Pavement Design: The Fatigue Subsystem

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In the report of the workshop on the structural design of asphalt concrete pavement systems (1), it was indicated that the most frequently occurring mode of distress in asphalt highway pavements in the United States is fatigue cracking associated with traffic loads. That report also indicated that sufficient information is available to permit estimation of that type of distress as a part of the design process, and the recommendation was made that that information be used to check proposed designs. To assist the designer to make such an estimate, this symposium was organized. Its primary objective was to demonstrate that a fatigue subsystem, such as the one shown in Figure 1, of the pavement design and management system is a design tool that can be implemented now. Accordingly, the symposium assembled a series of papers that attempt to bring together the necessary information required to ascertain the potential for fatigue cracking.

Already a number of agencies are using some of the techniques discussed as a part of their design systems. For example, Shell has incorporated the fatigue factor in its design procedure for highway pavements since 1963 and for airfield pavements since 1970. Two working procedures recently developed, one by Havens, Deen, and Southgate in Kentucky for highway pavements and the other by Witczak at The Asphalt Institute for airfield pavements, are included in this Special Report.

It should also be noted that fatigue cracking received considerable attention at the Third International Conference on the Structural Design of Asphalt Pavements; the results presented at that conference (2) reinforce the recommendation made at the HRB workshop (1).

This paper summarizes the steps required to determine whether a particular pavement section will be subject to fatigue from a series of traffic loads (applied during some specified time period). Each box shown in Figure 1 is considered in a stepwise progression, and authors whose papers in this Special Report contain detailed information are indicated so that the basis for a particular step is apparent.

An example is included to illustrate the application of this procedure to determine the potential for fatigue cracking in an existing pavement consisting of an asphalt concrete layer and a base and subbase composed of untreated granular material.

Essentially the process consists of checking a specific section (which may be developed by an existing design procedure such as the AASHO Interim Guides or the California procedure for highway pavements or the Corps of Engineers method for airfield pavements) to ascertain the potential for fatigue distress. If the analysis indicates that fatigue cracking may occur sooner than desired, the design may then be modified. In effect, that process parallels the conventional structural engineering approach in which a structure is selected (designed), its behavior under anticipated service conditions is analyzed, and its adequacy with respect to a specific distress criterion is determined.

In the procedure outlined here, the state of knowledge does not permit an estimate of the time (or amount of traffic loading) to cracking of some specific areal extent. (The paper by Finn discusses the relation between cracking and performance and indicates that at this stage there is no definitive relation between the areal extent of cracking and performance.) Rather, if the estimate is based on laboratory tests (e.g., those discussed by Pell), the time or traffic carried will be that associated with crack initiation; if, on the other hand, the estimate is based on fatigue curves developed from analysis of test roads (Witczak) or in-service pavements (Havens, Deen, and Southgate), then the amount of cracking should be somewhat comparable to that observed in the pavements from which the data were obtained. The approach taken here is a phenomenological one based in part on the principles of continuum mechanics. Although it cannot be implemented at this time, fracture mechanics concepts should assist in the prediction of areal cracking; and thus the topic has been included (Majidzadeh and Ramsamooj) as an example of a promising future development in that area of design.

A USABLE FATIGUE SUBSYSTEM

The information presented in this Special Report provides the necessary basis for the engineer to reasonably ensure that his asphalt pavement structure will carry the traffic estimated to be applied to the pavement within some time period. This section briefly summarizes the steps shown in Figure 1; information presented in the symposium papers is used as a basis.

Traffic

For highway pavements the following traffic information (Fig. 1, box 1) should be determined: axle load distribution; wheel and axle configurations of representative vehicles using facility; contact (or tire) pressures of the various classes of vehicles; distribution of truck traffic throughout day, month, and year; vehicular velocities; and lane distribution of truck traffic for multilane facilities. For airfield pavements, the required information includes gear configurations of representative aircraft using facility, contact (or tire) pressures of the various aircraft, aircraft weights as affected by length of flight and takeoff and landing operations, daily and seasonal variations in aircraft movements, lateral distribution of loads on taxiways (particularly) and runways and longitudinal load distribution on runways, and aircraft velocities.

Methodology for estimating traffic is discussed by Hudson, and Deacon suggests a procedure for discretizing the spectrum of loads to make the traffic factor more manageable in the analysis stage. To simplify the process even further, Havens, Deen, and Southgate use the concept of equivalent axle loads, reducing all traffic to a common parameter—passages of an 18,000-lb axle load. Witczak has made use of the same concept for airfield pavements, defining all aircraft in terms of equivalent passes of a fully loaded DC-8-63F aircraft.

Although lateral distribution of traffic across a lane of a highway pavement is of little import at this time, the lateral distribution of aircraft on both taxiways and runways should be considered to ensure an economical design; Witczak demonstrates how that may be accomplished.

Although the concept of equivalent passes (or equivalent axle loads) is used in the 2 design procedures that have been included, that simplification need not be made. As noted above, Deacon illustrates how the spectrum of loading can be reasonably analyzed.

Environment

Because the response of asphalt-bound materials is dependent on temperature, distributions of temperature within layers containing such materials should be determined. In addition, the influence of environment (Fig. 1, box 1) as it influences the water contents (or effective stresses) of materials constituting pavement sections should be ascertained. In nonfrost areas such determinations will be concerned with potential seasonal changes in water contents (or effective stresses) of the various materials in the pavement structure; in frost areas the effects of freezing and thawing must be considered as well.

Temperature

Pavement temperatures can be computed from weather data (Hudson). That is done by solving the heat conduction equation by numerical techniques, such as finite-difference procedures or finite-element procedure, or by closed-form techniques as presented by Barber. Alternatively, a representative temperature can be estimated by the procedure suggested by Havens, Deen, and Southgate or by Witczak.

Of the computation procedures, the finite-difference method appears to be the most versatile and is recommended for use with layers of different conductivities and for areas where freezing occurs.

All of the available data indicate that temperatures can be reasonably estimated, and there is, therefore, no deterrent to considering that environmental factor in design.

Moisture

One of the most important environmental effects is that of water, particularly because it influences the response of materials in the pavement section to load and because it may cause undesirable volume changes (the latter was not of concern in this symposium).

For design purposes, the influence of water may be considered by measuring the properties of materials at water contents that are assumed representative of those that may develop at some time subsequent to construction, generally a saturated condition (Havens, Deen, and Southgate). In some instances, such procedures result in soil conditions that are not representative of those that develop in the field. Accordingly, it is desirable to have alternative procedures available that provide the designer with estimates of expected in situ moisture conditions and an indication of how those conditions might develop. For example, the procedure might be some measure of the rate of increase (or decrease) in water content of the subgrade soil with time or seasonal variations in the water content of untreated granular materials.

For semi-arid areas or portions of the lower half of the United States where little or no freezing of the subgrade occurs, soil moisture suction considerations should provide a useful (and practical) approach to estimating the equilibrium moisture conditions in fine-grained soils underlying pavement sections with treated layers resting directly on the subgrade (that is briefly discussed by Hudson).

For conditions where freezing and thawing can occur, recent work by Bergan (3) provides a means of defining the subgrade characteristics for use in design.

Construction Requirements and Effects

The influence of construction (Fig. 1, box 4) is not well defined at present and represents an area in which effort should be placed. [Sherman details available information on variations that can occur in paving materials and paving structures (4).] The influence of the degree of compaction on the stiffness characteristics of both treated and untreated materials can be determined (Barksdale and Hicks), and the influence of void content of asphalt-bound materials on their fatigue response can be approximated (Pell). Those effects can be incorporated into the design process as will be seen subsequently.

