Comments on Modified Models for Maintenance and Rehabilitation Planning

MOSHE LIVNEH

Various agencies have adopted deterministic models to evaluate short-range and long-range maintenance and rehabilitation planning programs. For problems at network level, these models are expressed in terms of only a single pavement condition index (*PCI*), or present serviceability index (*PSI*). Unfortunately, a pavement state cannot be evaluated by a single parameter; thus, the use of a single parameter may lead occasionally to misleading results at the output stage of the planning program. Family models that take into account other pavement-state parameters are therefore needed. It is suggested that the *PCI* rating method be used together with three additional parameters, which are inserted in formulated models. These parameters are needed for governing the deterioration rate over time (or as a result of traffic); the structural rehabilitation solutions for various pavements at a given *PCI* value; and the ride-comfort properties of various pavements at a given *PCI* value. Various tentative values as gathered on the basis of airfield pavements are suggested. Naturally, these values should be considered as no more than preliminary ones, requiring more specific correlation work to specific site observations. It is, however, highly recommended that these values be used at least for conducting sensitivity analyses on the various outputs derived from any pavement management calculations.

Various agencies have adopted deterministic models to evaluate short-range and long-range maintenance and rehabilitation planning programs. Generally, three models are involved in this matter:

1. the deterioration model, i.e., the deterioration of a pavement over time under a given loading pattern, environmental conditions, and maintenance and rehabilitation policy;
2. the agency costs model, i.e., the maintenance and rehabilitation expenditures over time; and
3. the user costs model, i.e., the user costs concerning vehicle operation, travel time, comfort, and safety.

For network-level problems, the above three models are expressed only in terms of a single pavement condition index, usually the present serviceability index (*PSI*) or pavement condition index (*PCI*). A pavement state, however, cannot be evaluated by a single parameter. The deterioration mechanism involves these two interrelated mechanisms: the functional deterioration mechanism related to ride-quality states, and the structural deterioration mechanism related to bearing-capacity states. Roughness may be considered as the major characteristic for the functional deterioration, and the *PSI* may thus be regarded as a reasonable expression of the functional conditions. The rutting and cracking, on the other hand, may be considered as only a partial characteristic of the structural deterioration. This means that the *PCI* cannot be regarded as a single satisfactory expression of the structural conditions, and a supplementary index related to nondestructive tests (NDT) data or destructive tests (DT) data is needed.

Thus, the single-parameter approach, which uses *PSI* or *PCI* for expressing both functional and structural conditions, has significant limitations with respect to the accuracy of the output results; therefore, a modification of these models is needed even for network-level problems. This modification should involve at least another condition indicator, besides the *PSI* (roughness) and the *PCI*. This additional indicator is, of course, related to the structural strength of the pavement as derived from deflection-bowl measurements or from in situ bore-hole testing. The fundamentals of this basic approach are discussed herein.

Transportation Research Institute, Technion-Israel Institute of Technology, Haifa, Israel.

BASIC DETERIORATION FUNCTION

Various deterioration functions have been introduced over the years. However, not all of them seem to be logical deterioration functions. A logical deterioration function should recognize that a pavement's rate of deterioration changes during its service life and that the deterioration process proceeds in accordance with certain boundary conditions at the beginning and the end of its life. The general logical shape of this function is an S-shaped curve (Figure 1). "The deterioration curve shown in this figure illustrates that early in a pavement's life, it is stable and of very high quality, but with time it begins to deteriorate, reaching a point where this deterioration increases significantly over a short time span. As it approaches the end of its useful life, the deterioration rate begins to stabilize, but the pavement is considered to be in poor condition. Nevertheless, it supports some minor level of traffic." (*1*) It is reasonable to assume the following for this kind of behavior:

$$\frac{d(\Delta S/\Delta Sf)}{dt} = \frac{d(\Delta S/\Delta Sf)}{dn} = 0 \tag{1}$$

for $t = 0$ and $t = T$, or $n = 0$ and $n = N$.

For this equation, S and Sf are defined as follows:

$$\Delta S = S_0 - S \tag{2}$$

$$\Delta S_f = S_0 - S_f \tag{3}$$

where

S is the structural state of the pavement at a given time $- t$, or at a given number of ESAL $- n$;

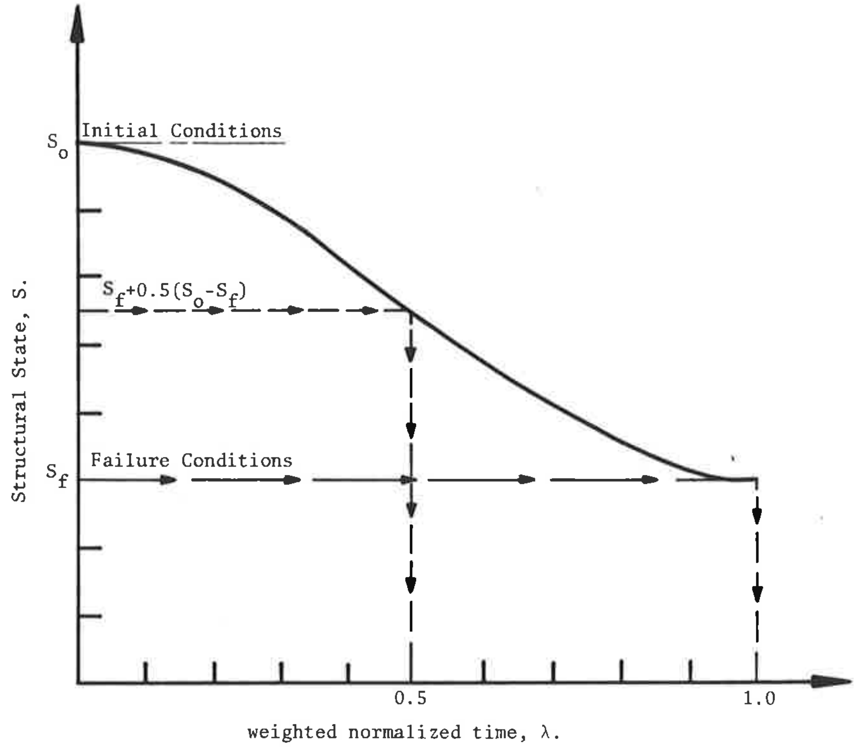


FIGURE 1 Basic S-shaped curve for pavement deterioration.

