STRUCTURAL EVALUATION OF HIGHWAY AND AIRFIELD PCC PAVEMENTS USING THE FALLING WEIGHT DEFLECTOMETER

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The evaluation of the fundamental structural properties of Portland Cement Concrete (PCC) pavements is presented. The evaluation is carried out in-situ and non-destructively using a Falling Weight Deflectometer (FWD). The recommended FWD testing procedure used on jointed PCC pavements is described. At each test point, the FWD load magnitude and deflection basin are stored on magnetic tape. This data is automatically analyzed by programs developed for the same microcomputer that controls the operation of the FWD. From the deflection basin measured at the interior of each slab, the elastic moduli of the PCC and of the foundation support are calculated. The typically non-linear response of the foundation support is considered in the back-calculation procedure, since this is often essential in order to obtain the correct PCC and foundation moduli. The structural condition of the joints and corners are determined using Westergaard's newest equations. The modulus of subgrade reaction (k-value) is calculated at the interior of the slab, and at the joints and corners. From the ratio of these values, the quality of the foundation support at joints and corners may be ascertained. Practical examples of the evaluation of joint and corner conditions on highway and airfield pavements are presented. The use of empirical relationships between the derived mechanistic material properties and pavement performance are also discussed. These relationships are user-controlled, variable inputs to the computer programs used. The programs may, therefore, be calibrated for any specific local environmental conditions.

Maintenance and rehabilitation of existing pavement systems are very costly. Decisions on what to do, and where and when to do it, should therefore be based on the most accurate information on the condition of the existing pavements obtainable.

The overall pavement condition depends on the FUNCTIONAL condition (primarily ride quality) and on the STRUCTURAL condition (load bearing capacity). This paper deals primarily with evaluation of the structural condition of Portland Cement Concrete (PCC) pavements, although the relationship between structural and functional condition is also addressed.

The paper describes how the Falling Weight Deflectometer (FWD) may be used to non-destructively determine the elastic moduli of the concrete and foundation support of PCC pavements. For jointed PCC pavements, the foundation support at the joints and corners as well as the degree of load transfer at joints may also be determined.

Once the fundamental structural properties have been determined, it becomes possible to calculate the critical stresses or strains in the concrete and foundation support.

Determination of the fundamental structural properties (elastic moduli, joint conditions) and calculations of the critical stresses are carried out using analytical (mechanistic) methods. But to decide on appropriate M & R measures, it is necessary that the future condition (both structural and functional) of the pavement can be predicted.

This paper discusses some empirical relationships between pavement response (critical stresses) and pavement performance (deteriorating structural condition and ride quality).

Derivation of Fundamental Structural Properties

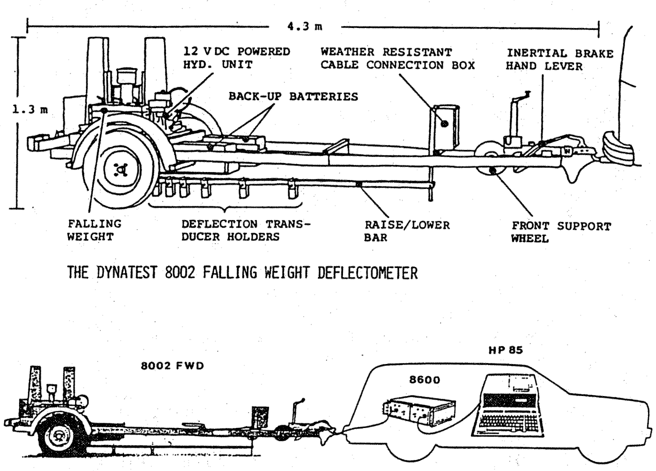
In order to evaluate the structural condition ("bearing capacity") of a jointed PCC pavement, the following structural properties must be determined:

1. The elastic PCC modulus.
2. The PCC thickness.
3. The elastic modulus (E-value), modulus of subgrade reaction (k-value), or CBR-value of the foundation support.
4. The degree of load transfer at joints.
5. The effective foundation support at joints and corners.

Ideally, the daily and yearly variations of these fundamental properties should also be considered, and some design procedures also require the temperature gradients in the PCC.

Most often, the PCC thickness will be known from construction records (at least the nominal thickness), and the remaining four of the above listed five parameters may then be determined non-destructively using the Falling Weight Deflectometer (FWD). This chapter describes the procedure used with one commercially available FWD, the Dynatest 8000 FWD Test System shown in Figure 1.

