DESIGN CONSIDERATIONS FOR CONTROL OF JOINT FAULTING OF UNDOWELED PAVEMENTS

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SYNOPSIS

Based on a study of the performance of undoweled concrete pavements, it is concluded that the degree of joint faulting depends on the number of heavy trucks carried, slab thickness, joint spacing, and the type of subbase and subgrade. The effect of different degrees of joint faulting on the quality of ride, or serviceability, of pavements is evaluated. A correlation of the faulting data provides an improved method for predicting the degree of faulting and serves as a more definite basis than previously available to decide whether or not a doweled joint design is required. The fault prediction model may also be used, with stated limitations, to estimate the required thickness of undoweled pavements designed thicker than conventional doweled pavements to compensate for the absence of dowels. The results are intended to apply to the design of highway and street pavements with short joint spacings.

INTRODUCTION

Twenty states currently specify pavements with undoweled joints and short joint spacings, either as the exclusive design or as an alternative to other types of concrete pavement design, for Interstate highways and major expressways. Generally, the performance of these undoweled pavements, which are in the 8-to 10-in. thickness range, has been satisfactory. However, in some cases, excessive joint faulting* has developed, which generally is associated with large volumes of heavy truck traffic. Undoweled pavements for lower class facilities such as primary and secondary roads - 7-or 8-in. thickness range - do not fault objectionably if they are not subjected to more than a moderate volume of heavy truck traffic. Undoweled pavements are often used for low-traffic facilities such as residential streets and farm-to-market roads -- usually less than 7 in. thick -- carrying low volumes of heavy trucks, and these generally do not fault to any serious degree or may show no faulting.

It thus is indicated that faulting of conventionally designed pavements, although a complex process,** is related to the accumulated number of passes of heavy trucks and to pavement thickness. The specific number of heavy trucks required to cause an objectionable degree of faulting has not been determined precisely. PCA has previously suggested[1], as a very rough guide, that dowels may be omitted if the volume of heavy trucks does not exceed approximately 300 per day (in two directions). This is meant to apply to pavements with short joint spacings - so that load transfer by aggregate interlock is effective - and normal slab thicknesses as determined by conventional design procedures.

In the many cases where undoweled joints have been used for airport pavements, performance experience has shown that faulting rarely occurs. One reason for this is that

*Faulting is the permanent vertical displacement of the slab on one side of a pavement joint with respect to the position of the slab on the other side of the joint. Typically, the slab is lower on the downstream traffic side of the joint.

**Mechanisms of joint faulting are discussed in References 2, 3 and 4.

faulting seems to be associated with only one-directional traffic and most runways and taxiways are two-directional, or at least not entirely one-directional. A rare case of a faulted runway pavement was observed at a southeastern U.S. airport where the traffic is almost entirely one-directional. Subjected to Boeing 727 aircraft loads, the portion of the runway that is 11 in. (280 mm) thick (somewhat thin for these loads) was badly faulted while the immediately adjacent 13-in. (330 mm) thick slabs in the same traffic path were not, indicating the influence of pavement thickness under equal conditions of traffic, subbase-subgrade, and climate. In this case, a pumpable subbase material contributed to the faulting problem.

In view of this general performance experience with undoweled pavements, it appears that the degree of faulting is strongly related to the number of passages of wheel loads that would be judged as heavy in relation to the slab thickness. Thus, to judge whether or not dowels are needed, it seems logical to consider the interrelationship among (1) magnitude of heavy loads, (2) number of passages of these loads, and (3) slab thickness. A corollary is that it may not be possible to estimate the specific number of heavy loads that will fault an undoweled pavement unless the pavement thickness is specified.

The major objective of this report is to give an improved estimate of the conditions for which dowels are, or are not, required at joints in unreinforced concrete pavements with short joint spacings. Based on a correlation of faulting data with traffic, pavement age and thickness, and other variables, a method for predicting the degree of faulting is presented. By this method, the designer, with given design information, including pavement thickness, can predict the degree of faulting and use this information to decide whether or not dowels are needed.

For the most part, the study concerns the design of highway and street pavements of unreinforced concrete slabs with relatively short joint spacings.

WAYS OF EXPRESSING THE DEGREE OF FAULTING

Investigators of pavement faulting have expressed the degree of faulting in different terms. Brokaw[5] used SVF (slope variance due to faulting) as a measure of faulting where the numerical values might range up to 14 or somewhat higher for a severely faulted pavement. He used SVF as a tie-in to Chloe slope variance to obtain a direct relation to the serviceability concepts of the AASHO Road Test[6] and AASHTO design procedure[7]. The method of calculating SVF from raw fault measurement data is given in Reference 8, page 136.

In studying the faulting of pavements in Georgia, Gulden[4] used FI (faulting index) as the yardstick, with numerical values ranging up to 25 or more for severely faulted pavements. FI of a pavement section is defined as the average accumulative amount of faulting for five consecutive joints expressed in 32nds of an inch. Thus, an FI of 20 when divided by 5 gives an average joint fault of 4/32 in. (3.2 mm). The selection of the FI as the yardstick is probably influenced by the effective procedure developed by Gulden for obtaining representative faulting measurements.

The relation between SVF and FI is given by the equation:

$$FI = 1.314[(SVF)(J)]^{0.5804} \quad \text{... Eq. 1}$$

where: FI = faulting index
SVF = slope variance, faulting
J = joint spacing, ft.

These investigators have chosen their yardsticks for valid reasons; however, the physical meaning of reported numerical values sometimes can not be readily interpreted until one is familiar with the system. As a result, the yardstick of faulting used in this paper is Favg., average fault, in 32nds of an inch (0.8 mm). (Favg. is numerically equivalent to the faulting index, FI, divided by 5).

FAULTING AND THE SLOPE VARIANCE - SERVICEABILITY CONCEPT

It is the author's impression that the effect of faulting on the quality of ride is not well described by the AASHTO serviceability model* involving PSI and slope variance.

This is illustrated in Fig. 1, where the upper curve represents the AASHTO effect of pavement roughtness and the lower curve represents a different concept of the effect of faulting.** It is suggested that faulting is a unique type of roughness that has to be analyzed differently from other forms of pavement roughness in evaluating its effect on pavement serviceability.

The concept is that, while the AASHTO model and the effect of faulting are perhaps not in disparity at low to moderate levels of faulting, severely faulted pavements affect serviceability much more adversely than indicated by the AASHTO model.

That this could be true is not surprising as the AASHTO relationship was established for pavements on which no significant faulting had developed and thus reflects the effects of other forms of roughness.