S_0 is the structural state of the pavement at $t = 0$ or $n = 0$; and

S_f is the structural state at failure, i.e., for $t = T$ or $n = N$.

There are, theoretically, many choices for the deterioration functions. The following considerations stated by Garcia-Diaz and Riggins (*2*) serve as useful guidelines for selecting a deterioration function:

1. The function must have a maximum value of traffic level (or time) equal to zero, and must be strictly decreasing as the traffic level increases.
2. The function cannot have negative values; indeed, if the structural value is standardized to be between 0 and 1, the particular function chosen cannot have values outside this range as traffic or time increases.
3. The function must have at least one parameter so that a family of pavements may be represented by different values of the parameter, or combinations of parameter values in the case of several parameters.
4. The structure of the function must be suitable for an efficient estimation procedure of the parameters on the basis of observed data.

These considerations lead to the following basic equations:

$$\frac{\Delta S}{\Delta S_f} = 3\,\lambda^2 - 2\,\lambda^3 \qquad (4)$$

$$\lambda = \left(\frac{t}{T}\right)^{\tau} \quad \text{or} \quad \lambda = \left(\frac{n}{N}\right)^{\phi} \qquad (5, 6)$$

where

τ is a nonlinear, site-specific constant, where time elapsed from construction action or last rehabilitation action is taken into account; and

ϕ is a nonlinear, site-specific constant, where number of ESAL from construction action or last rehabilitation action is taken into account.

It can be seen that equation 4 conforms to the following conditions:

(a) $\Delta S/\Delta S_f$ equals zero for λ equals zero;

(b) $\Delta S/\Delta S_f$ equals one for λ equals one;

(c) The first derivative $d(\Delta S/\Delta S_f)/d\lambda$ equals zero when λ equals zero, or when λ equals one.

In this deterioration model there are three unknown values to be calibrated with site observations: N (or T), S_f and λ (or τ). For any given two observed points, i.e., $\Delta S_1/\Delta S_f$ and $\Delta S_2/\Delta S_f$ at $t = t_1$ and $t = t_2$, respectively, τ and T may be derived as follows:

$$\tau = \log(\lambda_2/\lambda_1)/\log(t_1/t_2) \qquad (7)$$

$$T = \frac{t_1}{\lambda_1^{1/\tau}} = \frac{t_2}{\lambda_2^{1/\tau}} \qquad (8)$$

where λ_1 is the corresponding value for $\Delta S_1/\Delta S_f$ and λ_2 is the corresponding value for $\Delta S_2/\Delta S_f$.

The same applies for ϕ and N. When more than two observation points are available, the following values should be used:

$\Delta S_1/\Delta S_f = 0.3$ ($\lambda = 0.36$) and $\Delta S_2/\Delta S_f = 0.7$ ($\lambda = 0.62$)

It is important to emphasize that the main strength of the suggested function is in its capacity to change its S-shape with respect to traffic or time. In other words, different shapes of structural performance are gained for different values of ϕ or τ (Figures 2 and 3). In Figure 3, the a-line is similar to the

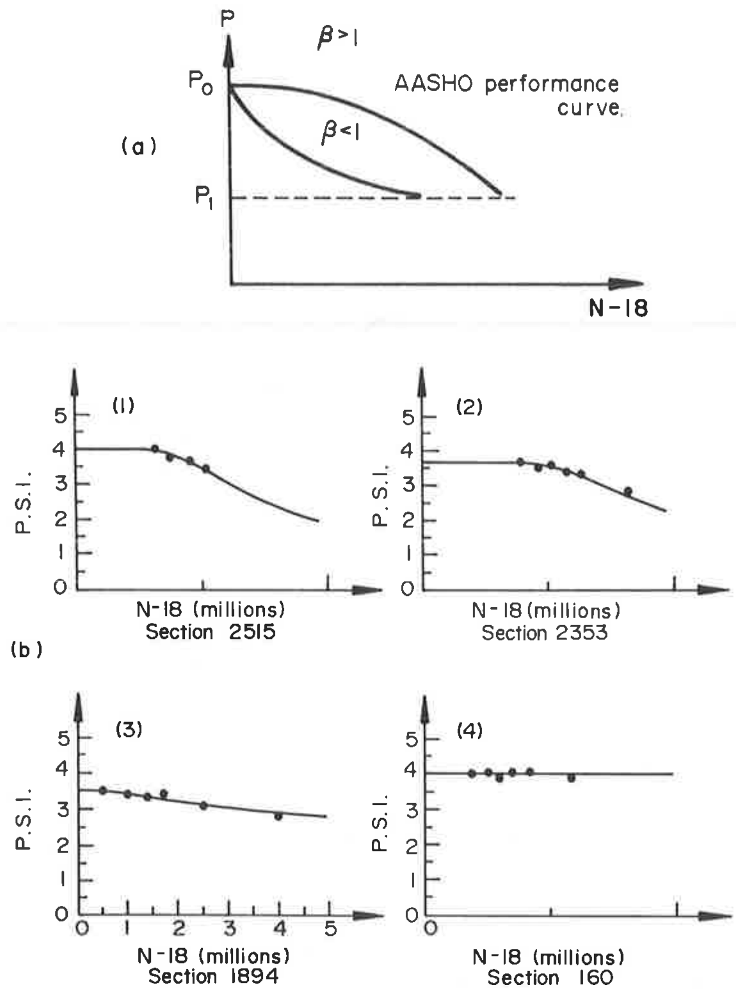


FIGURE 2 (a) The AASHTO performance curve; (b) Typical sections for the four cases of serviceability performance; (*2*).

American Association of State Highway and Transportation Officials (AASHTO) performance curve for $\beta > 1$, and the b-line is similar to the AASHTO performance curve for $\beta > 1$. Similarities exist between these same lines and other performance curves, such as those shown in Figure 4 (for the a-line of Figure 3) and in Figure 5 (for the b-line of Figure 3). This "strength" of the deterioration formula endows it with the ability to serve as a practical universal formula.

