ROAD MAINTENANCE STUDY COURSE

IN

THAILAND

1982

JAPAN INTERNATIONAL COOPERATION AGENCY

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Road Maintenance Study Cource

1.	Nature of Road Failure	Takashi Iijima
2.	Pavement Assessment Techniques	Takashi Kobayashi
3.	Assessment and Priority Maintenance	Tadashi Aida
4.	Road Maintenance Equipment	Kazuo Watanabe
5.	Construction Equipment Management	Kazuo Watanabe

ROAD MAINTENANCE STUDY COURSE

NATURE OF ROAD FAILURE

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Contents

Page

Page

1 Failure of asphalt pavement, and heating Required characteristics of asphalt mixtures

1-1 General

Our standard approach to the preventive and corrective maintenance of pavements starts with an analysis of the cause-and-effect relationships of pavement failures, that is, with the classification of the various surface conditions of the pavements in daily use by the causes. The causes of pavement failures are many; they include pavement design and workmanship, loads, environmental conditions, climatic conditions, etc. Besides, these usually happen, not independently, but in a declicate combination to cause road failures. Here we shall study the historical background about the classification of asphalt pavement failures, statistical classification of road failures in Japan, and what is required of asphalt mixtures to cope with the failures.

1-2 Classification of asphalt pavement failures

The history of the classification of asphalt pavement failures dates as far back as to 1924 when P. Hubbard tried a simple classification. Since then, there have been proposed a great variety of classification methods.

Failures
- Cracking
- Shoving

In 1955, B.A. Ballerge proposed a practical classification chart listing the failures and their causes in detail.
But it was reserved for N. Hveem to establish the foundation for a modern system of classification in 1962. A year later, he codified it as shown in Fig. 3-1.
As is clear from Fig. 3-1, he made an exhaustive list of ramified cause-and-effect relationships, locking into primary and secondary causes and their mechanisms.
In 1972, Monismith was the last man who brought forward a new

classification system. His system is as simple as shown in Fig. 3-2. While respecting the achievements of these researchers, the Ministry of Construction conducted extensive research and study on the modes of failures of the various pavements used on an experimental or practical basis, and has formulated a simple yet viable classification system according to which a definite improvement measure is forthcoming to any specific failure as demonstrated in Fig. 3-3. Fig. 3-3 lists the major causes of such asphalt pavement failures that can be identified visually. Usually, the faulty symptoms are accompanied by secondary failures. For example, the flow of surface course under item (1) in Fig. 3-3 may be accompanied by cracking, and the flow of subgrade and subbase referred to under tiem (2) often concurs with the cracking and disintegration. The major causes for these troubles include design and engineering workmanship, environmental and climatic conditions. The unevenness (5) which often eludes our attention is also a failure because the surface smoothness is indispensable for smooth, safe and swift traffic flow.

1-3 Requirements of asphalt mixtures

Considering the causes of asphalt pavement failures listed in Fig. 3-3, it is found that the asphalt mixtures should fulfill the following requirements.

(1) Stability
(2) Durability
(3) Flexibility
(4) Fatigue resistance
(5) Skid resistance
(6) Permeability or imperviousness
(7) Rupture strength (tensile strength)

In addition to these characteristics required of the pavement, the workability is also one of the most important assets of the asphalt mixtures. What counts most, however, is to follow a proper construction process, because any asphalt mixture will display the best performance in these characteristics and the maximum service life in its way.

1) Stability

The stability is the first thing to be considered in the mix proportioning design, and is defined as the resistance to deformation due to loading. The deformation here refers to the permanent set or plastic flow caused by the following factors.

(a) Lateral flow due to low-speed loading at a comparatively high temperature.

(b) Rutting owing to repeated traffic loads as in the lanes in which all the vehicles must stay during running.

In 1924, MacNanghton pointed out the following as factors governing the stability of sheet asphalt.

(a) Intermeshing of aggregate.

(b) Fractional resistance developed in the aggregate by contact.

(c) Cohesive power of asphalt as a maxtrix or ligament for the aggregate.

Hveem and Vallerga epitomized the factors affecting the stability. A revision of their listing is shown in Fig. 3-4.

2) Durability

The durability is defined as resistance to aging and weather as represented by exposure to rain, resistance to tear and wear due to traffic. The climatic factors include the following.

(a) Changes in the qualities of asphalt due to volatilization, oxidation, polymerization, dissociation, condensation, etc.

(b) Changes in the qualities of mixture under the influence of water and vapor.

Experience shows that it is necessary to form a well-tamped asphalt-rich, impervious dense-grade pavement for the purpose of minimizing the evil effects of climatic actions.
Asphalt must have a tensile strength high enough to bear up against the traction and chafing forces of vehicles, and must be perfectly integrated with the aggregate.

Equally important is the durability of aggregate. The aggregate is required to bear the brunt of harsh loads during construction work and at the same time after the road is open to traffic. It must also serve for an extended period without aging.

3) Flexibility

The flexibility is defined as a capability of preventing the asphalt mixture from chapping and cracking and of adapting the asphalt mixture to the deformation of subgrade, subbase course, etc. over a long period. Although the flexibility is one of the most outstanding characteristics given to the asphalt, it is often spoken of mistakenly as a combination of long-term plasticity and fatigue resistance against repeated loading.
A long-term subsidence is often invited in the form of settlement due to consolidation of weak or compressible subsoils or in the form of uneven settlement due to irregular ramming of substructure by traffic loads. The flexibility of the pavement will be improved by increasing the ratio of asphalt in the mixture and making the aggregate density lean (open grade).

4) Fatigue resistance

In a heavy traffic pavement, the asphalt mixture develops cracks because it undergoes elastic deformation at a high frequency.
In their early stages, these cracks seem to be of no consequence to the permanent set of the asphalt mixture. This kind of pavement failures was described by Hveem in detail. The cracks due to fatigue have much to do not only with the characteristics of asphalt mixture itself, but also with the thicknesses and characteristics of the courses of which the road is made up, the magnitude, amplitude and frequency of the load repeatedly applied on the road surfaces, and other various factors. Accordingly, the entire behavior of the pavement structure must be studied when we are to discuss the fatigue resistance. Every asphalt mixture cannot be immune to fatigue. When the fatigue cracks start is governed largely by whether the asphalt mixture is

subjected to a small yet frequent tensile stress or strain or to an infrequent yet large stress or strain.
So fat as our past studies on the serviceability of pavements show, it seems likely that the asphalt ratio and the density of asphalt mixture have a definitive effect upon the fatigue resistance; namely, that the larger the ratio of asphalt is, and the smaller the porosity is, the longer the life of the pavement results.

5) Skid resistance

The skid resistance is defined as a surface quality of the asphalt pavement mixture required to make a vehicle stop within a proper braking distance under various environmental conditions.
While there are many other factors that govern the skid resistance, such as surface contaminants like oil film, excessive water film, snow and ice, the pavement should be so designed and constructed to ensure an enough coefficient of sliding friction between the tires and the pavement surface.
Generally speaking, the skid resistance can be increased by improving the factors that govern the stability of pavement. Namely, it is required to reduce the asphalt mix ratio and use coarse aggregate resistant to abrasion.

6) Permeability

The permeability of asphalt mixture is designed as the quality or degree of being permeable to air, water and vapor. Usually, the pavement lasts longer the higher its imperviousness is to air, water and vapor. The permeability is nothing but the reciprocal of the durability discussed earlier. The imperviousness of the asphalt mixture may be improved by increased asphalt mix ratio, increased density and careful stamping.

7) Rupture strength (tensile strength)

The rupture strength is defined as the maximum strength that an asphalt mixture generates when it is applied with a tensile force. The rupture strength is governed by the temperature and loading time. Usually, the rupture strength becomes higher the lower the temperature is and the higher the loading speed is. The rupture strength is an important factor, particularly in case the road is liable to undergo heavy traffic loads when the subsurface courses are slacked under the repetitive freezing and melting as in early Spring though the surface course is frozen hard, or in case the possibilities of crack development due to thermal expansion and contraction of pavement composition, volumetric changes caused by the closure of pores around aggregate, and volumetric changes in the subsurface courses due to consolidation, etc. are to be assessed.

The characteristics referred to above are synergetic on one occasion and counterproductive on the other. Accordingly, these characteristics should be well coordinated in order to ensure the longevity of asphalt pavement against various failure factors while taking into full account the purposes for which the pavement is used, local topographic, geological, hydrological and climatic conditions, traffic requirements, etc. Fig. 3-5 is schematic representation of the relationships between the pavement characteristics, and is designed to offer something of a basis upon which to coordinate the characteristics of each specific asphalt mixture.