PSI's computed from fault measurements, or roadmeter data[9], of moderately faulted pavements may be in the range of 3.2 to 3.5***, in general agreement with the user's experience of riding comfort. PSI's computed in the same manner for severely faulted pavements - say an average fault of 1/4 in. (6.4 mm) - are usually not down to 2.5; the PSI values are in disagreement with experience of riding comfort and general opinions of tolerable faulting. The AASHTO model seriously underestimates the degree of faulting required to reduce serviceability to commonly accepted terminal levels (PSI of 2.5 or 2.0).

Agreement with this concept is implicit in the reports of others who have studied pavement faulting. Brokaw's criteria on allowable SVF and Gulden's numerical ratings (see

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*As described in Ref. 6, this was developed from a correlation of a panel's rating of the ride of concrete pavements with physical measurements of pavement roughness and other properties. For rigid pavements the relationship is:

$$PSI = 5.41 - 1.80 \log (SV + 1) - 0.09 \sqrt{C + P}$$

where: PSI = Present Serviceability Index
SV = slope variance, a measure of pavement roughness computed from Chloe profilometer measurements
C and P involve the amount of pavement cracking and patching

**The lower curve of Fig. 1 is plotted to show only the concept (shape of the curve); it is not intended to indicate the exact position of the curve, which may vary with different joint spacings and different peoples' opinions.

***The values of PSI computed depend somewhat on joint spacing.

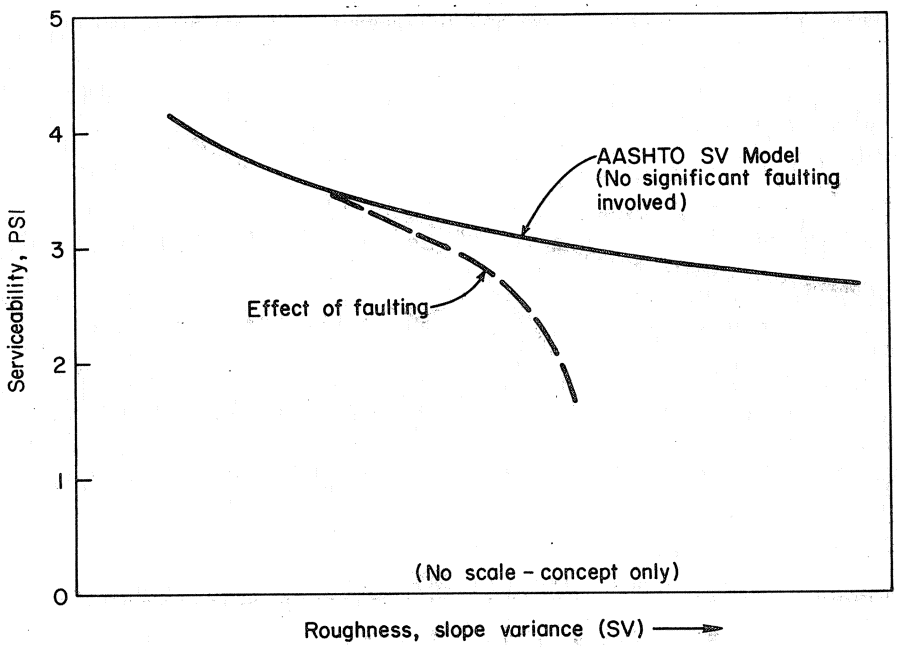


Fig. 1 Concept of How Faulting Affects Pavement Serviceability.

discussion under Faulting Criteria) imply that severe faulting has a much greater effect than indicated by the slope variance model.

A personal observation is that serviceability decreases rather uniformly and gradually as faulting increases; then, beyond a certain level of faulting - thought to be about 7/32 or 8/32 in. (5.6 - 6.4 mm) - the quality of the ride diminishes rapidly.

It is concluded that faulting is a unique type of roughness that needs to be evaluated further before it can be introduced into pavement serviceability concepts. The slope variance model does not seem adequate for faulted pavements for reasons previously discussed. Also, it cannot be consistently applied to pavements with different joint spacings. Computed slope variance is inversely proportional to joint spacing; it is doubtful that the highway user would accept a much greater magnitude of faulting at 30 ft (9 m) than at 15 or 20 ft (5 or 6 m). While some ideas have emerged on how faulting affects serviceability, additional research, including the opinions of highway users, is needed.

FAULTING CRITERIA

From his studies of pavements with joints spaced at 15 or 20 ft (5 or 6 m) intervals, Brokaw[5, 10] concluded that faulting creates an incipient stage of highway user dissatisfaction when SVF is greater than 12 and that this might be increased to an SVF of 14 for pavements with randomized, skewed joints. These SVF's correspond approximately to a fault range of:

FI	29 to 32	
Favg.	6	(4.8 mm)

Gulden in his studies of Georgia Interstate pavements with 30 ft joint spacings[4] set up the following ratings:

FI	Rating	Favg. (32nds in.)	
0-5.0	Excellent	0-1	(0-0.8 mm)
5.1-10.0	Very good	1-2	(0.8-1.6 mm)
10.1-15.0	Good	2-3	(1.6-2.4 mm)
15.1-20.0	Fair	3-4	(2.4-3.2 mm)
20.1-25.0	Poor	4-5	(3.2-4.0 mm)
>25.0	Very poor	>5	(>4.0 mm)

He also concluded that, for projects with areas having an FI of 20 or more, continuous maintenance will probably be necessary. For these pavements, it is noted that severe faulting is also accompanied by depression and cracking of the shoulder in the vicinity of the joint, cracking of the downstream slab, and settling of that portion of the slab. This is more objectionable and requires greater maintenance than the usual situation where pavements can reach a severely faulted state without cracking or other serious distress. Naturally, this situation influences the rating of the Georgia pavements.

For California pavements, Spellman, Woodstrom and Neal[11] said that on moderately faulted pavements - e.g., 0.12 to 0.25 in. (3 to 6 mm) - some vehicles may not be affected, but drivers of trucks and a number of lighter-weight cars may feel considerable discomfort.

Although not presented as a criterion, Schroeder in his study of Iowa's undoweled pavements[12] classified joints as "flat" or "faulted" depending on whether the fault was greater than 1/8 in. (3.2 mm).

Combining these concepts with the author's opinion on the quality of ride, it appears that a value somewhere in the average fault range of 5 or 6 32nds of an inch (4.0 or 4.8 mm) for many projects may be the dividing line between tolerable and intolerable faulting; this value is difficult to define exactly with universal agreement. However, it is probably agreed that an average fault of 4/32 in. (3.2 mm), if this is the only defect in the pavement, gives a tolerable ride (and does not reduce PSI greatly), while an average fault on the order of 7/32 or 8/32 in. (5.6 or 6.4 mm) is definitely intolerable, at least on a major, high-speed facility.