Design Structural Section

In the design of the structural section (Fig. 1, box 5), one can begin with a pavement cross section selected by an existing method of pavement design. Alternatively, a series of sections can be selected as described by Havens, Deen, and Southgate and by Witczak, and the specific section meeting all of the criteria considered in the design procedure (including fatigue) can be selected.

Materials Considerations

Available materials (Fig. 1, box 2), design asphalt concrete and other treated materials (Fig. 1, box 6), test asphalt concrete (Fig. 1, box 7), and test other pavement materials (Fig. 1, box 8) are interrelated. Related to them in the design process are the following: survey of subgrade soils traversed by the proposed route, selection of the most economical materials to be used in the pavement sections, and design of the asphalt concrete mixture and other treated materials. (By design is meant the selection of mix proportions, e.g., amount of asphalt in the case of asphalt-bound materials.) Emphasis in this symposium was placed only on those aspects of materials characterization necessary to examine the fatigue mode of distress. However, as noted by Pell, the design of the asphalt-concrete mixture to be used is related to the type of pavement in which the materials are used. Accordingly, it must be emphasized that mixture design and structural pavement design must be treated as part of the same process. Figure 1 shows that in stepwise progression.

Characterization procedures for asphalt concrete (Fig. 1, box 7) and the other paving materials (box 8) are discussed by Barksdale and Hicks. For untreated materials, a measure of stiffness (Deacon uses "measure of deformability" in the same connotation) termed the resilient modulus can be determined from a simple repeated load triaxial compression test defined as

$$E_R = \frac{\text{repeated axial stress}}{\text{recoverable axial strain}}$$

Such a technique is suggested by Witczak as one alternative for determination of subgrade stiffness in The Asphalt Institute design procedure.

If equipment necessary to determine such a modulus is not available, estimates from conventional tests can be used as noted by Barksdale and Hicks; i.e. (from Shell), E (in psi) = 1,500 CBR. The Kentucky procedure (Havens, Deen, and Southgate) makes use of that method, and Witczak suggests it as an alternative in The Asphalt Institute procedure. Witczak also suggests the use of approximate relations between E and the measure of stiffness determined from a plate-bearing test if such is available for an airfield pavement design.

For untreated granular materials, the same type of test can be used to determine a stiffness modulus as noted by Barksdale and Hicks. That modulus is dependent on the applied stresses

$$E_R = K \cdot \theta^n$$

where $\theta = \sigma_1 + \sigma_2 + \sigma_3$ (sum of principal stresses).

Alternatively, one can make use of the procedure suggested originally by the Shell investigators wherein the stiffness of the granular layer is proportional to the stiffness of the underlying material; i.e.,

$$(E_{gran}) = F(E_{subgrade})$$

where F = f(thickness of granular layer). In the Kentucky procedure, F is a function of the stiffness of both the subgrade and the asphalt-bound layer varying from 1 to 4 (Fig. 2, Havens, Deen, and Southgate).

Environmental influences must be considered when the response of untreated materials is assessed. Accordingly, proper water contents (or suctions) as well as the effects of freezing and thawing should be reflected in the stiffness measurements.

For asphalt-bound materials, stiffness as defined by the relation

$$S(t, T) = \frac{\sigma}{\epsilon}$$

where

$S(t, T)$ = mixture stiffness at a particular temperature and time of loading and
σ, ϵ = applied stress and strain,

can be used and either measured directly (Barksdale and Hicks), estimated by the Shell procedure, or estimated by a procedure such as that suggested by Witczak.

The stiffness characteristics of other treated materials are also briefly discussed by Barksdale and Hicks. It can thus be seen that sufficient guidelines are available to characterize materials for use in the fatigue subsystem.

Structural Analysis

To permit estimation of the potential for fatigue distress (Fig. 1, box 9) requires an estimation of the stress and deformations resulting from moving wheel loads on realistic representations of pavement structures. As noted in a number of the papers (Barksdale and Hicks; Deacon; Terrel; Havens, Deen, and Southgate; and Witczak), the assumption of the pavement responding as a layered elastic system appears reasonable at this time. Computer solutions are available (CHEV 5L, BISTRO or BISAR, and GCP-1) to facilitate determination of the stresses.

For the asphalt-bound layer, it must be recognized that, because of the temperature dependency of stiffness and the fact that temperatures vary throughout the day and the year, a simplified representation of that environmental influence must be obtained. Deacon discusses how this might be accomplished for the general case, and Witczak and Havens, Deen, and Southgate present specific procedures.

Determine Fatigue Properties

Considerable evidence is presented indicating that strain is a good damage determinant for fatigue (Fig. 1, box 10) in asphalt-bound materials (Pell; Terrel; Havens, Deen, and Southgate; and Witczak) and that an equation of the following form relates the intensity of strain to repetitions to fracture (Pell):

$$N_f = C\left(\frac{1}{\epsilon}\right)^m$$

Such a relation can be determined from laboratory tests (Pell) or from the analysis of the performance of in-service pavements (Havens, Deen, and Southgate, for Kentucky conditions, and Witczak, who used an analysis by Kingham of the AASHO Road Test pavements). Deacon suggests that a criterion based on the performance of in-service pavements may be more appropriate at this stage because such a criterion would necessarily include some amount of areal cracking.

Data are presented indicating that the distribution function for fatigue lives at a particular stress level can be represented as logarithmic normal (Pell). Use of a function of that type thus permits prediction of not only a mean fracture or service life but also the life corresponding to any desired confidence level.

Pell also notes that the linear summation of cycle ratios appears to be a reasonable cumulative damage hypothesis that permits the prediction of service life for a range in loading conditions from the results of simple-loading laboratory tests.

Fatigue Life Estimation

The paper by Deacon contains a step-by-step procedure whereby the fatigue life (Fig. 1, box 11) of a trial pavement section can be ascertained from the information obtained from the previous steps (Fig. 1, boxes 1 through 10). Essentially the procedure makes use of the linear summation of cycle ratios, which in simple form may be stated as

$$\sum_{i=1}^{i} \frac{n_i}{N_i} = D$$

where

n_i = number of applications at strain level i;
N_i = number of applications to cause failure in simple loading at strain level i; and
D = total cumulative damage.

In that relation, failure occurs when D equals or exceeds 1.0. Thus, the design procedure becomes one of checking the particular section to ensure that D is equal to or less than unity for the anticipated design conditions. [If the value of D is considerably less than one, the section may be underdesigned; when D is greater than one, a redesign (re-analysis) may be in order.] As Deacon notes, various temperatures as well as loading conditions can be considered and thus make the procedure adaptable to any environmental condition.

Terrel presents a number of examples illustrating the applicability of such an approach to analyze fatigue distress occurring in either trial or in-service pavements. The comparisons that he presents lend support to the use of this procedure for design purposes now.

In the design procedure described by Havens, Deen, and Southgate, the fatigue life and thus the design section is associated with the number of repetitions of 18,000-lb single axle load; some equivalency between the other loads and the 18,000-lb value is established. In the procedure described by Witczak, traffic is converted to equivalent passes of a DC-8-63F aircraft for design purposes. In this procedure, use is made of the linear summation of cycle ratios described above.

It must be emphasized that the estimate made by this technique is associated with no specific amount of cracking. If the analysis is based on laboratory fatigue tests, the traffic will be that associated with crack initiation and will provide a slightly conservative estimate. Comparisons of a number of different design procedures are shown in Figure 2 (5) to illustrate this point.

