 0.55 to a more conservative 0.50. Allowable load repetitions are shown in Fig. A3. The values are conservative with respect to flexural fatigue research on concrete.

Accumulation of the effects of repeated loads and mixed traffic is made on the basis of the Miner hypothesis[55] with sufficient conservatism to incorporate a very low probability of failure. Kesler,[56] in a summary of the fatigue properties of concrete, concludes that reasonable results can be obtained in this way. Ballinger's work[57] corroborated the Miner hypothesis but showed less reliable performance at high stress ratios of 0.70 or more; this is normally beyond the range used for pavement design purposes.

Use of these concepts for pavement design was initiated by PCA in 1933 and modified in view of additional information in 1966.[54] As discussed in Reference 58, the design procedure making use of the cumulative damage concept and the fatigue curve has given reliable thickness designs for highways and streets. These concepts, without specific use of the cumulative damage theory, have been applied to airport design since 1950.

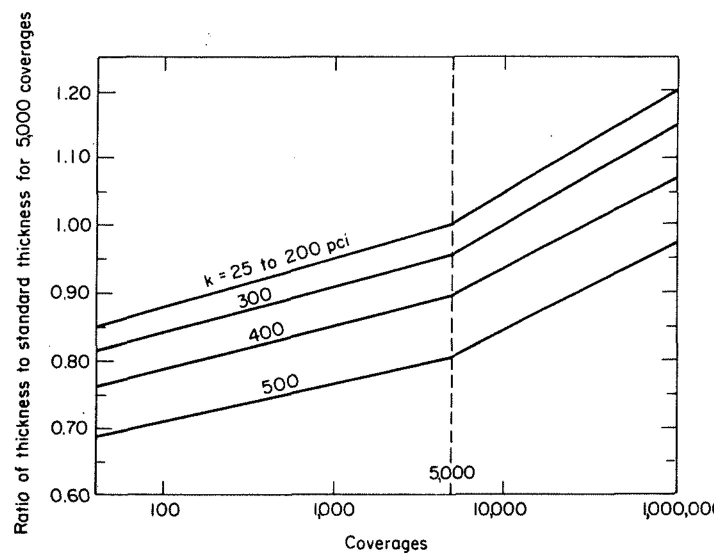


**Fig. A1. Relation between pavement thickness and allowable coverages.**

## Development of Fatigue Procedure for Airports

To compare fatigue relationships with coverage concepts, the effects of the variables (for example, traffic width and aircraft gear configuration) had to be translated in terms of fatigue effects. It was found that an excellent correlation exists when some conservatism is used in the fatigue approach by recognition of a realistic degree of variation in concrete strength. An additional adjustment in the procedure reflects the experience of the Corps of Engineers with pavements built on weak foundations.

The effects of these factors, discussed in detail in Reference 48, are described briefly in the following paragraphs.

### TRAFFIC WIDTH

In developing the correlation between fatigue and coverage procedures, the Corps of Engineers' traffic distribution curves were represented by normal distribution curves with various standard deviations. It was found that standard deviations of 24 in.* for taxiways and 16 ft. for runways fit the distribution curves, and these are suggested for design purposes. Thus, on taxiways, two-thirds of the time transverse placements of aircraft will fall within a 4-ft. width; on the central portion of runways, two-thirds will fall within a 32-ft. width. Data are provided for other standard deviations** in case the designer wishes to use different values based on results of traffic width studies currently in progress or proposed.

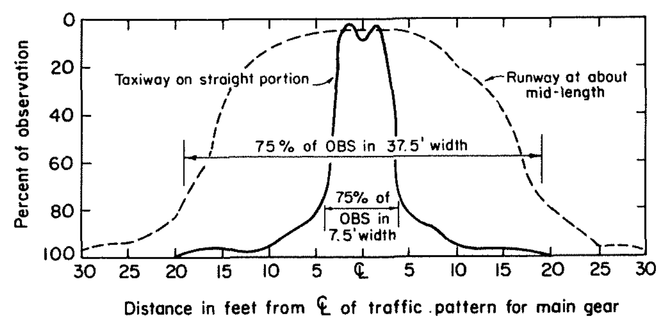


**Fig. A2. Traffic distribution patterns for dual and dual-tandem gear aircraft.**

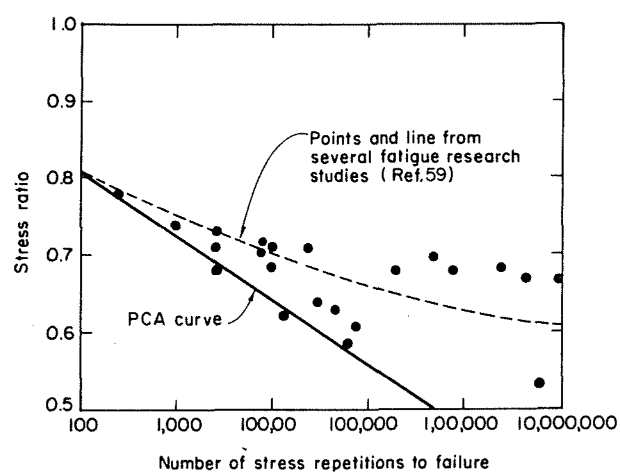


**Fig. A3. Fatigue curve for concrete subjected to flexural stresses.**

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*For taxiways, a standard deviation of 40 in. fits the distribution curve in the sense that 75 percent of the traffic falls within a traffic width of 7.5 ft. However, a standard deviation of 24 in. more closely approximates the actual shape of the curve within the traffic width.

**The relation between traffic width, $T$, as defined by the Corps of Engineers, and the standard deviation of traffic distribution, $\sigma$, is: $\sigma = (0.88)T/2$.

## LOAD REPETITION FACTOR

The load repetition factor (LRF) relates the number of aircraft passes over a given traffic width to an equivalent number of full-load stress repetitions that will give the same degree of fatigue consumption. The factor is analogous to the coverage factor used to convert aircraft operations to number of coverages.

Load repetition factors are determined from the complete stress profile and resulting fatigue consumption for various standard deviations of traffic distribution and for various $\ell$ values by a computer program that combines the PCA program for stresses,[20] the PCA fatigue curve,[59] and the normal probability curve. To save the designer the work of determining them himself, these factors are available from the Portland Cement Association. Table A1 lists factors for several aircraft. In a design problem, the expected number of full-load passes of a given aircraft is multiplied by the load repetition factor.

**Table A1. Load Repetition Factors for Several Aircraft**

| Aircraft | Load repetition factor (tentative design values) Taxiway | | Runway | |
|---|---|---|---|---|
| | $\sigma$ = 24 in. | $\sigma$ = 48 in. | $\sigma$ = 96 in. | $\sigma$ = 192 in. |
| DC-3 | 0.12 | 0.07 | 0.05 | 0.03 |
| B-727 | 0.41 | 0.23 | 0.13 | 0.09 |
| DC-8 and B-707 | 0.83 | 0.46 | 0.25 | 0.17 |
| B-747 | 0.58 | 0.38 | 0.33 | 0.28 |
| C5A | 0.74 | 0.61 | 0.37 | 0.25 |
| B-2707* | 0.52 | 0.39 | 0.22 | 0.16 |
| Concorde | 0.83 | 0.44 | 0.23 | 0.15 |
| DC-10-10 and L1011 | 0.57 | 0.40 | 0.22 | 0.12 |
| Future #4** | 1.33 | 0.84 | 0.44 | 0.24 |

*12-wheel gear, spacing: 3 sets 22x44x22 at 44 in., 2 post, 265-in. tread.
**Projected 1 million pound aircraft, dual-tandem gear 44x56 in., 4 post (2 tracking), 426-in. tread.

Load repetition factors reflect the effects of the configuration of all wheels and gears. If trailing wheels or gear induce a substantially separate stress repetition (depending on wheel spacing and the radius of relative stiffness of the pavement) the effect is included in the load repetition factor. Additional gear, not trailing, are also included; these have negligible effect for traffic widths representing taxiways and greater effect for runway traffic widths.

## VARIATION IN CONCRETE STRENGTH

Recognition of the variation of concrete strength is considered a realistic addition to the slab-thickness design procedure given in this appendix. Expected ranges of variations in the concrete's modulus of rupture have far greater effect than the usual variations in the properties of other materials—subgrade and subbase strength, layer thicknesses, etc. Typical ranges of variations in concrete strength are shown in Table A2.

Variation in concrete strength is introduced into the procedure by selecting a design modulus of rupture as follows:

$$DMR = MR_{90}\left(1 - \frac{V}{100}\right) M \qquad \text{(A2)}$$

where

$DMR$ = design modulus of rupture, psi
$MR_{90}$ = average modulus of rupture at 90 days, psi
$V$ = coefficient of variation of modulus of rupture, percent
$M$ = factor for the average modulus of rupture during design life, recognizing that concrete strength increases with age

Several combinations of $V$ and $M$, along with load repetition factors computed from selected standard deviations of traffic distribution, $\sigma$, permit a close correlation of fatigue and coverage results. Selection of conservative values for all three factors ($V$, $M$, and $\sigma$) will result in excessive conservatism in the procedure.

**Table A2. Variation of Concrete Strength**

| Rating of concrete control | Coefficient of variation, $V$, percent |
|---|---|
| Excellent | Below 10 |
| Good | 10 to 15 |
| Fair | 15 to 20 |
| Poor | Above 20 |

Note: Table is from Reference 61 and is based on compressive strength tests. Variations in flexural strength are expected to be similar.

The following values, which give close correlation between coverage and fatigue relationships, are suggested for design purposes: realistic values* for $V$ of between 10 and 18 percent; a conservative value* for $M$ of 1.10; and values for $\sigma$ of 24 in. for taxiways and 192 in. for runways.

## WEAK FOUNDATION SUPPORT

When supported by firm subbase and subgrade foundations, pavements continue to give serviceability for some time after initial cracks have developed. This is reflected in Fig.

*References 60 through 63 discuss variations and gains in concrete strength and refer to other sources of information on these topics.

A1(49) where more load repetitions are allowed for $k$ values greater than 200 pci, which represent reasonably strong subbase and subgrade support. Conservatively, for pavements carrying high traffic volumes, this additional serviceability is not recognized in the fatigue design procedure.

If reasonably strong subbase and subgrade support is not provided, pavement failures may be due to causes other than flexural failure (such as excessive subgrade strains, excessive transient and permanent subgrade deformations, subgrade pumping) that lead to loss of support or nonuniform support and then to the secondary result of failure of unsupported slabs or joints.

It is interesting to note that theory and experience agree on the principle that stronger subgrade, subbase support provides more assurance that the pavement will behave as designed by flexural methods: As the $k$ value increases, deflections and subgrade strains decrease at a faster rate than flexural stresses. An example is illustrated in Fig. A4. At safe flexural stresses, deflections and subgrade strains for a strong support are more likely to be within safe limits than for a weak subgrade where deflections and subgrade strains may be excessive even though slab flexural stresses are safe. This principle has greater effect for multiple-wheel gears.