In this paper, it is not intended to define the terminal faulting criteria exactly. This is left to the judgement of the design engineer, who should consider several factors including the type of facility, economic factors, and consequences of user dissatisfaction (the phone call factor). If he has the opportunity, the engineer should also ride pavements that are faulted to various degrees to get a first-hand impression.

The selection of less conservative criteria may be justified for lower types of pavement facilities such as residential streets, secondary roads, and industrial yards where conditions -- traffic volume and speed, economics, a less serious phone call factor -- might not demand as high a level of riding comfort as would be expected for an Interstate highway. Several examples observed are low and moderate truck traffic roads which, although now faulted an average of 1/4 in. (6.4 mm) or so, are still adequate after more than 20 years of service; there is no intent or pressure to rehabilitate them in the near future.

If an Interstate pavement or other major high-speed, high-traffic facility is being considered, the designer may wish to be conservative in selecting his criteria; however, the concept of zero faulting is not realistic since a faulting level up to 2 or 3/32 of an

inch (1.6 or 2.4 mm) is not noticeable to the average user. (It should be emphasized that the criteria represent terminal faulting -- the average fault at the end of 20 years or whatever the design period is -- and not the average degree of faulting during the design life).

DATA ANALYSIS FOR THE FAULT PREDICTION MODEL

Data on the faulting of undoweled pavements were collected from 404 sections of highways in 5 states. Sources were the report of Brokaw[10] for the data from Minnesota, Wisconsin, and North Dakota (pavements with 15 and 20 ft (4.6 and 6.1 m) joint spacings), the reports of Gulden[4] for Georgia data (30 ft (9.1 m) joint spacings), and an unpublished report of Brokaw for the California data (randomized joint spacings averaging 15.5 ft (4.7 m). Correlations of these data are compared with additional information on faulting from the Nussbaum and Lokken report of a survey in several states[13] and the report of Schroeder on the performance of undoweled pavements in Iowa[12].

The pavements included in the study, from 7 to 10 in. (178 to 254 mm) thick, ranged in age from 1 to 20 years and carried volumes of trucks ranging from a few up to 2,000 tractor-semi-trailers per day in one direction. For each pavement section the following data were available: the degree of faulting either in SVF, FI, or raw fault measurements, number of daily tractor-semi-trailer and combination trucks (T),* age of the pavement (A), pavement thickness (D), joint spacing, type of subbase, and, in rough terms, whether or not the subgrade was free-draining. The data points are plotted in Figs. 2 through 6.

In preliminary analysis of the data, it became apparent that SVF data and suggested criteria might not apply to pavements with a 30 ft (9.1 m) joint spacing. Normally this would be of no concern since the criteria were developed and are used for pavements with shorter joint spacings in the range of 15 to 20 ft (4.6 to 6.1 m). The same degree of faulting in terms of raw fault measurements would compute to a much lower SVF for a longer joint spacing; for example, an average fault of 5/32 in. (4.0 mm) computes to an SVF of 10.7 for a pavement with 15 ft (4.6 m) joint spacings, while the same degree of faulting in a pavement with 30 ft (9.1 m) joint spacings would compute to an SVF of only 5.3. In order to apply to the longer spacings, SVF values were converted to average fault values for the data analysis.

Preliminary multiple regression trials were run to indicate the effects of pavement thickness, age, and number of trucks. Since the data were from pavements having different joint spacings, different subgrades, and different types of subbases, an attempt was made to account also for these additional variables in the regression analysis.

Preliminary regression trials indicated that the influences of traffic, age, and pavement thickness on the degree of faulting were not linear. The data were fitted better by semi-logarithmic and logarithmic models. However, these models had a tendency to underpredict at both low and high degrees of faulting, being constrained by a greater mass of data in the middle range. In attempting to obtain a better fit, particularly in the high range which is of more practical significance, various regressions were run with some of the data in the middle and low range omitted; for example, some of the data represented the same pavement sections measured at different time periods. In some runs,

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*T, as used in this report, is the average number of these trucks per day over a stated time period. For the analysis, reported T values were discounted at an arbitrary 5% per year interest rate and the average was computed.

these were averaged; and, in some cases, pavements having Favg. values less than 1.4 or 1.6 (1.1 or 1.3 mm) were omitted.

The regression was further improved by use of a mixed model of the general form

$$F_{avg.} = a + \frac{T^{b_1} A^{b_2}}{D^{b_3}} B^{b_4} S^{b_5} J^{b_6} \quad \text{.....Eq. 2}$$

where: $F_{avg.}$ = average fault, 32nds of an inch
T = number of tractor-semi-trailer and combination trucks in one-direction (average number per day during A years of service)
A = age of pavement, years
D = pavement thickness, inches
B = type of subbase (1 for stabilized, 2 for granular)
S = type of subgrade (1 for good, 2 for poor*)
J = joint spacing, ft.
$a, b_1, b_2 \ldots$ = constants or regression coefficients

With adjustments of the numerical factors for subgrades and subbases and omission of cases for which the Favg. was less than 1.4 (1.1 mm), a correlation coefficient, R, of 0.866 was obtained.

In all these trials, the regression is significant at the 1% level. Considering the mixed sources of data, the large inherent variation that may be assumed to be associated with the truck traffic factor,** and the crude rating of subgrades and subbases, the correlation is surprisingly fair.

It is a shortcoming of the regression analyses that the data are not factorial -- that is, all levels of each variable in relation to levels of other variables are not fairly represented. Because of this, the final model was derived graphically, using the coefficients from the several regression analyses as an approximate guide. This was done also so that the prediction model would better fit cases in the moderate to high range of faulting since a greater degree of error in the low range would be of less practical significance in predicting the serviceable life of a pavement. The resulting prediction model is represented by the equation:

$$F_{avg.} = 1.29 + 48.95 \frac{(TA^2)^{0.465}}{D^{3.9}} S^{0.610} (J - 13.5)^{b_6} \quad \text{...Eq. 3}$$

where: b_6 = 0.241 for granular subbase;
0.037 for stabilized subbase

The lines in Figs. 2 through 6 illustrate the degree of fit of the model to the data points. The fit of Equation 3 to the data is characterized by a correlation coefficient of 0.878 and a standard error of 1/50 of an inch (0.6 mm). Although there is scatter, the fit is not bad when the actual physical magnitude of the deviations is considered.

*The terms "good" and "poor" refer to drainage characteristics.

**Obviously, the truck count can vary substantially from time to time, and the count at any specific time may not always represent the average. Also, there is a general tendency for the number of trucks and their axle loads to increase with time and for the proportion of heavy axle loads to increase as the truck count increases.