EXAMPLE

The example presented here illustrates the use of the procedure to check a pavement designed by the California procedure to ascertain whether it will be adequate to resist fatigue cracking for the 10-year period for which it was designed. The example is presented primarily to illustrate the concepts discussed at the symposium and has purposely been made brief. Details of the procedure may be found elsewhere (6).

The structural pavement section of this project east of Sacramento, California, on US-50 consists of 0.33 ft of asphalt concrete, 0.25 ft of asphalt-treated base, and 1.50 ft of untreated base and subbase on a subgrade of compacted dredger tailings.

Traffic

Data from the Perkins recording station, located near the test section, was obtained from the California Division of Highways. Average daily traffic for 1970 was reported to be as follows:

Vehicles	ADT	Percent of Trucks
All	25,000	
Trucks	1,400	100

Vehicles	ADT	Percent of Trucks
2 axle	800	57
3 axle	100	7
4 axle	100	7
5 or more axles	400	29

The Folsom project was placed in service either in late 1965 or early 1966. Because traffic data are not available for the earlier period, the 1970 data were reduced at a rate of 3 percent per year to obtain the 1966 ADT. Resulting estimates for trucks are as follows:

Axles	Number	Percent
2	708	57
3	87	7
4	87	7
5	223	18
6	137	11
Total	1,242	100

Truck traffic distribution throughout the day in hourly increments was estimated by the use of the average annual statewide data given in Table 1. Axle load groups (12 in this analysis) were related to operations in each classification by means of the monthly wheel load factors given in Table 2. Those factors permit the determination of wheel loads of a specific magnitude in each axle category.

Additional assumptions were as follows:

1. Truck operations are equally divided directionally;
2. Eighty-five percent of the operations in each direction occur in the lane adjacent to the shoulder (design lane); and
3. Truck operating speeds correspond to a time of loading of 0.02 sec in the asphalt concrete.

Analytically the traffic variables were related as follows:

$$AHT_{ij} = ADTT \cdot \frac{A_j}{100} \cdot \frac{HT_i}{100}$$

where

$ADTT$ = average daily truck traffic, 1 direction in design lane;
A_j = percentage of truck traffic of class j (2 axle, 3 axle, and so on);
HT_i = percentage of truck traffic in the hourly interval i (Table 1); and
AHT_{ij} = number of operations of class j in hour i (daily).

$$AXLD_{ik} = \sum_{j=1}^{S} (AHT_{ij} \cdot WLF_{kj})$$

where

WLF_{kj} = monthly wheel load factors to relate axle class j to axle load group k (Table 2); and
$AXLD_{ik}$ = matrix of the number of axle loads of group k in each hour i (on a monthly basis.

The $AXLD_{ik}$ values were expanded to an annual basis by incorporating climatic information as described in a subsequent section.

Environment

The climatic data used for the Folsom area (Table 3) were obtained from records

Figure 1. Diagram of a fatigue subsystem.

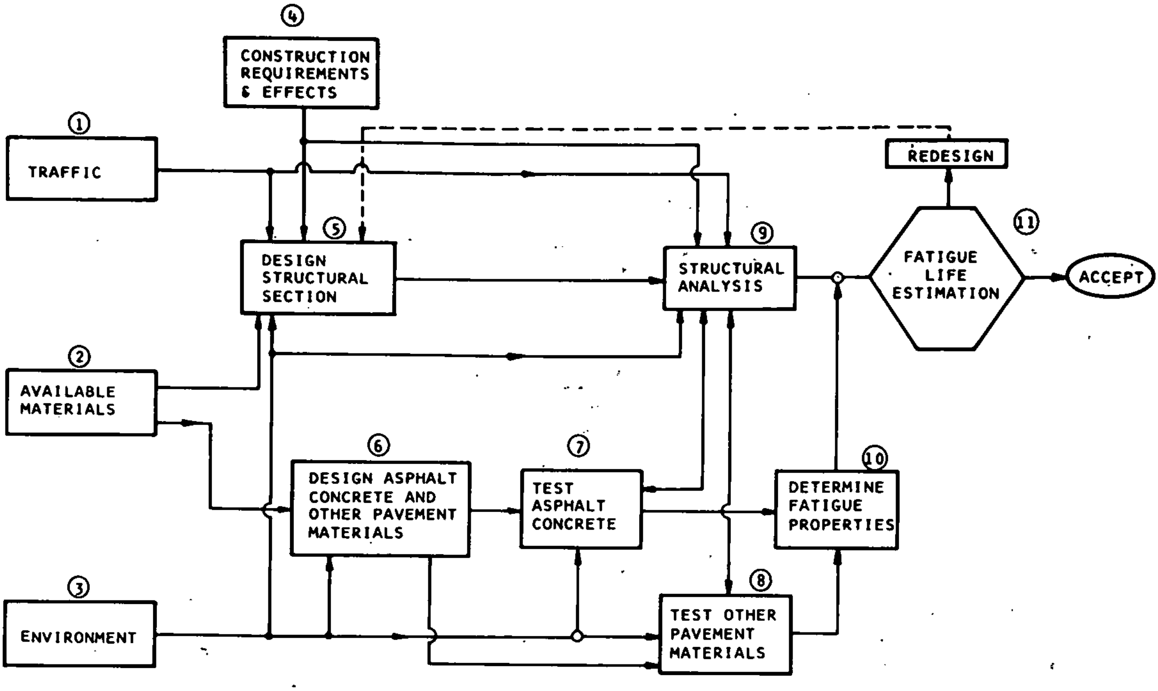


Figure 2. Comparison of thickness of asphalt concrete determined by existing procedures.

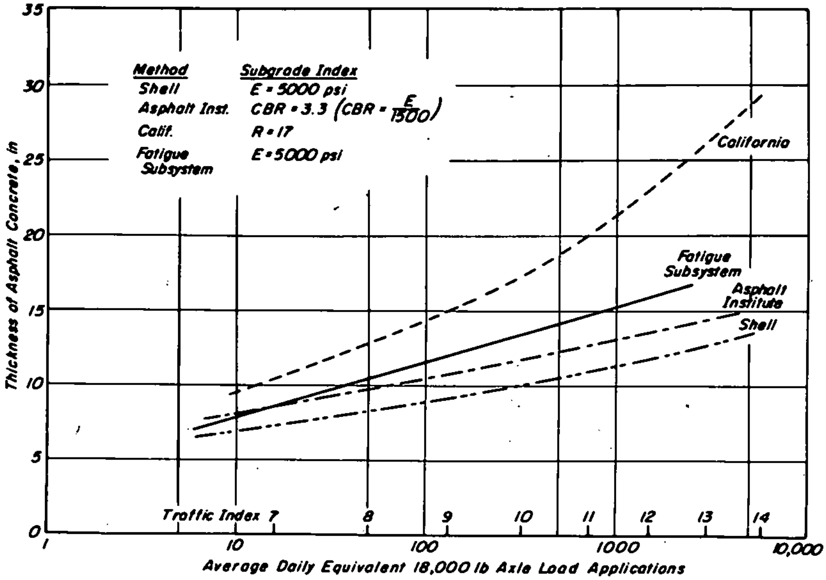


Table 1. Annual average daily statewide truck traffic in 1967.

Hour	Percent of Traffic	Hour	Percent of Traffic
12 to 1 a.m.	2.8	12 to 1 p.m.	5.2
1 to 2	2.7	1 to 2	5.6
2 to 3	2.9	2 to 3	5.7
3 to 4	3.1	3 to 4	5.7
4 to 5	3.5	4 to 5	5.4
5 to 6	4.1	5 to 6	4.5
6 to 7	4.2	6 to 7	3.8
7 to 8	4.3	7 to 8	3.5
8 to 9	4.8	8 to 9	3.3
9 to 10	5.0	9 to 10	3.2
10 to 11	5.2	10 to 11	3.3
11 to 12	5.2	11 to 12	3.0

Table 2. Monthly wheel load factors based on W-4 loadometer studies from 1966 to 1968.

