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EVALUATION OF THE FLEXIBLE PAVEMENT ON THE TOMEI EXPRESSWAY BY THE FATIGUE CRACKING CONCEPT

By *Takashi* KOBAYASHI* and *Eugene L.* SKOK, *Jr.***

1. INTRODUCTION

Highway pavement design is largely based on research and experience dealing with performance of pavement sections or structures consisting of various thicknesses and combinations of surface, base, and subbase layers over existing subgrade soils. Among those extensive studies, it is generally accepted that fatigue cracking of the bituminous layer is initiated by the tensile strain at the base of the asphalt layer under the repeated traffic loads and that the excessive deformation of the pavement is resulted by the excessive vertical compressive strain in the subgrade. Excessive rutting due to improper bituminous mix design under heavy traffic can exist. However, this should be taken care under the mixture design problem.

In this paper, an application of the fatigue concept to estimate the potential cracking of an in-service highway (The Tomei Expressway) is discussed in corporation with material and structural properties of the pavement, and traffic and/or environmental conditions. Main assumptions uterized in this paper are listed as below;

a) Among a number of possible causes of the pavement distress, only distress due to fatigue cracking of the asphalt layer is considered.
b) Fatigue cracking is initiated from the bottom strain of the asphalt layer.
c) Multi-layered elastic system was chosen to simulate the pavement behavior after comparison of measured and calculated values. Five layer system, four asphalt bound layers and a granular layer, was adopted for the multi-layered elastic system computation.
d) Property change of the asphalt cement due to aging is not considered.
e) Fatigue equation developed by the Asphalt Institute can be applied to the asphalt concrete of the Tomei Expressway.
f) Assumptions for the multi-layered elastic system analysis are described in Reference 3.
g) Estimation of the pavement life due to fatigue cracking is only applied to the investigated section (T-2 Section) on the Tomei Expressway.

The outline of this paper is briefly described as follows.

First, the results obtained from the field dynamic tests on an in-service highway are presented to describe the pavement behavior under a moving wheel load. Chapter two deals with the Tomei Expressway Pavement Research Program and its results taken from T-2 Section (Kawasaki-Yokohama, East bound).

Second, the multi-layered elastic system computer program developed by Chevron Research Corporations was used to compute the radial strain at the bottom of the asphalt layers and the deflection in each layer, these values are compared with those obtained from the field dynamic tests in February, 1968. Since a fairly good agreement was obtained when comparing the observed values in the winter and the computed values under the simulated conditions, the pavement responses under the other loading and environmental conditions were estimated using this simulation.

Third, the asphaltic concrete fatigue theory is introduced and the pavement life is estimated comparing the number of repetitions to failure obtained in the laboratory tests with the number of equivalent axle load applications on Test Section T-2. To determine the number of repetitions to failure of the asphalt concrete, the fatigue equation developed by the Asphalt Institute was chosen. Based on the radial strain at the bottom of the asphalt layers computed by the multi-layered elastic system and the Asphalt Institute

* Engineer, Nihon Doro Kodan, Tokyo First Bureau of Traffic Operations.
** Professor of Civil Engineering, University of Minnesota.

fatigue equation, axle load equivalency factors were developed considering the load and pavement temperature. An extensive traffic analysis was made using the traffic volume and weight distribution data obtained from the Tomei Expressway. These equivalency factors and traffic analyses developed were used to estimate the total equivalent axle load applications during the service life on Test Section T-2 and the number of years to failure of the three lanes of this section were thus determined.

2. THE TOMEI EXPRESSWAY PAVEMENT RESEARCH PROGRAM AND TEST RESULTS

(1) Pavement Research Program

In the late 1950's the American Association of State Highway Officials completed the third full-scale test of pavement behavior under controlled truck traffic. The AASHO Road Test Reports stimulated the interest in the structural design of highway pavements among the highway engineers in Japan. Under these circumstances, The Tomei Expressway Pavement Research Program was set up in order to modify the AASHO Road Test results to meet the specific conditions in Japan.

The testing program will be broken down into three parts, the construction measurements, the dynamic tests immediately after the completion of construction, and the periodic measurements after completion. In the following section, the test results obtained from the dynamic test and adopted for the analyses in this paper are briefly presented. Data are taken from the results on Test Section T-2. Detailed information is obtained from Reference 1.