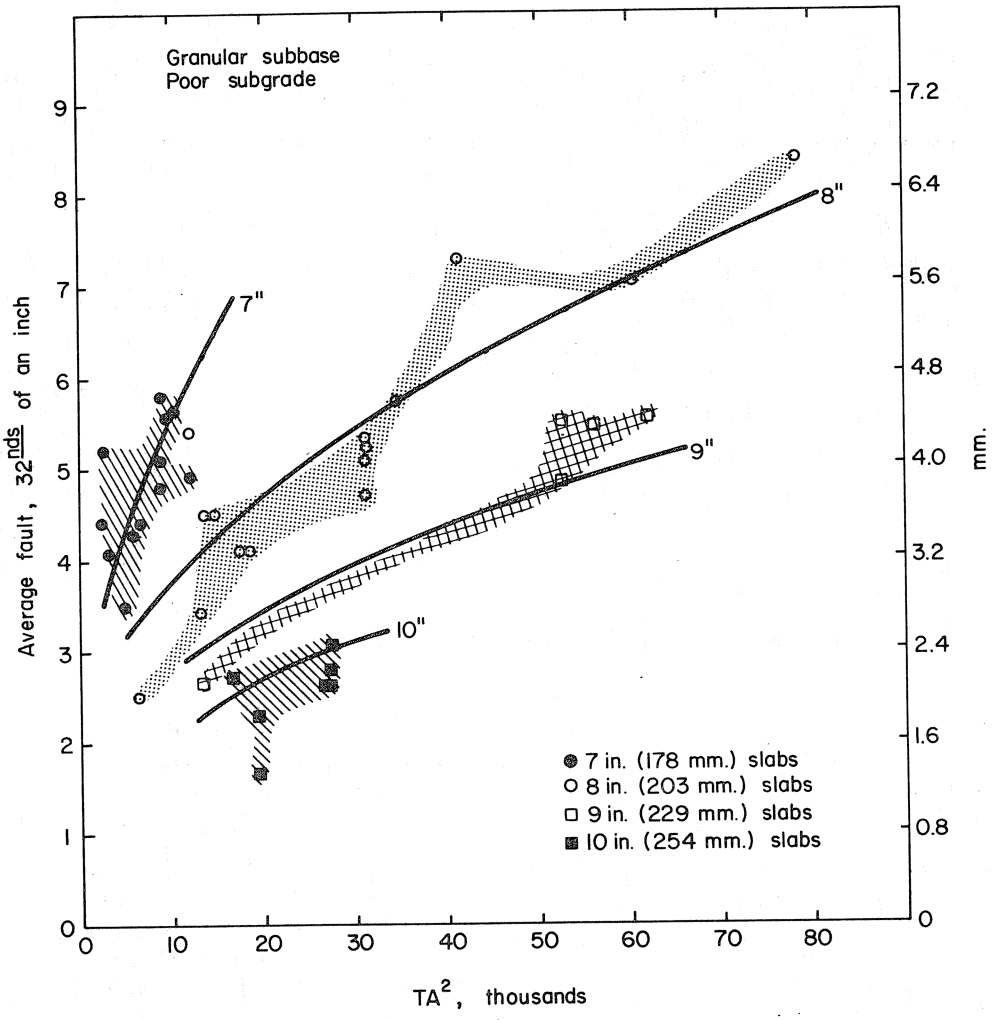


Fig. 2 Joint Faulting in Minnesota, North Dakota, and Wisconsin (20-ft joint spacing, poor subgrade, granular subbase).

Figs. 7 and 8, which are discussed in the next section, have been drawn up from Equation 3 to predict faulting at the end of 20 and 30-year service periods.

Another step in the data analysis was to compare predicted faults with those reported in independent studies. One of these was the study of Nussbaum and Lokken[13], which included undoweled pavement sections on Interstate and primary routes in five states. The fault prediction model estimates the reported faults well in Oklahoma, Iowa, and Georgia, but overestimates slightly the degree of faulting in Wisconsin and California. A comparison to data for Iowa's primary routes[12], most of them constructed without a subbase, indicates that the fault prediction model underestimates the faulting there. These comparisons are not fully conclusive because the truck volumes and fault measurements are not expressed in the same manner as in the fault prediction model.

The prediction model represents an extension of Brokaw's equations to include data for additional geographic areas and design conditions, and modifications that improve the accuracy of prediction over previously reported models.

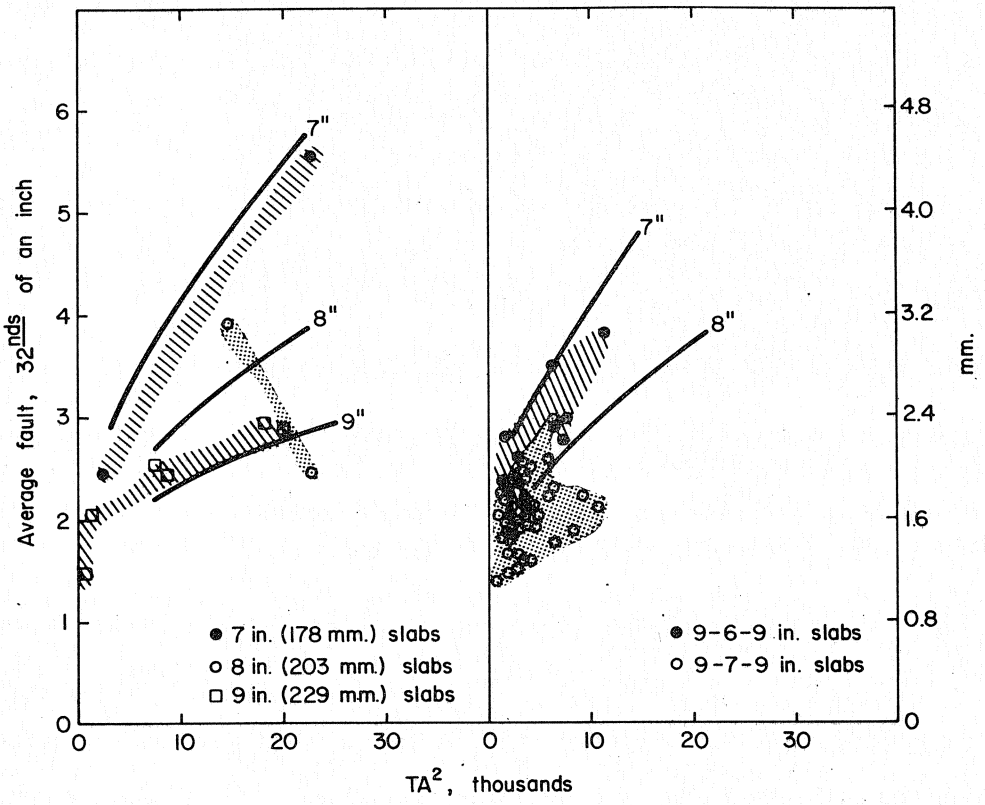


Fig. 3 Joint Faulting in Minnesota, North Dakota, and Wisconsin (15-ft joint spacing, poor subgrade, granular subbase).