2 Pavement design requirements, and material specifications

2-1 General

Usually, the hot asphalt mixture (hereinafter referred to as mixture) used for surface course and base course is required to have the following properties.

(1) Insusceptibility to deformation (flow) such as undulation and rutting. ....(Stability)

(2) Insusceptibility to cracking. ....(Flexibility)

(3) Property to prevent the vehicle from skidding. ....(Skid resistance)

(4) Resistance to embrittlement. ....(Aging resistance)

(5) Insusceptibility to breakup. ....(Imperviousness)

(6) Resistance to wear and tear by tire chains, etc. ....(Wear resistance)

(7) Ease of handling. ....(Workability)

With the exception of the workability, all these properties are concerned with the performance required of the pavement in use, and are the antithesis of evil phenomena caused by traffic and weather. The degree by which these phenomena assert themselves varies depending on the types, qualities and mix proportion of aggregate, filler and asphalt.

Now let us call the factors causing the evil phenomena "the external factors" and the factors governing the latent characteristics of the mixture "the internal factors." Then, the required practical characteristics of the mixture are determined by both the internal and external factors, and not simply by either internal or external factors. To attain a pertinent mix proportion, it is imperative to make clear the interrelations between the internal and external factors.

In designing a mix proportion, most of a designer's efforts is, it seems, expended to deal with the internal factors alone. But the designer has in his mind a set of criteria acquired by experience to assess the internal factors while taking into account the effects of external factors upon the internal factors.

In this textbook, we will learn the mix proportioning design with emphasis on the external factors. As regards the internal factors, the materials and their proportioning will be discussed in generalities.

The external factors are classified into two groups; one relating to traffic and the other to enviroonment.

Coming under the traffic-related group are such factors as traffic volume (particularly the traffic volume of heavy vehicles), destribution of loci along which vehicles pass, the number of lanes, the lane width, channelization, and other geometric design factors, including location of signal-controlled intersections, bus stops, curves, slopes and parking lots. The environment-related group includes ambient temperature, rainfalls, snowfalls, etc.

The internal factors governing the mix proportioning relate to the materials of which the mixture is composed, that is, the quality of aggregate and asphalt.

2-2 Pavement temperature

2-2-1 General

The asphalt mixture uses asphalt as a binder. Asphalt is thermoplastic, and the asphalt pavement softens at high temperatures and is liable to yield under wheel loads, leaving ruts on it.

The temperature of the asphalt pavement is governed by the ambient temperature, rainfalls, sunshine, and other climatic conditions. In this section, we shall see what the asphalt pavement temperature is in Japan, based on actual observations in various sites.

2-2-2 Measuring sites, and measured sections

In Japan, same example of the temperatures of the asphalt pavements were measured at the locations listed in Table 3-1.
Fig. 3-6 illustrates the cross sections of pavements measured in Chiba and Hokkaido. The climatic conditions in Chiba and Hokkaido are given in Table 3-2 by way of reference.

2-2-3 Measured results

(1) Daily temerature changes in the pavement

The temperature data taken within the Chiba Office of the Public Works Research Institute in 1961 are as shown in Fig. 3-7 and 3-8.
Figs. 3-9 and 3-10 show the typical pavement temperature changes due to sunshine and rain measured in January and August 1961. It is found that the temperature changes do not follow a sinusoidal curve, but show steep ups and downs in the daytime and moderate slopes in the nighttime. The depthwise temperature gradient is also large in the daytime; in August, the temperature gradient in the daytime is 2 to 3 times as large as that in the nighttime.
At the measuring position VI, the subgrade temperature shows a moderate change perennially with little or no diurnal change. In the rainy day, the maximum temperature is lower, the minimum temperature higher, and the diurnal temperature amplitude smaller, than in the sunny day.

(2) Frequency of maximum and minimum temperatures

Fig. 3-10 shows the frequency of maximum and minimum temperatures measured in Chiba over a year at positions I, III and V. It is found that the maximum temperature on the surface cause changed from 6°C to 60°C, and that the number of days showing a maximum temperature of 58 to 60°C was only 3 a year, or 1% of the total frequency. The number of days exceeding 50°C was 51 a year, or 15% of the total frequency. The number of days showing a minimum temperature of below

0°C was 22, or 7% of the total frequency.

The maximum temperature decreases with increases in the pavement depth; the maximum temperature at the bottom of the asphalt macadam pavement (10 cm deep) did not exceed 50°C. On the other hand, the minimum temperature rose with increase in the depth; at the bottom of the asphalt macadam pavement, there was not a single day registering sub-zero temperatures.

(3) Ambiment temperature and pavement temperature

The daily maximum and minimum temperatures measured at Chiba Weather Station were compared over a year with the corresponding maximum and minimum pavement temperatures. The results are as shown in Figs. 3-11 and 3-12. Chiba Weather Station is located a little seaward of the Chiba Office of the Public Works Research Institute, and the temperatures within the premises of the Chiba Office may have been more or less different from those measured at Chiba Weather Station. As the ambient temperatures were not measured at the Chiba Office, the weather data available from Chiba Weather Station were therefore used. From Figs. 3-11 and 3-12, it is evicent that the pavement temperature increases with increase in ambient temperature. What is noteworthy is that the pavement temperature changes ±10 degrees Centigrade from the same ambient temperature. The maximum pavement temperature was 1.7 times the maximum ambient temperature. The correlation between the minimum ambient temperatures and the minimum pavement temperatures given in Fig. 3-12 is stronger as compared with Fig. 3-11; namely, the change of the pavement temperature from the same ambient temperature was held within about ±3 degrees Centigrade, suggesting that the minimum pavement temperature can be expressed as 1.2 times the minimum ambient temperature.

Ohta, et al., reported their measurements as summarized in Figs. 3-13 and 3-14; it is found that their results show almost the same tendency as stated above. They derived the following from their measurements.

(a) Relationship between daily maximum ambient temperature and daily maximum pavement surface course temperature

Although the pavement temperature increases with increase in the ambient temperature, the rate of change in the pavement temperature to the ambient temperature is different from location to location. The daily maximum surface course temperature is about 1.6 times as high as the daily maximum ambient temperature on the average of all the measuring stations.

(b) Relationship between the daily maximum ambient temperature and the daily maximum intermediate course temperature

On the average of all the measuring stations, the daily maximum intermediate course temperature is about 1.4 times as high as the daily maximum ambient temperature.

(c) Relationship between the daily maximum ambient temperature and the daily maximum base course temperature

On the average of all the measuring stations, the daily maximum base course temperature is about 1.2 times as high as the daily maximum ambient temperature.

(d) Relationship between the daily minimum ambient temperature and the daily minimum pavement temperature

On the average of all the measuring stations, the daily minimum temperature in each course is nearly equal to the daily minimum ambient temperature.

(e) Differences between measured years

Although the relationships between ambient temperatures and pavement temperatures differed more or less depending on the measuring positions or stations, there seems to be little difference in such relations depending on the years of measurement (1966 and 1967).

(4) Annual percentage frequency distribution of pavement temperatures

The data measured in Chiba are given in the form of an annual percentage frequency distribution curve in Fig. 3-15 in which the pavement temperatures measured at an interval of two hours are divided into classes having an interval of 5°C and the annual frequency of each class is given in per cent. Fig. 3-16 is a cumulative percentage distribution of Fig. 3-15. Fig. 3-17 shows the percentage frequency of pavement temperatures in January, April, May, August, September and October. From Fig. 3-15, it is found that the annual percentage frequency distribution of pavement temperatures assumes a trapezoidal form, and that the mode lies in the range of (5°C to 10°C) to (30°C to 35°C) irrespective of the measuring positions. It is also found that the corresponding frequencies are held almost constant to show a plateau. The frequency at which the asphalt concrete surface course exceeds 50°C is about 1%, and the frequency at which the top of the surface course exceeds 50°C is 2.6%. Namely, the aggregate number of days per year when the pavement surface exceeds 50°C is only 9.5, and the aggregate number of days per year when the temperature 5 cm below the surface exceeds 50°C is only 3.7.

The aggregate number of days when the temperatures stand below zero can be calculated the same way. Within the range of 15°C to 35°C, these days are almost constant irrespective of the measuring positions.

It is quite interesting to note that the depth effect appears beyond this temperature range. From Fig. 3-17, it is found that the pavement surface temperatures will exceed in the period from May to September.

As demonstrated in Figs. 3-18 and 3-19, the data obtained by Ohta and his colleagues show almost the same tendency as above. Because they surveyed in Hokkaido, the mode is shifted to the range of -5°C to 20°C.
Compare Chiba and Hokkaido. It is found from the annual percentage frequency distribution of pavement temperatures that Hokkaido is about 10 degrees C lower than Chiba.