(2) Field Dynamic Test Results at the Completion of Construction

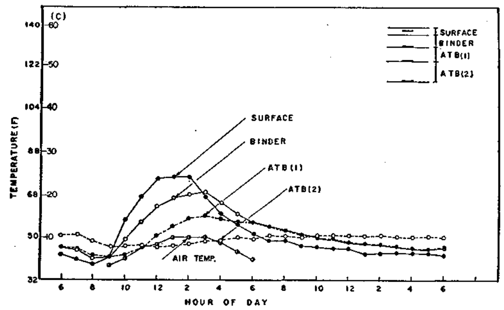

Fig. 1 Pavement Temperature Record (Winter).

The dynamic tests were made operating the test trucks with specified axle loads and specified travelling speeds. Specified axle loads are 14, 10, 6, and 2 ton. Specified travelling speeds are 72, 40, and 4 km/hr.

Fig. 1 illustrates a typical 24 hour record of the pavement temperature at four different depths in midwinter February 14, 1968. Similar temperature records in late fall and summer are adopted from Reference 2. These temperature conditions have been used in order to estimate the stiffness modulus of the asphalt layers in Chapter III.

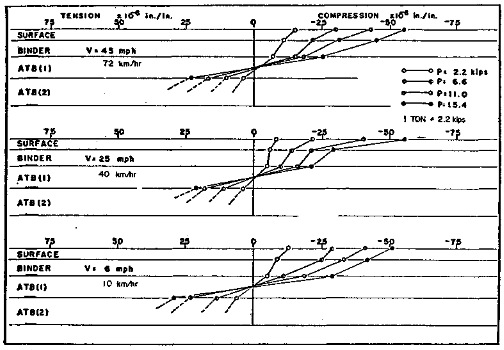

Fig. 2 Vertical Distribution of the Radial Strain in the Case of Four Different Axle Loads and Three Traveling Speeds.

Fig. 2 shows the vertical distribution of the radial strain in case of four different axle loads and three different travelling speeds on the center of the gauge point. Unfortunately the bottom face of ATB(2) strain gauge, which might gives us the most critical value for the bituminous layers and hence were most desired to obtain, were not operative even immediately after construction.

3. MULTI-LAYERED ELASTIC SYSTEM ANALYSIS

(1) Multi-Layered Elastic System Theory and the Computer Program

A flexible pavement consists of several layers which have very different material properties. Many investigators and researchers have been trying to deal with pavement structure as a simulated elastic layered system.

Burmister laid much of the ground work for solution of elastic layers on a semi-infinite elastic subgrade followed by Hank and Scrivner, Acum and Fox, and so forth.

The Chevron Research Corporation (3) has developed multi-layered pavement computer program. This program will give the distribution of stresses and deformations of such an n-layered system when subjected to a load uniformly distributed on a circular area on the free surface of the semi-infinite solid. The inter-faces between layers are considered to have full frictions. The program can be used for up to 15 layers.

Under the given conditions of the stiffness modulus, Poisson's ratio, layer thickness for each layer, and loading conditions (wheel load and tire inflation pressure), four components of stresses and strains (vertical, radial, tangential, and shear) can be output at any point in a pavement structure.

(2) Estimation of Stiffness Moduli for Bituminous Layers

Stiffness, as defined in this paper, is the relationship between stress and strain as a function of time of loading and temperature.

Estimation of the stiffness modulus was made by using a method of time-temperature superposition theory presented by Monismith (4) and Krokosky (5). As was noted familiar, the response of the asphaltic concrete is not only dependent on time, but also on temperature. Practical limitations in the laboratory normally do not permit the complete definition of stiffness over the wide range of times shown schematically in Fig. 3 at a particular temperature. It is, however, possible to measure the behavior for a smaller time range but a number of different temperatures. Measured characteristics for these conditions are shown schematically by the family of curves in Fig. 4. A special case occurs if, by a temperature change, the position of the modulus curve, but not its shape, is altered on the time scale. One of the family of the curves would be shifted horizontally by the shift factor, $a_T = t_T/t_0$, in order to obtain the extended time curve of compliance versus reduced time required to observe a phenomenon at temperature T, and t_0 is the required time to observe the same phenomenon at some reference temperature, T_0.

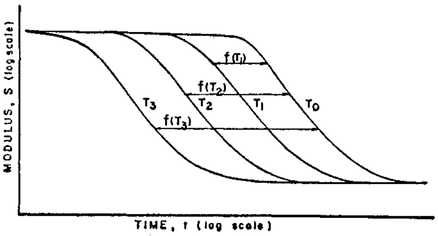

Fig. 3 Effect of Time and Temperature on the Stiffness Modulus for a Thermorheologically Simple Material (After Monismith et al.).