APPLICATION OF THE FAULT PREDICTION MODEL

The principle purpose of the fault prediction model is to give more specific guidance than previously available as to whether a particular pavement should be designed with doweled joints. After the designer has selected a slab thickness by using some established design procedure, he may predict the degree of faulting that would occur at the end of a selected time period. If, in his judgement, the predicted faulting is excessive,* dowels should be used.

While the prediction model equation may be used for various design life periods, Fig. 7 and Fig. 8 have been drawn up as example charts for 20 and 30-year designs. These charts predict the degree of faulting that would occur after 20 or 30 years of the specified heavy truck traffic (T on the chart represents the average number of tractor-semi-trailers and combinations per day in one direction).

*See previous discussion, Faulting Criteria.

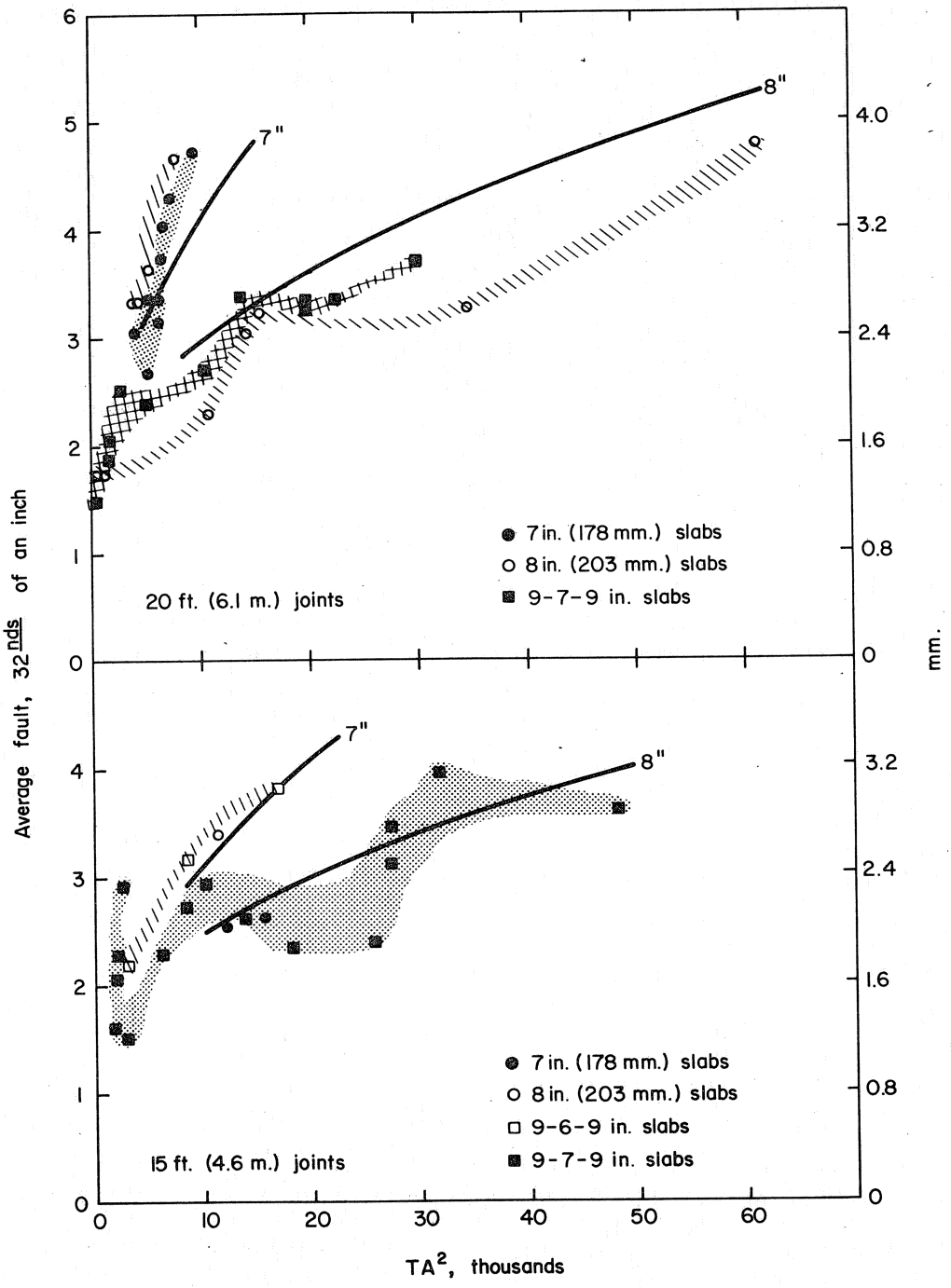


Fig. 4 Joint Faulting in Minnesota, North Dakota, and Wisconsin (good subgrade, granular subbase).

