# Control of Faulting Through Joint Load Transfer Design

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The development of mechanistic empirical algorithms for more realistic estimates of anticipated faulting in concrete pavements is described and evaluated. Earlier theoretical investigations are considered, interpreted through more recent finite element results, and calibrated using an extensive data base of field observations. A factor influencing faulting is the dowel-concrete bearing stress, for which an improved method of determination is presented. A procedure is outlined for assessing the need for dowels in both plain and reinforced jointed concrete pavements, also resulting in an estimate of the required bar diameter so that significant faulting is prevented. Application of the procedure is facilitated through use of program PFAULT, which can be implemented on a personal computer.

In the last several years, significant deterioration has been observed in many jointed plain or reinforced concrete pavements (JPCP or JRCP), even though most had been designed in accordance with conventional codes of practice. The major types of distress exhibited are usually faulting, spalling, and corner cracking. Lockup of joints is also fairly frequent and causes the opening of nearby transverse cracks with subsequent deterioration. Joint repair procedures are costly and have contributed to a large proportion of rehabilitation contracts. These observations call for a re-examination of accepted joint design methodologies in light of recent theoretical, analytical, and empirical data.

In the design and construction of doweled or undoweled joints for portland cement concrete (PCC) pavements, a disparity exists among the practices adopted by agencies in the 50 states, as well as among those reported from foreign countries. The main reasons for this are the following:

1. The state of the art with respect to the theoretical treatment of the pertinent problems is still fairly elementary and strictly applicable only to highly idealized conditions;
2. Climatic and geotechnical considerations vary widely from state to state and from country to country;
3. The number, frequency, magnitude, and geometry of applied traffic loadings are considerably different in each locality, whereas concepts used to reduce mixed traffic to a design traffic number are often theoretically unfounded and sometimes fundamentally flawed;
4. As a corollary of 1–3 above, a large degree of empiricism derived from local experience enters the design and construction approaches of each agency.

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This paper reports on some aspects of a broader FHWA study whose main objectives included the development of design guidelines for the prevention of faulting through the proper use of doweled joints in PCC pavements. The approach used to achieve this goal was mechanistic in nature, calibrated with empirical inputs stemming from an extensive data base of field observations. A major factor was the bearing stress developing at the dowel-concrete interface, for which a new method of determination is proposed. In addition, by using nonlinear multiple regression techniques, algorithms were developed for estimating transverse joint faulting, as a function of a wide range of pavement system parameters. A mechanistic evaluation of these statistical formulae has identified several limitations of the current state of the art, and has led to the formulation of pertinent recommendations for future research.

## ANALYTICAL METHODS FOR DOWELED JOINTS

The first rational procedure for the design of doweled joints in concrete pavements was presented by Westergaard in 1928 (*1*). This crude but ingenious method enabled engineers to base on theoretical principles such decisions as those pertaining to the number and spacing of dowels to be used. The analytical treatment assumed that a point load was applied midway between two dowels and that the deflected shape of the loaded side of the joint coincided at all points with the basin formed by the unloaded slab. Thus, all dowels were assumed to be perfectly rigid. The background for this method consisted entirely of Westergaard's earlier analytical studies of the one-slab problem (*2*). Nonetheless, two important new conclusions were reached:

1. Only the two, or at most four, dowels nearest to the load need be considered as active, since the contribution of more remote bars is negligible; and
2. Dowels are effective in reducing the bending stress developed in the loaded slab only if they are spaced closely enough (at less than 2 ft apart).

These two issues remained the prominent foci of the debate that followed in the next several decades, even to the present day.

The Arlington tests (*3*) provided the first documented opportunity for a field study of dowel performance. Their results corroborated Westergaard's conclusions, suggesting that a dowel spacing of closer than 2 ft may be necessary. As indicated by theory, using larger bars at a closer spacing will increase the stiffness of the dowels, thus enhancing their effectiveness in load transfer. This type of spacing should be done judiciously, however, since it may also cause a detrimental increase of restraint to longitudinal warping or curling. Thus, dowels that are too stiff may cause more distress in the pavement slab than would result from their complete omission (*4*).

In their independent investigations of the stress condition existing in and around the dowel bars, Grinter (*5*) and Bradbury (*6*) made reference to Westergaard's single-slab edge loading solution. Their analytical treatments, however, dispensed with Westergaard's original restrictive assumptions and were instead based on a method presented by Timoshenko and Lessels (*7*), which considers the dowel as an infinite beam encased in an elastic medium. This approach is sensitive to a parameter that has been difficult to determine with any degree of accuracy, namely, the modulus of dowel support ($K$). Despite early warnings that such calculations "should be taken as significant qualitatively rather than quantitatively" (*8*), the Timoshenko procedure had been used exclusively in related studies until the introduction of the finite element method (FEM) in the 1970s.