Figure 1. The Dynatest 8000 Falling Weight Deflectometer Test System.



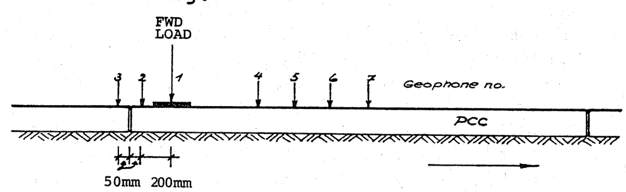
The FWD Measuring Procedure

In its standard configuration, the Dynatest FWD produces an impact load of 25-30 msec duration, and a peak force of up to 105 kN (24,000 lbs), by dropping a weight onto a specially designed buffer system. This load has been found to closely resemble the load from heavy moving wheels [1,2].

Both the load level and a series of seven simultaneous deflections are monitored for each FWD test. The deflections are measured at the surface of the pavement, from the center of the loading plate (through a small hole in the middle of it) out to a distance of 1.5 - 2 m (5-7 ft) from the center. This enables calculation of the elastic properties of each structural layer in the pavement, assuming the layer thicknesses are known. The elastic properties are calculated through a reverse, iterative procedure which matches up the measured FWD load and deflections against a unique set of material properties.

For each slab, a FWD test is carried out at the interior of the slab and at one or more joints and/or corners. During joint and corner testing, the sensors are positioned such that the deflections on both sides of the joint are measured (see Fig. 2).

Figure 2. Position of FWD Sensors for Joint Testing.



Preferably, the interior slab tests should be carried out during night/early morning conditions, or at other times when there is little or no temperature gradient in the slab, to ensure that the PCC and subgrade moduli are correctly determined. Testing of joints and corners should also be carried out during night/early morning conditions, when the slabs will be "curled" (raised upwards), and the joints open. The joints and corners will, therefore, be in their most critical state.

If the quality of foundation support at joints and corners need evaluation (eg, for void detection), additional tests should be carried out at the joints and corners when the slabs are not curled upwards, eg, during the afternoon, because an apparent lack of support during night/early morning testing may be due to upward curling alone.

Calculation Of Elastic Moduli

For each FWD test point, the deflection basin is analyzed using a special version of the ELMOD program (acronym for **E**valuation of **L**ayer **M**oduli and **O**verlay **D**esign [3]). This program is called ELCON (**EL**mod for **CON**crete).

Both two- and three-layer systems may be handled by the program, but in most cases PCC pavements must be treated as two-layer systems, because any base or subbase layer will have very little influence on the shape of the deflection basin compared to the influence of the PCC and subgrade layers.

The ELCON program operates on the same micro-computer that controls the FWD during field operations. The program makes use of the **M**ethod of **E**quivalent **T**hicknesses (MET) [4] to determine the unique set of layer moduli which will produce the same deflection basin as measured.

Most of the measured magnitudes of deflection are due to the response of the subgrade. It is therefore very important that the subgrade modulus is accurately determined. A small error in the subgrade modulus will lead to a very large error in the derived PCC modulus. For this reason, it is necessary to consider any existing non-linearity of the subgrade. This can be done quite easily with the MET, with the non-linearity of the subgrade expressed as:

$$E_m = C_o \times (\sigma_1 / \sigma')^N \qquad (1)$$

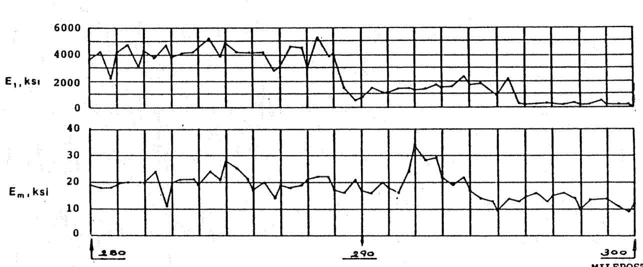
where E = the (in-situ) subgrade modulus at
σ_1 = major principal stress level, and
σ' = a reference stress, and
C_o and N are constants ($N \leq 0$).

The ELCON program automatically calculates C_o, N and E_m at the stress level imposed by the FWD.