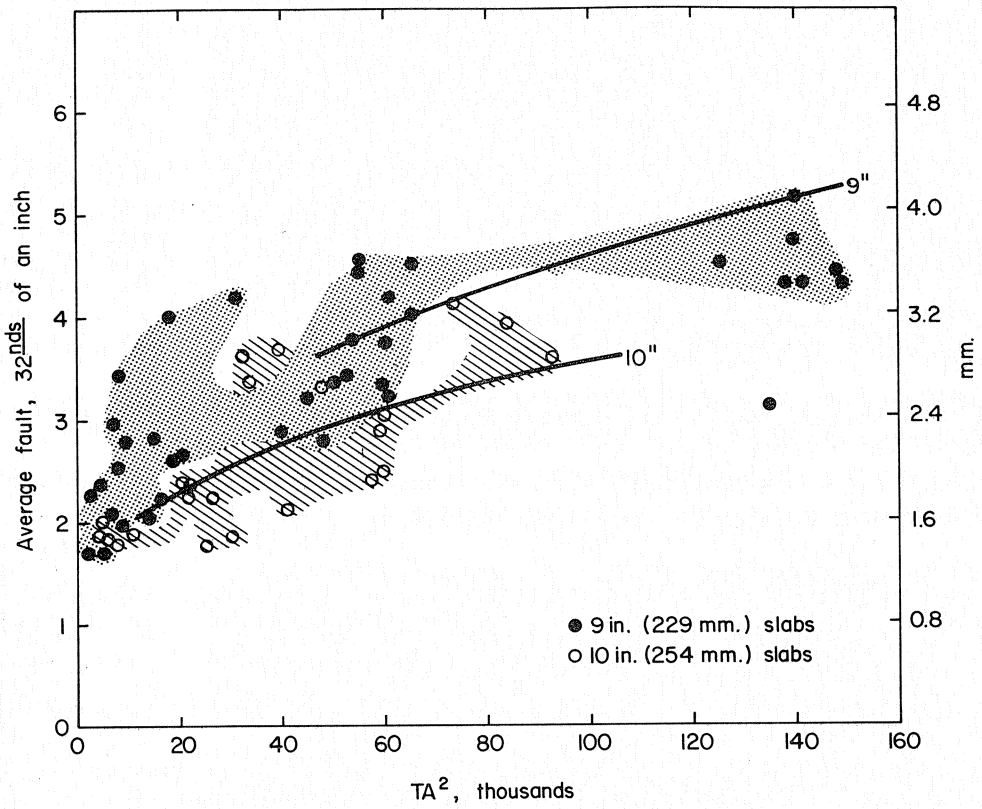


Fig. 5 Joint Faulting in Georgia (30 ft joint spacing, poor subgrade, stabilized subbase).

The dashed-arrow line on Fig. 7 gives an example of the use of the charts. Assume: (1) for a granular subbase and some specified traffic conditions, that an 8-in. (203 mm) slab thickness is indicated by an established design procedure; (2) 200 T's per day (one way) are estimated as the average number for the design period; (3) the selected design period is 20 years; (4) the subgrade soil is a fine-textured silty-clay (poor drainage); and (5) a 15-ft. (4.5 m) joint spacing is proposed. Following the example line through the chart with this sequence of variables, an average fault of 6/32 in. (4.8 mm) is predicted at the end of the 20-year period. For a high-speed, major facility this is considered excessive -- the pavement would reach an unsatisfactory level of faulting in a time period somewhat short of the intended design life. Doweled joints should therefore be specified to prevent faulting.

The following examples indicate that the model is in reasonable agreement with general knowledge about the performance of pavements. First, for many conventional pavements, it is said, based on faulting studies[10] that dowels should be used if the number of tractor-semi-trailers and combinations is more than about 300 per day in 2 directions (150 T's per day in one direction). The chart agrees with this rough criterion for slab thicknesses in the range of 8 to 9 in. (200 to 229 mm) and a fault criterion of 5 or 6 32nds in. (4 or 5 mm). The second example is a 6-in. (152 mm) thick residential street typically carrying not more than 1 or 2 heavy trucks per week (e.g., a garbage truck or ready-

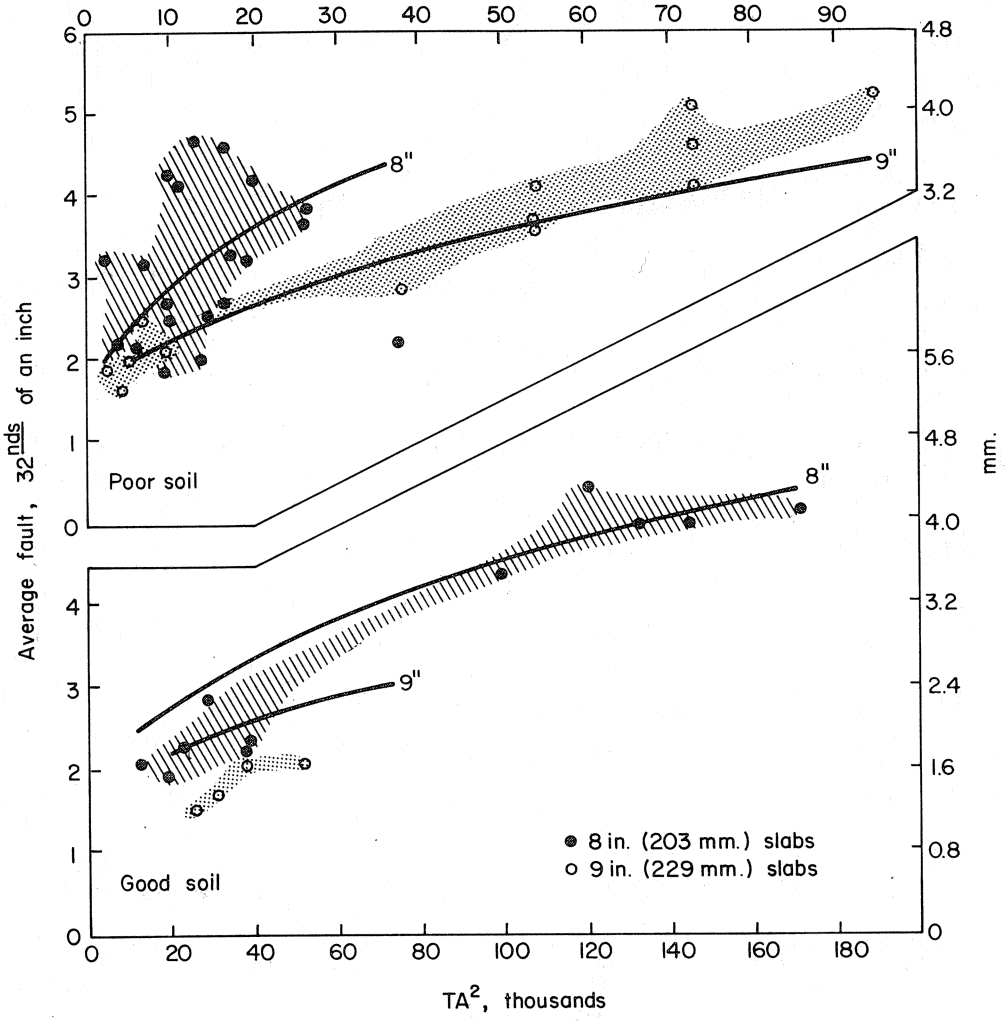


Fig. 6 Joint Faulting in California (random joint spacing averaging 15.5 ft, stabilized subbase).

mix truck with axle loads as heavy as tractor-semi-trailers). The chart shows that dowels are not needed and that faulting will not be perceptible. Pavement performance of undoweled residential streets generally confirms this indication.