Credit for the prominence of the Timoshenko analysis is generally given to the theoretical and experimental expositions by Friberg published in the late 1930s (*9,10*). This investigator presented a set of design equations for evaluating dowel deflections, moments, and stresses, provided the shear force transferred by the dowel could be determined (*10*). Thus, the concrete bearing stress ($\sigma_b$) arising under the dowel bar (responsible for spalling distress and dowel looseness) is given by the formula

$$\sigma_b = K\Delta_0 \tag{1}$$

where $K$ is the modulus of dowel support ($FL^{-3}$) and $\Delta_0$ is the deflection of the dowel with respect to the concrete at the face of the joint ($L$). Note that the primary dimensions are abbreviated herein as $L$ for length, $F$ for force, and, later, $\Theta$ for temperature. Deflection, $\Delta_0$, may be evaluated from

$$\Delta_o = \frac{P_i}{4\,\beta^3\,E_d\,I_d}(2 + \beta\omega) \tag{2}$$

where

$P_i$ = shear force acting on any particular dowel, transferred across the joint ($F$);
$\omega$ = width of joint opening ($L$);
$E_d$ = modulus of elasticity of the dowel bar ($FL^{-2}$); and
$I_d$ = moment of inertia of dowel bar cross-section ($L^4$).

For solid round bars,

$$\Delta_0 = \frac{\pi d^4}{64} \tag{3}$$

where $d$ is the dowel bar diameter ($L$) and $\beta$ is the relative stiffness of the dowel-concrete system ($L^{-1}$),

$$\Delta_0 = \left(\frac{Kd}{4E_d I_d}\right)^{1/4} \tag{4}$$

In deriving these equations, use was made once again of Westergaard's early theoretical works (*2*), leading Friberg to conclude that dowels at distances greater than an "effective length" ($e$) of 1.8 times the radius of relative stiffness of the slab-foundation system ($l$) from the point of application of the external load were inactive and did not influence the moment at the load point. Recall that $l$ is defined as

$$l = \left(\frac{Eh^3}{12(1 - \mu^2)k}\right)^{1/4} \tag{5}$$

where

$E$ = slab modulus of elasticity ($FL^{-2}$);
$\mu$ = slab Poisson's ratio;
$h$ = slab thickness ($L$); and
$k$ = modulus of subgrade reaction ($FL^{-3}$).

Friberg (*10*) was also the first investigator to suggest that the load transferred by each dowel could be reasonably assumed to decrease linearly with distance from the point of loading. Friberg's additional assumption of a value of 1,000,000 psi/in. for the modulus of dowel support ($K$) for all pavement systems elicited considerable discussion. Thus, Grinter postulated that $K$ ranged between 300,000 and 1,500,000 psi/in. but also anticipated a "maximum variation of a hundred-fold" in the value of this parameter (*11*). Less attention was paid to Friberg's assertion of an effective length of $1.8l$, despite the fact that this would not be in accordance with Westergaard's own conclusions (*1*). For a typical $l$-value of 36 in. and dowel spacing of 2 ft, Westergaard's assumption that only the two dowels closest to the load are active would correspond to an effective length of only $1.0l$. Data presented by Sutherland (*12*) also supported a shorter effective length.

A potential for a real breakthrough in analytical methods for doweled joints was created in the late 1970s with the introduction of the FEM. Although the capabilities of this versatile numerical tool are far from exhausted even to this day, several important observations have already been made. Tabatabaie et al. (*13,14*) were among the first to present a finite element model of the doweled joint; they concluded that "only the dowels within a distance $1.0l$ from the center of the load are effective in transferring the major part of the load." They concurred with Friberg's adoption of a linear approximation to the dowel shear force diagram, but suggested that this should begin with a maximum under the load and diminish to zero at a distance of $1.0l$ from this point.

Finite element studies by Tabatabaie also led to the conclusion that the dowel diameter ($d$) and concrete modulus of elasticity ($E$) have a very significant effect on the maximum dowel deflection and concrete bearing stress. Slab thickness ($h$) and subgrade modulus ($k$) play a much lesser role. Earlier laboratory investigations by Marcus (*15*) and Teller and Cashell (*16*) had also pointed out the same effects. The following relationship for the critical concrete bearing stress ($\sigma_{bc}$) was developed, "based on the result of two- and three-dimensional" finite element analyses (*14*):

$$\sigma_{bc} = \frac{(800 + 0.068\,E)}{d^{4/3}}(1 + 0.355\,\omega)sP\alpha_i \tag{6}$$

where

$s$ = dowel spacing ($L$);
$P$ = applied wheel load ($F$); and
$\alpha_i$ = load location coefficient (dimensions unclear), i.e., 0.0091 for edge load, 0.0116 for protected corner load, and 0.0163 for unprotected corner load.

All parameters in this expression employ the kip as the unit of force and the inch as the unit of length, whereas $\sigma_{bc}$ is expressed in pounds per square inch.