2-2-4 Discussions

In Japan, the stability test, the result of which underlies the mix proportioning design of the mixture, usually is conducted at 60°C which is generally considered as the maximum pavement temperature. But the number of days a year when the pavement temperature exceeds 55°C to 60°C is quite small. In addition, the maximum pavement temperature lower with increase in the depth; it is considered that the position 10 cm below surface will never exceed 50°C. On the other hand, the consolidation and plastic flow occur saliently in a very short period when the temperature is high as shown in Fig. 3-20. In order to improve the mix proportioning design against plastic flow, the following will be mandatory.

(1) To identify the effect of temperature on the plastic flow of asphalt mixture.

(2) To make clear the frequency (%) at which the temperature to be taken into account in (1) above is exceeded.

In addition, the traffic volume at such temperature will have to be clarified. Now let us discuss the above according to the measurements in Chiba.
As regards the effect of temperature on the plastic flow of asphalt mixture, we can obtain practically high-reliability quantitative data from a laboratory test.
What we should do for this purpose, for example, is to determine the relationship between the loading frequency and permanent set with the temperature and load taken as parameters as shown in Fig. 3-21.

Fig. 3-21 is a result of test conducted by the Transportation and Road Research Laboratory, U.K., by making use of a road machine. The specifications of the road machine are as shown in Table 3-3. With reference to Fig. 3-21, the effect of the temperature on the permanent set after the first 10,000 cycles of loading at 5,100 kg (which is close to the maximum legal wheel load in Japan) is as follows:

Taking 20 to 30°C as a basis, the temperature range of 30 to 40°C develops about 7 times as much permanent set, and the temperature range of 40 to 50°C about 20 times as much. By extrapolation, the temperature range of 50 to 60°C will develop about 40 times as much permanent set as the temperature range of 20 to 30°C. From the annual pavement temperature distribution in Chiba shown in Fig. 3-16, the temperature ranges, 20 to 30°C, 30 to 40°C, 40 to 50°C, and 50 to 60°C, account for 28%, 17%, 7% and 3%, respectively.

With respect to the example above, the effect of respective temperature ranges upon the permanent set can be calculated as shown in Table 3-4, provided that the traffic flow is assumed to be uniform throughout the year.

While the 30 to 40°C temperature range appears to be negligible from the viewpoint of high-temperature stability, the example here shows that its effect on the permanent set compares well with the 50 to 60°C range. This is because the width of the frequency distribution in the 30 to 40°C range is wide. This suggests that the rationalized mix proportioning design presupposes a stochastic analysis of the pavement temperatures upon permanent deformation based on the actual pavement temperature measurements along an intended route.

2-3 Traffic load

2-3-1 General

The traffic load is one of the primary factors not only for the designing of pavement structure, but also for the mix proportioning design of asphalt mixture because it overrides all other external factors in determining the requirements of the mix proportioning of the asphalt mixture.

In this section, we shall discuss the traffic volume, wheel load, wheel path, running speed, acceleration, deceleration etc. together with actual data.

2-3-2 Wheel load

In Japan, the Road Traffic Law and the Vehicles for Road Transportation Law require the gross tonnage of the vehicle to be less than 20 tons, the axle load to be less than 10 tons and the wheel load to be less than 5 tons. With the progress of economic activities, however, the overland transportation of cargo has been increasing at a spiraling rate. But those teamsters who overload their trucks with the knowledge of violating the laws are rife, because they are forced to do so for transportation economy. Since the latter half of the 1960s, positive measures have been taken to control overloaded trucks, but to little effect.

Japan's traffic conditions are characterized by the fact that the ratio of heavy trucks to the total traffic volume is high, and that most of such trucks are loaded in excess of the maximum allowable wheel load specified by law. This is truly a grave situation because the overloaded trucks run down the pavements prematurely and accelerate the plastic flow of asphalt roadways.

(1) Effect of wheel load on the breakdown of pavement

According to the Asphalt Pavement Design Manual, the pavement structure is designed for the total number of runs of vehicles over ten years as reduced to the number of 5-ton wheels (wheels/day/direction). The number of wheels in terms of 5-ton wheel load is given by the following formula.

$$N = \Sigma n \times \alpha_i \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Where, N: number of wheels in terms of 5-ton wheel load (wheels/day/direction)

n: number of wheels of each specific wheel load (P_i) (wheels/day/direction)

α_i: effect of P_i-ton wheel load on the breakdown of pavement as compared with that of 5-ton wheel load as unity. (Expressed in $(P_i/5)^4$)

The larger the N value is, the thicker the pavement must be. If the wheel load is increased by an extra loading of 20% to 6 tons, the breakdown effect will be increased by 2.1 times (= $(6/5)^4$). On the other hand, if the wheel load is 4 tons, the breakdown effect will be reduced by more than 50% ($(4/5)^4 = 0.4$). This is the reason why the pavement designer gives his attention to the traffic of large vehicles.
The pavement breakdown effect of wheel loads as expressed by α_i has been established by a sweeping evaluation of various factors, including the longitudinal surface irregularities, degree of cracking, degree of patching requirement, and rut depth, according to AASHO road testing methods. Taking the run depth alone, the breakdown effect of the wheel load will naturally be expressed by a different index.

(2) Actual measurements of wheel loads

In 1962, the Public Works Research Institute, Ministry of Construction, started the development of an automatic wheel load measuring system. In 1964, the first practical system was developed. Following this, the systems have been installed at various sites in Japan, contributing much toward unveiling the actual state of wheel load distribution.

The first measurements by the systems were made along the national roads (Routes 17, 4, and 6) in and around the Kanto area. At each site, the measurements were taken on the up lanes around the clock from November 24 to December 15, 1964. The results are as shown in Table 3-5. In Routes 4, 6 and 17, the maximum wheel loads were as high as 10 to 11 tons. As regards the large vehicles (ordinary trucks and large buses under load), the ratio of the vehicles exceeding 5 tons in wheel loads was 47% on Route 4, 60% on Route 6 and 71% on Route 17. Among others, Route 17 showed a high ratio of dump trucks to the total number of large vehicles.

The frequency distribution of the rear wheel loads of the loaded dump trucks running along Route 17 was as shown in Table 3-6. It is found that almost all loaded dump trucks show a wheel load of more than 5 tons, that the average wheel load is 7 tons, and that those exceeding a 8-ton wheel load account for as much as 14%.

In FY1972, a nation-wide survey on wheel loads was carried out. The results are as shown in Figs. 3-22, 3-24 and 3-25. It is found from Fig. 3-22 that the number of wheels exceeding a legal wheel load limit of 5 tons is 7.4 per 100 vehicles, and that the number of wheels exceeding 8 tons is about 1 per 1000 vehicles. Those exceeding 12 tons and even 14 tons were also noticed as in the past.

As shown in Fig. 3-24, the sites having registered 12 to 14 tons of wheel loads were the largest in number, and the sites having registered more than 14 tons in maximum wheel load, that is, 75% or more in excess of a design wheel load of 8 tons, accounted for one third of the total. Just as the past wheel load measurements, the surveys above show that even in FY1972, illegally loaded vehicles were seen at almost every site in Japan, suggesting that it is quite difficult to control the wheel loads below the legal limit.

The measurement of wheel loads has still been going on at 100 sites in Japan, and the data obtained from such sites are as shown in Figs. 3-25 through 3-28.

Table 3-7 is an enumeration of wheel loads by traffic volume. The measurement of wheel loads is of paramount importance not only for the design of pavement, but also for the analysis of pavement failures.

2-3-3 Wheel path

The rutting of asphalt pavement is more liable the more the path along which wheels pass comes to stay. The situation is even more amplified the narrower the lane width is and the heavier the traffic the pavement must convey. Once a rut is developed and even if its depth is only a few millimeters, the running wheels will gravitate into it, developing the so-called channelization phenomenon to aggravate the rutting. It is therefore important to project the degree of wheel load concentration along particular courses from the lane width and the number of lanes. Table 3-8 and Figs. 3-29 and 3-30 show the wheel load concentration data measured by the regional construction bureaus of the Ministry of Construction in 1972.

It was found that a histogram showing the relationship between the wheel passing position and the frequency of wheels assumes nearly a normal distribution.

Table 3-8 shows the mean value and standard deviation of the lefthand wheel passing position (a) as measured from the inside lane mark.

A survey conducted in FY1971 revealed that the standard deviation of the passing positions (a) is different between the 2-lane road and the four-lane road, and that there is some degree of correlation between the mean value and standard deviation of the values (a) on the one hand and the lane width on the other.

Thus, it was tried to determine the mean value of (mean value of (a)/lane width) and the mean value of (standard deviation of (a)/lane width) and to express the distribution of the wheel passing positions as a function of lane width (b).