A possible second application of the fault prediction model is the reverse of the first -- using it as a thickness design procedure for thick, undoweled pavements. For pavements of conventional thicknesses carrying heavy trucks and thus requiring dowels, the question arises as to how much thicker an undoweled pavement has to be to prevent an objectionable degree of faulting. A thicker pavement should tend to fault less since it would have lower deflections under load, a greater depth of aggregate interlock for load transfer at the joints, and would be subject to less warping. Recently, several thick, undoweled pavements have been constructed on major highways with thicknesses from 11 to 15 in. (280 to 380 mm). Normally the conventional design for these would

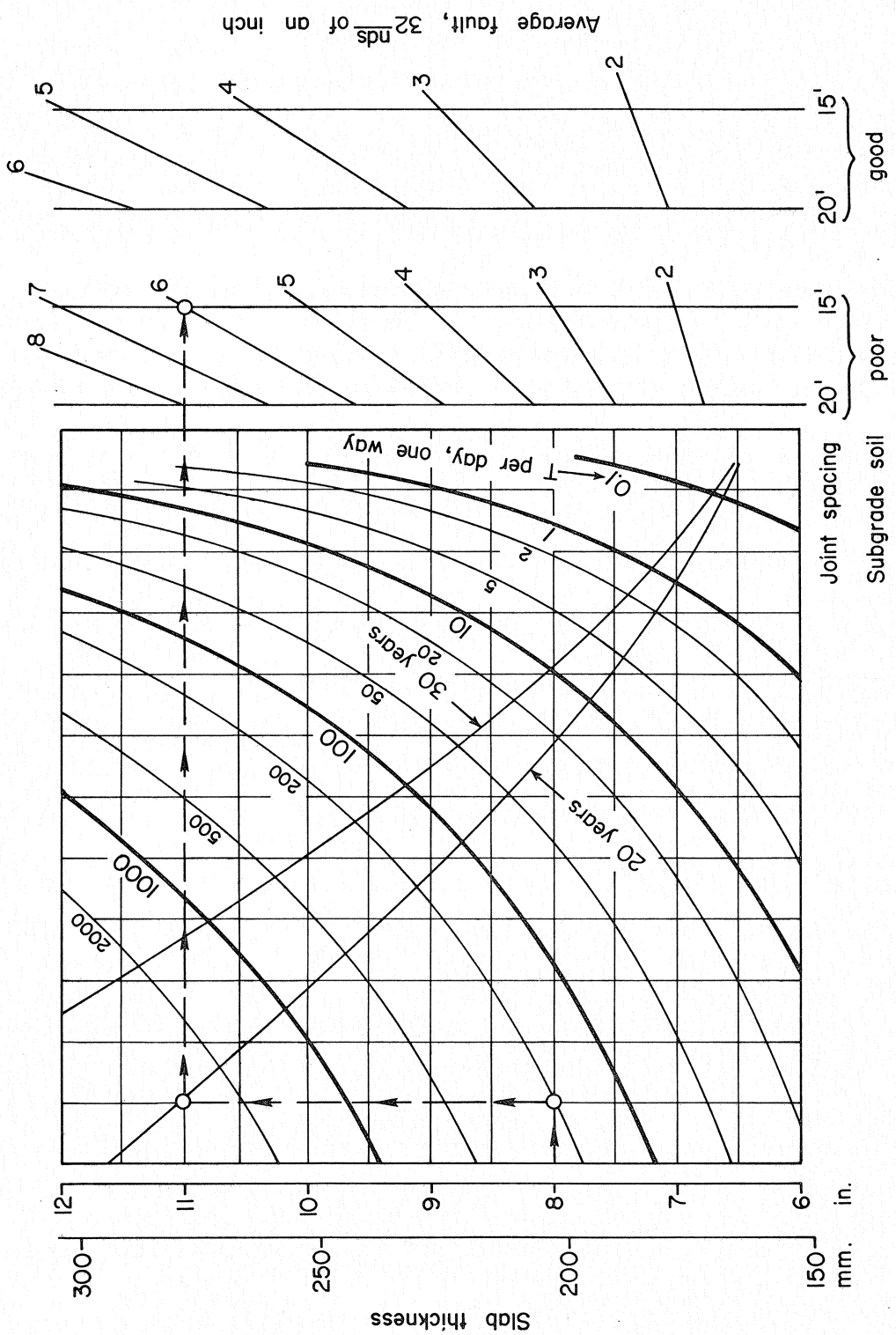


Fig. 7 Estimate of Degree of Joint Faulting at End of 20 and 30 Years (granular subbase).