The main drawback of the suggested function stems from the fixed value of 0.5 for $\Delta S/\Delta S_f$ when $\lambda = 0.5$. Although this drawback can be greatly rectified by adding more terms to equation 3 (such as λ^4, λ^5, etc.), it seems that for network-level problems, such additions can be practically disregarded.

PCI PERFORMANCE CURVE

This paper adopts the use of the *PCI* method, which is considered to be a reasonable approach to the analysis of pavement conditions as derived from the necessary visual inspections. The *PCI* rating itself is determined as described by Shahin and Kohn (*4*).

The conversion of equation 4 into *PCI* units is

$$\frac{\Delta PCI}{\Delta PCI_f} = 3\,\lambda^2 - 2\,\lambda^3 \qquad (9)$$

For reasons of simplicity, it is assumed that $\Delta\ PCI_f = 100$, thus

$$\frac{\Delta\ PCI}{100} = 3\,\lambda^2 - 2\,\lambda^3 \qquad (10)$$

In this simplified approach, the unknown parameters of Equation 10 are only two: ϕ (or τ) and N (or T). Preliminary ranges for the τ and T values can be derived from Figures 6 and 7. These two figures represent deterioration curves for flexible pavements; one has not been rehabilitated since the date of construction and the other was rehabilitated during its service life. Table 1 shows the values obtained for these curves.

The normal range of rehabilitation rates, both for the treated and the untreated pavements, is characterized by τ larger than 0.44 and smaller than 1.8 (Table 1). When τ is larger than 1.8, a low rate of deterioration is experienced; when is smaller than 0.44, a high rate of deterioration is experienced.

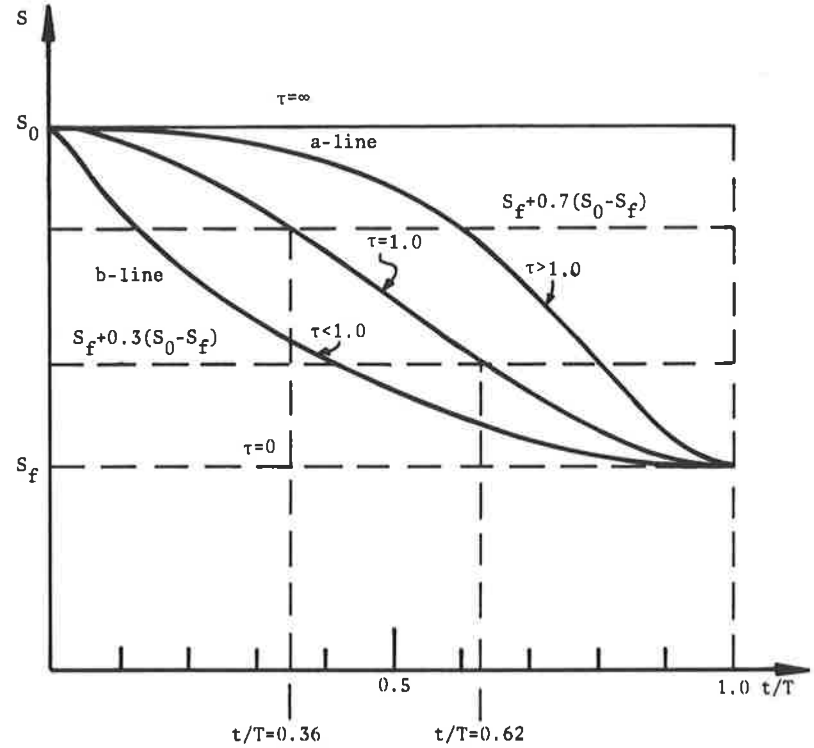


FIGURE 3 Various possible cases of pavement deterioration exposed by the τ parameter.

Contrary to the above-mentioned values, the derived T values do not correspond logically with the two rates of deterioration levels (Table 1). Therefore, a modified parameter has been introduced, the t value for Δ $PCI/100 = 0.7$, i.e., t_{30}.

In practice, the pavement may be considered as having reached its failure state when $PCI = 30$. Thus, it is seen from Table 1 that higher values for t_{30} are associated with low values of deterioration rates and that the t_{30} values for rehabilitated pavements are about half of those for unrehabilitated pavements.

In conclusion, τ and t_{30} are basic parameters indicating the deterioration curve of a given pavement. The better the pavement, the higher the τ and t_{30}. It is recommended that future studies attempt to correlate these parameters with the following variables:

1. structural design of the pavement;
2. environmental conditions of the pavement;
3. actual traffic loadings induced in the pavement; and
4. various routine maintenance activities performed in the pavement.

In the meantime, Table 1 can serve as a first approximation of the τ and t_{30} values.