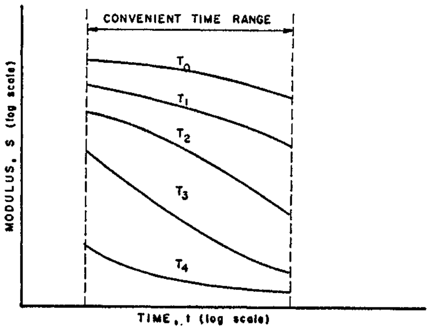

Fig. 4 Influence of Temperature on Modulus (After Finn).

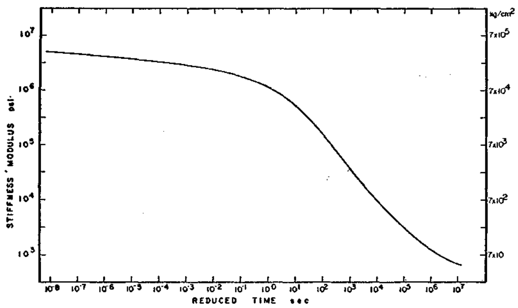

Fig. 5 Master Stiffness Modulus Curve at a Reference Temperature 40 F.

Using this principle, the master curve showing the stiffness modulus versus reduced time relationships was constructed from the data in Reference 4, and is illustrated in Fig. 5. Krokosky and Chen (5) reported the relationships between a shift factor and temperature in terms of the asphaltic concrete mixtures which have asphalt contents of 5, 6, and 7 percent as shown in Fig. 6. Using these two figures, the stiffness moduli were estimated as shown below.

Assuming the surface course temperature is 53°C (128F) and the asphalt content is 6.5 percent and loading time is 0.11 sec, a shift factor, $a_T = 3\times10^{-4}$, is read from Fig. 6. Then the point corresponding to 0.11 sec loading on the master curve was shifted to the right by 3×10^{-4}, and a stiffness modulus of 7×10^4 psi (4.9×10^3 kg/cm^2) was obtained.

In this manner, stiffness moduli for three seasons, four different time of loadings and four asphaltic layers used in the multi-layered computer program were determined.

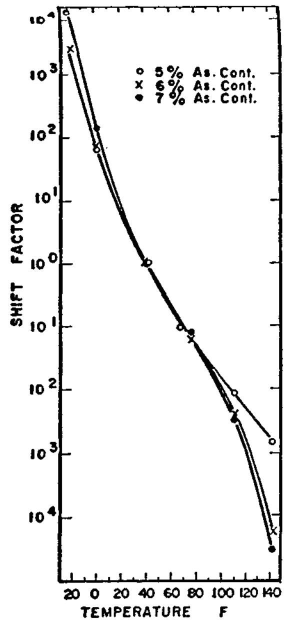

Fig. 6 Shift Factor a_T, for Compression Stress Relaxation (After Krokosky and Chen).

a) Selection of Pavement Temperature

As shown in Chapter II, three representative patterns of the pavement temperature in a year have been obtained. Temperature pattern in spring is assumed to be identical with that in fall. For the simplicity of computation of equivalent axle load applications stated later, the continuously changing temperature pattern in each season was broken down into the following three discrete temperature conditions.

① Day temperature condition; This refers to the day time, here considered 8:00 AM to 4:00 PM, pavement temperature conditions, in other words, the upper part of bituminous layers, has a higher temperature than the lower part of them.

② Evening temperature condition; This refers to the uniform pavement temperature or small amount of temperature difference between layers from 4:00 PM to 12:00 midnight.

③ Night temperature condition; This refers to the night time, 12:00 midnight to 8:00 AM, temperature condition. At this time the upper part of bituminous layers has a lower temperature than the lower layers.

b) Determination of Time of Loadings

The times of loadings which are associated with four travelling speeds of the test trucks were read off the oscillograph record from the field tests. The selected values are 0.11, 0.15, 0.68 and 1.8 sec for 72, 40, 4, and 1 km/hr travelling speeds, respectively.

(3) Presentation of Computed Values and Comparison with the Observed Values

Pavement simulation study was carried out by using the multi-layered elastic theory program with the working parameters briefly discussed in the previous section.

Since the computation was made under the nested-factorial design of four factors with three or four levels, many different analyses can be possible for the stress, strain and deformation data. The limitation of the space permits us to present only a strain analysis, mainly the radial strain at the bottom face of ATB(2), and surface deflection analysis in this chapter.

a) Strain Analysis

Fig. 7 indicates the vertical distribution of the radial strain under dual wheel load, P, $=7$ ton and travelling speed, V, $=72$ km/hr in three temtemperature conditions of three seasons. Observed values obtained from the field dynamic test on February 14, 1968, are also plotted on them. A fairly good agreement between observed and theoretical values can be seen in this case.