This revealed that in FY1972, the mean value and the standard deviation of the wheel passing positions (a) were respectively 0.73b and 0.09b for the two-lane roads and respectively 0.69b and 0.13b for the four-lane roads; namely, that in the four-lane roads, the passing positions were shifted a little inward as compared with the two-lane roads and had a larger dispersion than the two-lane roads. These are illustrated in Figs. 3-29 and 3-30. A lane-by-lane 50ton wheel load distribution survey revealed that on a six-lane expressway, the curb lane accounts for 87% of traffic, the middle lane for 12%, and the overtaking lane for 1%, and that on a four-lane highway, the curb lane accounts for 94%, and the overtaking lane for 6%. It is therefore evident that the measures against the rutting problems should be provided with emphasis on the steady-running lanes.

2-3-4 Running speed, acceleration and deceleration

The stability of asphalt mixture is spoken of in terms of stress and deformation. The relationship between stress and deformation is largely concerned not only with the temperature and loading speed, but with the loading mode as well. When the temperature and load are fixed constant, the deformation of the asphalt mixture will progress more the slower the loading speed is, and the larger the horizontal strain the loading develops.
In other words, the pavement surface is more susceptible to deformation as the vehicular speed is low and as the pavement surface undergoes thrust loads by acceleration and declaration.
Fig. 3-31 shows the relationship between the number of loadings (dynamic stability) and the loading time necessary for developing a unity deformation in the asphalt mixture. This relation was established by Sugawara, et al., who used a wheel tracking tester for measurement.
From Fig. 3-31, it is evident that the deformation of asphalt mixture is accelerated with decrease in the vehicular speed and with increase in the loading time.
This fits in with the fact that the roads usbjected to slow yet heavy traffic, the slopes, curves and the intersections and the positions before and after them are liable to cause pavement deformation.

2-4 Aggregate

2-4-1 General

The aggregate is classified into natural and artificial aggregates according to its production process. While the natural aggregate goes through with man's actions such as crushing, screening and washing, it is called so if its physico-chemical properties are held intact as they were at the quarry site. On the other hand, the aggregate which has undergone artificial process for its physico-chemical change is usually called the artificial aggregate.
The natural aggregate includes pit-run gravel, sand, crushings of rocks and boulders, crushed sand (screenings), and pulverized stone dust (filler).
The artificial aggregate includes lightweight aggregate and colored aggregate produced by melting ore in a special process, and byproducts such as blast furnace slag. The aggregate is also classified by size into coarse aggregate, fine aggregate and filler.
For asphalt pavement work, the aggregate the size of which is larger than a 2.5 mm mesh is called the coarse aggregate, and the smaller sizes are called the fine aggregate. The aggregate the bulk of which passes through a 0.074 mm mesh sieve is called the filler (dust). Usually stone dust comes under this category.
The mechanical properties, shapes and grain sizes of aggregate have a great bearing upon the properties of the asphalt mixture. The importance of the role played by the aggregate will be well understood if you know that the aggregate accounts for about 90 to 96% in weight and about 80% in volume of the asphalt mixture. This will speak volumes about how important the selection of aggregate will be in determining the properties of the pavement.
In recent years, the aggregate resources have been strained heavily, and the quality of aggregate available tody is nothing like so good as that we could have in the past.
Accordingly we are forced to make use of artificial aggregate.
One of the most important matters facing road engineers today and in future is how to secure high-quality aggregate for the surface

courses of truck roads.
The aggregate currently in wide use for asphalt pavements include crushed strone, sand and stone dust available from terranes.
Here we shall study the specifications and required properties of natural aggregate.

2-4-2 Specifications for aggregate

(1) Coarse aggregate

According to JIS A 5001 "Crushed Stone for Road," the quality of crushed stone is specified as follows.
The crushed stone shall be obtained from rocks of basalt, andesite, granite, hard sandstone, hard limestone or equivalent or from the boulders having 3 times as large as the maximum grain size of aggregate required.
The crushed stone shall be homogeneous, clean, hard, durable and free of dust, mud, organic matter and any other contaminants. A sample of crushed stone graded through a 5 mm to 13 mm mesh sive shall be tested according to the methods specified in JIS A 5001, and shall meet the requirements specified (see Table 3-9 hereof).
Though not specified in the text of JIS A 5001, the tolerance of the contaminants in the crushed stone for road use is given in its exposition as shown in Table 3-10 hereof.
In the Asphalt Pavement Design Manual, on the other hand, reference requirements concerning the durability and contaminant limits are shown as in Tables 3-11 and 3-12. The grading of crushed stone is specified in JIS A 5001 as shown in Table 3-13. It is also specified in JIS A 5001 that the crushed stone of the grading not meeting the requirements in Table 3-13 may be used if it meets the grading requirements of aggregate for an intended mixture when combined with other crushed stone, sand, stone dust or others.
All the above requirements are applies to the gravel. As regards slag, the above requirements are also applied with the exception of specific gravity and hygroscopicity.

If aggregate is hydrophilic, asphalt film may come off easily under the influence of water. The cohesion between aggregate and asphalt is related to the characteristics of both aggregate and asphalt. If the cohesion is poor, or is suspected to be poor, the cohesion tests must be conducted. Crushed stone is a local product, and the latitude of its selection is limited primarily from the viewpoint of economics. Even if moisture absorption is higher than a specified value of 3%, the aggregate should be used if it is available from a local source. In this case, however, its application methods should thoroughly be studied in advance through JIS tests, other engineering tests and experimental pavement work, etc. It is also important to inspect the crusher plant to make sure that the crushed stone of uniform quality is produced.

(2) Fine aggregate

River sand is widely used as fine aggregate. In addition, crushed sand (screenings), sea sand and hill sand are also used. According to the Asphalt Pavement Design Manual, the fine aggregate is required to have cleanliness, hardness, durability and grain size to meet specific purposes, and to be free from contaminants such as dust, mud, wood chips, etc.

The Design Manual has no provisions concerning durability, hardness, etc. This is probably due to the fact that sand is made of the "hard core" of rock that has weathered well. River sand varies in size from depth to depth even at the same extraction site. Thus, a careful preliminary survey is required for determination of grain size distribution. In some case, it will be necessary to assess the applicability of pit-run sand according to the Concrete Mix Proportioning Standard prepared by the Japan Society of Civil Engineers or the provisions for the screenings specified in JIS A 5001, "Crushed Stone for Road."

(3) Filler (stone dust)

Filler (stone dust) is produced by pulverizing limestone or igneous rock. It is required to be less than 1.0% in moisture content and free from behing bulked. The standard grading is as specified in Table 3-14.
The filler produced by pulverizing igneous rock is required to meet Table 3-15.
In addition, Portland cement and slaked lime are available as filler. In recent years, byproducts from various industries are used as filler. Asbestos may be classed among filler.
Filler fills up the voids in the asphalt mixture to increase its density, reduce permeability to water, and to prevent the displacement of aggregate. When mixed up well with asphalt, filler increases the toughness of asphalt mixture, reduces the thermal sensitivity of the mixture, improves other physical properties, and reinforces the mixture as a whole.
Particularly, slaked lime serves as an antistripping agent for asphalt as explained later.

2-4-3 Properties of aggregate

(1) Physical properties of aggregate

(a) Specific gravity and moisture absorption

Usually, the specific gravity and the moisture absorption act contrary to each other. If the minerals of which the aggregate is made are the same, the aggregate will showa smaller moisture absorption the larger the specific gravity it has. The aggregate which is large in specific gravity and small in moisture abosrption is dense, and is generally hard and durable. The quality of aggregate can therefore be evaluated to some degree by analyzing the results of the measurement of specific gravity and moisture absorption.
Gravel, crushed boulders, sedimentary sand (ground sand, hill sand) often contain flaky stones, pumice and other

impurities which are smaller in specific gravity than other useful hard grains. Thus, by checking the specific gravity, we can judge the applicability of the aggregate. The moisture absorption is an important index showing the surface and internal porosity of aggregate. The larger the moisture absorption, the lower the strength of the aggregate, and the lower the resistance to the frosting and melting actions.
The above, however, should be interpreted simply as a generalization. There is such aggregate that shows a high moisture absorption, yet is hard and durable.
The moisture absorption of aggregate is said to influence the durability of asphalt mixture. If aggregate used has a high moisture absorption, it absorbs part of asphalt during construction and continues to absorb asphalt with time even after the road is put into commission. As a result, the asphalt coating over the grains of aggregate becomes thinner and thinner, increasing the percentage of voids in the mixture. In effect, such aggregate turns the asphalt mixture into an asphalt-lean one. In recent years, however, high-quality aggregate is hard to come by, and we often are forced to make locally available aggregate do, however high its moisture absorption may be. In such a case, we must rack our brains in mix proportioning design and engineering. Various symptioms appear depending on the moisture content in aggregate and the quantity of asphalt used. Increasing the amount of asphalt in anticipation of absorption into aggregate does not always work wel. Ideally speaking, aggregate should have a proper degree of porosity as it take in asphalt to a degree for increased cohesion with asphalt matrix to improve imperviousness and durability of the mixture as a sholw.