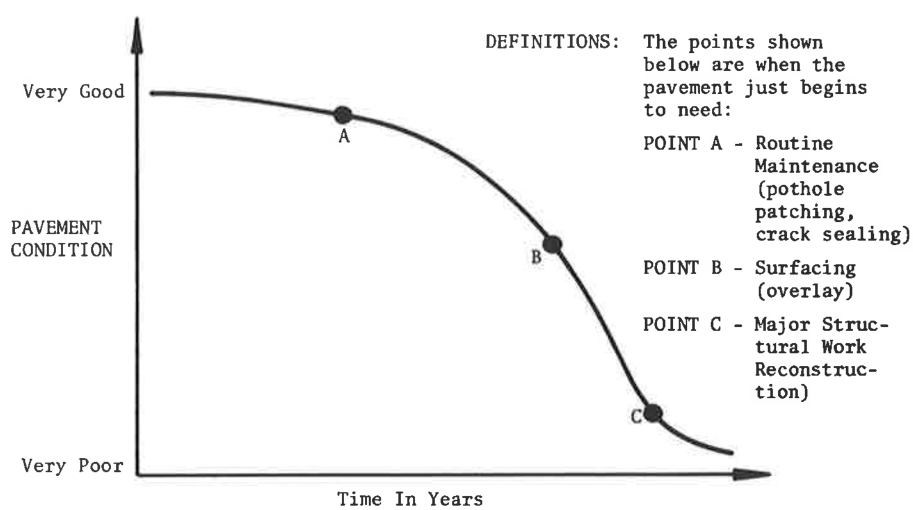
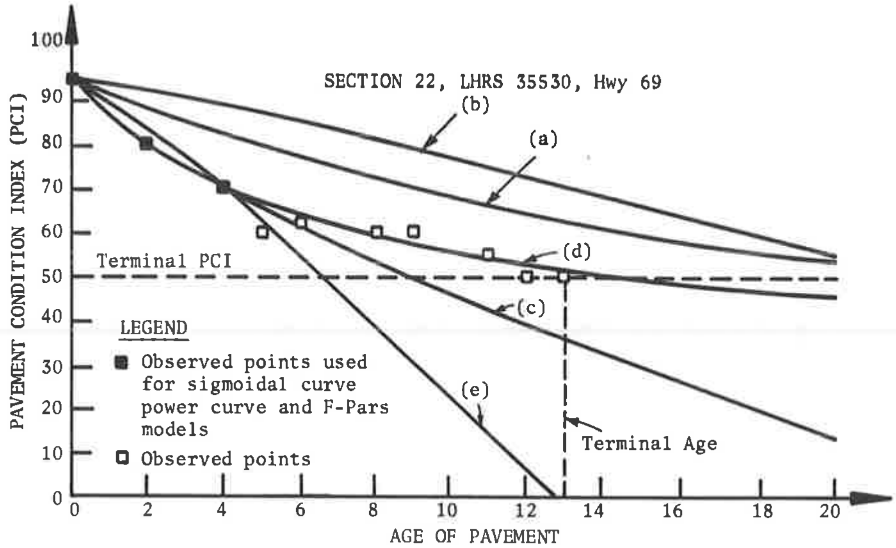


FIGURE 4 Typical asphaltic pavement life cycle (*1*).



curve (a) OPAC model (mechanistically derived)
curve (b) PARS model (empirical, pavement classes)
curve (c) Power curve (empirical, site-specific)
curve (d) Sigmoidal curve (empirical, site-specific)
curve (e) Factored PARS model (Bayesian approach, site-specific)

FIGURE 5 Example of model comparisons (*3*).

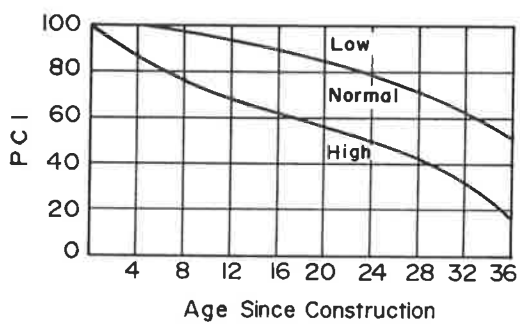


FIGURE 6 Determination of long-term rate of deterioration for asphalt concrete pavements (*4*).

RELATIONSHIP BETWEEN STRUCTURAL DETERIORATION AND *PCI* FOR THE AGENCY COSTS MODEL

The agency costs of repairs and rehabilitation are not a function of the *PCI* value assigned, by itself, to a given pavement. Two different pavements may reach the same *PCI* value, yet the repair or rehabilitation needs might be substantially different. In Figure 8, Pavement A has reached the same *PCI* value as Pavement B. The deterioration of Pavement A, however, has been mainly a result of cracking (i.e., failure of the upper layers), while the deterioration of Pavement B has been mainly the result of rutting (i.e., failure of both upper and

FIGURE 7 Determination of long-term rate of deterioration for asphalt concrete (AC) pavements overlay over AC pavements (*4*).

TABLE 1 T, t_{30}, AND τ VALUES FOR THE DETERIORATION CURVES GIVEN IN FIGURES 6 AND 7

Figure No.	6		7	
Rate of Deterioration	Low	High	Low	High
τ	1.80	0.44	1.64	0.44
T (Years)	53	100	33	50
t_{30} (Years) for PCI = 30	41	34	25	17

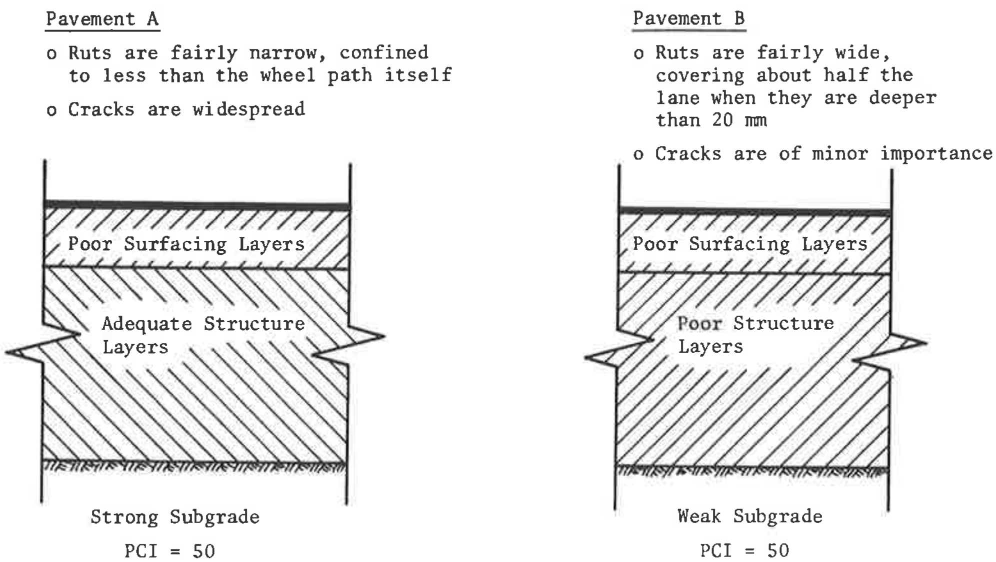


FIGURE 8 Example of two pavements reaching the same *PCI* value due to two different distress mechanisms.

lower layers). Thus, Pavement A requires no more than the rehabilitation of the asphalt layers through scrubbing and relaying of a new asphalt layer. Pavement B, on the other hand, requires extended rehabilitation action; all layers have to be upgraded, or new asphalt layers at a relatively great thickness have to be added. Consequently, the rehabilitation costs for Pavement B are higher than for Pavement A, despite both having an identical *PCI*. It is reasonable, on the other hand, to assume that overlay costs are mainly a function of a strengthening factor ΔSN, where ΔSN is the reduction value in the structural number of the pavement. When SN is expressed in terms of an equivalent asphaltic layer thickness, ΔSN denotes the needed asphaltic overlay (Figure 9). For this reason, the *PCI* values and the *SN* values should be interrelated, that is

$$U = 1 - \frac{\Delta SN}{SN_f} = f\left(\frac{\Delta PCI}{100}\right) \tag{11}$$

where
U is the utility value of the pavement and SN and SN_f are defined in equations 12 and 13.