Although it is not advisable to build a pavement to carry heavy multigeared aircraft on a weak subgrade, allowance is made for the subgrade-induced slab failure mode in the proposed procedure by adapting Corps of Engineers' curves for $k$ values of 200 pci or less. These substantially reduce the allowable number of load repetitions for very low $k$ values. To simplify use of this modification, the total fatigue consumption computed in Table A3 is increased by multiplying by a factor from Table A5 for the corresponding value of subbase, subgrade support.

For well-designed, heavy-duty pavements, a good subbase with a $k$ value of at least 200 pci should be provided so that this modification would not apply.

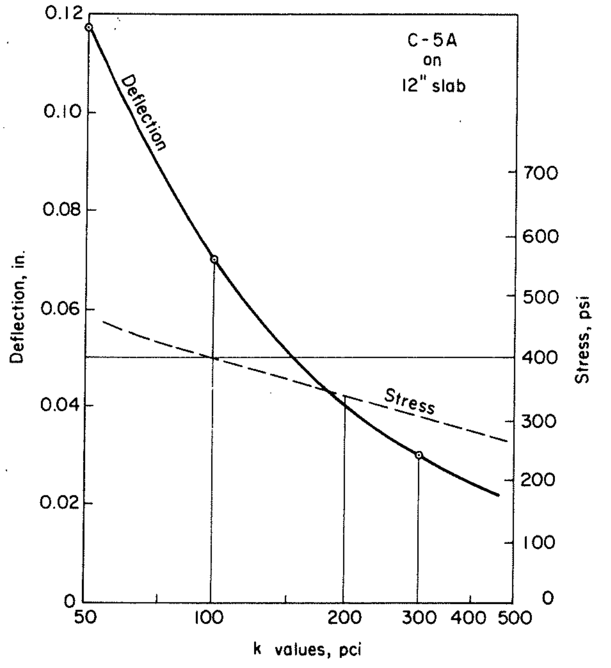



**Fig. A4. Effect of foundation support on deflection and stress.**

## Use of Fatigue Procedure, Mixed Traffic

### DESIGN

An example of use of the fatigue method for analysis of mixed traffic is given in the design form shown here as Table A3. It was assumed that a specific forecast of aircraft types and expected number of operations* was made for the design life. (Data for several future and stretch aircraft also were assumed.) The design form conveniently incorporates aspects of the fatigue design method. Stresses are determined in the usual way from PCA stress charts for specific aircraft, and allowable load repetitions are listed in the table.

*Number of operations is expressed as full-load passes. This usually includes departures only because, for jet aircraft, arriving-aircraft gross weights are about 25 percent less than maximum gross weight. The lower stresses would not induce fatigue consumption unless the stress ratio were greater than 0.50. In special cases where arriving aircraft exceed this stress ratio, they can be included as separate aircraft of lighter weight.

Total structural capacity used by all the aircraft,* column 8 of Table A3, should ideally not exceed 100 percent for an adequate design. For a 16-in. pavement, the structural capacity used is 83 percent; this represents an adequate design. If for a thickness of 15.5 in., however, more than 200 percent of the structural capacity is used, the design would be inadequate.

As explained, a good subbase should be provided for heavy-duty pavements carrying near-capacity traffic including multigeared aircraft. If the subgrade, subbase support value is low, experience indicates that the pavement's structural capacity is reduced. Due to failure in the foundation, the pavement may not be able to carry as many loads as predicted by flexural design methods. The factors in Table A5 are used to modify the structural capacity of pavements on foundations having $k$ values of less than 200 pci. They are multipliers of the total structural capacity used computed on the design sheet. The basis for this modification is given in more detail elsewhere in this appendix.

### EVALUATION

A similar analysis would apply for the evaluation of an existing pavement's future structural capacity, or to determine the capacity to carry a limited number of overloads where data on past traffic is known or can be estimated.

## Load Repetitions and Safety Factors

It is of interest to compare numerical values obtained by

*Landing gear of future aircraft may be widely spaced and may not track in the same path as other aircraft, or only the inboard gear may track in the path of other aircraft. In this case, separate summations may be appropriate for areas of highly channelized traffic.

**Table A3. Pavement Design for Mixed Traffic**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| | Pavement: Taxiway C | | | Traffic: 2 Million Departures | | | |
| | Slab thickness: 16.0 in. <br> $k$ value: 300 pci | | | 90-day modulus of rupture: 650 psi <br> $V$ = 18%, $M$ = 1.10, $(1- V/100)M$ = 0.90 <br> Design modulus of rupture (DMR) = 585 psi | | | |
| Aircraft (1) | Stress, psi (2) | Stress ratio (3) | Expected number of departures (4) | LRF (5) | Fatigue repetitions (6) | Allowable repetitions (7) | Structural capacity used, percent (8) |
| Future #4* | 354 | 0.61 | 1,500 | 1.33 | 2,000 | 24,000 | 8.3 |
| B-2707** | 332 | 0.57 | 9,600 | 0.52 | 4,990 | 75,000 | 6.7 |
| DC-10-X† | 330 | 0.56 | 32,000 | 0.57 | 18,200 | 100,000 | 18.2 |
| L1011-X† | 324 | 0.55 | 15,000 | 0.57 | 8,550 | 130,000 | 6.6 |
| B747-X† | 336 | 0.57 | 35,000 | 0.58 | 20,300 | 75,000 | 27.1 |
| DC-8-63 | 305 | 0.52 | 57,500 | 0.83 | 47,700 | 300,000 | 15.9 |
| B707 | 285 | 0.49 | 84,000 | 0.83 | 69,700 | – | 0 |
| B747 | 280 | 0.48 | 38,000 | 0.58 | 22,000 | – | 0 |
| DC-10-10 | 275 | 0.47 | 90,000 | 0.57 | 51,300 | – | 0 |
| L1011 | 270 | 0.46 | 24,000 | 0.57 | 13,700 | – | 0 |
| B727 | 265 | 0.45 | 387,000 | 0.41 | 158,000 | – | 0 |
| Other | <270 | <0.50 | 1,227,000 | – | – | – | 0 |
| | | | | | Structural Capacity Used, Total | | 82.8 |

Columns 1 and 4 – From traffic projection
Column 2 – From PCA design charts
Column 3 – Stress ÷ DMR
Column 5 – Values from Table A1
Column 6 – Column 4 X Column 5
Column 7 – Values from Table A4
Column 8 – Column 6 ÷ Column 7 X 100

*Projected Future Aircraft No. 4, 1 million lb. gross weight, gear 44x56 in., 4 post (2 tracking).
**12-wheel gear, spacing: 22x44x22 at 44 and 44 in.
†Projected future stretch versions, gross weight assumed 20 percent greater.

**Table A4. Stress Ratios and Allowable Load Repetitions**

| Stress* ratio | Allowable repetitions | Stress ratio | Allowable repetitions |
|---|---|---|---|
| 0.51** | 400,000 | 0.63 | 14,000 |
| 0.52 | 300,000 | 0.64 | 11,000 |
| 0.53 | 240,000 | 0.65 | 8,000 |
| 0.54 | 180,000 | 0.66 | 6,000 |
| 0.55 | 130,000 | 0.67 | 4,500 |
| 0.56 | 100,000 | 0.68 | 3,500 |
| 0.57 | 75,000 | 0.69 | 2,500 |
| 0.58 | 57,000 | 0.70 | 2,000 |
| 0.59 | 42,000 | 0.71 | 1,500 |
| 0.60 | 32,000 | 0.72 | 1,100 |
| 0.61 | 24,000 | 0.73 | 850 |
| 0.62 | 18,000 | 0.74 | 650 |

*Load stress divided by modulus of rupture.
**Unlimited repetitions for stress ratios of 0.50 or less.

the fatigue procedure with the recommended values of safety factors used in conventional design problems. For this purpose, safety factors have been computed by the fatigue procedure for a taxiway and runway for two operational levels: (1) capacity traffic, and (2) occasional operations of the design aircraft.

| Pavement | Total operations | LRF | Fatigue repetitions | Working stress ratio | Safety factors |
|---|---|---|---|---|---|
| Taxiway | 4 million (capacity) | 0.83 | 665,000 | 0.50 | 2.0 |
| | 100,000 (occasional) | 0.83 | 16,600 | 0.62 | 1.6 |
| Runway central portion | 4 million (capacity) | 0.17 | 136,000 | 0.55 | 1.8 |
| | 100,000 (occasional) | 0.17 | 3,400 | 0.68 | 1.5 |

(Assumptions: DC-8 and B-707 are design aircraft; 40 percent of operations are design aircraft and 50 percent are departures; $\sigma$ = 24 and 192 in.)

These safety factors, computed as the reciprocal of the allowable stress ratio based on the fatigue analysis, are in reasonable agreement with those specified in Chapter 2.

## Slab Thicknesses for Critical and Noncritical Areas

Further use of the fatigue method is illustrated by the following example computing thicknesses of pavements for a runway, taxiway, and apron. (Assumptions: B-747 aircraft; 175,000 departures; foundation $k$ value of 300 pci; design modulus of rupture of 630 psi.)

| Pavement | Traffic width as $\sigma$ | LRF | Fatigue repetitions | Working stress ratio | Slab thickness, in. |
|---|---|---|---|---|---|
| Taxiway, apron, runway ends, and turnoffs | 24 in. | 0.58 | 102,000 | 0.55 | 13.5 |
| Runway, central portion (excluding turnoffs) | 16 ft. (192 in.) | 0.28 | 49,000 | 0.58 | 13.0 |

In this example for a Boeing 747, the slab thickness for noncritical areas is 96 percent of that for critical areas. This value will vary depending on the wheel configuration of aircraft. For example, for a DC-8 and B-707 the value is about 90 percent, which is in reasonable agreement with Corps of Engineers and Federal Aviation Administration design recommendations.

It is important to note that, for mixed traffic and a greater proportion of design aircraft with complex gear, the required slab thickness for runways would approach that for taxiways.

The fatigue procedure is intended primarily as a method for handling mixed traffic. The examples in this section for a single aircraft are given to demonstrate reasonable agreement with past design experience.

**Table A5. Adjustment for Weak Foundation Support**

| *k* value, pci* | Multiply "Structural Capacity Used" in Table A3 by: |
|---|---|
| 50 | 8.0 |
| 75 | 5.4 |
| 100 | 3.7 |
| 150 | 1.9 |
| 200 | 1.0 |

**k* value on surface of foundation layer(s).

## APPENDIX B

### ADJUSTMENT IN $\ell$ VALUE FOR HEAVY-DUTY PAVEMENTS

This appendix provides a needed modification for the computation of stresses, deflections, and subgrade pressures for thick airport pavements carrying heavy aircraft with multiwheeled landing gear. The need for this modification arises from the question of evaluating the effect of a subbase in the analytical model of the pavement system.

#### Basis for Adjustment

Conventional methods of computing pavement response to loads, either by the dense-liquid subgrade assumption or the elastic-solid subgrade assumption, assume that the subbase and subgrade reaction is evaluated by a single modulus, $k$ or $E$. By this assumption, the radius of relative stiffness, $\ell$, is decreased when a subbase layer is used since the subbase and subgrade support is greater than that for the subgrade alone. This concept has satisfactorily given the approximate pavement response under past conditions of load configurations and pavement thicknesses. However, for emerging and predicted future aircraft with complex gear configurations and for multiwheeled gear operating on thick pavements, an adjustment is appropriate.