(b) Hardness of aggregate

The hardness of aggregate here refers to resistance to wear and fracture. The resistance to wear and fracture is particularly important in ensuring the stability of asphalt mixture. The aggregate in the asphalt mixture is required to support the traffic loads and at the same time to communicate them to the under layers. The aggregate undergoes ramming force during road construction and after the road is open to traffic, and will be broken and worn down. Thus, it is required to be as hard and tough as possible. The grains of aggregate gall and chafe each other when they are loaded, compressed or forced to slide and roll, and will be triturated. The degree of reduction in grain size is governed by the size of external load and the resistance of grains to fracture, friction and wear, etc., and is also related with the original size of the grains. Coarse aggregate-rich asphalt mixture is liable to cause this phenomenon. If the mixture uses fragile aggregate, the asphalt matrix is liable to be broken. And if consolidated excessively, the asphalt surface course will become liable to run. As explained above, the aggregate should preferably be as hard and tough against crushing and wearing forces as possible. In evaluating the hardness and wear resistance, however, such a hardness testing method as used for rocks is not employed, but usually a Los Angeles tester is used for determining the wear loss. The popularity of Los Angeles tester is due to the fact that it produces a viable result reflection the effects of flaky, flat and spinal grains.

(c) Surface texture of aggregate

The surface texture is as important for the stability of asphalt mixture as the appearance (cubic, flat, slender). Rough-surface grains are more immobile than smooth-surface grains, and improve the stability of asphalt mixture. But this should also be interpreted as a simple generalization. The grains which appear rough and irregular may become fluid when mixed with asphalt. On the other hand, there are such smooth-surface grains that show a large sliding resistance when mixed with aspahlt. In other words, there is no decisive method of evaluating the surface texture of aggregate so far as the present state of the art is concerned. In this respect, futher study will be needed.

(d) Durability and heating stability

The durability here refers to the resistance to cryoturbation, thermal actions and hydraulic actions. In order to evaluate the durability, there is usually employed a stability test using solutions of sodium sulfate and magnesium sulfate. Although it is unknown to what degree the test results reflect the service conditions of asphalt pavement, this stability test has long been incorporated as a standard practice.
The aggregate which shows a low moisture absorption has little or no problem from the viewpoint of durability, but if the aggregate is highly hydrophilic and prone to frosting and melting actions, the evaluation by this test method will make sense.
In the hot mix process, the aggregate is heated by a drier at high temperatures until dry, and is liable to be overheated. In an asphalt plant drier, the aggregate may be heated at more than 500°C even for a short period. Under such heat, the aggregate may be

embrittled or denaturalized, and may affect the durability of the pavement. Fig. 3-32 offers a case in point; ten different aggregates are compared with reference to fracture strength measured according to BS 812 between before and after heating for about 1 hour in electric furnaces controlled at 300°C and 600°C. Almost every aggregate is affected by heating, and the test values are above the 1:1 line. Although there is no significant difference in fracture value between the aggregates, it is evident that the fracture value is higher when heated at 600°C than when heated at 300°C. In actuality, however, the aggregate is rarely subjected to such high temperatures for as long as one hour. But, the plant often heats up the aggregate at a high rate, and the thermal stress the aggregate must take during heating in the plant will be higher than it will take at a laboratory test. For this reason, it will be reasonable to measure and judge the aggregate strength after a heating test.

(e) Conhesion (anti-stripping property)

When exposed to repeated traffic loads in the presence of water, the asphalt mixture is liable to disintegrate as asphalt tends to separate from aggregate. This phenomenon is one of the major causes of asphalt pavement failures, seriously impairing the stability and durability of asphalt pavement. This phenomenon is not only closely related to the physico-chemical properties of aggregate and asphalt, but also is governed by various factors, including mixing conditions, ingredients of mixture, and service conditions.
The aggregate is classified by its degree of affinity for water into hydrophilic and hydrophobic types.
The hydrophilic type gets wet with water than with asphalt, and is liable to part with asphalt. Granite,

quartz prophyry, quartzite, and other silicious acid igneous rocks or sedimentary rocks formed therefrom. On the other hand, basic aggregate from basalt, gabbro, basic igneous rocks and limestone goes well with asphalt, and is least subject to the influence of water. In the appendix of the Asphalt Pavement Design Manual, there is specified an asphalt film stripping test for the purpose of evaluating the cohesive power between coarse aggregate and asphalt. But this method leaves much to be desired. In recent years, ASTM and AASHTO have employed an immersion compression test for asphalt mixture, though this still needs improvement in view of true representation of practical aggregate requirements. In Japan, the immersed Marshal tester method is specified in the Asphalt Pavement Design Manual and the Specifications formulated by the Japan Highway Public Corporation.

According to a survey conducted by the Ministry of Construction, it is found that the separation of asphalt mixture has a grave effect on the cracking, rutting and thus on the maintenance of road. There are available two methods for the prevention of separation; one in which materials and mix proportion are properly selected and the other in which a particular process is used to cut off water. For example, it is desirable to use slaked lime and Portland cement as part of fillter and to mix anti-stripping agent or tar into asphalt. It is also desirable to use asphalt of a low penetration.

(2) Shapes and dimensions

(a) Angularity

Angular and irregularly shaped aggregate is rather hard to compact than round aggregate. But when rolled well, brreciated grains intermesh each other and become stable mechanically. The angularity of aggregate is an important factor for ensuring the stability of asphalt mixture. It is reported that the meshing becomes firmer the closer the shape is to cube. The stability of open or leangrade asphalt mixture is solely dependent on the meshing because grains are nearly in point contact with each other. In the mixture, like Topeka, which contains fine aggregate much, the angularity has a salient effect on the stability. It is reported that when the ratio of screenings to the total volume of fine aggregate is increased to about 30%, the stability is improved pronouncedly. It should be noted however that if the ratio of screenings is increased too much, the stability will be impaired because the existing rolling processes cannot consolidate the mixture satisfactorily. In Japan, there is no proper way of evaluating the angularity. But it is specified that the quarry-run blocks which crushed stone is to be produced should have a size at least three times as large as the maximum size of crushed stone. It is also specified that the crushed gravel, more than 90% of which remains on a 5 mm mesh sieve, should have at least one fractured face on each of the grains which account for more than 40% in weight. The Japan Highway Public Corporation requires that the crushed gravel, more than 90% of which remains on a 2.5 mm mesh sieve, should be used. In this way, the angularity of aggregate is specified indirectly.

(b) Flatness and slenderness

Aggregate containing flat or slender grains much is inferior in intermeshing, and impairs the stability of asphalt mixture. Fig. 3-33 shows an example. It is found that the mixture using flat aggregate alone by 10% is down about 70% in Marshal stability compared with the mixture using high-qualitynear-cubic aggregate.

It is reported that when a mixture containing quantities of slender and flat aggregate is given a gyratory motion - a kind of kneading action -, the aggregate which has laid on the side stands up. This suggests that this kind of aggregate may wander about when the pavement is in service.

It is also reported that this kind of aggregate is liable to load the screen in the asphalt plant. While it is required to evaluate the detrimental effect of slenderness and flatness, there is no unified standard in Japan to deal with this problem. Table 3-36 are specifications set forth in the Asphalt Pavement Design Manual and the Japan Highway Public Corporation's Specification.

(c) Maximum grain size

The grading and the maximum grain size of aggregate should be determined properly depending on the construction process to be applied, pavement thickness and environmental conditions for the purpose of ensuring improved pavement stability. The maximum grain size is expressed by the smallest mesh size of the sieves through which at least 95% of grains in weight can be passed. The mixing and grading of the mixture becomes difficult with increase in the maximum grain size. On the other hand, if the maximum grain size is reduced to too small a value, the ramming will become hard to cause the flow of mixture at the time of rolling, though the mixing and grading will be improved.

By the rule of thumb, the maximum grain sixe usually is set at a half to a quarter of the pavement thickness.

$D = \frac{1}{2}d$ to $\frac{1}{4}d$ (D: maximum grain size; d: thickness per course)

The optimum maximum grain size is mainly concerned with the thickness of a course to be constructed.

According to the Asphalt Pavement Design Manual, the optimum grain size is specified to be 13 to 15 mm. This will probably due to the fact that such grains ensure well-grainted, uniform and dense road surfaces and at the same time are easy to place by machine, and that the mechanical stability (internal friction angle) of asphalt mixture is governed by grain size (content of coarse aggregate) rather than by the difference in maximum grain size.