Axle Load (kips)	2-Axle	3-Axle	4-Axle	5-Axle	6 or More Axles
Under 3	4.126	0.161	1.034	0.511	1.450
3 to 7	17.786	16.425	20.904	19.658	21.740
7 to 8	2.083	6.342	7.498	6.785	8.172
8 to 12	3.816	12.814	18.334	16.777	32.766
12 to 16	1.503	6.955	8.203	16.669	19.998
16 to 18	0.537	1.927	3.322	12.947	4.459
18 to 20	0.142	0.340	0.627	1.934	0.585
20 to 22	0.004	0.033	0.101	0.058	0.724
22 to 24	0.003	0.004	–	0.027	0.227
24 to 26	–	–	–	0.006	0.204
26 to 30	–	–	0.009	0.011	0.399
30 to 35	–	–	–	–	–

made at the Sacramento airport, the nearest location to the test site for which records were available for long periods of time.

Materials Characterization

Four sampling locations were selected, and slabs of asphalt concrete were taken for laboratory measurements of stiffness.

Granular base and subbase materials were obtained at the same locations to determine in-place densities, water contents, and gradations. Additional samples for preparation of laboratory test specimens were obtained from the plant supplying the materials to the job site. Subgrade samples were also obtained from the 4 test locations for in situ densities, water contents, and gradations.

Subgrade Soil

Subgrade soils were described as a combination of weathered slate, lava conglomerate, and silty clay at 2 of the sampling locations and as dredger tailings, red silty clay, and cemented cobbles at the other 2 locations. The descriptions were supplied by staff of the Materials and Research Department, July 1967. Atterberg limits for the materials are as follows:

Location	Liquid Limit	Plastic Limit	Plastic Index	Unified Classification
1	33	20	13	CL
3	25	19	6	CL
4	30	29	1	ML

Samples for laboratory resilience testing were prepared by separating the soil into 3 fractions: passing No. 4 sieve (approximately 70 percent), $^3/_4$ in. by No. 4 (approximately 10 percent), and retained on $^3/_4$ in. sieve (approximately 20 percent). The weight of material passing the No. 4 sieve equaled that of the field samples, while material greater than $^3/_4$ in. was wasted and replaced with an equivalent weight of $^3/_4$ in. by No. 4 material. The modified material was then compacted by kneading compaction into 4-in. diameter by 8-in. high specimens, and a series was prepared to cover the range of dry densities and water contents observed in the field sampling program.

Repeated load tests were conducted using a constant cell pressure of 3 psi and a repeated deviator stress ranging from 1 to 5 psi applied at a frequency of 20 repetitions per minute and a duration of 0.1 sec. Resilient moduli were ascertained from recoverable deformations measured over the center 4 in. of the specimens with dual LVDT's clamped to the membrane surrounding each specimen after 1,000 stress repetitions.

From the tests (Table 4) isolines of resilient moduli could be developed (Fig. 3) for a repeated deviator stress of 2 psi. The development of such relations for a range in deviator stresses makes it possible to define a modulus versus deviator stress relation for any specified water content and dry density.

Subbase

The subbase material was a well-rounded gravel conforming to California specifications for a class 1 aggregate subbase.

Prior to specimen preparation, all material was separated into individual fractions by dry sieving, and material retained on the $^3/_4$-in. sieve was wasted. As with the subgrade soil, the percentage by weight passing the No. 4 sieve was maintained for the laboratory specimens the same as for the field samples. In addition, the plus $^3/_4$-in. material was replaced by an equivalent weight of $^3/_4$ in. by No. 4 material.

Specimens for the repeated load triaxial compression tests were prepared by vibratory compaction at water contents representative of those existing in the roadway at the time of sampling and to densities corrected from field values to account for the modified grading.

Table 3. Climatic data.

Month	Avg Air Temperature	Daily Air Temperature Range	Avg Wind Velocity	Insolation	Sky Cover
Jan.	45.2	16.0	8.7	182.0	7.1
Feb.	49.2	18.8	8.7	287.0	5.9
Mar.	53.4	22.8	9.7	426.0	5.5
Apr.	58.4	26.1	9.3	547.0	4.6
May	69.0	28.5	9.8	642.0	3.9
June	70.5	32.1	10.5	701.0	2.0
July	75.4	36.0	9.7	685.0	0.9
Aug.	74.1	35.6	9.7	621.0	1.3
Sept.	71.6	33.2	8.3	506.0	1.7
Oct.	63.5	28.2	7.4	374.0	3.2
Nov.	52.9	22.6	6.8	248.0	5.4
Dec.	46.4	16.5	7.4	157.0	6.9

Table 4. Repeated load test results for subgrade soil.

Site	Test	Dry Density (lb/ft^3)	Water Content (percent)	Resilient Modulus (psi × 10^{-3}) 1-psi σ_d	2-psi σ_d	3-psi σ_d	Poisson Ratio at σ_d = 3 psi
1	1	133.0	9.6	2.5	1.9	2.2	—
	2	132.2	9.7	7.8	5.2	4.2	0.43
	3	128.2	10.1	1.5	1.7	2.1	0.45
	4	127.4	7.3	2.0	2.6	2.2	—
2	1	134.1	9.8	—	14.0	14.0	—
3	1	137.4	8.6	16.0	7.6	9.0	0.58
	2	122.8	9.1	15.2	14.3	13.9	0.49
	4	139.0	5.8	63.8	54.0	46.8	0.33
	5	136.2	6.3	95.0	56.0	47.6	—
	6	138.4	7.6	63.8	56.1	55.4	0.42
4	1	130.7	9.3	15.9	19.7	12.4	—
	2	130.5	9.8	5.4	6.1	6.4	0.52
	3	130.6	9.4	8.3	6.4	7.2	0.50

Note: Confining pressure = 3 psi except for test site 2 material where confining pressure = 2 psi.