When a subbase is used, especially a strong subbase, it is understood intuitively that the load-spreading capability of the pavement is increased—in effect, that the radius of relative stiffness is increased. The significance of this for multiwheeled landing gear is that the effects of wheel interaction are increased, rather than decreased as conventionally assumed. While the effect of one or two closely spaced wheels may be approximated with no correction applied, the effect of additional wheels nearby is underestimated. The degree of error increases with the number of wheels in the landing gear, with increased ratios of subbase to subgrade strength and with increased subbase thickness.

As a result, an adjustment in the design procedure has been developed and is recommended for use in heavy-duty pavement design. The adjustment has negligible effect for single- and dual-wheel landing gear, some effect for dual-tandem gear, and substantial effect for aircraft with more than four wheels per gear.

Computations of stresses, deflections, and subgrade pressures are carried out based on the assumption that the radius of relative stiffness, $\ell$, is increased by the factor indicated in Fig. B1. The figure represents the results of a study of several analytical methods[64-67] and correlation with pavement loading test results.[68-75]

Basically the formulation involves the parameters in a three-layer analysis. However, it was found that the analysis could be simplified for practical use if computation is based on a given $\ell$ value. The only additional parameters needed are $h_2$, subbase thickness, and $E_2$, subbase modulus. This is possible because the other parameters are already included in the $\ell$ value or have constant or negligible effect in a practical design problem.*

## Determination of Load Stresses, Deflections, and Subgrade Pressures

The adjustment in $\ell$ value is determined by use of Fig. B1 for given values of subbase thickness, $h_2$, and subbase modulus, $E_2$. The adjusted $\ell'$ value is computed by multiplying a given $\ell$ value for no subbase (see Table D1) by the factor $r$. For convenience, the chart gives values for direct use with the dense liquid subgrade assumption.** Stresses for specific aircraft are then determined from Fig. B2. Deflections and subgrade pressures can be computed based on the adjusted $\ell$ value.

The subbase modulus value, $E_2$, intended for use in Fig. B1 is determined from plate-loading tests# or by laboratory tests† on the subbase material. As an estimate, or when testing is not feasible, typical values reported in the literature may be used. Ranges of values that give good agreement with experimental loading tests are indicated in Fig. B1 for granular and cement-treated subbases. These are suggested for design purposes providing the materials meet specifications and requirements given in Chapter 1. For other stabilized subbase materials, the modulus is determined by tests on the specific material or is selected based on values reported for similar materials meeting the same specifications.

The use of Fig. B2 to determine stress for a specific aircraft is illustrated in the following example:

| | |
|---|---|
| Boeing 747, gear load | = 166,500 lb. |
| Concrete slab, $h_1$ | = 12 in., $E_1$ = 4 X $10^6$ psi |
| Cement-treated subbase, $h_2$ | = 7 in., $E_2$ = 1 X $10^6$ psi |
| Clay subgrade, $k_3$ | = 100 pci |
| $\ell$ (no subbase) | = 49.27 in. (Table D1) |
| $r$, for $h_1/h_2 = 1.7$ | = 1.15 (Fig. B1) |
| $\ell' = r\ell = 1.15 \times 49.27$ | = 56.7 in. |

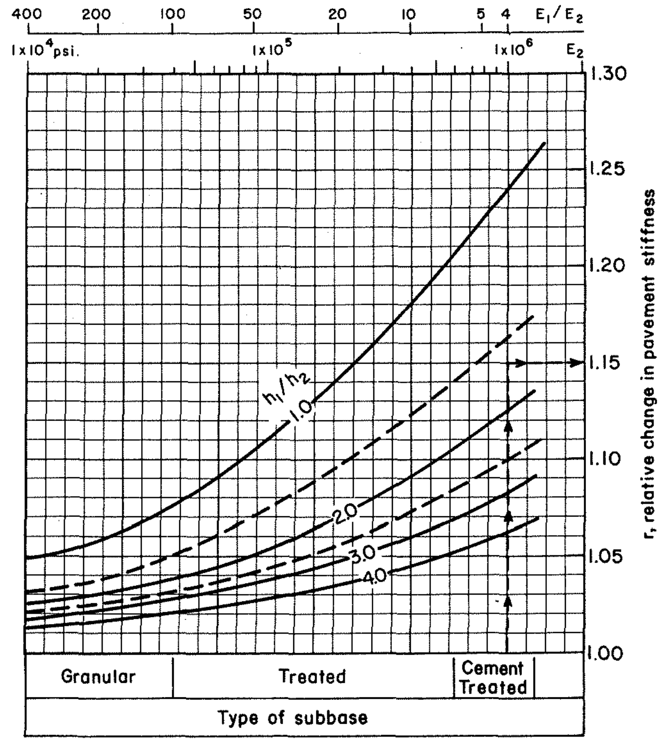



**Fig. B1. Change in pavement stiffness due to use of a subbase layer.**

Entering Fig. B2 with $\ell'$ of 56.7, intersect load line for B-747; proceed to $r$ value line of 1.15; then to pavement thickness of 12 in.; and to flexural stress of 410 psi.

If the aircraft gear load is different from that in Fig. B2, the stress value is corrected in direct proportion to the gear load; that is, if the gear load is 10 percent greater than that shown, the stress found from Fig. B2 is increased by 10 percent.††

---

*Parameters used in the development of the procedure are:
$E_1/E_2$ = ratio of elastic modulus of slab to that of subbase
$E_2/E_3$ = ratio of elastic modulus of subbase to that of subgrade
$k_2/k_3$ = ratio of $k$ values determined by plate-loading tests on subbase and subgrade
$h_1/h_2$ = ratio of slab thickness to subbase thickness
$a/h_2$ = ratio of radius of loaded area to subbase thickness
The ratio $E_1/E_2$ is shown at the top of Fig. B1 and can be used directly, or scaled as $E_2$ alone if a value for $E_1$ is assumed ($E_1$ = $4 \times 10^6$ psi is assumed in the figure). The effect of $E_2/E_3$ is included through the relationship to the ratio of $k$ values by the formulations given in Reference 66 using a plate radius, $a$, of 15 in. Size of tire contact area was found to have constant effect as long as the radius of contact area is less than slab thickness—the usual situation for airport pavement design.

**Basically the factor, $r$, shown in Fig. B1, expresses the change in flexural rigidity of the pavement when a subbase is added, as $\sqrt[4]{D'/D}$, where $D = E_1 h_1^3/[12(1-\mu^2)]$. For the elastic solid subgrade assumption, $\ell'/\ell$ is proportional to $\sqrt[3]{D'/D}$ and is thus equal to $r^{4/3}$. Using this modification, stresses, deflections, and subgrade pressures can be computed for the elastic solid subgrade case.

#$E_2$ values are computed by using the analysis of Burmister[65] or Ali[66] from elastic rebound deflections determined by repetitive, 30-in. plate-load tests. First, $E_3$, modulus of the subgrade, is computed from the elastic deflection obtained in the test on the subgrade. To compute $E_2$, the ratio of $E_2/E_3$ is determined by two-layer analysis from the elastic deflection on top of the subbase (in this case, $E_2$ represents the modulus of the top layer). When only static (nonrepetitive) deflection data are available, tests show that static deflections can be used only for strong, stabilized subbases (PCA tests on cement-treated subbases[74]) and that, for granular subbases and relatively weak subgrades, the deflections should be decreased by a factor of 1.77 (AASHO Road Test,[9] Fig. 165).

†For stabilized subbase materials, the appropriate values are determined by the dynamic modulus test or the static modulus in flexure test. Methods are described in Reference 69.

††The direct proportionality between load and stress is exact if tire contact area is not changed. If contact area is changed slightly, this relationship is closely approximated but not exact. (See "Effect of Contact Area" in Appendix C.)

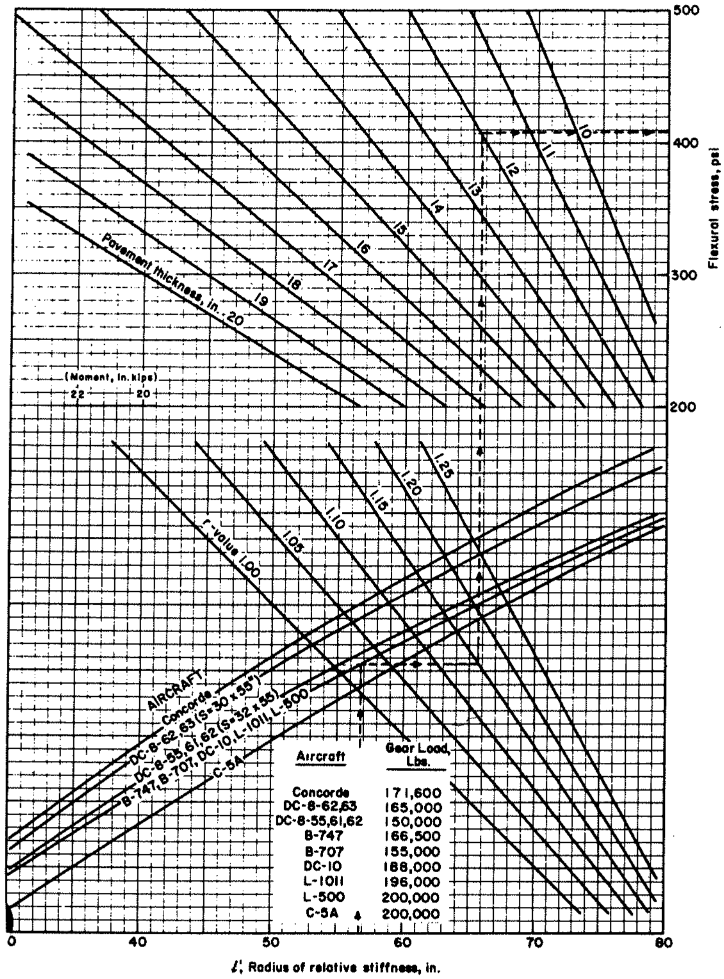



Fig. B2. Flexural stresses for multiwheeled aircraft.

For future aircraft, or for aircraft not shown in Fig. B2, a load line can be added by the following method: First, for the specific aircraft, the relationship between bending moment and $\ell$ is determined from computer program or influence charts using a sufficient number of $\ell$ values to cover the range of design consideration. For example, assume that the following data were determined for a Boeing 747 with a gear load of 166,500 lb.:

| $\ell$, in. | Bending moment, $M$, in.-lb. |
|---|---|
| 30 | 7,770 |
| 40 | 10,330 |
| 50 | 12,750 |
| 60 | 14,940 |
| 70 | 16,920 |
| 80 | 18,710 |

Then, these data are plotted as in Fig. B2 with $\ell$ on the horizontal scale and $M$ on the left vertical scale. (The vertical scale for $M$ is the projection of the moment scale from the 1.0 $r$ line.) The remainder of the chart is not changed. (For the elastic solid subgrade assumption, the same procedure with Figs. B1 and B2 is used except that the $r$ value is increased by the exponent 4/3. See footnote regarding the factor $r$ in this section. Deflections and subgrade pressures are also determined on the basis of the adjusted stiffness value.)