(3) Grain size of aggregate

The grain size of aggregate is one of the most important factors that affect the stability and workability of asphalt mixture. In Japan, most of asphalt mixtures use continuously graded aggregate as in coarse, dense and fine grade asphalt concrete. Gap-graded asphalt concrete (coarse and dense) is also used.

The mixtures are termed variously, signifying how important the grading is. In the 1967 edition of the Asphalt Pavement Design manual, the grading of aggregate was specified only for one class of coarse grade asphalt concrete, two classes of dense grade asphalt concrete (maximum grain size: 20 mm and 13 mm) and revised Topeka. In recent years, however, the types of mixtures have been diversified reflecting a sharp rise in demand for mixtures. The demand for flow-resistant mixtures is particularly strong. Entrepreneurs has mushroomed to meet his demand, to the extent they may disturb the asphalt pavement technology. Concerned about the situation, the Ministry of Construction conducted a survey through various institutions in the country.

According to the recommendations of the commissioned institutions, the Ministry of Construction published a new manual (1978 edition) for standardization, labor-saving and rationalization of pavement engineering. The grading of aggregate to be used for a specific mixture should be

determined after due consideration of the types and purposes for which the surface course, base course, etc. are used, traffic conditions, climatic conditions, etc. In the new design manual, the types of mixtures and their selection standards are specified as shown in Table 3-17, and the grading of mixtures as shown in Table 3-18. As footnoted in Table 3-17, the mixtures are classified by the quantity of aggregate which passes through a 2.5 mm mesh sieve.
The coarse grade, dense grade and fine grade are respectively up to 30%, 30% to 50% and 50% up in terms of undersize quantity. Discrete gradings are called the gap gradings.
In handling the gradings of mixtures, the 2.5 mm mesh is an important point dividing between coarse and fine aggregate and also a characteristic point for job control. The more the quantity of the 2.5 mm mesh undersizes, the finer the road surface texture. If the undersize quantity becomes smaller, the road surface becomes coarser.

2-5 Asphalt and asphalt quantity

2-5-1 General

For crude oil, Japan in mostly dependent on Mid-East countries such as Iran, Saudi Arabia, United Arab Emirates and Kuwait. The crude oils originating in the Mid-East are rich in paraffin, and the asphalt produced therefrom are said not so good for pavement use. European countries are also importing from the Mid-East, and are using asphalts without complaint. Japan's track record for over 20 years in the use of Mid-East asphalts for pavement shows no particular problem. In this section, we shall learn the specifications for the paving grade asphalt and their meanings in brief.

2-5-2 Specifications for paving grade petroleum asphalt and their transition

(1) Specifications

The engineering qualities of asphalt are specified in JIS K 2207 (Petroleum Asphalt). On the other hand, the qualities of paving grade petroleum asphalt have so far been established by the Japan Road Association.

Up until now, the Japan Road Association has published the Asphalt Pavement Design Manual four times, the first edition being in 1961 (Table 3-19), the second in 1967 (Table 3-20), the third in 1975 (Table 3-21) and the fourth in 1978 (Table 3-22).

(2) In Transition

In the 1961 edition, the paving grade asphalt was classified into three types A, B and C according to low-temperature ductility. Type A usually is produced from asphalt-base crude oil, Type B from mixed base crude oil, and Type C from mixed base crude oil which contains a comparatively large quantity of paraffin. In the selection of asphalts according to this manual, the trends were in favor of Type A and B, shunning away Type C. The quantitative analysis method of paraffin content in asphalt was incomplete, and the opinions came that there would be unnecessary to classify asphalt by thermal ductility only if we use asphalt with care, and that the method of measurement be simplified. Thus, the classification by ductility was repealed, and the manual was revised in 1967.

In the 1967 edition, the specifications for softening point, evaporation, penetration after evaporation, and flash point were all tightened as compared with the 1961 edition. It was specified that the ductility test be conducted at 15°C only to eliminate Type C specified in the 1961 edition.

Table 3-21 was finalized after being designated as tentative upon the recommendations of the Pavement Committee in June 1972.

Serious rutting failures were noticed among the pavements constructed according to the 1967 edition. In order to against this trouble, specific penetration after evaporation was introduced, together with membrane weight loss at heat, penetration after membrane heating and specific gravity for tightened control of softening point, penetration after evaporation and flashing point, for the purpose of improving the asphalt quality as compared with the 1961 edition.

Table 3-22 shows the specifications of paving grade petroleum asphalt currently in use in Japan.

2-5-3 Purposes and significance of specification tests

(1) Penetration test

This is carried out mainly for knowing the consistency (hardness) of asphalt at normal temperature. The penetration is used for the classification of asphalt.

When combined with the softening point test results or when measured under various temperature conditions for different test durations, the penetration will serve as an effective measure in evaluating the thermal sensitivity and elasticity (mechanical characteristics) of asphalt.

(2) Softening point test

The softening point is defined as the temperature representing a specific consistency (hardness) under a specific condition.

Asphalt softens at heat, and will start flowing.

It the softening point test (R & B test), the temperature at which asphalt presents a specific state of fluidity is measured.

(3) Ductility test

Usually, the ductility is considered as an index showing the degree of cohesion to aggregate or flexibility of pavement. But there are opinions that the ductility cannot be a measure for representing the required quality of paving asphalt because asphalt exists in the form of membrane in the pavement.

(4) Evaporation test, and post-evaporation penetration test

The paving grade asphalt is heated during and after construction. If heated for a long time, the asphalt will send off part of its constituents and will harden.
The evaporation test is designed to show the quantity of volatile components in asphalt. The post-evaporation penetration test is designed to check the degree of hardening after heating for the purpose of eliminating such asphalt that ages quickly.

(5) Thin-film oven test and penetration test after thin-film oven test.

These tests are designed to evaluate the aging characteristics of asphalt and to eliminate such asphalt that ages quickly.
Asphalt is heated, fused and mixed with heated aggregate. From this stage on, asphalt exists in the form of membrane is asphalt mixture. The depth of the asphalt sample used in the evaporation test is about 30 mm because the container used for penetration test is directly used for the evaporation test. In the membrane heating test, however, the sample depth is about 3 mm; namely, the test conditions are by far severer than in the evaporation test.

(6) Post-evaporation penetration test

This is a new test which has been incorporated since the 1975 edition. This kind of test isnot employed by other countries. In this test, the absolute value of the anti-aging property of asphalt is measured.

2-6 Conclusions

In this chapter, the factors affecting the mix proportioning design have been divided into external and internal ones. The pavement temperature and traffic load have been taken up as external factors, and the aggregate, asphalt and the grading of aggregate as internal factors. As regards the internal factors, the engineering specifications and their significance have been touched upon briefly. As regards the external factors, actual data on pavement temperatures have been presented and analyzed.
The following is a summary of this chapter.

(1) The maximum asphalt pavement surface temperature in Chiba is 60°C.

(2) The maximum asphalt pavement surface temperature in Chiba is expressed as 1.7 times the maximum ambient temperature with a dispersion of ±10°C. The minimum pavement surface temperature in Chiba is expressed as 1.2 times the minimum ambient temperature with a dispersion of as small as ±3°C.

(3) From the annual cumulative percentage distribution of pavement temperatures, it is found that the frequency at which the asphalt concrete temperature in Chiba exceeds 50°C is about 1%. The frequency of the pavement surface temperature exceeding 50°C is about 3%. The frequency is about 7% for 40°C to 50°C, about 17% for 30°C to 40°C, and about 28% for 20°C to 30°C.

(4) The pavement temperature in Chiba is about 10 degrees C higher than that in Hokkaido.

(5) According to the tests conducted by the Road Research Laboratory, London, it is found that the ffect of pavement surface temperature on the permanent set takes place almost qually between three temperature ranges - 30°C to 40°C, 40°C to 50°C, and 50°C up.

(6) Thanks to the development of the running vehicle weighing system, the wheel load distribution on the roads in Japan has been made clear. It is found that the vehicles loaded in excess of the legal limits are rife in Japan as compared with other countries. The effects of the distribution of wheel running paths and the vehicle speeds on the pavement failures are also discussed.