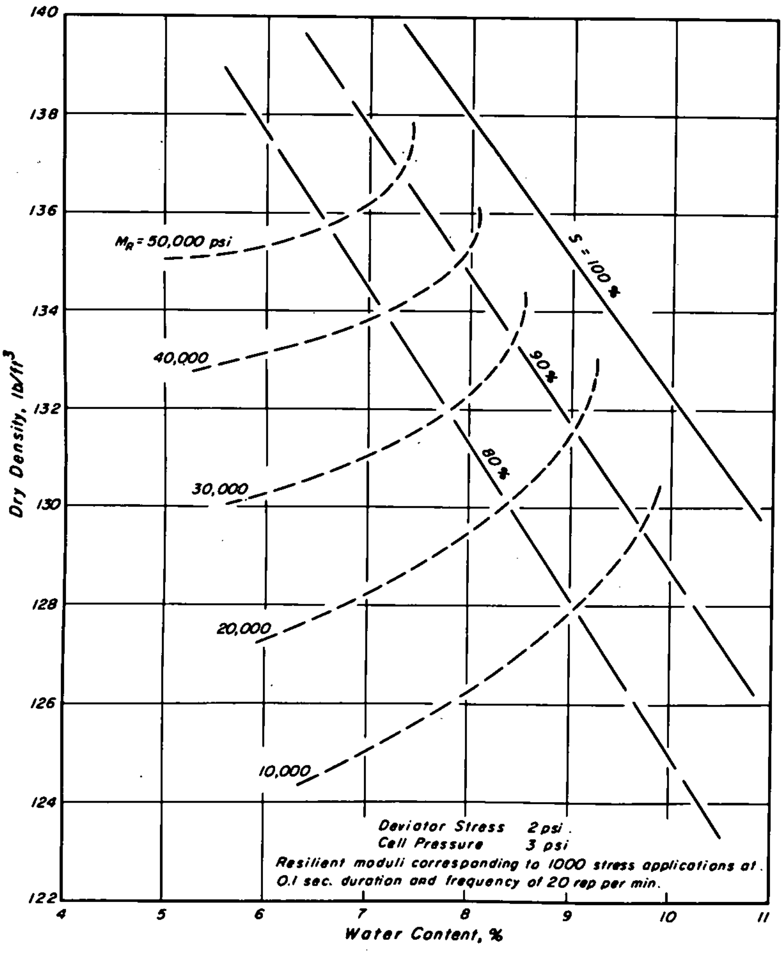


Figure 3. Water content-dry density-resilient modulus relation for subgrade soil.

The compacted specimens were subjected to a range of repeatedly applied deviator stresses and a range in static cell pressures (σ_3). Both axial and circumferential deformations were measured with LVDT's clamped to the central portion of the specimen. Resilient modulus and resilient Poisson's ratio were determined after 200 axial stress repetitions at a particular state of stress for a particular specimen. The effects of stress history were minimized by first subjecting each specimen to 200 stress repetitions at an intermediate stress ratio, then at a range of stress ratios from low to high, and then at the ratios in reverse.

Linear regression analyses of the logarithm of modulus, M_R versus the sum of principal (total) stresses θ, were obtained for each of the 9 test series and for all of the data together. Although the results for the individual series were generally good (correlation coefficients greater than 0.85), the results for all data together were somewhat poorer and reflected the influence of variations in density and water content.

For the pavement analysis, the equation based on all data

$$M_R = 7730\,(\theta)^{0.46}$$

has been used (correlation coefficient of 0.68).

The dependence of Poisson's ratio on stress was also ascertained, and the following relation has been used in the pavement analysis:

$$\nu = 0.13 + 0.05\,(\sigma_1/\sigma_3)$$

Base

The base material is a crushed gravel conforming to California specification for a class 2 aggregate base.

Resilient moduli were determined in the same manner as for the subbase material, and a linear regression using all the test data resulted in the following relationship:

$$M_R = 3470(\theta)^{0.65}$$

The correlation coefficient was 0.96, indicating less scatter than for subbase data.

For Poisson's ratio, the following relation was established.

$$\nu = 0.16 + 0.08\,(\sigma_1/\sigma_3)$$

Asphalt Concrete Surface

Stiffness characteristics of the asphalt concrete from the pavement specimens were measured in the laboratory at only 1 temperature, 68 F, and 1 time of loading, 0.1 sec. To define the stiffness over the range of temperatures encountered in the field and for a range in loading times, use was made of the Shell procedure and the following recovered asphalt properties for both base and surface courses (control section):

Location	Course	Penetration at 77 F (dmm)	Ring and Ball Softening Point (deg F)
1	Surface	38	132
1	Base	31	135
2	Surface	26	141
2	Base	28	138

The resulting relations between mixture stiffness and temperature for times of loading of 0.02 sec (fast traffic) and 0.1 sec (slow traffic) are shown in Figure 4.

Poisson's ratio was assumed to vary from 0.30 to 0.35 for the temperature range encountered.

In addition, for estimates of the stiffness in situ as a function of temperature using Barber's solution, the asphalt concrete was assumed to have the following thermal characteristics: thermal conductivity = 0.70 Btu/ft^2/hour/deg F/ft; specific heat = 0.22 Btu/lb · deg F; and surface coefficient = 0.85.

Fatigue Characteristics of Asphalt Concrete

Fatigue data for the mixtures used in the project were obtained by testing specimens recovered from the road and by testing laboratory prepared specimens. Those data provide a basis for selecting the appropriate data to be used in the performance estimate.

For the Folsom pavement, which had a 7-in. asphalt concrete layer, stiffnesses were estimated to range from about 100,000 to 2,000,000 psi. For those conditions, the controlled-stress mode of loading provides a conservative estimate of fatigue life. It is probable that an intermediate mode of loading (as defined by the mode factor, 6) would be appropriate. To determine that condition (or conditions), mode factors were computed for a range in asphalt concrete stiffnesses and loads as shown in Figure 5. Computed values ranged from about -0.7 to +0.4 depending on the wheel load and mix stiffness.

Strain versus fatigue life relations for intermediate modes of loading will lie between the limits defined by the controlled-stress and controlled-strain modes of loading. For that analysis, it was assumed that an intermediate mode could be defined by a direct interpolation between the limiting relationships; e.g., the strain-fatigue life relation corresponding to a mode factor of zero would lie midway between the 2 curves.

Some measure of the difference was obtained in controlled-strain tests that were performed at 68 F; results are shown in Figure 6. Comparison of the controlled-stress and controlled-strain data (68 F) are shown in Figure 7. Lines representing the intermediate modes are also shown. The data are based on specimens whose stiffnesses ranged from 160,000 to 170,000 psi. To cover the range in stiffnesses anticipated required some adjustment in the data, and a series of relations like that shown in Figure 7 are required to cover the range of stiffness expected in service.

For example, for a stiffness of 150,000 psi (the lowest value used in the analysis), the controlled-stress relation was steepened slightly and the controlled-strain relation was flattened. Similar adjustments were also made for other stiffnesses. At stiffnesses greater than 700,000 psi, the controlled-stress and controlled-strain lines were assumed to coincide (that decision being based on the experience of other investigators, e.g., Pell).

Structural Analysis

Figure 8 shows the pavement section as well as the material characteristics and other variables considered in the analysis. For convenience the 7-in. asphalt concrete section was considered as 1 layer. The stiffness of the asphalt concrete and the resilient properties of the subgrade were considered to be the primary variables affecting the deflection of the pavement under load. The CHEV 5L (w/iteration) program was used. Because the program only considers a variable modulus in the granular layers, manual iteration was also required to account for the variability in Poisson's ratio.

Deflections computed by this process are shown in Figure 9 as a function of asphalt concrete stiffness. Included are the deflections measured in January 1968. For the temperature conditions at the time of measurement (air temperature = 48 F, and surface temperature = 53 F), the asphalt concrete stiffness was estimated to be about 1,000,000 psi (time of loading of 0.1 sec).

Comparison of the measured deflections with computed values indicates that the dry, high-density subgrade condition is most representative of the in situ condition. Hence, those subgrade properties have been used in the estimate of fatigue life discussed in the next section.

Figure 4. Computed relations between mixture stiffness and temperature.