## PROPOSED METHOD FOR MAXIMUM BEARING STRESS DETERMINATION

Recent research has provided additional evidence supporting the conclusion reached by Tabatabaie and others that the effective length over which dowels are active in load transfer is considerably shorter than was assumed by Friberg. Joints designed under the assumption that all dowels within $1.8l$ from the applied load are effective have exhibited unacceptable performance (e.g., substantial faulting (*17*)), indicating that a more conservative approach is necessary. Therefore, a comparison was conducted during the present study between Tabatabaie's formula (which assumes an effective length of $1.0l$) and Friberg's original equation. This comparison is shown in Figure 1 for a typical PCC pavement section under a single-wheel 9-kip edge load. The pertinent system parameters were as follows: $E$ = 4,000,000 psi; $\mu$ = 0.15; $h$ = 10 in.; $k$ = 50 psi/in.; $l$ = 51.1 in.; $d$ = 0.75 in.; $s$ = 12 in.; $K = 1.5 \times 10^6$ psi/in.; $E_d = 29 \times 10^6$ psi; $\omega$ = 0.2 in. The assumed percentage of load transferred across the joint ranged from 0 to

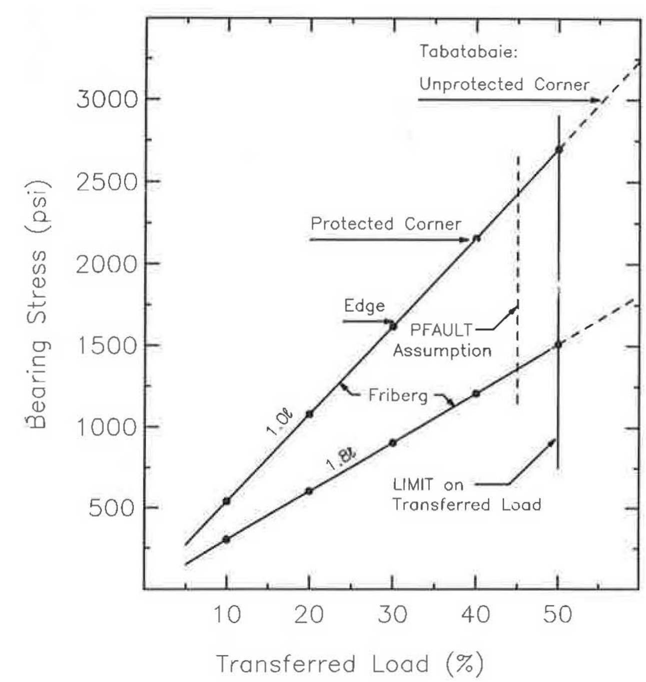


**FIGURE 1 Bearing stress formula for a typical PCC pavement section under a single-wheel 9-kip edge load. Data are adapted from Friberg and Tabatabaie (*9,10,13,14*).**

the maximum value of 50 percent. The maximum bearing stress, calculated using Friberg's equation on the basis of his linear diminution assumption over an effective length, was $1.8l$. This approach yields a straight line, whose slope is entirely dependent on the assumed effective length. Therefore, decreasing this length to $1.0l$ results in a second straight line, located above the $1.8l$ line and having a steeper slope. Clearly, a more conservative estimate of the maximum bearing stress (i.e., a greater value) is obtained with the shorter assumed effective length.

The actual percentage of load transferred across a doweled joint may be estimated by comparison of Friberg's predictions to the calculated maximum bearing stress according to Tabatabaie et al. (*14*). These percentages are also shown in Figure 1, for the three loading locations considered, i.e. loading at an edge, at a protected corner, and at an unprotected corner. It is apparent that a high assumed percentage of load transferred (close to 50 percent) leads to a conservative estimate of the maximum bearing stress. A value of 45 percent was adopted in this study to account for the possibility of some dowel looseness.

It is interesting to note that both the Friberg and Tabatabaie formulae for the determination of bearing stress can be rewritten as

$$\sigma_b = A(\text{structural}) \times B(\text{load}) \tag{7}$$

The first term, $A$, is entirely determined by the structural characteristics of the pavement system, whereas the second term, $B$, quantifies the transferred load. It would be reasonable to expect that at least the $A$-term should be the same according to the two equations. This, however, is not true, as illustrated by the following calculations for the case considered above:

1. Tabatabaie:

$$A = \frac{(800 + 0.068\ E)}{1000\ d^{4/3}} (1 + 0.355\ \omega)$$

$$= 1.6849 \text{ (units unclear, but should be in.}^{-2}) \tag{8}$$

2. Friberg:

$$A = \frac{K(2 + \beta\omega)}{4\beta^3 E_d I_d}$$

$$= 2.5812 \text{ in.}^{-2} \tag{9}$$

The discrepancy in this case is on the order of 35 percent. Friberg's term is considered superior, however, since it is theoretically based and dimensionally consistent. Note that parameters $K$ and $E_d$ do not enter Tabatabaie's formula. Dowel support was not explicitly prescribed in his three-dimensional finite element analysis, whereas a value of 1,500,000 psi/in. was assumed as "conventional" in his two-dimensional investigations. A value of $E_d$ of 29,000,000 psi was assumed by Tabatabaie.