Due to the large influence of the subgrade on the measured deflections, it is also very important that the deflections are measured at a load level similar to that resulting from a heavy truck (or aircraft) wheel(s), and that deflections measured at large distances from the loading center (> approx. 3 feet) are measured very accurately. With the Dynatest FWD, deflections are measured to an accuracy of 2-3 microns (approx. 0.1 mils).

An example ELCON output taken from data obtained between Mileposts (MP) 280 and 300 (WB) on I-94 in North Dakota are plotted in Figure 3. The moduli are given in kips (lbs x 1000) per square inch (ksi). E_1 is the modulus of Layer 1, which, from MP 280 -> 289 consists of Jointed Portland Cement Concrete Pavement (JCP), and from MP 289 -> MP 295.5 of PCC overlayed by Asphalt Concrete (AC). Here, E_1 represents the combined AC+PCC modulus. MP 295.5 - 300 is a simple AC-section, with no PCC.

Figure 3. Example Moduli Plot, I-94 (westbound) in North Dakota.



From MP 280 -> 289, the modulus of the PCC is generally around 4,000 ksi, which is quite normal for an intact layer of concrete. At some points, considerably lower moduli are found, as low as about 2,000 ksi, indicating either crack initiation or an insufficient thickness of PCC.

E_m is the modulus of the foundation support, ie, corresponding to measurements taken on the top of the subbase. The modulus of foundation support is seen to vary betwen 9 ksi and 34 ksi. Using the approximate "rule of thumb" relationship:

$$CBR = E_m/1.5 \text{ (in ksi)} , \qquad (2)$$

this corresponds to CBR values between 6% and 23%.

<u>Determination Of Joint & Corner Conditions</u>

To evaluate the conditions at the joints and corners, ELCON makes use of Westergaard's newest equations [5]. Using the PCC modulus determined from the elastic analysis, the program first calculates the modulus of subgrade reaction (the k-value of a Winkler subgrade) at the center of the slab. The program then assumes the same PCC modulus at the joints and corners as was derived from the interior slab test, and calculates the k-values at these positions. The degree of load transfer is also calculated and considered in calculating the k-values at joints.

If the k-value is low at a joint or corner compared to the value derived at the center, this is an indication of poor foundation support. For measurements carried out during night or early morning, the lack of support could be due partly to upward curling of the slab and partly to the presence of a void at the joint. Measurements carried out during the afternoon, or when there is no temperature gradient in the slab, should therefore be used to determine the quality of foundation support (uninfluenced by upward slab curling) at joints and corners.

The degree of load transfer between slabs is also calculated using Westergaard's equations. The percentage load transfer, %LT, is calculated such that:

$$Zj - Z'j = \left(1 - \frac{\%LT}{100}\right) \times (Ze - Z'e), \qquad (3)$$

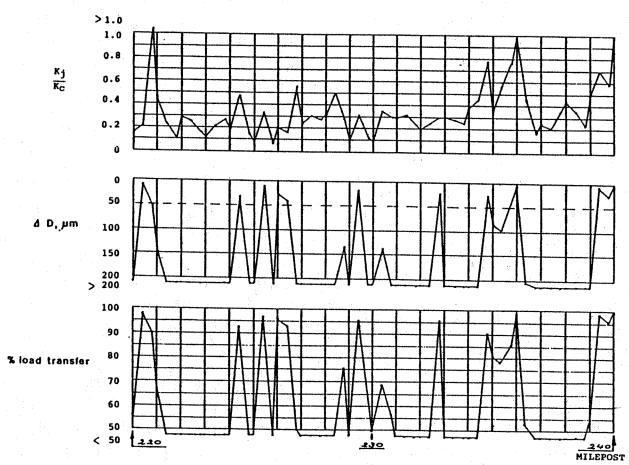
where Zj and Z'j are the deflections of the loaded and unloaded slabs at the joint, and Ze and Z'e are the corresponding deflections that would occur at the same points if the joint had no capacity for load transfer at all. Sometimes a simplified measure of the degree of load transfer is calculated as the ratio of Zj/Z'j, given in percent. This value may easily be determined from the relationship:

$$\frac{Z'j}{Zj} \times 100\% = \frac{\%LT}{200 - \%LT} \times 100\% \qquad (4)$$

The foundation support and the degree of load transfer at the joints will depend on the climatic conditions. The critical conditions are during wet and cold seasons, and during the spring thaw for regions with frost penetration.