Examples of U for a given pavement are shown in Figure 10 (*6*). The shape of the S-curves for U values suggests similar formulations for the *PCI* curves; therefore

$$\frac{\Delta SN}{\Delta SN_f} = 3\left(\frac{\Delta PCI}{100}\right)^{2\mu} - 2\left(\frac{\Delta PCI}{100}\right)^{3\mu} \tag{12}$$

$$\Delta SN = SN_0 - SN \tag{13}$$

$$\Delta SN_f = SN_0 - SN_f \tag{14}$$

where
SN is the effective structural number of the pavement at a given time; expressed in terms of an equivalent asphaltic layer thickness;
SN_0 is the initial effective structural number;
SN_f is the effective structural number at failure (equals about half SN_0, as estimated according to 7); and
μ is a nonlinear constant, specific to a given pavement.

The curves shown in Figure 10 lead to the following values of μ:

μ equals 0.8 for primary runways, and
μ equals 1.2 for secondary runways.

It should be recognized that the values of μ are associated with the strength characteristics of the pavement layers: the longer the life expectancy of the pavement, the higher the value of μ (Figure 11). These conditions hold also for the pavements illustrated in Figure 10, as secondary runways usually have a greater life expectancy than primary runways (taking into account the real traffic that is being induced in these two categories of pavements). Actual traffic for secondary runways is usually less than that estimated in the design. Thus, the cost of repairs or rehabilitation are related to the *PCI* values, together with a strength parameter, defined here as μ. This can also be explained by the following: As the *PCI* value is a measure of the failure level exposed on the surface of the pavement only, the μ value determines the involvement of the various layers in this failure. In other words, the μ value indicates how "deeply" the exposed failure picture of the surface has "penetrated" into the pavement's lower layers.

The use of strength parameters only, such as deflections and deflection bowls, may also sometimes lead to unreliable calculations of repair or rehabilitation costs. Deflections, in general, are good indicators of very poor conditions, but they play a dominant role where other conditions exist. This is particularly true in pavements whose base courses have failed without any excess distress on their subgrade. In such pavements, the deflection remains slight, although they need to be overlaid (see shaded area in Figure 12). For this reason, use of *PCI* values is essential but should be combined with the deflection criteria, as suggested in this paper.

The above example has been taken from observations of airfield pavements, but it can also apply to highway pavements. However, for a specific job, the μ values have to be calibrated with field observations and testing at various strengths, taking into account the list of local factors men-

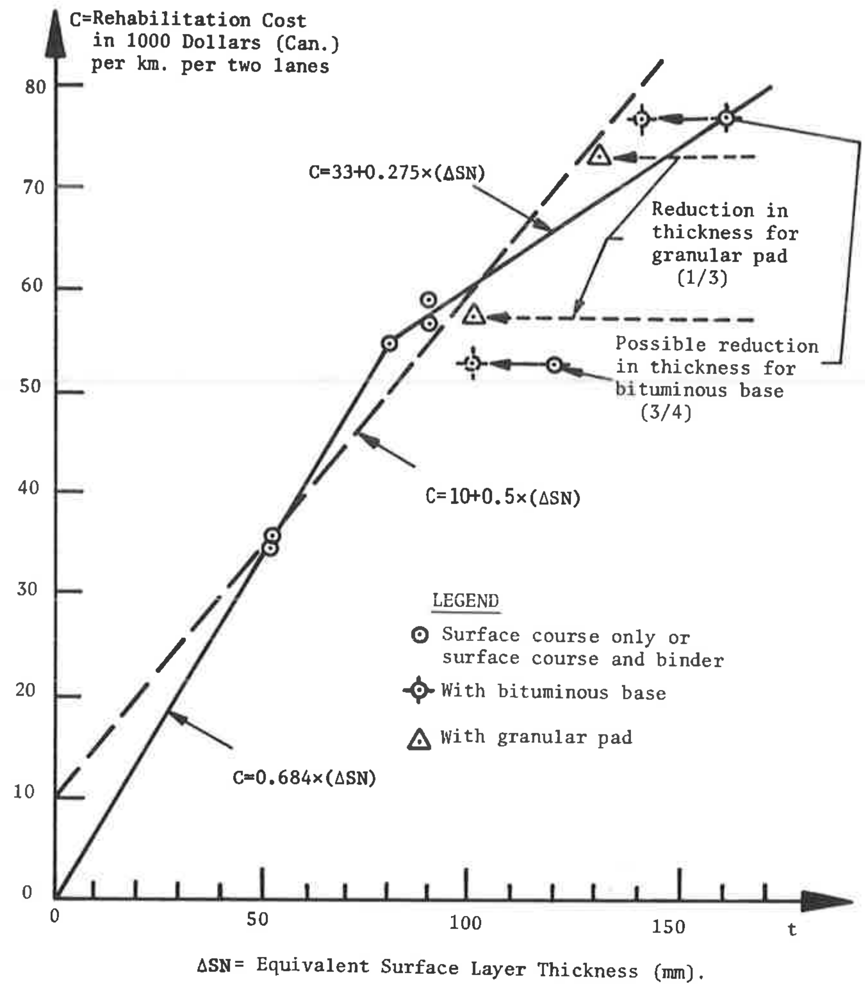


FIGURE 9 Agency cost function from 1984 cost data (*5*).