## APPENDIX C

### COMPUTER PROGRAM FOR AIRPORT PAVEMENT DESIGN

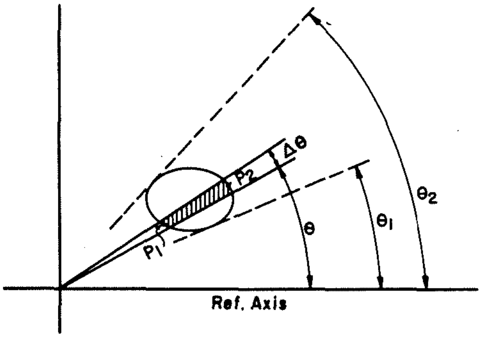
When design charts for specific aircraft are not available, pavement stresses can be computed by the PCA computer program described here (and more completely in Reference 20) or by the influence charts described in Appendix D. For aircraft with complex gear configurations, the computer program offers obvious advantages in accuracy and timesaving.

### Basis of Program

The computer program, available from Portland Cement Association, is written in Fortran IV language for the IBM 1130. It is based on Equation 23 in Reference 76; this equation is used in modified form in Fig. C1.

Series expansions are used to evaluate the Hankel functions in the moment equation. The moment for the shaded area in Fig. C1, a portion of a tire contact imprint, is found by subtracting the moment of the sector to $p_1$ from the moment of the sector to $p_2$. The moment for the total imprint area is the summation of the sector differences between the limiting rays $\theta_1$ and $\theta_2$.

An elliptical contact area is used instead of a rectangle with rounded ends as used with the influence-chart method. The two shapes have exactly the same length and contact area. Influence-chart counts indicate that there is no appreciable difference in the moments determined with the two shapes.




$$M = \frac{q\ell^2}{8}\,\mathrm{RE}\left[(1+\mu)\Delta\theta\frac{p}{\ell}\sqrt{i}\,H_1^1\!\left(\frac{\sqrt{i}\,p}{\ell}\right) + (1-\mu)\,2\sin\Delta\theta\cos 2\theta \cdot \left\{\frac{p}{2\ell}\sqrt{i}\,H_1^1\!\left(\frac{\sqrt{i}\,p}{\ell}\right) + H_0^1\!\left(\frac{\sqrt{i}\,p}{\ell}\right) - 0.5\right\}\right]$$

where: $M$ = moment at the origin in the direction of reference axis
$q$ = contact pressure, psi
$\ell$ = radius of relative stiffness, in.
$\mu$ = Poisson's ratio
$H_o^1$, $H_1^1$ = Hankel functions of order zero and one

***Fig. C1. Bending moment of loaded sector.***

**Table C1. Location and Direction of Maximum Moment**

| Aircraft | ℓ, in. | XMAX, in. | YMAX, in. | Max. angle, deg. | Gear | See Fig. C2 |
|---|---|---|---|---|---|---|
| 727-100 Main gear | 30 | 0 | -2.7 | 0 | Dual-wheel gear | Sketch b |
| | 50 | 0 | -3.1 | 0 | | |
| | 70 | 0 | -3.3 | 0 | | |
| | 90 | 0 | -3.6 | 0 | | |
| 737-200 Main gear | 30 | 0 | -2.3 | 0 | | |
| | 50 | 0 | -2.7 | 0 | | |
| | 70 | 0 | -2.9 | 0 | | |
| | 90 | 0 | -3.0 | 0 | | |
| DC-8-55 Main gear | 30 | 0 | -0.8 | 60.4 | Dual-tandem gear | Sketch c |
| | 50 | -1.1 | -1.1 | 64.8 | | |
| | 70 | -1.5 | -1.2 | 65.7 | | |
| | 90 | -1.7 | -1.3 | 66.0 | | |
| 707-320C Main gear | 30 | 0 | -0.6 | 58.5 | | |
| | 50 | -1.0 | -1.0 | 62.1 | | |
| | 70 | -1.4 | -1.1 | 62.9 | | |
| | 90 | -1.6 | -1.2 | 63.2 | | |
| C-5A Main gear* | 30 | -0.5 | -1.6 | 23.4 | Other gear | Sketch d |
| | 50 | -1.0 | -1.6 | 23.1 | | |
| | 70 | -1.2 | -1.4 | 20.8 | | |
| | 90 | -1.3 | -1.2 | 20.0 | | |
| C-5A Nose gear* | 30 | 0 | -0.4 | 0 | | Sketch e |
| | 50 | 0 | -0.9 | 0 | | |
| | 70 | 0 | -1.1 | 0 | | |
| | 90 | 0 | -1.2 | 0 | | |
| L-2000 Main gear* | 30 | 0 | -1.0 | 90 | | Sketch f |
| | 50 | 0 | -1.8 | 90 | | |
| | 70 | 0 | -2.1 | 90 | | |
| | 90 | 0 | -2.3 | 90 | | |

*Wheel spacing and contact area as given in Reference 20 for aircraft in preliminary design stage.

The area of load influence extends to a radius of 3ℓ in the computer program as compared to 2ℓ in the influence charts. Since negative moment reaches a maximum at 2ℓ, significant influence may not be included by the influence charts for contact areas extending past 2ℓ. Since the function approaches zero at 3ℓ, the wheels past 3ℓ can be disregarded without appreciable error.

Computer output includes the location and direction of maximum bending moment and the maximum stress at this position. These are computed by differentiating the sum of moments for all wheels.

## Applications

The program can be operated in any of the following four modes:

1. Thickness Design. With given aircraft data, maximum stresses are tabulated for various pavement thicknesses within the proper design range. From the data, the designer can select the design thickness for the load safety factors he has chosen. A sample output for a Boeing 727 is shown in Example 1 at the end of this appendix.
2. Pavement Evaluation. For an existing pavement with known thickness and subgrade strength, the program gives the maximum stress for the specified loading condition. This mode is used by the designer to determine if an existing pavement is structurally adequate for operation of a particular aircraft. Evaluation data for a Lockheed L-2000* are shown in Example 2.
3. Generation of Data for Design Charts. This mode generates moment values corresponding to a series of ℓ values. The data are used to construct design charts for specific aircraft. A condensed output for a Lockheed C-5A is shown in Example 3. (Contact pressure is deleted from the computations so that the results can be used with any desired pressure and gear load.)
4. General Analysis. This is a basic mode that does not maximize the moment with respect to the position of the landing gear. It is used for studies of the properties of the moment function as selected parameters are varied. Moments for each wheel and total moments are listed for incremented ℓ values and angles of rotation of the landing gear, while other factors can be varied in the input data. Example 4 lists output for the main gear of a DC-8, rotated about the maximum-moment angle.

## Location and Direction of Maximum Moment

The location and direction of maximum moment and stress are listed in Table C1 for several aircraft. These data serve as a guide for users of influence charts in reducing the trial-and-error procedure of finding the maximum stress.

To indicate the location and direction of maximum bending, the results are reported superficially as if the gear imprints have been shifted and rotated. XMAX and YMAX

*Formerly proposed version of a supersonic transport.

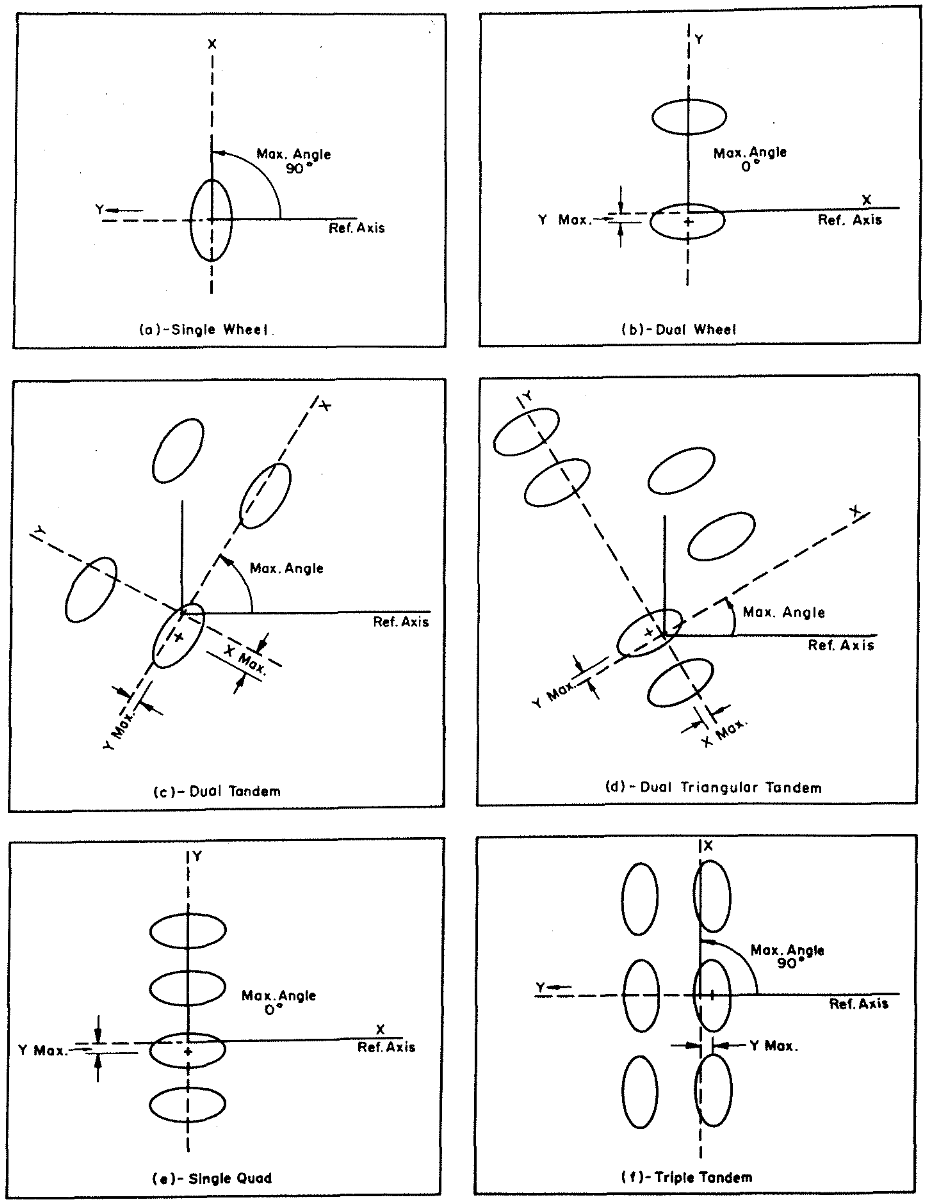



**Fig. C2. Maximum moment positions for several types of landing gear.**

represent the shift of the gear from an assumed original position; that is, the longitudinal axis of the aircraft in the direction of the reference axis. The dimensions are not scaled to $\ell$ values. MAX. ANGLE is the counterclockwise rotation of the gear in degrees from the original position. With this convention, the reference axis and the origin of the moment function represent, respectively, the direction and location of maximum bending.