Figure 4 shows a very poor section of I-94, again in North Dakota (from MP 220 -> 240, EB). The upper plot is the ratio of the k-value at the joint to the k-value at the center, k_j/k_i. If this ratio is less than approximately 0.8, then the joint has a poor foundation support. For the section shown in the figure, some joints have a k-value ratio as low as 0.1.

Figure 4. Example of Poor Joint Conditions, I-94 (eastbound) in North Dakota.

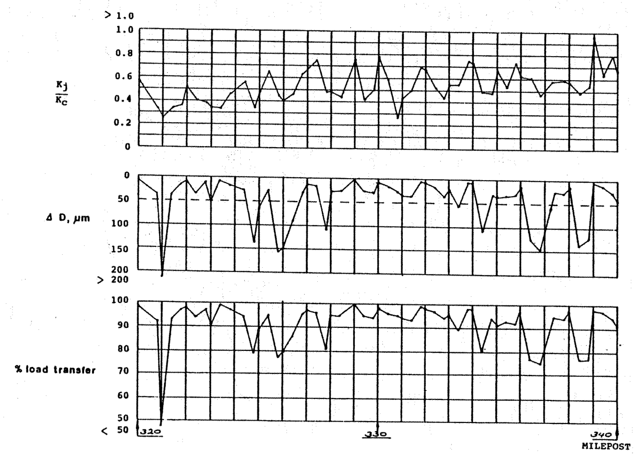


The middle plot is of the differential deflection, ΔD, in microns (mm x .001 = 0.04 mils), corresponding to a standard 18-kip axle load. The Asphalt Institute [6] suggests a maximum permissible value of 50 microns (2 mils). If the differential deflection is larger than this value, stabilization of the joint may be required before overlay. For the section shown in Figure 4, only very few joints had differential deflections less than 50 microns, and many had differential deflections of more than four times this amount.

The lower plot depicts the percentage load transfer. If it is less than 70%, the joint has a poor load transfer. This plot is, naturally, very similar to that of the differential deflections.

Figure 5 shows another I-94 section from MP 320 -> 340 where the joint condition is considerably better, especially with respect to load transfer.

Figure 5. Examples of Varying Joint Conditions, I-94 (eastbound) in North Dakota.



Examples from the evaluation of a reinforced, jointed PCC runway are shown in Figures 6 and 7.

Figure 6. Example of Joint & Corner Conditions on a JCP Runway, Stations 275' - 7425'.

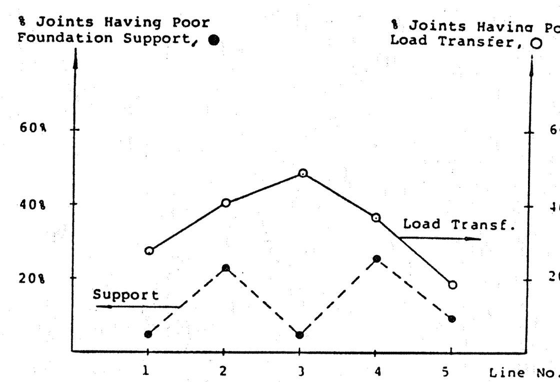
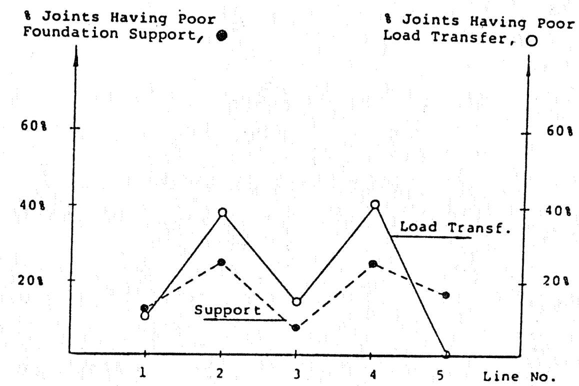


Figure 7. Example of Joint & Corner Conditions on a JCP Runway, Stations 7425' - 9200'.



The percentage of joints and corners having poor support (k_j or k_c/k_i < 0.8) and the percentage having poor load transfer (LT < 70%) are shown. The measurements were carried out along 5 parallel lines. Lines 1 and 5 were halfway between the main loaded slabs and the edges of the runway. Lines 2 and 4 were within the heaviest loaded area (where the main aircraft gears generally pass), while Line 3 was immediately right-of-center. It is interesting to note that the damage at the joints is clearly load associated. (Lines 2 and 4 are much poorer than Lines 1, 3, and 5).