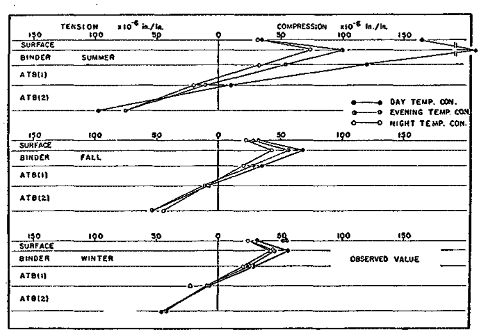

Fig. 7 Vertical Distribution of the Theoretical Radial Strain in the Three Temperature Conditions in the Three Seasons (P=7.0 ton, V=72 km/hr).

In both cases, the maximum strain is seen at the bottom face of the ATB(2). The season effect is illustrated by the fact that the tensile strain in summer season is larger than those in other two seasons, whereas those of fall and winter are almost same values. This does not necessarily mean, however, that the most critical season for the asphaltic pavements is summer because, as will be seen in the next chapter, the higher the temperature is, the longer life predicted for the asphalt concrete mixture under the same amount of strain repetitions. Likely day (surface maximum) temperature condition causes the higher strain as expected.

b) Deflection Analysis

Fig. 8 shows the surface deflection versus wheel load relationships with three temperature conditions in summer, fall and winter. Most of the time a linear approximation between two relationships can be seen. Solid dots on the figure indicate the measured deflections obtained from the

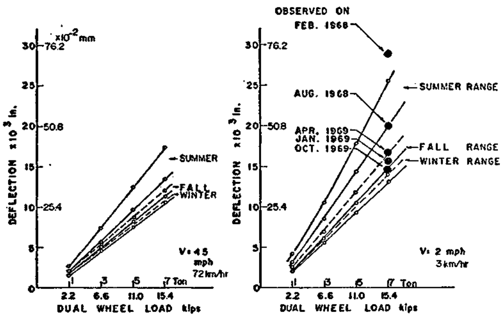

Fig. 8 Theoretical Deflection vs. Wheel Load Relationship.

periodic field measurements. Good agreement between theoretical and observed values can also be seen except the high observed value which were measured at the field test immediately after the completion of construction.

4. ESTIMATION OF THE PAVEMENT LIFE

(1) Fatigue Theory

The word "Fatigue" is defined as "phenomenon of flexure under repeated or fluctuating stress having a maximum value less than the tensile strength of the material."

In the laboratory, fatigue behavior of materials such as asphaltic concrete has been determined in a number of different ways. Two of the most common are constant stress or load and constant strain or deflection tests. In both tests, the relationships between stress or strain and number of repetitions to failure are essentially straight line on the logarithmic paper or expressed as follows:

$$N=K(\varepsilon)^n \quad \cdots\cdots (1)$$

where N=number of repetitions to failure
ε =initial strain
K, n=constants

This simple stress or strain fatigue phenomenon can be extended to the compound stress or strain case which are expected on the actual pavement. Assume that the specimen is subjected to N_1 repetitions of load condition 1; damage will be accumulated in the specimen as represented by the ordinate D_1 in Fig. 9. If these repetitions are

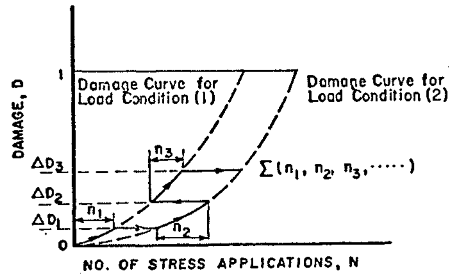

Fig. 9 Procedure to Summarize Cumulative Damage from Simple Load Condition (After Finn).

followed by N_2 repetitions of load condition 2, additional damage, D_2, will result. This process can continue until

$$\sum_{i=1}^{m} \Delta D_i = 1 \quad \cdots\cdots (2)$$

and the service life under compound loading becomes

$$N_s = \sum_{i=1}^{m} n_i \quad \cdots\cdots (3)$$

Therefore, once the relationships between stress or strain and the number of repetitions to failure and the number of axle load applications which cause the specific level of strain in the asphaltic concrete mixes are known, the pavement life until the fatigue failure can be estimated.