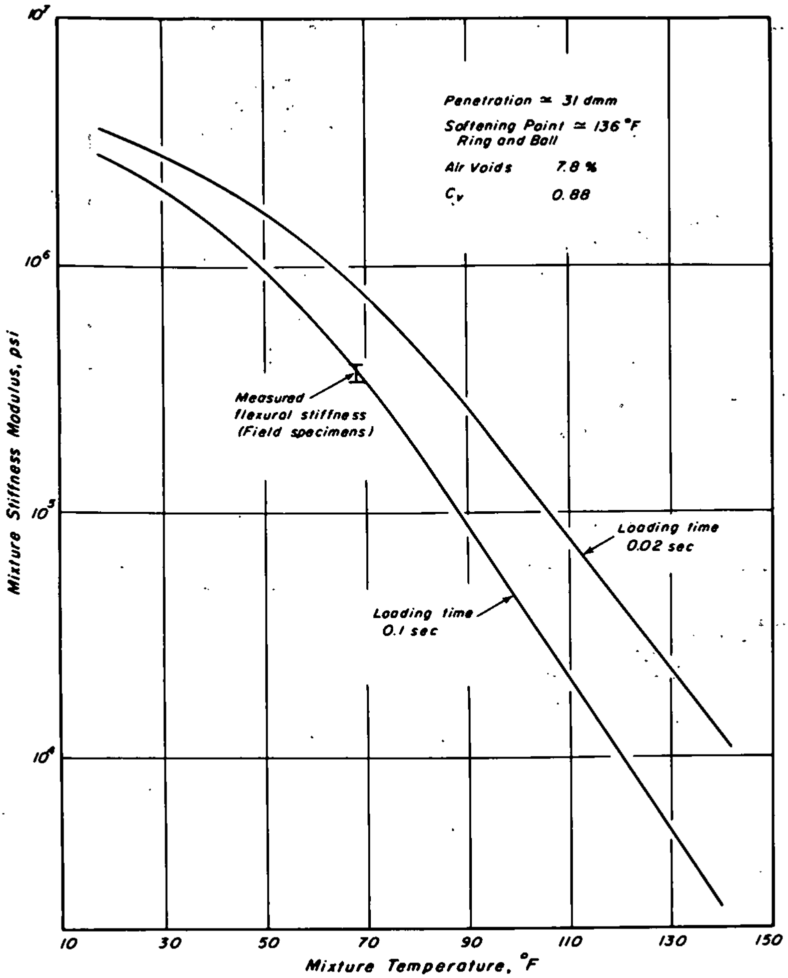


Figure 5. Mode factors as function of asphalt concrete stiffness and axle load.

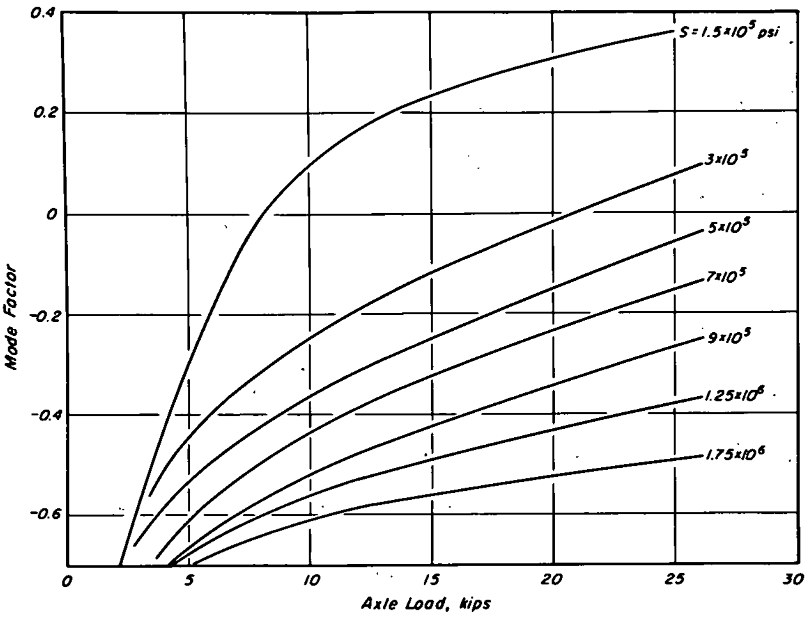


Figure 6. Initial bending strain versus service life in controlled-strain (deflection) tests on laboratory-prepared specimens.

Figure 7. Fatigue curves for series of modes of loading.

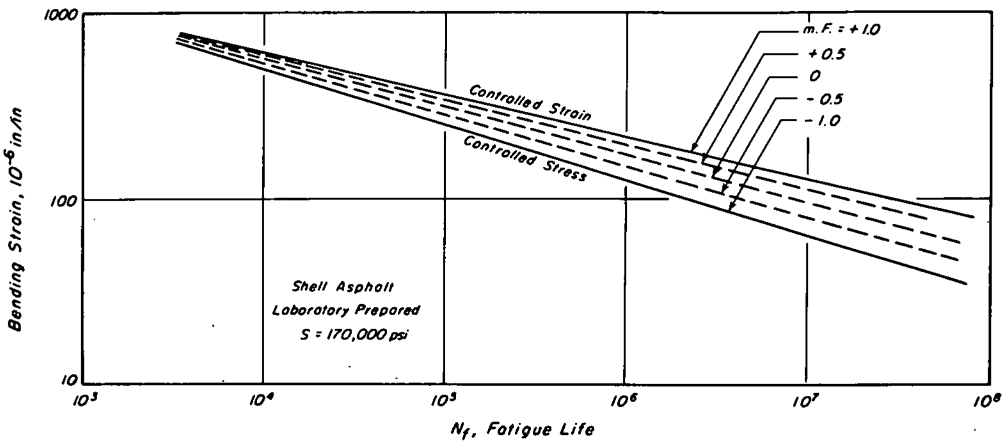
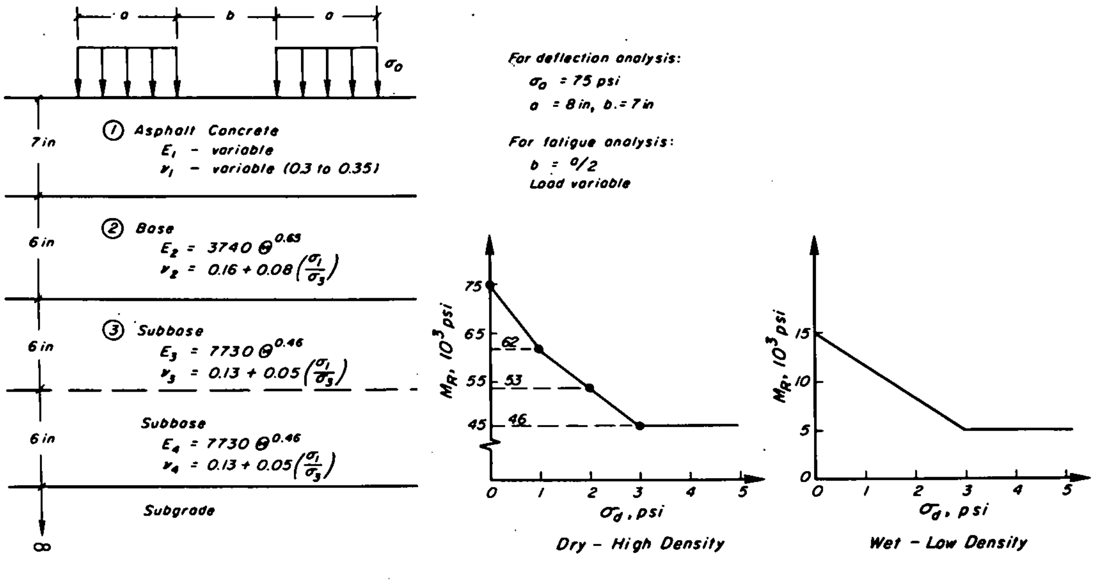


Figure 8. Pavement section used in analysis.



During this analysis, it was observed that the vertical stresses at the subgrade surface were of sufficient magnitude (at least 3 psi) so that the modulus could be considered constant. That simplifying assumption has been used in all subsequent calculations.