Predicting Future Condition

Once the fundamental properties of the PCC pavement have been determined, as described in the previous sections, the maximum tensile stresses in the concrete caused by the design load can then be calculated. Many different methods for calculating these stresses exist, ranging from Westergaard's simple equations to very sophisticated finite element methods. It is also possible to consider stresses caused by temperature gradients or other environmental causes.

If the fatigue relationship for the concrete is known, it then becomes possible to calculate the permissible number of (repeated) loads. The cumulative damage caused by different gear loads may be summed using Miner's law. It is less certain how the dynamic traffic stresses and the semi-static temperature stresses should be superimposed, but different methods exist (eg, the Modified Goodman Diagram [7] or Aas-Jakobsen's Equation [8]).

Traditionally, fatigue relationships have been determined by testing PCC specimens in the laboratory. Most often a relationship of the type:

$$N = \left\{\frac{f_c}{\sigma}\right\}^p \tag{5}$$

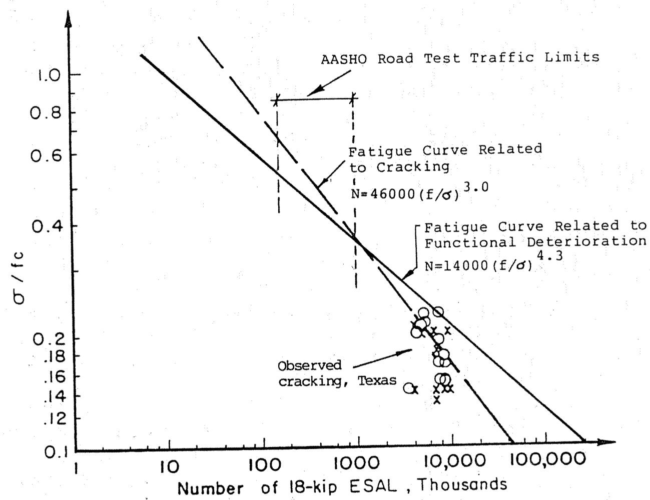
where N = the permissible number of loads,
σ = the maximum tensile stress in the concrete,
f_c = the modulus of rupture, and
p = a constant

is employed.

Most laboratory tests result in a "p"-value (or power) between 20 and 30. The Portland Cement Association method for rigid airport pavement design, for example, uses a p-value of approximately 20.

Fatigue relationships may also be derived from full-scale pavement testing or from studies of in-service pavements. Vesic and Saxena [9] analyzed the AASHTO Road Test and found a power of 4 (rather than 20 or 30). This was confirmed by MacLeod (10) from a study of in-service pavements in California. A "reverse" analysis of Road Note 29's empirical method of design of rigid pavements indicates a power of even less than four. Figure 8 shows two relationships, both derived from the extended AASHTO Road Test. One relationship utilized Class 3 or 4 cracking as failure criteria, while the other employed a decrease of PSR to 2.5 as service life criteria.

Figure 8. Comparison of PCC Fatigue Curves.



It is quite evident that fatigue relationships developed in the laboratory cannot be directly applied to in-service conditions. At the present time, therefore, pavement performance in terms of cracking or functional deterioration must be empirically related to analytically derived critical stresses or strains.

To calculate remaining life (and needed overlay thickness), the ELCON program makes use of the relationship:

$$N = A \times \left\{\frac{f_c}{\sigma}\right\}^p, \tag{6}$$

where A and p are empirically derived constants.

A, p (and f_c) are user-controlled input parameters to the program. The failure criteria may be either structural (cracking) or functional (roughness).

Alternatively, the modulus of rupture, f_c, may be calculated by the program from the PCC modulus determined in the elastic layered analysis using the relationship:

$$f_c = B \times (E/E')^C, \tag{7}$$

where E = the modulus of PCC,
E' = a reference modulus, and
B and C are constants, where C may assume different values, depending on whether E is greater than or less than E'.

The modulus of the foundation support (the E_m-value) and the modulus of subgrade reaction (the k-value) may be varied as a function of season in the ELCON program, either as an exponential relationship typical for areas with frost and spring thaw, or as a sinusoidal relationship more appropriate for regions with cyclic wet and dry periods. If the PCC has an AC overlay, or if overlaying with AC is considered for rehabilitation, the AC modulus is adjusted for seasonal temperature variations as well.