(2) Fatigue Equation Developed by the Asphalt Institute

Recently, the Asphalt Institute (6) has developed the relationships between the strain and the number of repetitions to failure (fatigue equation) in a more comprehensive way which are shown in Eq. (4).

$$N_f = K_q \times (\varepsilon)^c \;; \quad K_q = A(B)^{t_p^{d_0}} \quad \cdots\cdots (4)$$

where N_f=number of repetitions to failure
ε =strain
A =3.09×10^{-27} (constant)

B = 1.0268 (constant)
t_p = average pavement temperature, F
d_0 = 1.46 (constant)
C = 7.0 (constant)

The interesting feature of this equation is the indication of pavement temperature effect as well as strain level. For the purpose of the later sections, the number of repetitions to rupture in the combination of nine temperature conditions and nine axle load categories is computed.

(3) Traffic Data Analysis

a) Past and Present Traffic Data

① Traffic Volume: The three busiest sections of the Tomei Expressway were first partially opened to traffic on April 25, 1968, and after a couple of stage openings, it was completely opened on May 26, 1969. Monthly average daily traffic (MADT) per three lanes in one direction was 8,657 in May, 1968, and increased very rapidly to 26,874 after two and one-half years. The rapid increase of MADT during the first couple of years, especially in the case of step opening, seems to be natural. It is interesting, however, that essentially the same amount of traffic increase has been continuing after full completion.

Table 1

AXLE LOAD DISTRIBUTION ON TEST SECTION T-2

AXLE CATEGORY (kg)		AXLE LOAD DISTRIBUTION IN			
		Lane 1	Lane 2	Lane 3	Total
<1,500	Veh.	7,106	9,362	4,280	20,748
	%*	36.1	48.3	82.8	46.1
	%**	34.2	45.1	20.7	100.0
1,500–2,999	Veh.	7,765	8,692	769	17,226
	%*	39.4	44.8	14.9	38.9
	%**	45.0	50.5	4.5	100.0
3,000–4,999	Veh.	1,951	771	68	2,790
	%*	9.9	4.0	1.3	6.3
	%**	69.9	27.7	2.4	100.0
5,000–6,999	Veh.	1,301	260	15	1,576
	%*	6.6	1.3	0.3	3.6
	%**	82.5	16.5	1.0	100.0
7,000–8,999	Veh.	541	118	18	677
	%*	2.7	0.6	0.3	1.5
	%**	79.9	17.4	2.7	100.0
9,000–10,999	Veh.	408	121	19	548
	%*	2.1	0.6	0.4	1.2
	%**	74.5	22.0	3.5	100.0
11,000–12,999	Veh.	288	38	2	328
	%*	1.5	0.2	0.0	0.7
	%**	88.3	11.6	0.6	100.0
13,000–14,999	Veh.	187	16	1	204
	%*	0.9	0.1	0.0	0.5
	%**	91.7	7.8	0.5	100.0
>15,000	Veh.	164	14	0	178
	%*	0.8	0.1	0.0	0.4
	%**	92.1	7.9	0.0	100.0
Total	Veh.	19,771	19,392	5,172	44,275
	%*	100.0	100.0	100.0	100.0
	%**	44.5	43.8	11.7	100.0

* Percent in each lane.
** Percent in each axle category.

From this high traffic increase and the fact that the Meishin Expressway, which is five years older than the Tomei Expressway, still has a 15 to 20 percent annual increase of the total traffic, it is expected that the capacity for T-2 Section will be reached in a very few years.

Fig. 10 indicates the annual traffic volume increase ratio in the past year in five vehicle types.

② Axle Load Distribution: Table 1 indicates the axle load distribution measured in February and March, 1970, in each lane of Test Section T-2.

The other parameter required to make this analysis is the lane distribution of axle loads. The data show the ratio of the total axles to each lane to be 44.5: 43.8: 11.7 for lane 1: lane 2: lane 3, respectively. However, the ratio of the truck axles heavier than 3 ton is 76.8: 21.2: 2.0 for lane 1: lane 2: lane 3, respectively. This axle load distribution might cause the unbalance of the pavement damage when the pavement design is identical for lane to lane.

b) Projection of Traffic Characteristics into the Future

Based on the data in the previous section and other information, several traffic characteristics were projected into the future to determine the equivalent axle load applications on Test Section T-2 as follows.