To consider the effects of temperature on stiffness, we developed a simplified procedure in which the computer is used. The procedure permits determination of the temperature T_i and stiffness S_i (Fig. 10) at the beginning and end of each hour of the day (25 values per day) at each specified depth d_i.

If w_i is defined as

$$w_i = \left(\frac{d_i - d_{i-1}}{2}\right) + \left(\frac{d_{i+1} - d_i}{2}\right)$$

(w_i and w_n must be adjusted for the boundaries), then w_i is the increment of total depth that each T_i and S_i represent. Means of temperature and stiffness can then be determined.

$$T_{mean} = \left(\sum_{i=1}^{n} T_i \cdot w_i\right) \Big/ D$$

$$S_{mean} = \left(\sum_{i=1}^{n} S_i \cdot w_i\right) \Big/ D$$

Twenty-five such values are computed for each day.

The values for the beginning and end of each hour are averaged, and a representative value of the stiffness of the asphalt concrete during the hourly increment is obtained. Because the rate of change of stiffness in any mass of asphalt concrete is not large, this averaging was considered to be a reasonable estimate of the mean stiffness during the 1-hour interval.

Use of the mean stiffness also permits the full depth of asphalt concrete to be represented by a single modulus. Although there are some advantages to that simplification (as compared to the characterization of the asphalt concrete as 2 or more layers), use of the single value has some limitations. For example, a system consisting of soft-stiff layers versus one of stiff-soft layers, both with the same mean, does not exhibit the same stresses and deformations. However, for the approximately sinusoidal distribution of temperature with time used in the analysis, where the heating period is nearly balanced with the cooling period, differences tend to be compensated.

The computations result in 24 x 12 = 288 values of stiffness modulus to represent the daily and seasonal variation of stiffness modulus. Those values were then grouped, and the frequency matrix given in Table 5 was prepared. The frequencies are the number of months that a given interval of 1 hour had a mean stiffness in the group shown.

If these frequencies are designated $f_{i\ell}$, then traffic and stiffness can be related by

$$ADL_{\ell k} = \sum_{i=1}^{24} (f_{i\ell} \times AXLD_{ik})$$

where $ADL_{\ell k}$ = annual number of applications of axle load group k to the pavement when the stiffness occurs in stiffness group ℓ.

Stresses and deformations at the base of the asphalt concrete were determined by the multilayer linear elastic solution for a range of axle loads and asphalt concrete stiffnesses.

Results of those computations are shown in Figure 11 in the form of strain versus axle load with isolines of constant stiffness. Thus, the strain corresponding to each of the 12 axle load groups and each of the 7 stiffness groups can be obtained.

Figure 9. Computed and observed pavement deflections.

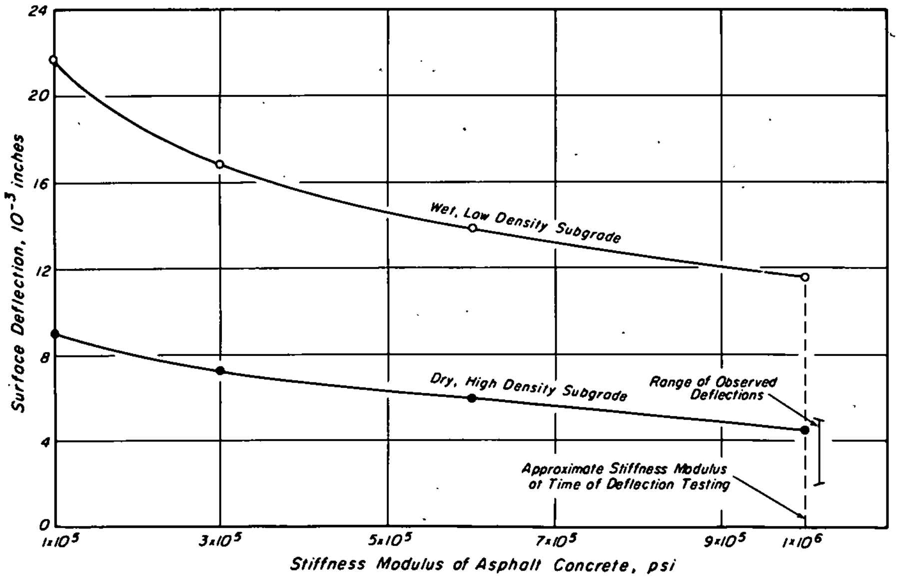


Figure 10. Notation for stiffness and temperature determinations.

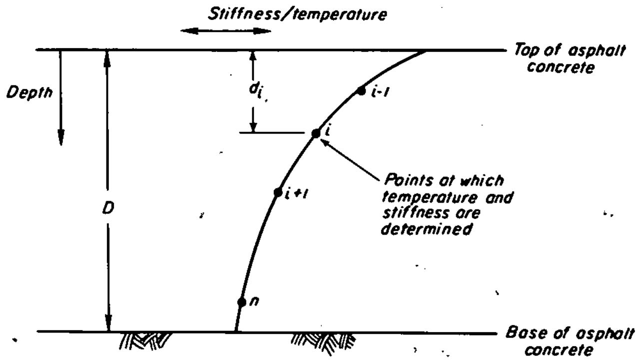


Table 5. Frequency of occurrence of pavement stiffness.

	Stiffness Group (psi × 10^6)						
	1.5 to 2.0	1.0 to 1.5	0.8 to 1.0	0.6 to 0.8	0.4 to 0.6	0.2 to 0.4	0.1 to 0.2
	Midpoint Stiffness (psi × 10^6)						
Time	1.75	1.25	0.90	0.70	0.50	0.30	0.15
12 to 1 a.m.	3	3	1	1	3	1	-
1 to 2	3	3	2	2	2	-	-
2 to 3	3	3	2	2	2	-	-
3 to 4	4	2	2	2	2	-	-
4 to 5	4	2	2	2	2	-	-
5 to 6	4	2	2	2	2	-	-
6 to 7	4	2	2	2	2	-	-
7 to 8	3	3	2	2	2	-	-
8 to 9	3	3	2	2	2	-	-
9 to 10	3	3	2	1	2	1	-
10 to 11	3	2	2	1	3	1	-
11 to 12	3	2	1	2	2	2	-
12 to 1 p.m.	2	3	1	2	1	3	-
1 to 2	2	3	-	2	1	3	1
2 to 3	2	3	-	2	1	3	1
3 to 4	2	3	-	1	2	2	2
4 to 5	2	2	1	1	2	2	2
5 to 6	2	3	-	1	2	1	3
6 to 7	2	3	-	1	2	1	3
7 to 8	2	3	-	2	1	2	2
8 to 9	2	3	-	2	1	2	2
9 to 10	2	3	1	2	-	4	-
10 to 11	2	3	1	2	2	2	-
11 to 12	3	1	2	1	2	2	-

Figure 11. Bending strain on underside of asphalt layer and axle load for stiffness range.