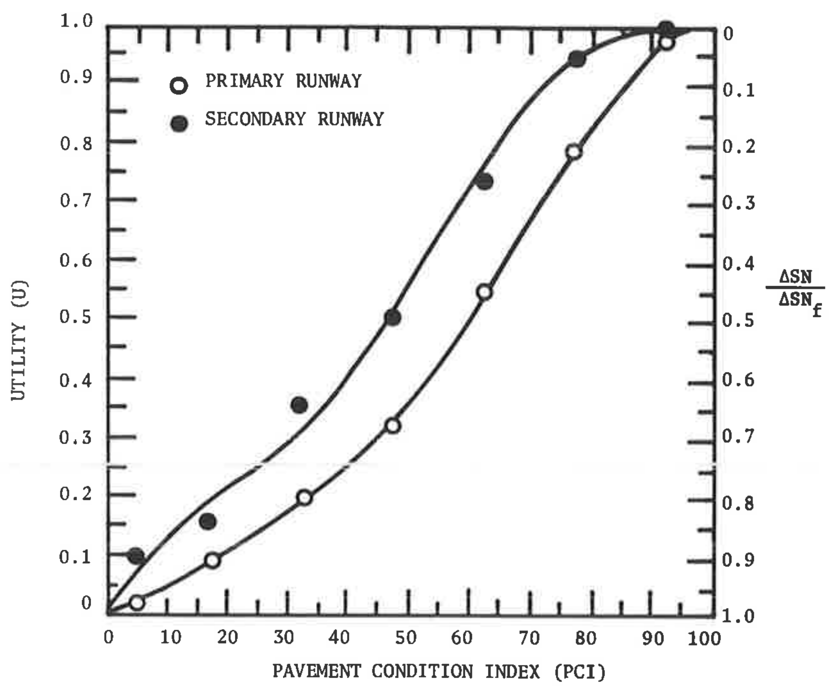


FIGURE 10 Pavement utility curve for runways (*6*).

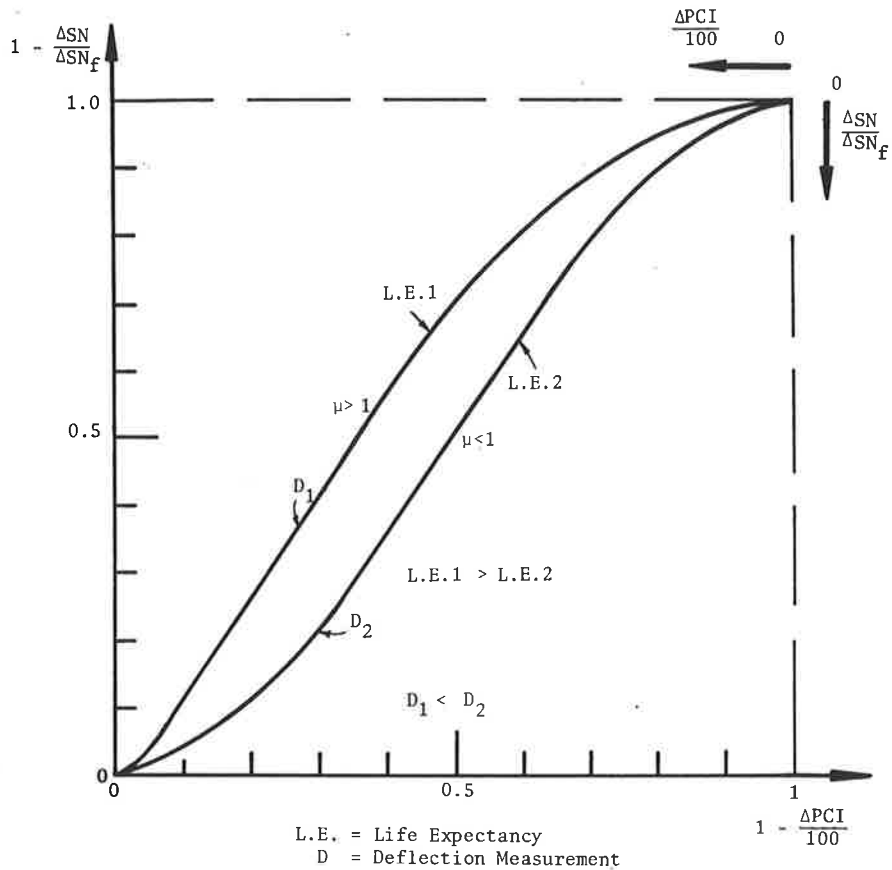


FIGURE 11 Utility model for pavements.

tioned under "PCI Performance Curve." When such data are not available, the μ values given in this section may be used as a first approximation. Higher values of μ are to be assigned to pavements with major cracking phenomena, and lower values of μ to pavements with major rutting phenomena.

RELATIONSHIP BETWEEN *PSI* OR ROUGHNESS AND THE USER COSTS MODEL

The user costs model is highly related to the road surface roughness. The literature suggests various functions using

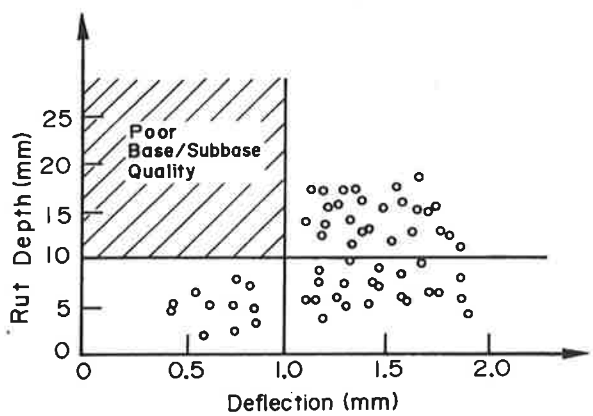


FIGURE 12 Example of deflection-rut depth relationship with demarcated poor base of subbase region (*7*).

roughness or *PSI* as the single dependent parameter, but it does not indicate how the roughness or the *PSI* values are related to the *PCI* values. This interrelationship is necessary when the major input is the *PCI* value of the pavement.