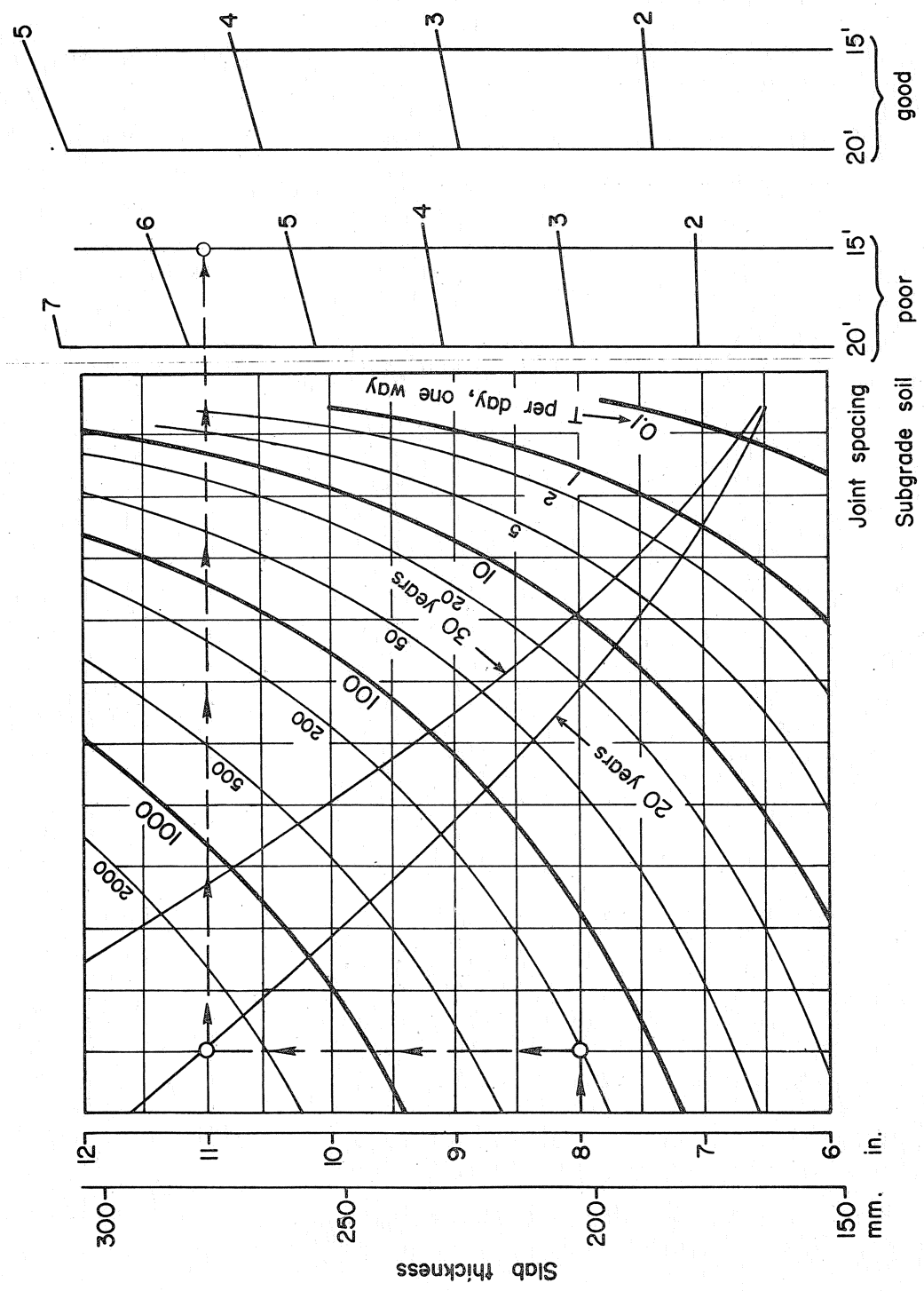


Fig. 8 Estimate of Degree of Joint Faulting at End of 20 and 30 Years (stabilized subbase).

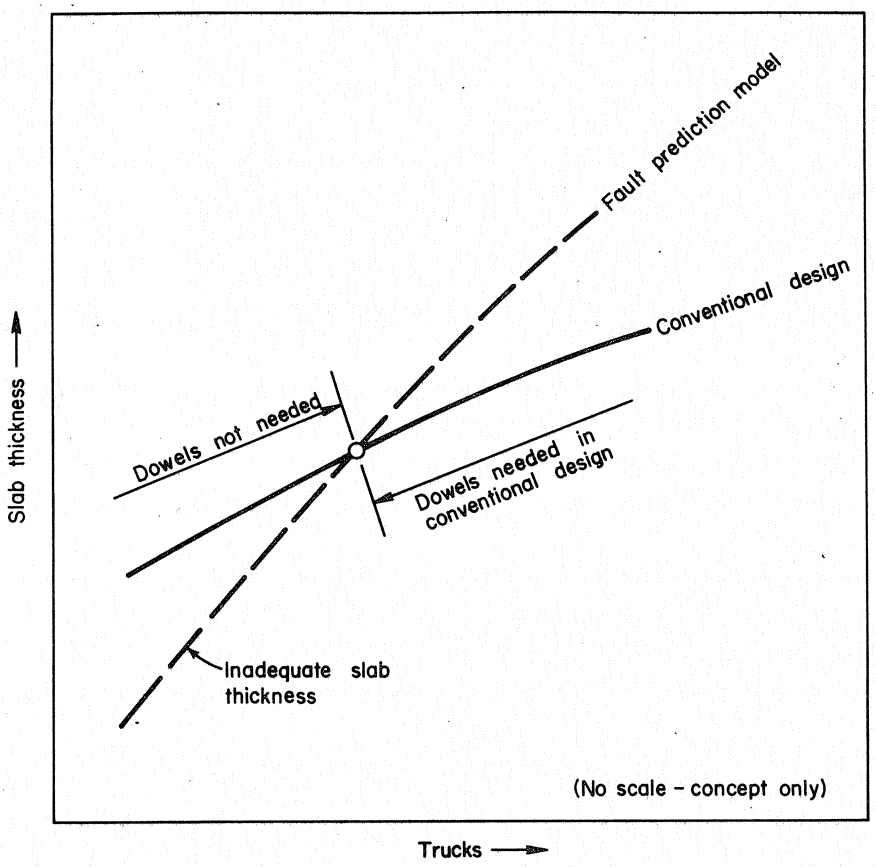


Fig. 9 Concept of Fault Prediction Model.

have been doweled pavements in the 8 to 10 in. (200 to 250 mm) thickness range. It is tempting to extrapolate the findings of this paper to pavement thicknesses greater than the 7-to 10-in. (180 - 250 mm) thicknesses included in the performance study. Recognizing that such extrapolations have not been verified by experience, as the thick pavements are too new to yield useful information, the fault performance model may serve only as a preliminary guide in this relatively unexplored thickness-design problem area.

It is not intended that the model be applied to the thickness design of undoweled, thin pavements. As shown in Fig. 9, the fault prediction model may indicate slab thicknesses considerably less than those indicated by conventional thickness-design procedures for light-traffic facilities. Such thin pavements are likely to fail by a mechanism other than faulting (e.g., flexural cracking due to over-loading) long before they reach a level of objectionable faulting.

Several limitations of the fault prediction model are recognized. The results apply only to conventionally designed pavements and not to pavements with special design features

such as lean concrete or other non-erodible subbase, concrete shoulders, drainage installations, etc., that are used to control the faulting mechanism.* Even for conventionally designed pavements, the model is not expected to be a finalized equation that will fit all conditions. Certainly, there are important effects such as climate, drainage conditions, joint sealing, type of subgrade and subbase that are presently not well defined. However, in view of the fair fit to the available data, the model appears to be useful and indicates the possibility of developing more refined relationships for local areas and specific conditions.

SUMMARY

The main objective of this report is to give an improved estimate of the conditions for which dowels are, or are not, required at joints in concrete pavements.

To accomplish this, the faulting behavior of undoweled pavements was studied. Fault measurements on several hundred pavement sections in 5 states were analyzed in terms of number of heavy truck loads carried, slab thickness, and other variables. The correlation of these data was compared to additional data from 6 states.

It is concluded that the degree of faulting is directly related to the accumulated number of heavy trucks, inversely related to slab thickness, and, to a lesser extent, depends on pavement joint spacing and type of subbase and subgrade.

Various criteria for the tolerable degree of faulting are discussed. Based on the author's observations of ride quality and roadmeter PSI measurements of faulted pavements, it is concluded that the effect of faulting on quality of ride is not well defined by AASHTO serviceability and slope variance concepts. Faulting seems to be a special type of pavement roughness which should be analyzed differently from other forms of roughness in evaluating its effect on pavement serviceability.

Based on a correlation of the faulting data, a method for predicting the degree of faulting is presented. It is intended that the designer, given design information including pavement thickness, be able to predict the degree of faulting and use this information in his decision on whether or not dowels are needed.

A possible second application of the fault prediction model is to use it as a thickness-design procedure for undoweled pavements, that is, pavements designed thicker than conventional doweled pavements to compensate for the absence of dowels.

The fault prediction model, which is an extension and modification of models presented by other investigators, appears to result in improved estimates of faulting of pavements for a greater range of design variables.

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*Discussions of several special design features to control faulting are given in the reports of Woodstrom[3] and Gulden[4].

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