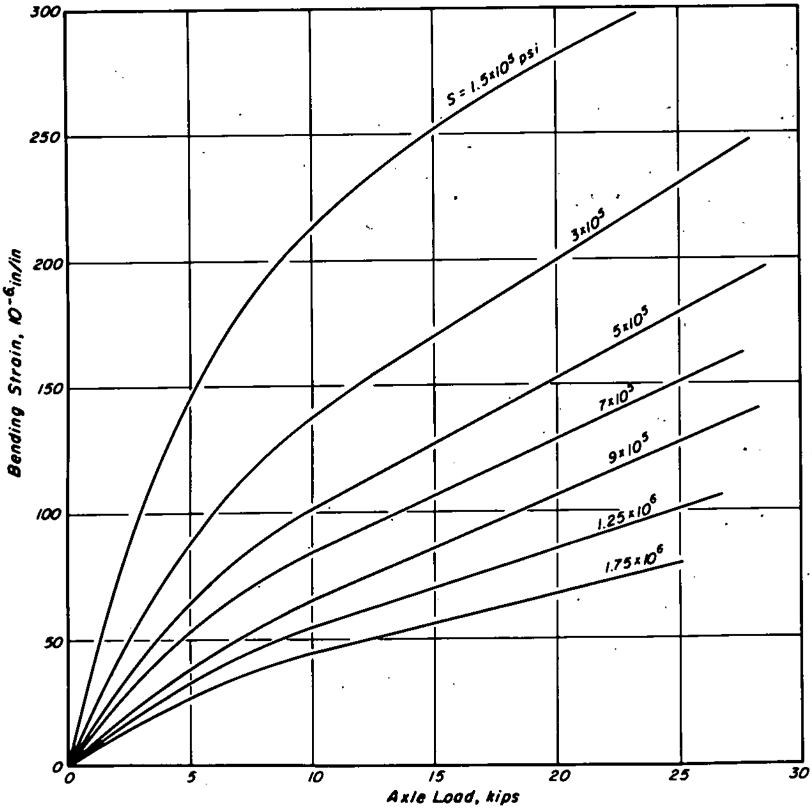


Table 6. Fatigue life matrix.

Axle Load	1.75[a]			1.25[a]			0.90[a]			0.70[a]			0.50[a]			0.30[a]			0.15[a]		
	ϵ	MF	N_f[b]	ϵ	MF	N_f[b]	ϵ	MF	N_f[b]	ϵ	MF	N_f[b]	ϵ	MF	N_f[b]	ϵ	MF	N_f[b]	ϵ	MF	N_f[b]
1.5-1	9	-1	10^{99}	11	-1	10^{99}	13	-1	10^{99}	18	-1	10^{99}	23	-1	10^{99}	30	-1	10^{99}	54	-1	10^{99}
5-2	26	-0.8	10^{99}	33	-0.7	10^{99}	39	-0.65	10^{99}	53	-0.6	10^{99}	64	-0.5	10^{99}	87	-0.4	7.0	146	-0.3	1.6
7.5-3	37	-0.65	10^{99}	46	-0.6	10^{99}	54	-0.6	10^{99}	71	-0.5	6.5	86	-0.45	4.5	116	-0.3	2.4	165	0.0	0.95
10-4	45	-0.6	10^{99}	55	-0.55	10^{99}	65	-0.50	10^{99}	84	-0.45	3.6	102	-0.35	2.7	138	-0.25	1.4	212	0.1	0.72
14-5	54	-0.55	10^{99}	66	-0.50	10^{99}	81	-0.95	2.75	101	-0.35	1.8	122	-0.3	1.45	163	-0.15	0.95	246	0.2	0.38
17-6	61	-0.55	10^{99}	76	-0.50	2.8	93	-0.4	1.7	115	-0.3	1.15	136	-0.2	1.0	180	-0.1	6.0	264	0.3	0.33
19	65	-0.55	10^{99}	82	-0.45	1.95	102	-0.35	1.2	124	-0.25	0.84	148	-0.2	0.71	194	-0.05	0.52	275	0.3	0.29
21	70	-0.5	3.3	88	-0.45	1.5	110	-0.35	0.76	132	-0.2	0.68	158	-0.15	0.58	206	0.0	0.44	285	0.3	0.24
23	75	-0.5	2.5	94	-0.4	1.1	118	-0.3	0.62	141	-0.2	0.54	168	-0.1	0.49	218	0.05	0.33	295	0.35	0.22
25	75	-0.5	2.1	100	-0.4	0.88	126	-0.3	0.50	150	-0.15	0.43	178	-0.05	0.40	232	0.05	0.23	304	0.35	0.20
28	86	-0.5	1.4	108	-0.35	0.64	138	-0.2	0.36	163	-0.1	0.32	194	-0.0	0.28	250	0.1	0.27	318	0.4	0.22

Note: Matrix based on Shell laboratory data where $N_f = \infty$ at $\epsilon < 70 \times 10^{-6}$ in./in.

[a]Stiffness, psi x 10^6.

[b]Except for the values 10^{99}, all other values are x 10^6.

Fatigue Life Determination

Data necessary to complete the fatigue analysis were set up in the form given in Table 6. The strain corresponding to each stiffness modulus and axle load group was first obtained from data shown in Figure 11. A mode factor was then obtained from data shown in Figure 5; the number of repetitions to failure for the specific strain level and mode factor could then be obtained from a series of charts of the form shown in Figure 7. These repetitions were designated $ENNF_{\ell k}$, where ℓ is the stiffness group and k is the axle load group. Then,

$$\sum_{\ell}\sum_{k} (ADL_{\ell k}/ENNF_{\ell k}) = \text{annual damage}$$

Traffic values were then incremented by an expansion factor (3 percent), and the process was repeated until the total damage was equal to one.

Fatigue life predictions were made by the use of 2 sets of strain-repetitions to failure relations.

1. Fatigue life was based on tests of laboratory results. Individual estimated lines were linearly extrapolated to the full range of strain computed. These data resulted in a predicted fatigue life of 9.5 years.
2. The same data were used as in the preceding relation except that the assumption was made that strains less than 70×10^{-6} in./in. caused no fatigue damage (6). The purpose of that calculation was to estimate the cumulative effect that the large number of small loads have on the fatigue life estimate. Those data resulted in a predicted fatigue life of 12 years.

Discussion of Example

In this example, the California Division of Highways had designed the pavement for a 10-year life. Based on the measured traffic data for 1970 and reasonable extrapolations to both the initial and later stages of its service life, the pavement can be expected to perform reasonably from a fatigue standpoint. That is, based on the mean fatigue data, a service life in the range 9 to 12 years appears feasible.

This example demonstrates the feasibility of design to preclude fatigue and illustrates many of the concepts described in detail in the subsequent papers in this report.

SUMMARY

The technology necessary to design airfield and highway pavements to minimize fatigue distress was presented at the symposium. The purpose of this introductory discussion is to develop a framework for the papers and to demonstrate that the results of research efforts during the past 10 years can be put into practice to make reasoned engineering decisions regarding the behavior of pavements relative to load-associated cracking.

The information discussed here holds the potential (as compared with and when used with conventional procedures) to assist the engineer to better utilize available materials; improve the reliability of the design estimate; consider loadings larger than those now applied, if required; better define the role of construction; and analyze existing pavements.

REFERENCES

1. Structural Design of Asphalt Concrete Pavement Systems. Highway Research Board Spec. Rept. 126, 1971.
2. Proceedings. Third International Conference on the Structural Design of Asphalt Pavements, Vol. 1, University of Michigan, Ann Arbor, 1972.

3. Bergan, A. T. Some Considerations in the Design of Asphalt Concrete Pavements for Cold Regions. Univ. of California, Berkeley, PhD dissertation, Sept. 1972.
4. Sherman, G. B. In Situ Materials Variability. Highway Research Board Spec. Rept. 126, 1971, pp. 180-188.
5. Monismith, C. L., and McLean, D. B. Symposium in Technology of Thick Lift Construction: Structural Design Considerations. Proc., AAPT, 1972.
6. Monismith, C. L., Epps, J. A., Kasianchuk, D. A., and McLean, D. B. Asphalt Mixture Behavior in Repeated Flexure. Univ. of California, Berkeley, Rept. TE 70-5, 1971.