Sketches of the gear positions corresponding to the tabular data are shown in Fig. C2. For example, the maximum position for the dual-wheel gear of the Boeing 727 and 737 is with the gear moved 2 or 3 in. downward (negative Y direction). For the DC-8, the maximum position is at 0 to 2 in. to the left (negative X direction), 0 to 1.3 in. downward (negative Y direction), and at a rotation angle that varies from 60 to 70 deg.

## Effect of Contact Area

Tire manufacturers' recommendations give data on desirable tire deflections, inflation pressures, and contact areas for specified wheel loads. In a design problem, tire contact area is assumed* to equal the wheel load divided by the tire pressure. It is usually further assumed that with a change in design load (as in a modified version of the aircraft) a dif-

*Actual contact areas are somewhat smaller than those computed from inflation pressure due to the effect of tire sidewall stiffness. Some designers compute contact area as 94 percent of the area computed from tire-inflation pressure. If the data are available, it is better to determine contact area from relationships between tire deflections and wheel load and then compute contact pressure by dividing load by contact area.

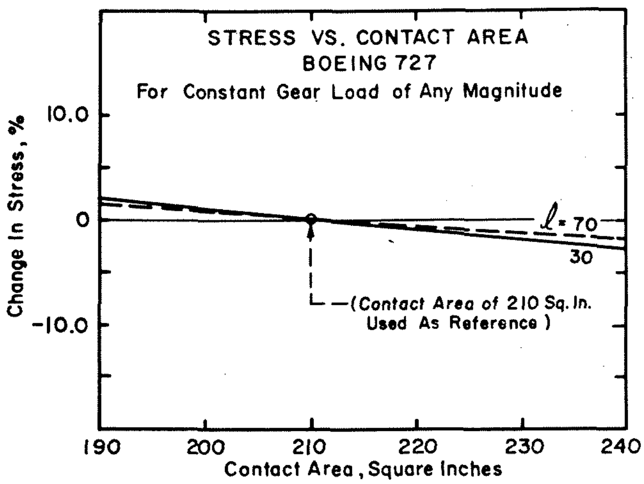



Fig. C3. Effect of tire contact area.

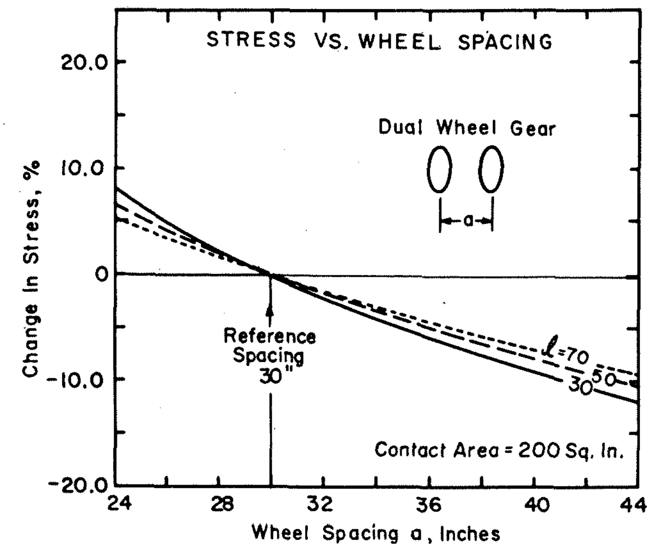



Fig. C4. Effect of dual-wheel spacing.

ferent inflation pressure is specified so that the original tire deflection and contact area are maintained. With the same contact area, stress can be computed in exact proportion to gear load and a separate analysis by computer or influence chart is not required.

In some situations, however, the designer may wish to use a contact area different from that originally assumed. If the contact area is different, stress is not exactly proportional to load. A separate computer or influence chart analysis is required to determine if significant error results.

As an example of the magnitude of this effect, data for a Boeing 727 were analyzed for a range of contact areas covering those in common use. The results are shown in Fig. C3. When the data are expressed as change in stress (stress for a specified contact area compared to that for a different contact area under equal load), the relationships shown in Fig. C3 apply for any gear load.

In this example, the conclusion is that the stress level is not particularly sensitive to changes in contact area. A stress change of less than 5 percent is indicated as contact area is varied over a wide range. It is also noted that the radius of relative stiffness has only a minor effect.

From these or similar data, the designer can estimate the approximate error introduced by using a different contact area.

## Effect of Wheel Spacing

In the preliminary stages of aircraft design, the engineer may wish to study the effect of various wheel spacings on pavement stresses. To illustrate application of the computer program to this problem, a limited study was made for dual gear and dual-tandem gear. Some of the results are shown in Figs. C4 and C5, where data are expressed as relative stress so that the relationships can be applied for any gear load.

Fig. C4 shows the effect of spacing for dual wheels. Generally, for each inch that spacing is increased, stress is reduced by slightly less than 1 percent. Changing the dual

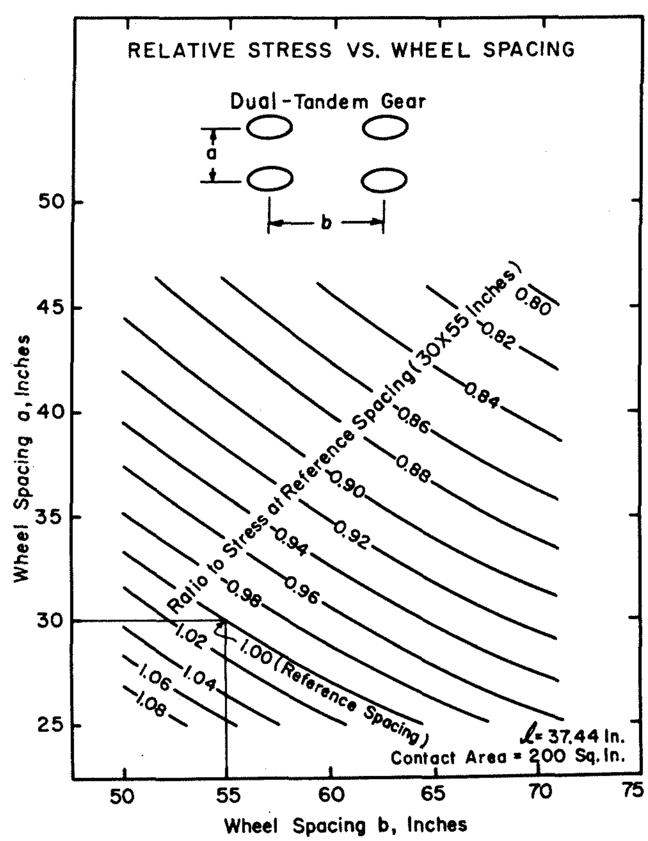



Fig. C5. Effect of dual-tandem spacing.

spacing from 30 to 36 in. reduces the stress by about 5 percent. Only minor effects are indicated for the other variables: radius of relative stiffness and contact area.

Fig. C5 shows the effect of spacing for dual-tandem gear. Change in the distance between dual wheels, dimension *a*, changes stress at the rate of about 1 percent per inch; changes in tandem spacing, dimension *b*, have slightly less effect. For equal loads, increasing the dual-tandem gear spacing from 30x55 in. to 36x60 in. decreases stress by about 9 percent. Although only one value of radius of relative stiffness and one contact area were investigated, only minor effects are deduced for these variables for the reasons stated previously.

From these data, the gear designer can optimize flotation properties within the practical range permitted by other aircraft design considerations. With computer analysis, similar results can be conveniently obtained for more complex gear configurations.

**Example 1.**
**Output for Mode 1.**
**Boeing 727.**

```
                         AIRPORT PAVEMENT DESIGN

UNITS  POUNDS INCHES DEGREES

AIRCRAFT     GEAR     NO. OF WHLS.     CONTACT AREA     CONTACT PRESSURE
 B-727       MAIN          2              237.60              145.00

COORDINATES OF WHLS.
 NO.        X         Y
  1        0.00      0.00
  2        0.00     34.00

MODE      K SUBBASE SUBGRADE
  1             300.00


                        THICKNESS    MAX. STRESS
                           8.0          631.2
                           8.5          579.1
                           9.0          534.0
                           9.5          494.6
                          10.0          459.8
                          10.5          427.9
                          11.0          399.5
                          11.5          374.1
                          12.0          351.3
                          12.5          330.8
                          13.0          312.2
                          13.5          294.7
                          14.0          278.8
```

```
UNITS  POUNDS INCHES DEGREES

AIRCRAFT      GEAR     NO. OF WHLS.     CONTACT AREA     CONTACT PRESSURE
 L-2000       MAIN          6              238.00             163.00

COORDINATES OF WHLS.
 NO.         X          Y
  1        0.00       0.00
  2        0.00      37.00
  3      -56.00       0.00
  4      -56.00      37.00
  5       56.00       0.00
  6       56.00      37.00

MODE      K SUBBASE SUBGRADE     PAVEMENT THICKNESS
  2            300.00                  12.0

     RAD. REL. STIFF.   37.43

          WHL. NO.  1      F   45.9232    CODE 0      COUNT   327.6
          WHL. NO.  2      F    0.4991    CODE 0      COUNT     3.5
          WHL. NO.  3      F    6.7373    CODE 0      COUNT    48.0
          WHL. NO.  4      F    1.8416    CODE 0      COUNT    13.1
          WHL. NO.  5      F    6.7372    CODE 0      COUNT    48.0
          WHL. NO.  6      F    1.8416    CODE 0      COUNT    13.1
                    TOTAL F   63.5802         TOTAL COUNT   453.6

          XMAX  0.0      YMAX -1.4      MAX. ANGLE   90.0

          MAX. STRESS  431.8
```

$$F = \frac{\text{Bending Moment}}{\text{Contact Pressure}}$$

$$\text{Stress} = \frac{6.0(\text{Contact Pressure})(F)}{(\text{Slab Thickness})^2}$$

$$\text{Count} = \frac{10{,}000(F)}{(\text{Rad. Rel. Stiff.})^2}$$

**Example 2.**
**Output for Mode 2.**
**Lockheed L-2000.**

```
                    AIRPORT PAVEMENT DESIGN

UNITS  POUNDS INCHES DEGREES

AIRCRAFT      GEAR     NO. OF WHLS.     CONTACT AREA
  C5A         MAIN          6              190.00

COORDINATES OF WHLS.
 NO.         X          Y
  1        0.00       0.00
  2        0.00     -36.40
  3        0.00     111.20
  4        0.00     147.60
  5       64.00      32.47
  6       64.00      78.72

MODE
  3

RAD. REL. STIFF.         F         COUNT      XMAX      YMAX     MAX. ANGLE
        30            38.8670      431.8      -0.5       1.6        23.4
        40            49.6881      310.5      -0.8       1.6        25.6
        50            60.5385      242.1      -1.0       1.6        23.1
        60            71.4143      198.3      -1.1       1.5        21.4
        70            81.8919      167.1      -1.2       1.4        20.8
        80            91.8275      143.4      -1.2       1.3        20.3
        90           101.1835      124.9      -1.3       1.2        20.0
```