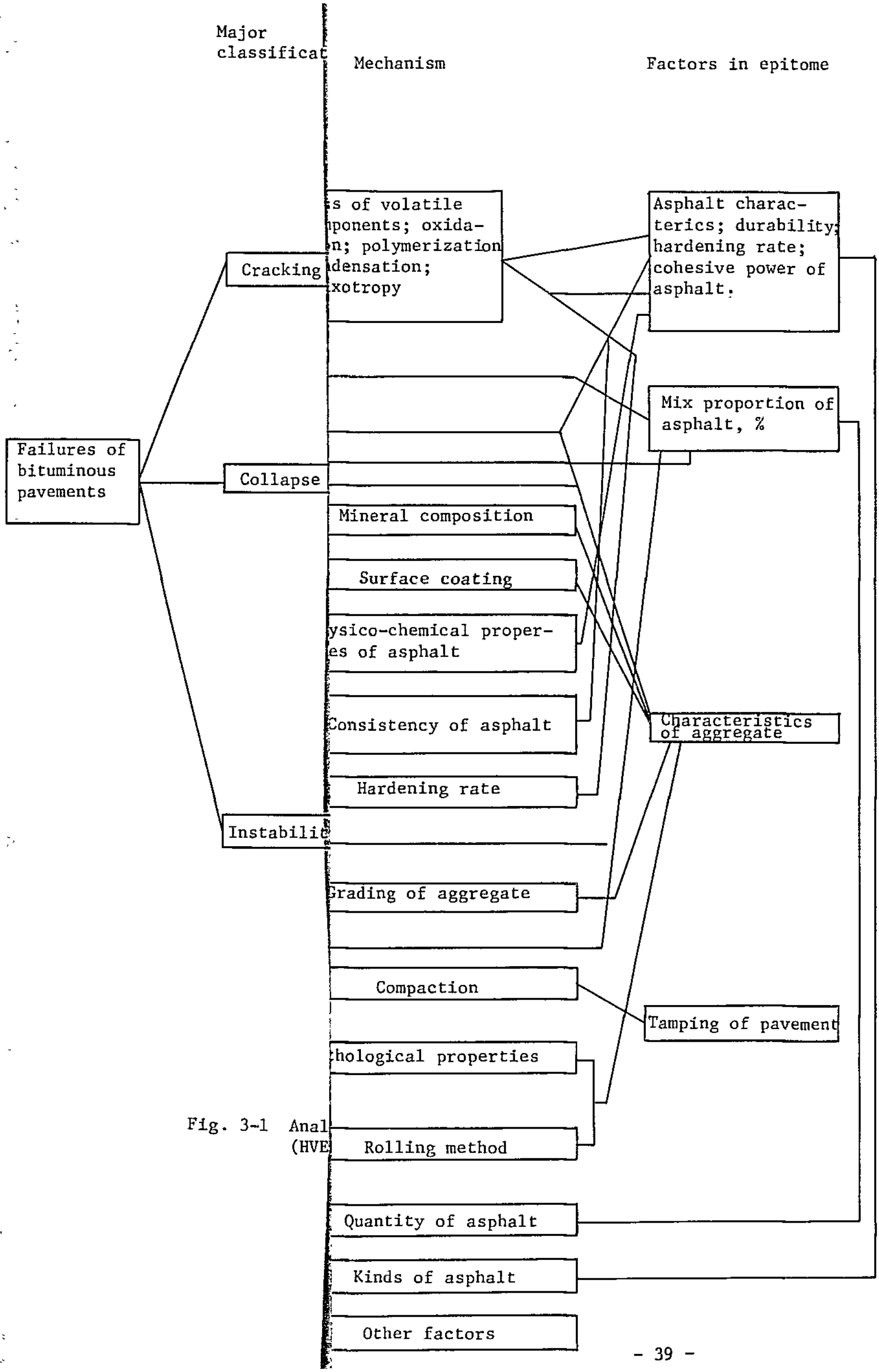

Fig. 3-1 Anal
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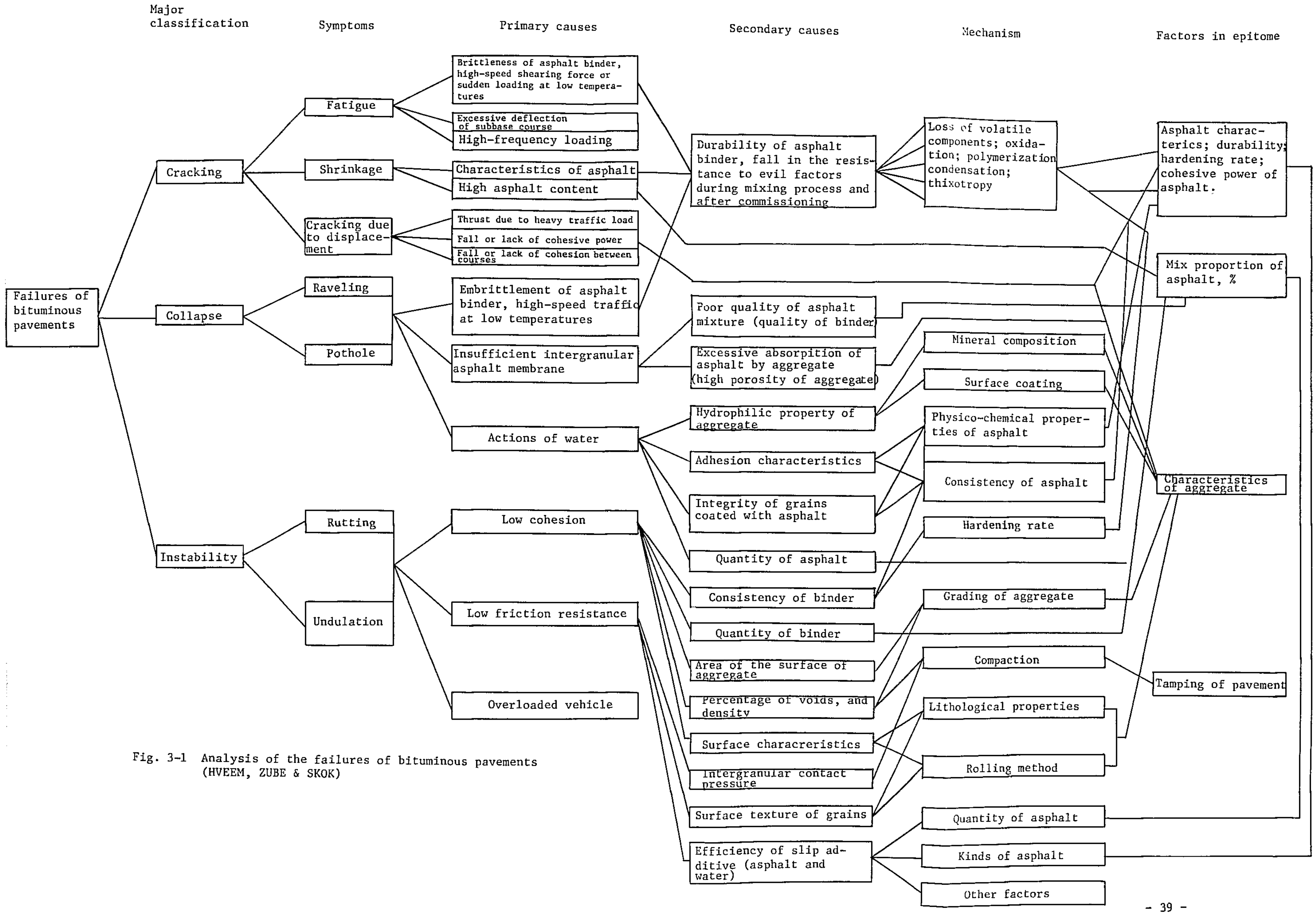

Fig. 3-1 Analysis of the failures of bituminous pavements (HVEEM, ZUBE & SKOK)

Type of failure	Faulty symptoms		An example of destructive mechanisms
Fracture	Cracking	Causes relating to traffic load	Repetitive load (fatigue)
			Excessive load
			Slip (in horizontal direction)
		Cause not relating to traffic load	Change in moisture content
			Temperature changes
			Shrinkage
Distortion	Permanent Deformation	Causes relating to traffic load	Excessive load (shearing stree)
			Time-dependent deformation (creep)
			Rolling (compaction)
		Causes not relating to traffic load	Swelling
			Consolidation of the courses below surface course
Disintegration	Raveling; stripping	Causes relating to traffic load	Wear by traffic
			Separation of aggregate
		Causes not relating to traffic load	Cohexive power (lack of bond)
			Chemical reactions
			Durability of asphalt

Fig. 3-2 Classification of asphalt pavement failures (Monismith)