For the sake of simplicity, some agencies assume the following:

$$PCI = 20\ PSI \tag{15}$$

This approximation, however, may lead to substantially unreliable results, especially in the following case. "The propagation of alligator cracks in wheelpath areas of a flexible pavement indicates lack of structural strength, requiring immediate attention in order to support imposed traffic. This condition, however, may not necessarily be reflected in poor ride-quality measurements for some undetermined period of time" (*9*). Hence, in this case, low *PCI* values may be associated with high *PSI* values. In this context, it should be recalled that ride-quality measurements dominate the *PSI* value (Table 2).

Table 2 and the boundary conditions suggest that *PCI* and *PSI* are associated in the following manner (Table 3).

A logical function that fits the values given in Table 3 should have the following characteristics:

1. The first derivative of *PCI* with respect to *PSI* should be low for high *PSI* values and should then increase gradually, where major deterioration is mainly exposed by cracking. This is due to the fact that cracked pavements may have good ride quality (Table 2).

TABLE 2 EXAMPLES OF AC PAVEMENTS WITH EXCELLENT RIDE QUALITIES BUT WITH OBVIOUS STRUCTURAL PROBLEMS (9)

Site	PSI Due to:			Structural Conditions
	Ride Score (Californian Method)	Ride Quality Only	Ride Quality & Other Pavement Deficiencies	
A	20	3.35	3.00	Alligator cracking > ¼" wide over > 45% of pavement area. Many patches and pumping
B	17	3.50	3.30	Alligator cracking ¼" wide over > 45% of pavement area. No patching recorded.
C	13	3.75	3.50	Alligator cracking ¼" over > 45% of pavement area. Patching is recorded in 4-8% of area

2. The first derivative of *PCI* with respect to *PSI* should be high for high *PSI* values and then should decrease gradually where major deterioration is exposed mainly by rutting. This is due to the fact that even small rutting has a significant effect on ride quality.

Thus, the most suitable function is:

$$PCI = 100 \left(\frac{PSI}{5}\right)^{\delta} \quad (16)$$

where

δ is the nonlinear constant specific to a given pavement.

Equation 16 is illustrated in Figure 13. The convex curve ($\delta < 1$) is the governing function for the rutting case, and the concave curve ($\delta > 1$) is the governing function for the cracking case.

From Table 3, the following values of δ are derived:

δ equals 0.7 for the extreme rutting case,
δ equals 1.0 for the intermediate case, and
δ equals 1.4 for the extreme cracking case.

TABLE 3 *PSI* AND *PCI* VALUES IN DIFFERENT FAILURE MECHANISMS

PSI	PCI Value is Due To:		
	Mostly Cracking	PCI for: Cracking & Rutting	Mostly Rutting
0	0	0	0
3	50	60	70
5	100	100	100

If a relationship between roughness values and *PCI* is needed, the following equation can be used (*10*):

$$IRI = 5.5 \ln \left(\frac{5}{PSI}\right) \quad (17)$$

where *IRI* is the international roughness index.

The combination of equation 16 with equation 17 leads to the following relationship (see Figure 14):

$$IRI = \frac{5.5}{\delta} \ln \left(\frac{100}{PCI}\right) \quad (18)$$

Bartell and Kampe (*10*) have presented the conversion of *IRI* values into other roughness indices.

Equations 16 and 18 may be used to calculate user costs, which are given as a function of *PSI* or as a function of the roughness index.

The δ value used in these calculations should be in correlation with field observations; however, the above-mentioned values may be utilized as a first approximation.

SUMMARY AND CONCLUSIONS

This paper demonstrates the necessity of several pavement condition parameters to evaluate short-range and long-range maintenance and rehabilitation planning programs. The use of a single parameter only, such as *PSI* or *PCI*, for indicating pavement performance may lead sometimes to misleading results. These parameters should, therefore, be accompanied by other pavement condition parameters:

- τ and t_{30} (or ϕ and n_{30}) for generating the deterioration rate over time (or traffic), as given by the *PCI* values;