① Traffic Volume and Vehicle Type Distribution: The traffic volume and vehicle type distribution projections up to 1980 are listed in Table 2. First of all, from the data shown in Fig. 10, the annual increase from 1971 to 1980 for each vehicle type were extrapolated and given in the first row on each vehicle type in Table 2. Next, the average daily traffic volume for each vehicle type in a specific year in the future was computed by multiplying an annual increase ratio for that type of vehicle for that year by the previous year's volume starting in 1971. When the number of vehicles within each class are added together and AADT for that year is obtained. For instance, the average daily traffic volume of a single unit (2-axle) truck in 1972 was determined by $3{,}741 \times 1.20 = 4{,}489$ vehicles per day and given in the second row of each vehicle type in Table 2. The value 3,741 is the measured number of vehicles of that type in 1971. Vehicle type distribution shown in the third row for each vehicle was computed by using the projected AADT and the volume of each vehicle type.

In the above computation, the capacity of this section was set as 54,000 vehicles per day on

Table 2

PROJECTION OF VEHICLE TYPE DISTRIBUTION AND TRAFFIC VOLUME ON TEST SECTION T-2

Vehicle Type		VEHICLE TYPE DISTRIBUTION AND TRAFFIC VOLUME IN THE YEAR OF						
		1970	1971	1972	1973	1974	1975	1976, 1977, 1978, 1979, 1980
Truck	Inc.*	1.39	1.26	1.20	1.17	1.14	1.00	
	ADT	2969	3741	4489	5252	5998	5998	SAME AS LEFT
(2-Ax.)	%**	11.1	11.3	11.1	10.8	10.5	10.5	
Truck	Inc.	1.76	1.36	1.26	1.20	1.15	1.00	
	ADT	1325	1802	2270	2725	3133	3133	
(3-Ax.)	%	5.0	5.4	5.6	5.6	5.5	5.5	
Semi-Trailer (3-Ax.)	Inc.	1.76	1.36	1.26	1.20	1.15	1.00	
	ADT	139	189	238	286	329	329	
	%	0.5	0.6	0.6	0.6	0.6	0.6	
Semi-Trailer (5-Ax.)	Inc.	1.52	1.18	1.15	1.12	1.10	1.00	
	ADT	35	41	47	53	58	58	
	%	0.1	0.1	0.1	0.1	0.1	0.1	
Bus	Inc.	1.20	1.06	1.06	1.06	1.06	1.00	
	ADT	448	475	503	534	565	565	
	%	1.6	1.4	1.2	1.1	1.0	1.0	
Passenger Car	Inc.	1.26	1.26	1.24	1.21	1.19	1.00	
	ADT	21807	26890	32881	39663	47173	47173	
	%	81.6	81.2	81.3	81.8	82.4	82.4	
Total	Inc.	1.29	1.24	1.22	1.20	1.18	0.00	
	ADT	26724	33138	40428	48513	57246	57246	
	%	100.0	100.0	100.0	100.0	100.0	100.0	

* Inc. = Annual Increase Ratio

% = Vehicle Type Distribution

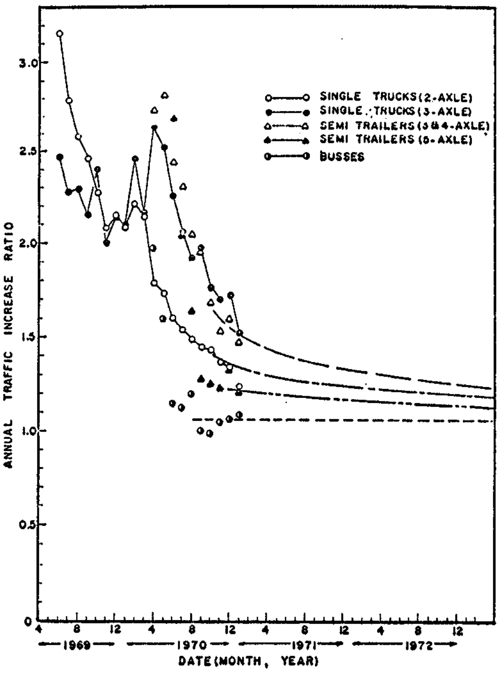

Fig. 10 Record of Annual Traffic Increase Rotio.

three lanes in one direction based on the observed data on the Meishin Expressway. Under this assumption, the capacity will be reached in 1974 as shown in Table 2 and after that year a constant volume was estimated every year.

② Hour of Day Fluctuation and Hour Factors: For predicting the equivalent axle load applications in the future the hour of the day fluctuation of the axle weight applications related to the hour factor was introduced rather than that of the traffic volume.