Turning now to the load $B$-term in these equations, it is possible to express this as follows:

$$B = P_t = P \times \text{TLE} \times f_d \tag{10}$$

where TLE is the transferred load efficiency, expressing the ratio of the load transferred across the entire length of joint ($P_T$) to the total applied load ($P$), that is,

$$\text{TLE} = \frac{P_T}{P} \times 100 \text{ percent} \tag{11}$$

and $f_d$ is a dimensionless distribution factor indicating how much of the total transferred load acts on any given dowel bar (usually on the critical bar), i.e.,

$$f_d = \frac{P_i}{P_T} \times 100 \text{ percent} \tag{12}$$

Note that the distribution factor ($f_d$) does not depend on the amount of load transferred but is entirely the consequence of the dowel spacing and of the assumptions regarding the effective length and the linear diminution of the dowel shear forces with distance from the applied load. For the case considered in Figure 1, Friberg's linear diminution approach leads to distribution factor values for the critical dowel ($f_{dc}$), of 13 percent for an effective length of 1.8$l$, and 23 percent for 1.0$l$. This illustrates once again the conservative nature of the shorter effective length. Tabatabaie, on the other hand, defines the transferred load by any given dowel as:

$$B = P_i = \alpha_i s P \tag{13}$$

In this expression, TLE is not explicitly stated, whereas the load location is accounted for. Note that here both $P$ and $P_i$ are expressed in pounds, and $s$ is expressed in inches, whereas $\alpha_i$ takes the values quoted above. Assuming TLE = 45 percent, Tabatabaie's equation yields the following distribution factors for the critical dowel:

$f_{dc}$ = 24 percent for edge loading,

31 percent for protected corner loading, and

43 percent for unprotected corner loading.

Friberg presumably considered edge loading only, and for this loading condition good agreement is observed. The distribution factor for an unprotected corner is almost twice as large as that for edge loading. This observation is confirmed by results shown in Figure 2, obtained using a recently modified version of finite element computer program ILLI-SLAB (Ioannides and Korovesis from a paper in this Record). The pavement section considered in developing Figure 1 was retained; a contact pressure of 90 psi was assumed. First, a single-wheel load was applied at the edge or at the corner of the slab. Because the broader FHWA study was primarily concerned with 18-kip single axle loading (dual tires), the variation of the distribution factor under such a load (applied at the corner) was also examined. The effective length ($e$) for both single-wheel loadings (i.e., at the edge and at the unprotected corner) is found to be closer to 1.0$l$, instead of to 1.8$l$, as concluded by Friberg (*10*). On the other hand, the single-axle (dual-wheel) load at the corner leads to $e \approx 2.0l$, indicating that the effective length is not constant but it is sensitive to the gear configuration.

Referring to the geometry of the transferred load distribution diagram assumed by Friberg, it can be shown that the value of $f_{dc}$ can be determined to a good approximation using, for edge loading,

$$f_{dc} = \frac{s}{e} \tag{14}$$

and for corner loading,

$$f_{dc} = \frac{2s}{e + s} \tag{15}$$

Both Figure 2 and Equations 14 and 15 suggest that for the case of a single-wheel load, the critical distribution factor pertaining to a corner load is almost twice as large as the corresponding edge loading value. Furthermore, under corner loading conditions, a single-wheel leads to responses similar to those obtained using a single-axle (dual-wheel) load, since