**Example 3.**
**Condensed**
**Output for Mode 3.**
**Lockheed C-5A.**

```
AIRCRAFT    GEAR     NO. OF WHLS.
  DC-8      MAIN          4

COORDINATES OF WHLS.
 NO.       X        Y
  1       0.00     0.00
  2       0.00    30.00                          CONTACT AREA
  3      55.00     0.00                             207.28
  4      55.00    30.00

MODE
  4

ROTATE FROM  58.1 TO  68.1 DEGREES

ROTATION ANGLE   58.1
     RAD. REL. STIFF.  37.44

          WHL. NO.  1     F  40.9883   CODE 0     COUNT  292.4
          WHL. NO.  2     F   5.9242   CODE 0     COUNT   42.2
          WHL. NO.  3     F   3.3946   CODE 0     COUNT   24.2
          WHL. NO.  4     F   4.5667   CODE 0     COUNT   32.5
                    TOTAL F  54.8739        TOTAL COUNT  391.4

ROTATION ANGLE   63.1
     RAD. REL. STIFF.  37.44

          WHL. NO.  1     F  41.1960   CODE 0     COUNT  293.8
          WHL. NO.  2     F   5.0456   CODE 0     COUNT   35.9
          WHL. NO.  3     F   4.1142   CODE 0     COUNT   29.3
          WHL. NO.  4     F   4.5835   CODE 0     COUNT   32.6
                    TOTAL F  54.9394        TOTAL COUNT  391.9

ROTATION ANGLE   68.1
     RAD. REL. STIFF.  37.44

          WHL. NO.  1     F  41.3787   CODE 0     COUNT  295.1
          WHL. NO.  2     F   4.2727   CODE 0     COUNT   30.4
          WHL. NO.  3     F   4.7472   CODE 0     COUNT   33.8
          WHL. NO.  4     F   4.4679   CODE 0     COUNT   31.8
                    TOTAL F  54.8666        TOTAL COUNT  391.4
```

**Example 4.**
**Output for Mode 4.**
**Douglas DC-8.**

$$F = \frac{\text{Bending Moment}}{\text{Contact Pressure}}$$

$$\text{Stress} = \frac{6.0(\text{Contact Pressure})(F)}{(\text{Slab Thickness})^2}$$

$$\text{Count} = \frac{10{,}000(F)}{(\text{Rad. Rel. Stiff.})^2}$$ (for comparison with Influence Chart)

## APPENDIX D

### INFLUENCE CHARTS

This appendix is for researchers or designers with special problems for which design charts are not published. When computer facilities are not available, special studies can be undertaken with influence charts (such as effects of gear configurations and design for special vehicles).

Influence charts for deflections and moments of pavement slabs were included in a paper presented at the 1949 Spring Meeting of the American Society of Civil Engineers.[21]

The influence charts are of particular benefit to designers of concrete airport pavement to handle any type of landing gear, particularly the multiple-wheel arrangements now in use. They represent two different assumptions with regard to the subgrade—that of a dense liquid and that of a deep elastic solid. They are for deflections and moments at an edge, near an edge, and at a point near the middle (interior) of a large slab.

The design method presented here employs only the influence chart for moment, a small reproduction of which is shown as Fig. D1. This chart represents the assumption of a dense liquid subgrade and is for loads near the interior of a large slab. When all joints have adequate load transfer, conditions at any point within the pavement are nearly the same as though that point were near the interior of a large slab. Since free outside edges are not a critical location in airport pavements and since load transfer is provided at all joints, the influence charts for a free edge or near an edge are not needed.

### Procedures for Using

These steps are the suggested procedure for using the influence charts:

1. Draw an imprint of tire or tires on transparent paper to the scale of the appropriate influence chart.
2. Place drawing on the chart according to location of load with respect to the values desired.
3. Count the blocks on the chart covered by the tire imprints.

Bending moment is then computed from the intensity of loading, a factor expressing properties of subgrade and slab, and the number of blocks covered by the imprints.

The following example illustrates use of the influence chart for moment:

To determine the stress in a concrete pavement of known or assumed thickness on a given subgrade under a given load, certain facts must be known about the load (weight, spacing of wheels for multiple-wheel landing gears, tire pressure, and size and shape of tire imprint).

For this example, it is assumed the plane is equipped with dual-tandem landing gear carrying a total load of 150,000 lb., with wheel spacings as in Fig. D2. Tire pressure is 158 psi.

To find the size and shape of the tire imprints, first determine the contact area of each tire. This is assumed to equal tire load divided by tire pressure or $\frac{37,500}{158} = 237.3$ sq.in. The shape of the tire is taken to be as shown in Fig. D3, a rectangle with rounded ends, with the width equal to six-tenths of the length. The actual length ($L$) and width ($W$) of each tire imprint can be computed by the formula given in Fig. D3, in this case

$$L = \sqrt{\frac{\text{Area}}{0.5227}} = \sqrt{\frac{237.3}{0.5227}} = 21.31 \text{ in.} \qquad \text{(D1)}$$

$$W = 0.6L = 0.6 \times 21.31 \text{ in.} = 12.79 \text{ in.}$$

Next, the properties of subgrade and pavement must be determined or assumed before the proper scale for the drawing can be computed. Notice that the scale of the influence charts is based on $\ell$, the radius of relative stiffness of the pavement, which in turn is defined by the formula:

$$\ell = \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}} \qquad \text{(D2)}$$

where

$E$ = Young's modulus of the concrete, psi
$h$ = slab thickness, inches
$\mu$ = Poisson's ratio for the concrete
$k$ = subgrade modulus, pci

In this example the following values are used:
$E$ = 4,000,000 psi  $h$ = 16 in.  $\mu$ = 0.15
$k$ = 100 lb. per cubic inch

As shown in Table D1, the $\ell$ value for this pavement structure is 61.13 in.

The outline of the tire imprints can now be drawn to scale on tracing paper, using the following relationship:

$$\frac{\text{Tracing dimensions } (L \text{ and } W)}{\text{Actual imprint dimension}} : \frac{\text{graphical } \ell \text{ on chart}}{\ell \text{ of pavement}}$$

or

$$\text{Tracing } L = \frac{\text{Actual } L \times \ell \text{ on chart}}{\ell \text{ of pavement}}$$

$$= \frac{21.31 \text{ in.} \times 10 \text{ in.*}}{61.13 \text{ in.}} = 3.49 \text{ in.}$$

$$\text{Tracing } W = 0.6L = 0.6 \times 3.49 = 2.09 \text{ in.}$$

After the dimensions of one wheel are determined, compute the spacings between tires and draw all four tires on tracing paper to scale.

The resulting drawing is superimposed on the influence chart for moment, $M_n$. The influence chart gives the moment at the origin of the chart in the $n$ direction, regardless of the location of the load. The point under a load at which maximum stress occurs can be found by placing the tracing

*Ten inches is the size of graphical $\ell$ on large-scale charts normally used. The large charts are available from the Portland Cement Association.

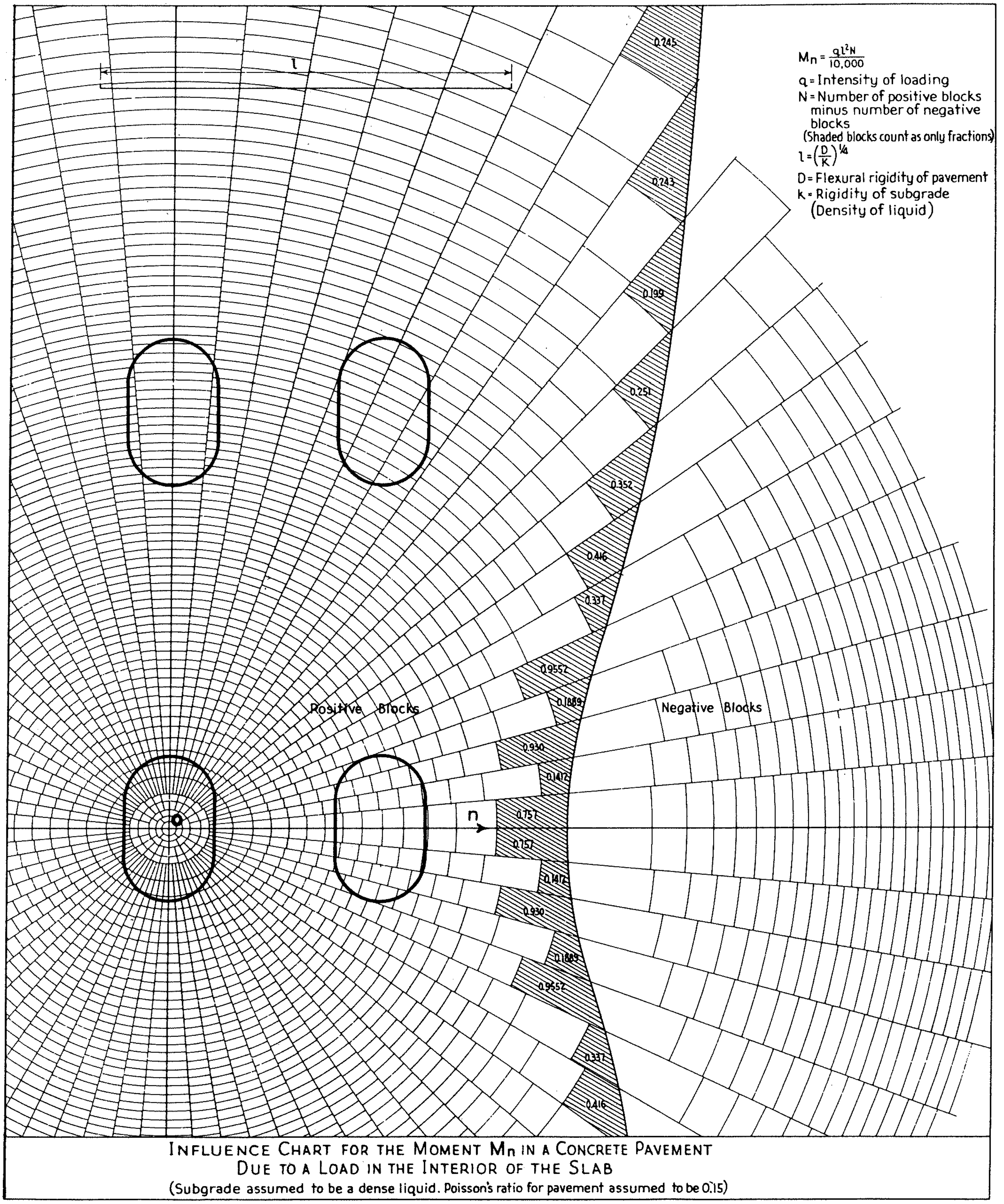



**Fig. D1. Influence chart for moment due to load at interior of concrete slab.**

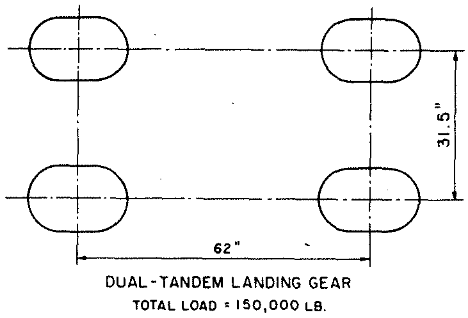
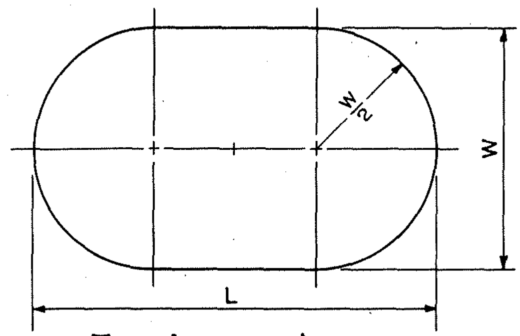