The hour factor is simply defined as the ratio of the axle applications of a specific axle category during a specific hour of the day to the total day applications of the same axle category. They are derived from the traffic weight study on Test Section T-2.

③ Month of Year Fluctuation and Season Factors: The season factor was defined as the ratio of the axle applications of a specific axle category during a specific season in a year to the total year applications of the same axle category. Two season factors for passenger cars and trucks were derived from the MADT data on the Tomei Expressway and they were assumed to be independent of the year.

④ Axle Weight Distribution: Axle weight distribution in the future was assumed to be the same as that shown in Table 1.

⑤ Lane Distribution of the Axle Load: Lane distribution of the axle load varies with time as well as axle load category. Although it is expected that the yearly variation of the lane distribution is closely related to the traffic volume, only two different lane distribution coefficients for each axle load category were estimated. One is the distribution for the first five years which was derived from the data shown in Table 1, and the other is the one for the rest of years which was estimated from inspection of Table 1 and considering that the traffic volume could be reaching the capacity of the highway. The assumption that the greater number of trucks will travel on lane 2 and lane 3 is considered reasonable.

(4) Computation of Equivalent 10 ton Axle Load Applications

a) Equivalency of the Axle Load Based on the Tensile Strain

Axle load equivalency is, in general, defined as the ratio of the destructive effect of a specific axle load to that of a base axle load. Several sets of axle load equivalency factors have been developed and used by different organizations. In this paper, the equivalency factors were developed based on the asphalt concrete tensile strain considering the axle load and the pavement temperature as shown in Eq. (5).

$$EF=\frac{N_f(\varepsilon_0, t_0)}{N_f(\varepsilon, t)} \quad \cdots\cdots\cdots(5)$$

where EF = equivalency factor in terms of strain level and pavement temperature

$N_f(\varepsilon_0, t_0)$ = number of repetitions to rupture under base strain (ε_0) and base temperature (t_0) conditions determined by Eq. (4)

$N_f(\varepsilon, t)$ = number of repetitions to rupture under a specific strain (ε) and a specific temperature (t) conditions determined by Eq. (4)

In the discussion for this paper, the strain developed at the bottom face of ATB(2) was only considered because it is the critical value. The tensile strain developed under a 10 ton load at 72 km/hr travelling speed with a winter day temperature condition was selected as the base strain because 10 ton is the maximum legal limit of the axle load at present and the comprehensive dynamic field tests were only run in the winter. These conditions result in $\varepsilon_0=30\times10^{-6}$ cm/cm and $t_0=11°C$ as the base strain and pavement temperature, respectively.

b) Computer Program for the Computation of Equivalent 10 ton Axle Load Applications

A computer program called N10AXLE was set up to compute the equivalent 10 ton axle load applications using the information developed in the previous sections. Yearly equivalent 10 ton axle load applications are computed by Eq. (6).

$$EAL_1=\sum_{j=1}^{9}\sum_{i=1}^{9}(EF_{ij}\times HF_{ij}\times SF_{ij}\times AX_j \times 365\times L_{1-j}) \quad \cdots\cdots\cdots(6)$$

where EAL_1 = yearly equivalent 10 ton axle load applications on lane 1.

EF_{ij} = equivalency factor for temperature, i, and axle load, j

HF_{ij} = hour factor for temperature, i, and axle load, j

SF_{ij} = season factor for temperature, i, and axle load, j

AX_j = number of axles for axle load, j

L_{1-j} = lane distribution coefficient on lane 1 for axle load, j

c) Results of Equivalent Axle Load Computations

The results of equivalent axle load computations for the first 13 years (up to 1980) are shown in Table 3.

(5) Estimation of Pavement Life and Conclusions

Table 3 indicates that equivalent axle load applications on lanes 1, 2, and 3 in 1968 were 1.32×10^6, 1.28×10^5, and 3.02×10^3, respectively. These applications, however, will rapidly increase to 5.76×10^6, 3.41×10^6, and 5.04×10^5, respectively in 1974 and after that year the same numbers of axle load applications as those in 1974 will be expected every year. Summations of axle load applications for the first 13 years give 6.60×10^7, 2.88×10^7, and 4.02×10^6 for lanes 1, 2, and 3, respectively. Comparing these values to the number of repetitions to failure, 5.30×10^8, developed by the fatigue equation under the strain level of 30×10^{-6} cm/cm and the temperature of 11°C, none of the three lanes will fail by fatigue of the asphalt concrete in the first 13 years for lane 1 and more than 100 years for lanes 2 and 3.