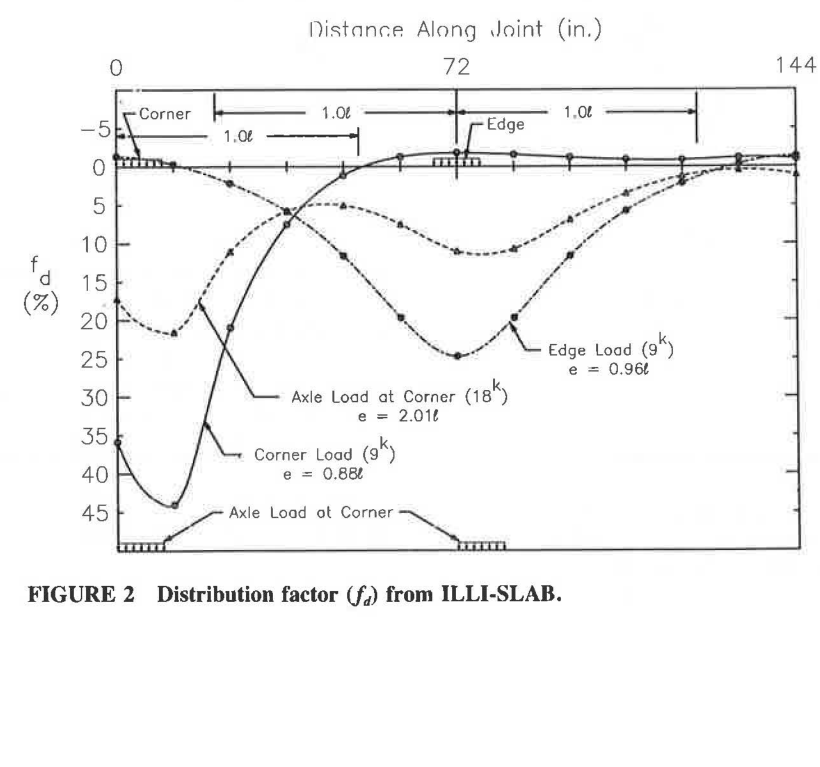


**FIGURE 2 Distribution factor ($f_d$) from ILLI-SLAB.**

the product ($f_{dc} \times P$) in Equation 10 is approximately the same in both cases. Consequently, a single wheel applied at the corner can be regarded as the critical (design) loading condition.

Using Equation 15 for $f_{dc}$ in the load $B$-term, and Friberg's structural $A$-term, the maximum bearing stress may be determined in a manner that accounts for both the location of the load and the rest of the parameters entering Friberg's theoretical development. Results obtained during this investigation using ILLI-SLAB indicate that TLE is only slightly affected by load location and is generally about 42 ± 1 percent. The assumed value of 45 percent is, therefore, somewhat conservative, as desired.

The proposed bearing stress determination method outlined above has been incorporated into an interactive computer program called PFAULT, prepared for FHWA for the estimation of faulting (*18*). Given below is the formula used in PFAULT to determine the critical bearing stress:

$$\sigma_{bc} = \frac{K(2 + \beta\omega)}{4\beta^3 E_d I_d} \times P \times \mathrm{TLE} \times f_{dc} \tag{16}$$

in which $f_{dc}$ is determined by using Equation 15, assuming $e = 1.0l$. The dowel spacing is set to 12 in. since the PFAULT data base contained only such sections. As noted above, $P$ = 9,000 lbs, and TLE = 45 percent. The joint width ($\omega$) is calculated by using Equation 17, as described below. The following inputs are tentatively adopted since they had also been assumed in analyzing the field data that provided the data base for PFAULT: $K$ = 1,500,000 psi/in.; $E_d = 29 \times 10^6$ psi.

The sensitivity of the proposed method for determining the maximum bearing stress was investigated for a wide range of $d$, $s$, $h$, and $k$ values (*18*). Dowel diameter ($d$) clearly emerged as the most important of these variables. The reduction in $\sigma_{bc}$ with increasing $d$ is particularly dramatic at the smaller dowel diameters ($d \leq 1$ in., say). The sensitivity of $\sigma_{bc}$ to the rest of the parameters considered is considerably smaller. It follows, therefore, that since the required dowel diameter is greatly affected by the assumed value of modulus of dowel support ($K$) it is imperative that a good estimate of the latter be obtained before proceeding to a faulting calculation. Unfortunately, no method exists today for this purpose.

## CALIBRATION OF MECHANISTIC-EMPIRICAL FAULTING ALGORITHMS

In conventional practice, the amount of transverse joint faulting for PCC pavements is commonly estimated by developing mechanistic-empirical algorithms by using a data base containing in-service pavement data. Accordingly, the NCHRP Project 1-19 (COPES) data base (*17*) was employed in the calibration of PFAULT. Nonlinear multiple regression techniques from the SPSS software package (*19*) were applied in describing the variation of measured faulting with respect to three mechanistic parameters—concrete dowel bearing stress, joint opening, and corner deflection. This can contribute toward more realistic and accurate estimates of anticipated transverse joint faulting, in both doweled and undoweled pavements. Guidelines for dowel design may also be formulated on the basis of such a mechanistic-empirical approach.

### Estimating Faulting in Doweled Pavements

The COPES data base (*17*) contains information from doweled pavement sections from the following four states: Illinois, Louisiana, Minnesota, and Nebraska. Both JPCP and JRCP are included. In addition, 12 sections of JRCP from the experiment at Rothsay, Minnesota, were incorporated into the PFAULT data base. These were designed with 27-ft joint spacing over granular, asphalt, and cement-treated bases.

The maximum concrete bearing stress was computed using the new proposed method of calculation. As indicated by Equation 16, this involves the joint width ($\omega$), which is known to be a highly variable parameter (*20*). The mean value of the joint opening caused by temperature and moisture changes in the slab is computed in PFAULT using the following expression:

$$\omega = CL\,(0.5\alpha\Delta T + \varepsilon_s) \tag{17}$$

where

$\alpha$ = thermal coefficient of contraction of concrete ($\Theta^{-1}$) (e.g., $5.0 \times 10^{-6}$ ε/°F);
$\varepsilon_s$ = drying shrinkage coefficient of concrete (approximately $0.5 - 2.5 \times 10^{-4}$ ε, specified as $1.5 \times 10^{-4}$ ε in PFAULT);
$L$ = transverse joint spacing ($L$);
$\Delta T$ = temperature range ($\Theta$), i.e., maximum mean daily air temperature in July minus minimum mean daily air temperature in January; and
$C$ = dimensionless empirical adjustment factor caused by slab-base frictional restraint (PFAULT assumes that $C$ = 0.65 for stabilized base, and $C$ = 0.80 for granular base).