The design loads may be either a single wheel, a dual wheel, or two dual wheels in tandem. Twelve different gear types may be specified for each evaluation. This is especially useful for airfield pavement evaluation because of the wide variety of existing aircraft gear configurations.

The damage of each gear load, for each season (max. 12 seasons), is summed using Miner's law. From this, the remaining life of the pavement is calculated. If overlaying is considered as a possible rehabilitation measure, the needed thickness for a given design period may also be calculated by the ELCON program for each FWD test point.

The method described above relates pavement performance (structural or functional) to the maximum tensile stress in the concrete. Sometimes JCP will deteriorate functionally (ie, loss of ride quality) due to faulting, even though it may be structurally sound. In this case, it seems less reasonable to relate pavement performance to the tensile stress in the PCC. Using the maximum tensile stress in Continuously Reinforced Concrete Pavements (CRCP) also appears doubtful. The purpose of the reinforcement in CRCP is to control the closely spaced cracking typical of these pavements, and cracking of the concrete is, therefore, not an appropriate failure criteria (although Class 3 or 4 cracking of the "layer", rather than fatigue cracking of the concrete material, could be empirically employed).

Use of the maximum tensile stress in the concrete also appears somewhat doubtful when the power, p, in the fatigue law is about 4 for in-service pavements and 20 or more for the concrete itself when tested in the laboratory.

Therefore, a good case can be made for a functional performance relationship, based on the maximum compressive stress on the foundation support, rather than the maximum tensile stress in the concrete. In the ELCON program, the following relationship is used:

$$\sigma = A \times (R \times N)^B \times (E/E')^C, \qquad (8)$$

where

σ = the permissible stress on the unbound layer at the interior of the slab,
R = the regional factor,
N = the number of load repetitions to cause the PSR to decrease from 4.5 to 2.5,
E = the modulus of the foundation support,
E' = a reference modulus, and
A, B, and C are constants.

Based on an analysis of the extended AASHTO Road Test and on a structural pavement evaluation of more than 1000 lane miles of Interstate Highway in North Dakota, the following tentative values of A, B, and C are suggested:

A = 70 MPa (E' = 160 MPa)
B = -0.47
C = 1 when E is > E' (160 MPa), and 1.16 when E < E'.

These values have been used to calculate the residual life of 50 sections of JCP and 62 sections of CRCP of I-94 and I-29 in North Dakota. The residual life distributions are shown in histograms for JCP in Figure 9 and for CRCP in Figure 10. A normal distribution is superimposed on the histograms.

Figure 9. Normal Distribution of Residual Life of 50 JCP Sections, I-94 and I-29 in North Dakota.

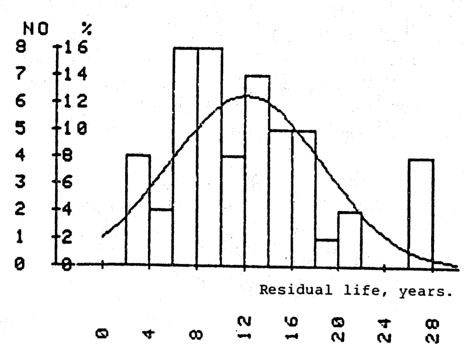
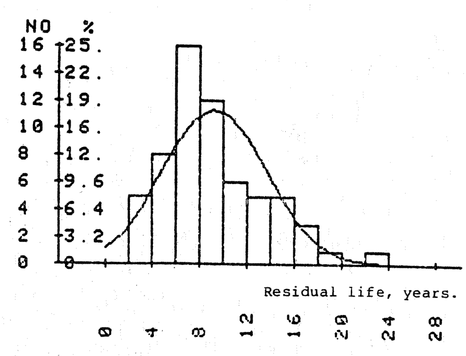


Figure 10. Normal Distribution of Residual Life of 62 CRCP Sections, I-94 and I-29 in North Dakota.



For JCP the residual life ranges from 2.6 to 27.5 years and for CRCP from 2.6 to 23.Ø. Most of these pavements were constructed between 1958 and 1974, with thicknesses of 9.25" - 1Ø.25" for JCP and 7.25" - 8.25" for CRCP. The mean residual life is 12.1 years for JCP and 9.2 years for CRCP.