Present pavement performance of T-2 Section is in extreamly good condition with no indication of both fatigue cracking and severe rut. The latest performance properties in lane 1 are summerrized as below.

Crack ratio = 0.0%

Maximum rut depth = 10 mm (Outer wheel path)

Profile index = 0.0 cm/km (Excluding roughness due to the culvert box)

Substituting these values into Eq. 2.2 in Reference 7, approximate Present Serviceability Index can be computed as below;

Table 3

PROJECTION OF AXLE LOAD APPLICATIONS ON TEST SECTION T-2

YEAR	NUMBER OF APPLICATIONS IN ONE YEAR				SUM OF APPLICATIONS			
	Lane 1	Lane 2	Lane 3	Total	Lane 1	Lane 2	Lane 3	Total
1968	1.32×10^{6}	1.27×10^{5}	4.13×10^{3}	1.45×10^{6}	1.32×10^{6}	1.27×10^{5}	4.13×10^{3}	1.45×10^{6}
1969	2.95×10^{6}	2.84×10^{5}	9.22×10^{3}	3.24×10^{6}	4.27×10^{6}	4.11×10^{5}	1.33×10^{4}	4.69×10^{6}
1970	4.30×10^{6}	4.14×10^{5}	1.34×10^{4}	4.72×10^{6}	8.57×10^{6}	8.25×10^{5}	2.68×10^{4}	9.42×10^{6}
1971	5.46×10^{6}	5.25×10^{5}	1.71×10^{4}	6.00×10^{6}	1.40×10^{7}	1.35×10^{6}	4.38×10^{4}	1.54×10^{7}
1972	6.60×10^{6}	6.35×10^{5}	2.06×10^{4}	7.25×10^{6}	2.06×10^{7}	1.99×10^{6}	6.44×10^{4}	2.27×10^{7}
1973	5.06×10^{6}	2.99×10^{6}	4.43×10^{5}	8.50×10^{6}	2.57×10^{7}	4.98×10^{6}	5.08×10^{5}	3.12×10^{7}
1974	5.76×10^{6}	3.41×10^{6}	5.04×10^{5}	9.68×10^{6}	3.14×10^{7}	8.39×10^{6}	1.01×10^{6}	4.08×10^{7}
1975	5.76×10^{6}	3.41×10^{6}	5.04×10^{5}	9.68×10^{6}	3.72×10^{7}	1.18×10^{7}	1.52×10^{6}	5.05×10^{7}
1976	5.76×10^{6}	3.41×10^{6}	5.04×10^{5}	9.68×10^{6}	4.30×10^{7}	1.52×10^{7}	2.02×10^{6}	6.02×10^{7}
1977	5.76×10^{6}	3.41×10^{6}	5.04×10^{5}	9.68×10^{6}	4.88×10^{7}	1.86×10^{7}	2.53×10^{6}	6.99×10^{7}
1978	5.76×10^{6}	3.41×10^{6}	5.04×10^{5}	9.68×10^{6}	5.45×10^{7}	2.20×10^{7}	3.03×10^{6}	7.96×10^{7}
1979	5.76×10^{6}	3.41×10^{6}	5.04×10^{5}	9.68×10^{6}	6.03×10^{7}	2.54×10^{7}	3.53×10^{6}	8.92×10^{7}
1980	5.76×10^{6}	$3{:}41\times10^{6}$	5.04×10^{5}	9.68×10^{6}	6.60×10^{7}	2.88×10^{7}	4.04×10^{6}	9.89×10^{7}

$$\begin{aligned}PSI&=4.57-0.447\sigma-0.316RD\\&=4.57-0.447(12+PrI)/43-0.316RD\\&=4.57-0.447(12+0.0)/43-0.316\times1.0=\underline{4.13}\end{aligned}$$

Considering the above performance data with the 6×10^{6} 10 ton equivalent axle applications so far, it might be concluded that the simulation study gives a good estimation at this time of moment.

The primary emphasis of this paper has been to illustrate a procedure whereby the fatigue concept may be incorporated into pavement design using available theory and collected information on an in-service highway. From the analyses so far, it was concluded that this section of the pavement would have the service life of 36 years for lane 1 and practically indefinite number of years for lanes 2 and 3. However, there might be the possibility of the potential for cracking in a shorter period than calculated, since the bottom strain of ATB(2) was not measured directly and the calculated strain could not strictly be varidated.

Another possibility for the potential cracking in a shorter time might be the asphalt aging and condition changes like seasonal change of saturation in the granular layer. This kind of uncontrolled variables should be incorporated in the further refinement of this study in the future.

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