**Fig. D2. Dual-tandem gear configuration.**




TIRE IMPRINT AREA
Imprint area of tire assumed in design problem
Where W=.6L Area=.5227 $L^2$

**Fig. D3. Shape of tire-contact area.**

**Table D1. Values of $\ell$, Radius of Relative Stiffness,* in Inches**

| $h$ in in. | $k$ = 50 | $k$ = 100 | $k$ = 150 | $k$ = 200 | $k$ = 250 | $k$ = 300 | $k$ = 350 | $k$ = 400 | $k$ = 500 |
|---|---|---|---|---|---|---|---|---|---|
| 6 | 34.84 | 29.30 | 26.47 | 24.63 | 23.30 | 22.26 | 21.42 | 20.72 | 19.59 |
| 6.5 | 36.99 | 31.11 | 28.11 | 26.16 | 24.74 | 23.64 | 22.74 | 22.00 | 20.80 |
| 7 | 39.11 | 32.89 | 29.72 | 27.65 | 26.15 | 24.99 | 24.04 | 23.25 | 21.99 |
| 7.5 | 41.19 | 34.63 | 31.29 | 29.12 | 27.54 | 26.32 | 25.32 | 24.49 | 23.16 |
| 8 | 43.23 | 36.35 | 32.85 | 30.57 | 28.91 | 27.62 | 26.58 | 25.70 | 24.31 |
| 8.5 | 45.24 | 38.04 | 34.37 | 31.99 | 30.25 | 28.91 | 27.81 | 26.90 | 25.44 |
| 9 | 47.22 | 39.71 | 35.88 | 33.39 | 31.58 | 30.17 | 29.03 | 28.08 | 26.55 |
| 9.5 | 49.17 | 41.35 | 37.36 | 34.77 | 32.89 | 31.42 | 30.23 | 29.24 | 27.65 |
| 10 | 51.10 | 42.97 | 38.83 | 36.14 | 34.17 | 32.65 | 31.42 | 30.39 | 28.74 |
| 10.5 | 53.01 | 44.57 | 40.28 | 37.48 | 35.45 | 33.87 | 32.59 | 31.52 | 29.81 |
| 11 | 54.89 | 46.16 | 41.71 | 38.81 | 36.71 | 35.07 | 33.75 | 32.64 | 30.87 |
| 11.5 | 56.75 | 47.72 | 43.12 | 40.13 | 37.95 | 36.26 | 34.89 | 33.74 | 31.91 |
| 12 | 58.59 | 49.27 | 44.52 | 41.43 | 39.18 | 37.44 | 36.02 | 34.84 | 32.95 |
| 12.5 | 60.41 | 50.80 | 45.90 | 42.72 | 40.40 | 38.60 | 37.14 | 35.92 | 33.97 |
| 13 | 62.22 | 52.32 | 47.27 | 43.99 | 41.61 | 39.75 | 38.25 | 36.99 | 34.99 |
| 13.5 | 64.00 | 53.82 | 48.63 | 45.26 | 42.80 | 40.89 | 39.35 | 38.06 | 35.99 |
| 14 | 65.77 | 55.31 | 49.98 | 46.51 | 43.98 | 42.02 | 40.44 | 39.11 | 36.99 |
| 14.5 | 67.53 | 56.78 | 51.31 | 47.75 | 45.16 | 43.15 | 41.51 | 40.15 | 37.97 |
| 15 | 69.27 | 58.25 | 52.63 | 48.98 | 46.32 | 44.26 | 42.58 | 41.19 | 38.95 |
| 15.5 | 70.99 | 59.70 | 53.94 | 50.20 | 47.47 | 45.36 | 43.64 | 42.21 | 39.92 |
| 16 | 72.70 | 61.13 | 55.24 | 51.41 | 48.62 | 46.45 | 44.70 | 43.23 | 40.88 |
| 16.5 | 74.40 | 62.56 | 56.53 | 52.61 | 49.75 | 47.54 | 45.74 | 44.24 | 41.84 |
| 17 | 76.08 | 63.98 | 57.81 | 53.80 | 50.88 | 48.61 | 46.77 | 45.24 | 42.78 |
| 17.5 | 77.75 | 65.38 | 59.48 | 54.98 | 52.00 | 49.68 | 47.80 | 46.23 | 43.72 |
| 18 | 79.41 | 66.78 | 60.35 | 56.16 | 53.11 | 50.74 | 48.82 | 47.22 | 44.66 |
| 19 | 82.70 | 69.54 | 62.84 | 58.48 | 55.31 | 52.84 | 50.84 | 49.17 | 46.51 |
| 20 | 85.95 | 72.27 | 65.30 | 60.77 | 57.47 | 54.92 | 52.84 | 51.10 | 48.33 |
| 21 | 89.15 | 74.97 | 67.74 | 63.04 | 59.62 | 56.96 | 54.81 | 53.01 | 50.13 |
| 22 | 92.31 | 77.63 | 70.14 | 65.28 | 61.73 | 58.98 | 56.75 | 54.89 | 51.91 |
| 23 | 95.44 | 80.26 | 72.52 | 67.49 | 63.83 | 60.98 | 58.68 | 56.75 | 53.67 |
| 24 | 98.54 | 82.86 | 74.87 | 69.68 | 65.90 | 62.96 | 60.58 | 58.59 | 55.41 |

*For $E$ = 4,000,000 psi and $\mu$ = 0.15

$$\ell = \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}} = 24.1652\sqrt[4]{\frac{h^3}{k}}$$

in different positions on the chart and counting the enclosed blocks at each location until a maximum number is obtained. For guidance in determining the position for maximum moment, see Fig. C2 and "Location and Direction of Maximum Moment" in Appendix C.

The number of blocks enclosed in each tire is then determined by first counting all whole blocks and then all fractional blocks. Negative blocks are subtracted. The value $N$ is the total number of blocks in all four tire prints, in this case 239.

The moment at point 0 is then computed by the formula:

$$M_n = \frac{q\ell^2 N}{10{,}000} \tag{D3}$$

where

$q$ = intensity of loading—assumed to equal tire pressure for design purposes (158 psi)

$\ell$ = radius of relative stiffness (61.13 in.)

$N$ = number of blocks enclosed (239)

Therefore:

$$M_n = \frac{158 \times (61.13)^2 \times 239}{10{,}000} = 14{,}111 \text{ in.lb.}$$

The stress in the slab, at point 0, equals the moment divided by the section modulus of the slab ($h^2/6$), or

$$\text{Stress} = M_n \Big/ \frac{h^2}{6} = \frac{14{,}111 \times 6}{(16)^2} = 331 \text{ psi}$$

The same procedure is followed for influence charts for deflection and subgrade pressure, using the formula given on the influence chart.

## REFERENCES


1. Seed, H. B., Mitchell, J. K., and Chan, C. K., "The Strength of Compacted Cohesive Soils," American Society of Civil Engineers, Research Conference on Shear Strength of Cohesive Soils, 1961. *Reprint No. 96*, Institute of Transportation and Traffic Engineering, University of California.
2. Seed, H. B., and Chan, C. K., "Structure and Strength Characteristics—Compacted Clays, A Symposium," *Transactions*, American Society of Civil Engineers, Paper No. 3246, Vol. 126, Part 1, 1961, pages 1344 to 1425.
3. Dubose, Lawrence A., *Evaluating Taylor Marl Clay for Improved Use in Subgrades*, Research Report 35, Texas Engineering Experiment Station, March 1952.
4. McDowell, Chester, "The Relationship of Laboratory Testing to Design for Pavements and Structures on Expansive Soils," *Quarterly of the Colorado School of Mines*, Vol. 54, No. 4, October 1959.
5. Parcher, James B.; and Liu, Ping Chang, "Some Swelling Characteristics of Compacted Clays," *Journal Soil Mechanics and Foundation Division, Proceedings of the American Society of Civil Engineers*, Vol. 91, No. SM3, Part 1, May 1965.
6. Holtz, W. G., "Expansive Soils—Properties and Problems," *Quarterly of the Colorado School of Mines*, Vol. 54, No. 4, October 1959.
7. *Soil-Cement Laboratory Handbook*, Portland Cement Association, Skokie, Illinois, 1971.
8. Christensen, A. P., *Cement Modification of Clay Soils*, Portland Cement Association Research and Development Bulletin RD2, 1969.