The logarithms of the residual lives are also shown in histograms on Figures 11 and 12, with a log-normal distribution superimposed. Not surprisingly, a better fit is obtained with a log-normal distribution. Using the log-normal distribution, then, the mean residual life is 1Ø.9 years for JCP and 8.3 years for CRCP.

Figure 11. Logarithmic Distribution of Residual Life of 5Ø JCP Sections, I-94 and I-29 in North Dakota.

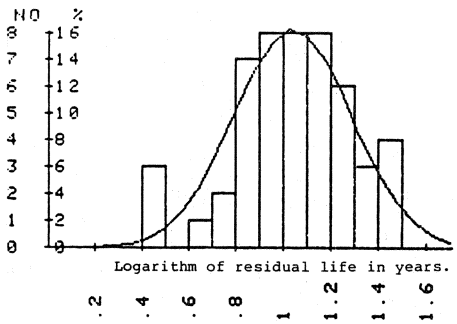
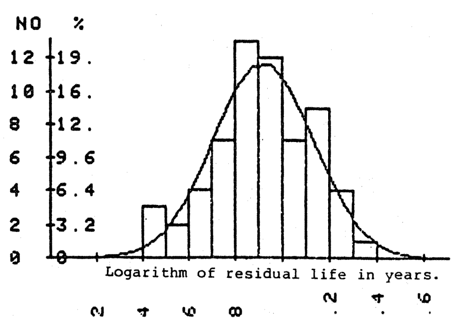


Figure 12. Logarithmic Distribution of Residual Life of 62 CRCP Sections, I-94 and I-29 in North Dakota.



If an empirical relationship between response and functional performance of the type given in Equation (8) could be verified through full scale testing, or through observations of in-service pavements, it could then be used for predicting pavement performance in a pavement management system.

Conclusion

Thanks to the development of suitable hardware (the Falling Weight Deflectometer) and software (eg, the ELMOD and ELCON programs) during the last decade, it is now possible to determine the fundamental structural properties of rigid highway and airfield pavements.

The modulus of elasticity (E-value) of PCC and foundation support may be determined in-situ, non-destructively and under realistic loading conditions using the FWD. The E-values are important inputs to overlay design methods using the analytical-empirical (or mechanistic) procedure presented. For design methods based on CBR-values, the modulus of the foundation support (E_m) may be converted to a CBR-value using the well-known empirical relationship: CBR = E_m/1.5 ksi.

The modulus of the PCC may be used directly to indicate whether the concrete is structurally sound. A structural deterioration of the concrete, crack initiation, or insufficient layer thickness will result in a lower derived modulus of the PCC layer than would be expected of sound concrete.

The modulus of subgrade reaction (the k-value) may likewise be determined, and this can, of course, be used in design methods using a Winkler subgrade. The main purpose of evaluating the k-values is to determine the quality of foundation support at joints and corners. A void under a joint or a corner will result in a lower k-value at this location than under the center of the slab. If the ratio of k-values between a joint and interior slab is less than Ø.6 - Ø.8, the joint support is poor. It should be noted that joint support should be measured when the slabs are not curled upwards due to temperature gradients.

The degree of load transfer at joints is another important parameter. This is determined from FWD tests at the joint with the sensors positioned such that the deflections on both sides of the joint are monitored.

After the fundamental physical properties of JCP or CRCP have been determined, critical stresses or strains in the PCC and in the foundation support may be calculated under any combination or configuration of loads. The most critical step in the analytical-empirical design procedure, however, is not the calculation of pavement response parameters, but the empirical relationship(s) between pavement response and pavement performance. Some relationships are suggested. These are based on either the maximum tensile stress in the concrete or the maximum compressive stress in the foundation support, and they are related to either structural deterioration (Class 3 or 4 cracking) or to functional deterioration (decrease in PSR), but these relationships should be considered as tentative only, since they presently need to be more widely calibrated in various regions and under different climatic conditions.

The authors of this paper consider it of utmost importance that a major research effort be directed towards developing more reliable relationships between pavement response to load and pavement performance. Some major dilemmas still exist in this area. Why, for example, does the fatigue law for concrete tested in the laboratory have a power of 20 to 30, while that of in-service concrete pavements is only 3 to 4? The development of more reliable empirical relationships between response and performance is important, both on the project level for designing overlays or other rehabilitation measures, and at the network level for use in pavement management systems.

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