Equation 17 is a modification of a regression equation, based on "limited field data," presented by Darter (*21*) to provide an approximate estimate of $\omega$ "despite the many complexities involved." Introduction of the factor of 0.5 in Equation 17 aims at reflecting a more realistic average condition of joint opening. The faulting algorithm established in this manner for doweled pavements is as follows:

$$\begin{aligned}\mathrm{FAULT} = \mathrm{ESAL}^{0.5377}\,(2.2073 &+ 0.002171\,\sigma_{bc}^{0.4918}\\ &+ 0.0003292\,L^{1.0793} - 2.1397\,k^{0.01305})\end{aligned} \tag{18}$$

Statistics: $R^2 = 0.53$; $SEE = 0.05$ in.; $n = 280$.

In this formula, ESAL denotes the cumulative 18-kip equivalent single-axle load applications (in millions), $\sigma_{bc}$ is expressed in pounds per square inch, and $L$ in feet. The effective modulus of subgrade reaction ($k$) is provided in pounds per square inch per inch, whereas FAULT is expressed in inches. Several climatic variables (e.g., precipitation and freezing index) were also introduced into the regression analysis, but they did not show any statistical significance. A plot of observed versus estimated faulting is presented in Figure 3.

Among the four parameters used for estimating FAULT, a sensitivity analysis showed that $\sigma_{bc}$ and $L$ are the most significant. Adjusting the joint spacing, slab thickness, and

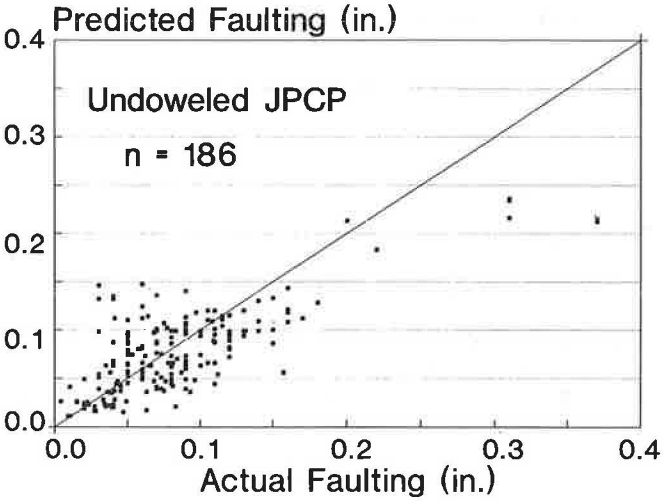


**FIGURE 3 Predicted versus actual faulting: nationwide model for undoweled pavements (PFAULT version 1.2).**

subgrade support to keep the ratio of $L/l$ below 6, for instance, may therefore help in fault control in JPCP (*22*).

The expression in Equation 18 is primarily a descriptive tool whose predictive capability is not high. This is especially true when a small number of particular sections are considered. As with all statistical algorithms, however, agreement between observed and estimated responses may be expected to increase as the number of test cases considered increases. A review of similar statistically based tools, such as PDMAP, COLD, and PMARP, indicated that reliability problems are widespread, particularly when these are applied to geographical regions other than those included in the original data base (*23,24*). Therefore, the following major deficiencies of the faulting algorithm in Equation 18 must be kept in mind by prospective PFAULT users:

1. Because of the limited number of climatic zones in the data base, climatic variables did not appear significant enough to warrant their explicit inclusion in the formula derived.
2. A variety of other situations existed, in which the range of some of the variables was not sufficient (e.g., permeable base type, subgrade type, edge support, and subdrainage).
3. The possible interactions between individual factors entering the algorithm are ignored.
4. The formula involves the ESAL concept, which is urgently in need of reconsideration (*25*).
5. A number of the implicit assumptions made during the development of this statistical algorithm and its application in PFAULT have been mere intelligent and educated guesses.

In examining the PFAULT data base, deficiencies in the state of the art have dictated choices that may or may not apply to any one situation. In order of their importance, such selections include a universal value of the modulus of dowel support ($K = 1.5 \times 10^6$ psi/in.); a constant value of the radius of the applied load ($a = 5.64$ in.); an empirical-statistical method for calculating joint width (Equation 17); and fixed values for the effective length ($e = 1.0l$) and transferred load efficiency (TLE = 45 percent). The method employed in the determination of the subgrade modulus $k$ was also arbitrary and inconsistent. This method was often based on correlations with soil classification groups or other soil properties, whereas on other occasions it was determined by application of elastic theory. The procedure followed involved increasing $k$ when a treated base was used, as recommended by the Portland Cement Association. This may be a conventional approach, but it has severe limitations as revealed by finite element investigations (*26*), especially for unbonded slab/base interfaces. Use of Equation 18 in cases involving undue extrapolation beyond the data range used in its generation is, therefore, to be avoided. This is particularly true for open-graded drainable bases.