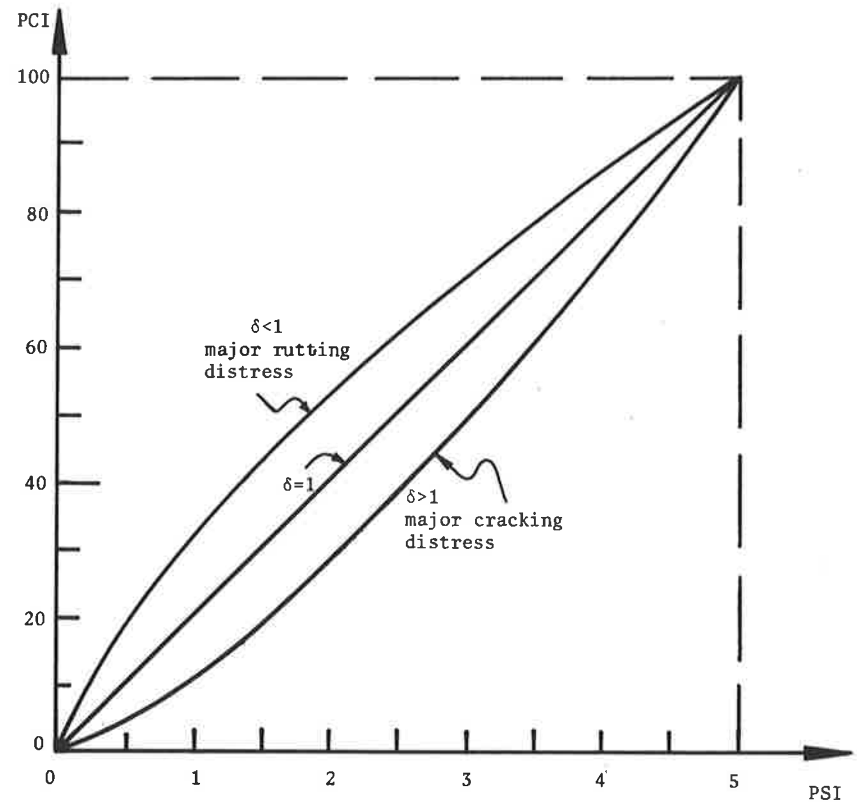


FIGURE 13 **The *PCI-PSI* model for various modes of distress.**

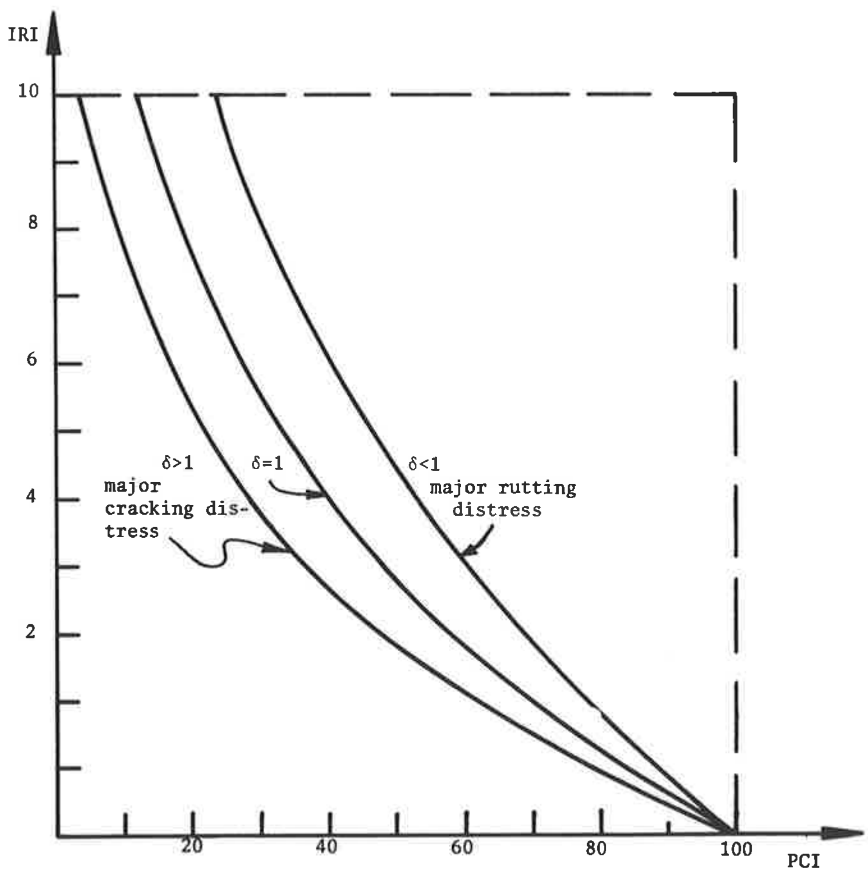


FIGURE 14 **The relationship between *IRI* and *PCI* for various modes of distress.**

- μ for governing the structural rehabilitation solutions for various pavements exhibiting the same *PCI* value;
- δ for governing the ride-comfort properties of various pavements exhibiting the same *PCI* value.

Tentative values for these parameters are suggested in this paper, with a possible relationship between the μ and δ values: μ equals 1.2 and δ equals 1.4 for the cracking case; μ equals 0.8 and δ equals 0.7 for the rutting case. Naturally, these values should be regarded as preliminary ones, coming from airfield pavements, and more specific correlation with specific site observations are needed. It is highly recommended, however, that these values be used for conducting at least sensitivity analyses on the various outputs derived from any pavement management calculations.

This paper does not deal with the possible variations of the μ and δ values with time. These are topics for a separate discussion.

ACKNOWLEDGMENT

This paper was prepared while the author stayed at the Massachusetts Institute of Technology, Center for Construction Research and Education within the Department of Civil Engineering, as part of a sabbatical leave from the Technion-Israel Institute of Technology. The generous assistance of MIT, and, in particular, Professor Fred Moavenzadeh, is gratefully acknowledged.

REFERENCES

1. D. S. Turner, J. V. Walters, T. C. Glover, and E. R. Mansfield. An Asphalt Paving Rating System Based on Highway Maintenance Engineers Experience. Presented at 65th Annual Meeting of the Transportation Research Board, Washington, D.C., 1986.
2. A. Garcia-Diaz and M. Riggins. Serviceability and Distress Methodology for Predicting Pavement Performance. *Transportation Research Record 997*, TRB, National Research Council, Washington, D.C., 1984.
3. J. J. Hajek, W. A. Phang, A. Prakash, and G. A. Wrong. Performance Prediction for Pavement. *Proc., North American Pavement Management Conference*, Toronto, Ontario, Canada, Vol. 1, 1985.
4. M. Y. Shahin, and S. D. Kohn. *Pavement Maintenance Management for Roads and Parking Lots*. Technical Report M-294. U.S. Army Corps of Engineers, Construction Engineering Research Laboratory, Oct. 1981.
5. F. W. Jung. *Modelling of Life Cycle Costs of Pavement Rehabilitation*. Research and Development Branch, Ministry of Transportation and Communication, Ontario, Canada, Nov. 1985.
6. M. Y. Shahin, S. D. John, R. L. Lytton, and W. F. McFarland. Pavement Management and Budget Optimization Using Incremental Benefit-Cost Technique. *Proc., North American Pavement Management Conference*, Toronto, Ontario, Canada, Vol. 1, March 1985.
7. C. A. Van Gurp, A. A. A. Molenaar, and F. J. M. Van Velzen. Testing the Delft University Pavement Management System. *Transportation Research Record 997*, TRB, National Research Council, Washington, D.C., 1984.
8. National Institute for Transport and Road Research (NITRR). *Technical Recommendations for Highways, Bituminous Pavement Rehabilitation Design*. Draft TRH 12, Pretoria, South Africa, 1983.
9. C. Bartell, and K. Kampe. Development of the California Pavement Management System. *Volume 1: System Description*. Report FHWA-CA-HM-7139-78-03. FHWA, U.S. Department of Transportation, Oct. 1978.
10. W. D. O. Paterson. Relationship of the International Roughness Index to Other Measurements of Roughness and Riding Quality. Presented at 65th Annual Meeting of the TRB, Washington, Jan. 1986.

Publication of this paper sponsored by the Committee on Pavement Management Systems.