9. *The AASHO Road Test, Report 5, Pavement Research*, Highway Research Board Special Report 61E, 1962.
10. *Soil-Cement Construction Handbook*, Portland Cement Association, 1969.
11. *Suggested Specifications for Soil-Cement Base Course*, Portland Cement Association, 1969.
12. *Soil-Cement Slope Protection for Earth Dams: Planning and Design*, Portland Cement Association, 1971.
13. Childs, L. D., "Effect of Granular and Soil-Cement Subbases on Load Capacity of Concrete Pavement Slabs," *Journal of the PCA Research and Development Laboratories*, Vol. 2, No. 2, May 1960.
14. Colley, B. E., and Nowlen, W. J., *Performance of Subbases for Concrete Pavements Under Repetitive Loading*, Bulletin D23, Research and Development Laboratories, Portland Cement Association, 1958.
15. Colley, B. E., and Humphrey, H. A., "Aggregate Interlock at Joints in Concrete Pavements," *Highway Research Record*, Number 189, 1967, pages 1 to 18.
16. Palmer, L. A., "Field Loading Tests for the Evaluation of the Wheel Load Capacities of Airport Pavements," *Proceedings*, American Society for Testing and Materials, 1947, page 1152.
17. *The AASHO Road Test, Report 6, Special Studies*, Highway Research Board Special Report 61F, 1962.
18. Vander Veen, C., "Loading Tests on Concrete Slabs at Schiphol Airport," *Proceedings of Third International Conference on Soil Mechanics and Foundation Engineering*, Switzerland, 1953, pages 133 to 141.
19. "Results of Modulus of Subgrade Reaction Determination at the AASHO Road Test Site By Means of Pavement Volumetric Displacement Test," U.S. Army Engineer Division, Ohio River, Corps of Engineers, Ohio River Division Laboratories, Cincinnati, Ohio, April 1962.
20. Packard, Robert G., *Computer Program for Airport Pavement Design*, Portland Cement Association, 1967.
21. Pickett, Gerald; and Ray, Gordon K., "Influence Charts for Concrete Pavements," *American Society of Civil Engineers Transactions*, Paper No. 2425, Vol. 116, 1951, pages 49 to 73.
22. *Model Tests to Determine Optimum Key Dimensions for Keyed Construction Joints*, U.S. Army, Corps of Engineers, Ohio River Division Laboratory, June 1954, 21 pages.
23. *Department of the Army (U.S.) Technical Manual TM 5-824-3.*
24. Anderson, A. A., "Expansion Joint Practice in Highway Construction," *Transactions*, American Society of Civil Engineers, Vol. 114, Paper No. 2384, 1949.
25. "Guide to Joint Sealants for Concrete Structures," reported by ACI Committee 504, *Journal of the American Concrete Institute*, No. 7, *Proceedings*, Vol. 67, Title No. 67-31, July 1970, pages 489 to 536.
26. Schutz, R. J., "Shape Factors in Joint Design," *Civil Engineering*, October 1962.
27. Tons, Egons, "Factors in Joint Seal Design," *Highway Research Record 80*, Highway Research Board, 1965.
28. Tons, Egons, "Materials and Geometry in Joint Seals," *Adhesive Ages*, September 1965.
29. Cook, J. P., and Lewis, R. M., *Evaluation of Pavement Joint and Crack Sealing Materials and Practices*, National Cooperative Highway Research Program Report No. 38, 1967.
30. *Design and Construction, Continuously Reinforced Concrete Pavement*, Continuously Reinforced Pavement Group, sponsored by Concrete Reinforcing Steel Institute and Committee of Steel Bar Producers of the American Iron & Steel Institute, 1968.
31. McCullough, Benjamin F., *Design Manual for Continuously Reinforced Concrete Pavement*, United States Steel Corporation, 1968.
32. "A Design Procedure for Continuously Reinforced Concrete Pavements for Highways," reported by ACI Subcommittee VII, Title No. 69-32, *Journal*, American Concrete Institute, Vol. 69, June 1972, pages 309 to 319.
33. Vetter, C. P., "Stresses in Reinforced Concrete Due to Volume Changes," *Transactions*, American Society of Civil Engineers, Vol. 98, 1933.
34. Zuk, William, *Analysis of Special Problems in Continuously Reinforced Concrete Pavements*, Highway Research Board Bulletin 214, 1959.
35. *AASHO Interim Guide for the Design of Rigid Pavement Structures*, AASHO Committee on Design, American Association of State Highway Officials, April 1962.
36. McCullough, B. F., and Ledbetter, W. B., "LTS Design of Continuously Reinforced Concrete," *Transactions*, American Society of Civil Engineers, Paper No. 3357, Vol. 127, Part IV, 1962.
37. Friberg, B. F., "Frictional Resistance Under Concrete Pavements and Restraint Stresses in Long Reinforced Slabs," *Proceedings*, Highway Research Board, 1954.
38. *Test Investigations of Lap Splices of Reinforcing Steel in Continuously Reinforced Concrete Pavement*, Concrete Reinforcing Steel Institute Committee on Continuously Reinforced Pavement, Bulletin No. 3.
39. Kelley, E. F., "Applications of the Results of Research to the Structural Design of Concrete Pavements," *Public Roads*, Vol. 20, 1939.
40. *Design of Concrete Pavements for Airports*, Wire Reinforcement Institute, 1960.
41. "Design of Concrete Overlays for Pavements," reported by Subcommittee VIII, ACI Committee 325, Title No. 64-40, *Journal*, American Concrete Institute, Vol. 64, August 1967, pages 470 to 474.
42. Hutchinson, Ronald L., and Wathen, Thurman R., "Strengthening Existing Airport Pavements," *Journal of Aero-Space Transport Division, Proceedings of the American Society of Civil Engineers*, Vol. 88, No. AT1, August 1962, pages 77 to 98.
43. Mellinger, Frank M., "Structural Design of Concrete Overlays," *ACI Journal, Proceedings*, Vol. 60, No. 2, February 1963, pages 225 to 236.
44. "Rigid Airfield Pavements," *Engineering and Design Manual*, EM1110-45-303, U.S. Department of the Army, Office of the Chief of Engineers, February 1958.
45. *Bonded Concrete Resurfacing*, Portland Cement Association, 1960.
46. "Guide for the Use of Epoxy Compounds with Con-

crete," ACI Committee 503, *ACI Journal, Proceedings*, Vol. 59, No. 9, September 1962, pages 1121 to 1142.
47. Westall, W. G., "Concrete Overlays on Asphalt Pavements," *Highway Research News* (22), Highway Research Board, February 1966, pages 52 to 57.
48. Packard, Robert G., "Fatigue Concepts for Concrete Airport Pavement Design," *Transportation Engineering Journal*, ASCE Vol. 100, No. TE3, August 1974, pages 567-582.
49. *A Method for Estimating the Life of Rigid Airfield Pavements*, Technical Report No. 4-23, U.S. Army Engineer Division, Corps of Engineers, Ohio River Division Laboratory, Cincinnati, Ohio, March 1962.
50. *General Provisions for Airfield Design*, TM5-824-1, AFM 88-6, Chapter 1, Departments of the Army and Air Force.
51. Hutchinson, R. L., *Basis for Rigid Pavement Design for Military Airfields*, Miscellaneous Paper No. 5-7, Department of the Army, Ohio River Division Laboratories, May 1966.
52. Mellinger, F. M., and Ahlvin, R. G., "Pavement Design for Commercial Jet Aircraft," American Society of Civil Engineers, *Journal of Air Transport Division*, Vol. 85, No. AT2, May 1959, pages 29 to 43.
53. *Rigid Pavements for Airfields, Other Than Army*, TM5-824-3, AFM 88-6, Chapter 3, Departments of the Army and Air Force, January 1970.
54. *Thickness Design for Concrete Pavements*, Portland Cement Association, 1966.
55. Miner, M. A., "Cumulative Damage in Fatigue," *Transactions*, American Society of Mechanical Engineers, Vol. 67, 1945, pages A159 to A164.
56. Kesler, Clyde E., "Fatigue and Fracture of Concrete," *Stanton Walker Lecture Series of the Materials Sciences*, National Sand and Gravel Association and National Ready Mixed Concrete Association, 1970.
57. Ballinger, Craig A., "The Cumulative Fatigue Damage Characteristics of Plain Concrete," *Highway Research Record 370*, Highway Research Board, 1971, pages 48-60.
58. Fordyce, Phil, and Yrjanson, W. A., "Modern Design of Concrete Pavements," American Society of Civil Engineers, *Transportation Engineering Journal*, Vol. 95, No. TE3, Proc. Paper 6726, August 1969, pages 407 to 438.
59. Fordyce, Phil, and Packard, R. G., "Concrete Pavement Design," paper presented at 49th Annual Meeting, American Association of State Highway Officials, Committee on Design, October 1963. (Available from Portland Cement Association.)
60. Haddad, Gilbert; and Freedman, Sidney, "Statistical Product Control," *Modern Concrete*, Parts 1 through 4, March, April, May, and June 1970; also reprinted by Portland Cement Association, 1970.
61. Recommended Practice for Evaluating Compression Test Results of Field Concrete, American Concrete Institute, *Standard 214-65*.
62. Withey, M. O., "Fifty-Year Compression Test of Concrete," *Journal*, American Concrete Institute, Vol. 58, No. 6, December 1961.
63. Gonnerman, H. F., and Lerch, William, *Changes in Characteristics of Portland Cement as Exhibited by Laboratory Tests Over the Period 1904 to 1950*, Special Technical Publication No. 127, American Society for Testing and Materials, 1952.
64. Odemark, Nils, *Investigations as to the Elastic Properties of Soils and Design of Pavements According to the Theory of Elasticity*, Statens Väginstitut, Stockholm, 1949, 100 pages. (In Swedish with English summary.)
65. Burmister, Donald M., "The Theory of Stresses and Displacements in Layered Systems and Applications to Design of Airport Runways," *Proceedings*, Highway Research Board, Vol. 23, 1943, pages 126 to 148.
66. Ali, G. A., "Influence of Variations in Poisson's Ratio in the Deflection Function for Elastic Layered Systems," Master of Science Thesis, Northwestern University, Evanston, Illinois, 1967.
67. Jones, A., "Tables of Stresses in Three-Layer Elastic Systems," *Highway Research Board Bulletin 342*, 1962, pages 176 to 214.
68. Childs, L. D., Colley, B. E., and Kapernick, J. W., "Tests to Evaluate Concrete Pavement Subbases," *Proceedings ASCE*, Paper No. 1297, Vol. 83 (HW-3), July 1957, pages 1 to 41; also PCA Development Department Bulletin D11.
69. Felt, E. J., and Abrams, M. S., "Strength and Elastic Properties of Compacted Soil-Cement Mixtures," *ASTM Special Technical Publication No. 206*, American Society for Testing and Materials, 1957, pages 152 to 178; also PCA Development Department Bulletin D16.
70. Childs, L. D., and Kapernick, J. W., "Tests of Concrete Pavement Slabs on Gravel Subbases," *Proceedings ASCE*, Vol. 84 (HW 3), October 1958; also PCA Development Department Bulletin D21.
71. Childs, L. D., and Kapernick, J. W., "Tests of Concrete Pavements on Crushed Stone Subbases," *Proceedings ASCE*, Proc. Paper No. 3497, Vol. 89 (HW-1), April 1963, pages 57 to 80; also PCA Development Department Bulletin D65.
72. Childs, L. D., "Tests of Concrete Pavement Slabs on Cement-Treated Subbases," *Highway Research Record 60*, Highway Research Board, 1963, pages 39 to 58; also PCA Development Department Bulletin D86.
73. Nussbaum, P. J., and Larsen, T. J., "Load-Deflection Characteristics of Soil-Cement Pavements," *Highway Research Record 86*, Highway Research Board, 1965, pages 1 to 14; also PCA Development Department Bulletin D96.
74. Childs, L. D., "Cement-Treated Subbases for Concrete Pavements," *Highway Research Record 189*, Highway Research Board, 1967, pages 19 to 43; also PCA Development Department Bulletin D125.
75. Larsen, T. J., Nussbaum, P. J., and Colley, B. E., *Research on Thickness Design for Soil-Cement Pavements*, PCA Development Department Bulletin D142, 1969.
76. Pickett, Gerald; Raville, Milton E.; Janes, William C.; and McCormick, Frank J., *Deflections, Moments, and Reactive Pressures for Concrete Pavements*, Kansas State College Bulletin No. 65, October 1951.

KEYWORDS: airports, cement-treated subbases, concrete pavements, evaluation, fatigue, joints, overlay, safety factor, stresses, structural design, subbases, subgrades, thickness, traffic.

ABSTRACT: Procedures given for design and evaluation of concrete airport pavements. Design topics include: subgrades and subbases; slab thickness; plain, reinforced, and continuously reinforced pavement; joint design; and concrete overlays. Special design topics include a fatigue analysis for mixed aircraft loads, determination of pavement stiffness for heavy-duty pavements, and use of computer program and influence charts.

REFERENCE: *Design of Concrete Airport Pavement* (EB050.03P), Portland Cement Association, 1973.

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