As a result of the limitations noted, the $R^2$ value of the algorithm in Equation 18 is rather low. Previous attempts at deriving similar algorithms were also fraught with similar weaknesses (*18,21*). It may be expected that substantial progress in the state of the art in mechanistic analysis methods would allow the elimination of the sources of the deficiencies outlined above. In the meantime, a much simpler predictive algorithm proposed by Snyder (*27*) appears promising. This algorithm involves a single independent variable, namely load transfer efficiency in terms of deflection ($LTE_\delta$, defined as the ratio of the deflections on the unloaded and loaded sides of a joint), and displays a considerably higher $R^2$ value. Snyder's proposal is as follows:

$$\text{FAULT} = 23.37 - 0.1288\ \text{LTE}_\delta + 141{,}900\ \text{LTE}_\delta^{-3.807} \tag{19}$$

Statistics: $R^2 = 0.691$; $SEE = 0.057$ in.; $n = 140$.

In this expression, FAULT is expressed in hundredths of an inch, and $LTE_\delta$ is expressed in percent. Note that this relationship is linear, since the third term is negligible for $LTE_\delta$ values in excess of 20 percent. A significant improvement of Snyder's model can result if the endurance of $LTE_\delta$ over time is investigated and incorporated into the proposed faulting algorithm.

### Estimating Faulting in Undoweled Pavements

Prevention of faulting in undoweled pavements relies on effective load transfer through aggregate interlock. The degree and long-term endurance of load transfer by aggregate interlock is affected by the size of joint opening, slab support, thickness of slab, coarseness, angularity and hardness of concrete aggregates, and number of applied load repetitions (Ioannides and Korovesis from a paper in this Record). The slab corner deflection ($\delta_c$) may provide a useful indicator of ensuing deterioration of load transfer efficiency by aggregate interlock and of impending distress because of pumping and faulting (*17*). According to Westergaard (*2*), this deflection is given by

$$\delta_c = \frac{P}{kl^2}\left[1.1 - 0.88\left(\sqrt{2}\ a/l\right)\right] \tag{20}$$

in which $a$ denotes the radius of the applied load ($L$). A refined expressio ı for $\delta_c$ (based on finite element results) was presented by Ioannides et al. (*28*). PFAULT assumes that $a$ = 5.64 in., which for $P$ = 9,000 lb corresponds to a contact pressure of 90 psi. Furthermore, erodibility factors for the base and subbase materials (ERODF) are introduced. These are similar to the loss of support coefficients recommended by AASHTO (*29*), as follows:

1. Granular material: ERODF = 2.5;
2. Asphalt-treated base: ERODF = 2.0;
3. Cement-treated base (without granular subbase): ERODF = 1.5;
4. Cement-treated base (with granular subbase): ERODF = 1.0; and
5. Lean concrete: ERODF = 0.5.

Regrettably, no permeable base courses were included in the PFAULT data base, even though these are expected to have a major effect on reducing faulting.

The nonlinear regression technique referred to above for doweled pavements was employed in a similar fashion for undoweled pavements. Only plain concrete pavement sections were available in the NCHRP 1-19 data base for undoweled pavements (*17*). The data base included pavement sections from the following states: Georgia, Illinois, Louisiana, Utah, and California. Two extra sections from New Jersey and Michigan were also added. The data base was expanded by 24 additional sections from California, which included the following conditions: half-joint spacing (7.8 ft), thicker slab (11.4 in.), and lean concrete base (effective $k$-value = 591 psi/in.). Furthermore, 12 sections from the experiment at Rothsay, Minn., were also incorporated. These sections had 27-ft joint spacing over granular-, asphalt-, and cement-treated bases. The resulting nationwide algorithm for estimating faulting in undoweled pavements is given as follows:

$$\begin{aligned}\text{FAULT} = \text{ESAL}^{0.3157}[&0.4531 + 0.3367\,\omega^{0.3322} \\ &- 0.5376(100\,\sigma_c)^{-0.008437} \\ &+ 0.0009092\,FI^{0.5998} + 0.004654\,\text{ERODF} \\ &- 0.03608\,\text{EDGESUP} \\ &- 0.01087\,\text{SOILCRS} \\ &- 0.009467\,\text{DRAIN}]\end{aligned} \qquad (21)$$

Statistics: $R^2$ = 0.55; $SEE$ = 0.03 in.; $n$ = 186.

In this formula

$FI$ = mean air-freezing index in °F days;
EDGESUP = numerical indicator of type of edge support, e.g., 0, if no edge support exists and 1, if edge beam/tied concrete shoulder exists;
SOILCRS = numerical indicator of AASHTO subgrade soil classification, e.g., 0, if A-4 to A-7 and 1, if A-1 to A-3; and
DRAIN = numerical indicator of drainage provided, e.g., 0, if no edge subdrains exist and 1, if edge subdrains exist.

Note that in Equation 21, the mean transverse joint opening ($\omega$) and the Westergaard corner deflection ($\delta_c$) are expressed in inches. Figure 4 shows a plot of observed versus estimated faulting for this formula. A sensitivity study revealed that the effects of $\omega$, ERODF, and EDGESUP are the most pronounced (*18*).

The comments made above with respect to the nature of statistical algorithms and the limitations of Equation 18 apply here as well. Although the effect of some climatic variables is included in this case, caution is still warranted in the application of the resulting algorithm. Unwarranted extrapolation beyond the data range from which Equation 21 was generated is to be avoided, as always. Nonetheless, this mechanistic-empirical tool may be useful in long-term performance evaluations and design applications within the range of data.

The interactive computer program PFAULT, mentioned above, incorporates both faulting algorithms developed in this study, i.e., Equations 18 and 21. The program is available in an IBM-PC compatible version with pertinent documentation. An input guide is included in the report by Heinrichs et al. (*18*).

## GUIDELINES FOR THE USE OF DOWELS

Mechanistic-empirical algorithms for estimating faulting can be used in assessing the need for dowel bars and determining the required diameter for pavements similar to those included in the data base used in their development. The application of the PFAULT formulae in this manner is described below.

### Jointed Reinforced Concrete Pavements

Dowels are always recommended for JRCP caused by longer joint spacing and expected wider joint opening. The required dowel diameter may be determined using PFAULT, if a threshold faulting level is selected. According to the NCHRP 1-19 data base (*17*), joint faulting in rough pavements [present serviceability index (PSI) ≤ 3.0] was about 0.26 in. In view of the weakness of the correlation in Equation 18, a design faulting magnitude of approximately 0.13 in. is suggested, subject to further verification.

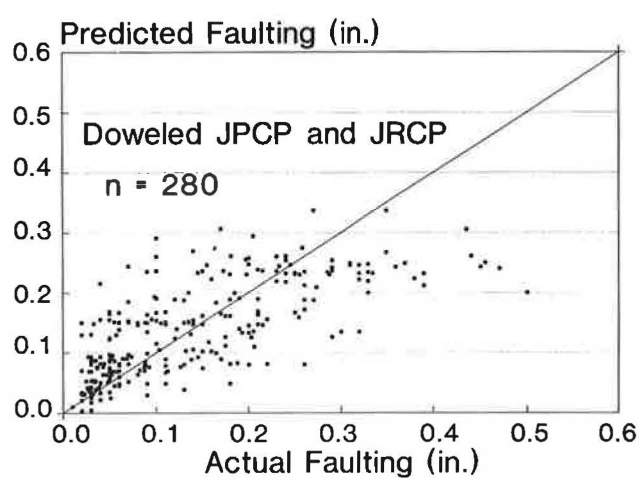


**FIGURE 4 Predicted versus actual faulting: nationwide model for doweled pavements (PFAULT version 1.2).**

### Jointed Plain Concrete Pavements

Dowels often are omitted in the design of JPCP, and aggregate interlock is relied on for load transfer. Aggregate interlock alone often is inadequate, and many such pavements develop serious pumping and faulting. The need for dowels may be assessed by using PFAULT. Referring again to the NCHRP Project 1-19 data base (*17*), JPCP offered a rough ride and exhibited faulting of about 0.13 in. A threshold value of about 0.07 in. may be appropriate for design purposes, subject to further verification. If faulting is excessive, the required dowel diameter may be determined as outlined above for JRCP.

## CONCLUSIONS

This paper presents some of the findings of a study conducted for the FHWA, to provide a synthesis of the current state of the art with respect to PCC pavement design, incorporating the results of recent pertinent investigations (*18*). In particular, design guidelines are outlined for assessing the need for load transfer at transverse joints, so that significant faulting is prevented. The proposed procedure can be used in determining whether dowels are needed and in selecting their appropriate size. Theoretical considerations and interpretation of an extensive field data base have led to the development of descriptive mechanistic-empirical algorithms for estimating the expected transverse joint faulting in both plain and doweled concrete pavements. Within the constraints imposed by their statistical nature, the application of these tools is facilitated by the user friendly micro-computer program PFAULT. At the heart of this procedure is a new method for calculating the maximum bearing stress developing at the dowel-concrete interface, which couples earlier theoretical investigations with more recent results from finite element studies.

## ACKNOWLEDGMENTS

The investigations for this paper were supported in part by a grant from FHWA, U.S. Department of Transportation. The Project Manager was Stephen Forster. The authors also acknowledge the contributions of K. W. Heinrichs and M.-J. Liu, formerly graduate research assistants at the University of Illinois, in the statistical analyses performed.

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*Publication of this paper sponsored by Committee on Rigid Pavement Design.*