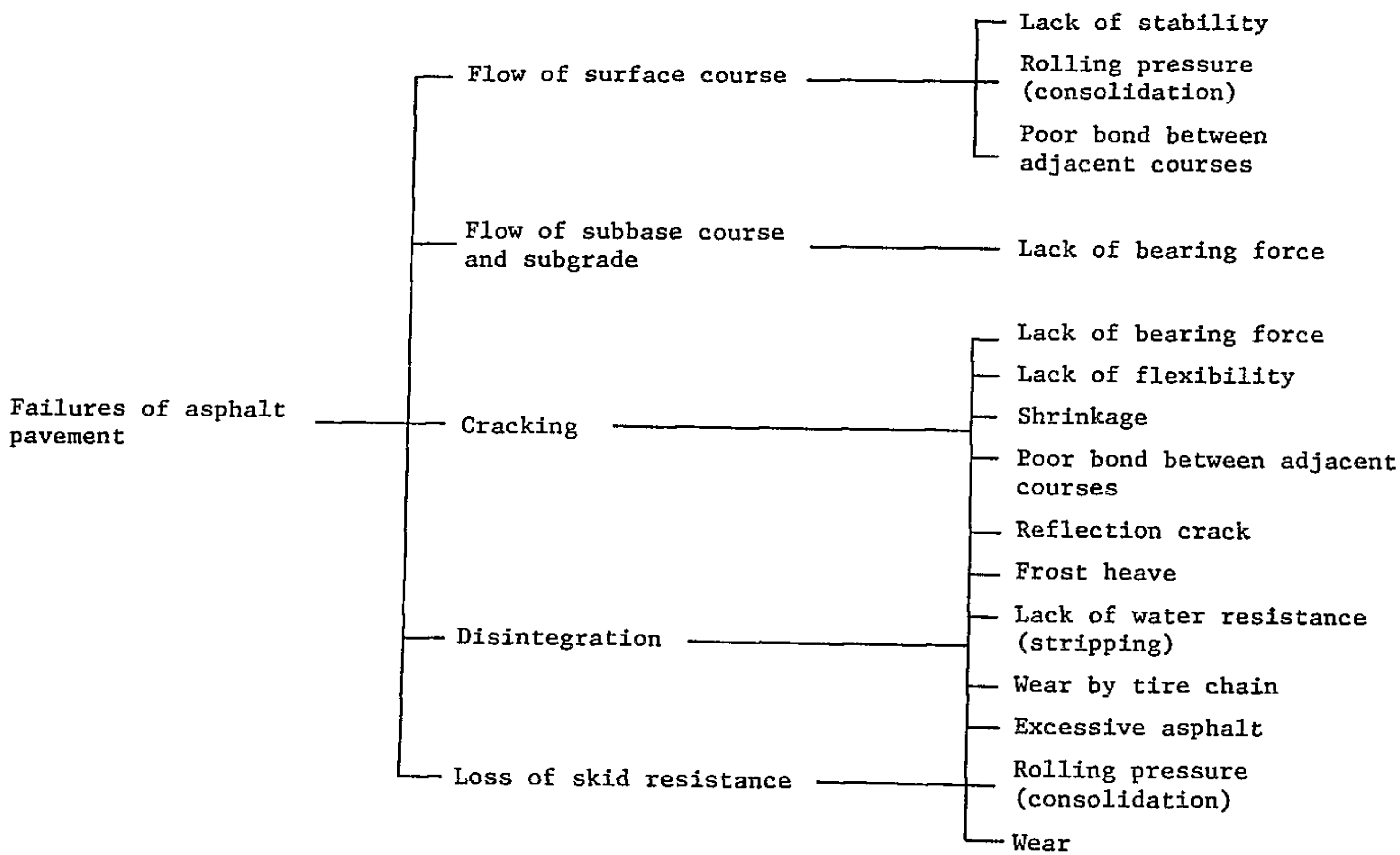

Fig. 3-3 Classification of asphalt pavement failures (by Saburo Matsuno)

- Stability (resistance to permanent deformation)
  - Friction
    - Intergranular friction (solids)
      1. Surface texture of aggregate (roughness)
      2. Intergranular contact pressure (by tamping and loading)
      3. Asphalt quantity (at intergranular contact points)
      4. Angularity of aggregate (intermeshing of aggregate grains)
    - Mass viscosity (liquids)
  - Cohesion (tensile strength)
    1. Rheological properties of asphalt
       - (a) Initial consistency
       - (b) Temperature
       - (c) Loading rate
       - (d) Effect of time and aging on consistency
       - (e) Chemical composition
    2. Grading and area of surface of aggregate
    3. Density of aggregate
    4. Cohesion between asphalt and aggregate
  - Inertia (resistance to motion)
    1. Magnitude of load (vehicle weight)
    2. Loading rate and time (vehicle speed)
    3. Mass of pavement mixture

Fig. 3-4 Factors affecting stability (by Hveem and Vallerga)

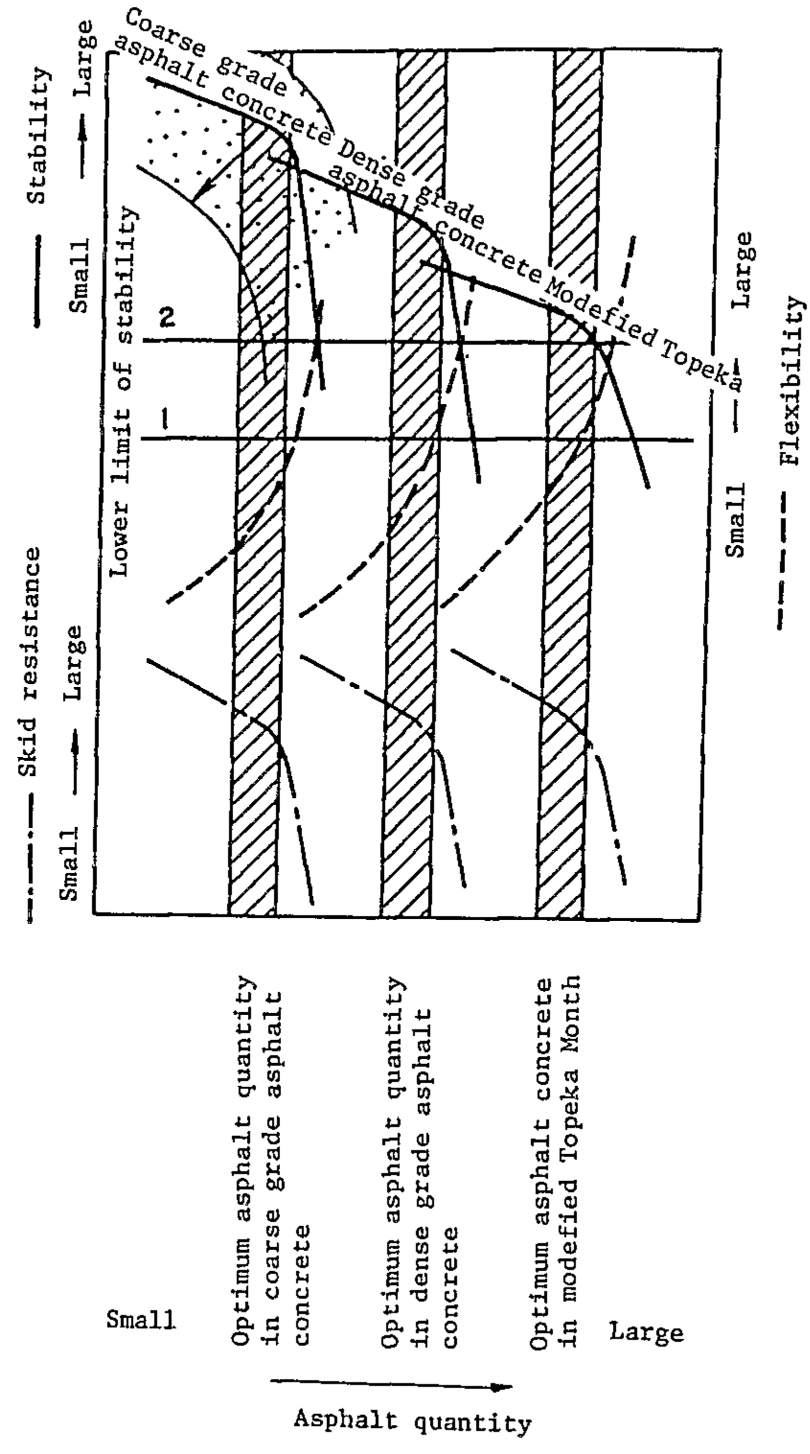

Fig. 3-5 Asphalt quantity vs. properties of mixtures

Table 3-1 Measurement of pavement temperatures

Reporter	Location			Measuring period	Method of measurement
Saburo Matsuno	Chiba: within the premises of Chiba Branch Office, Public Works Research Institute			Jan. to Dec., 1961	Automatic dot-printing temperature recorder, using a copper-constantan thermocouple (JIS Class 0.75) consisting of enameled and rubber-insulated 1.0 mm copper wire and constantan wire.
Masaaki Ohta	Hokkaido			1966 to 1967	Electric resistance thermometer
	Measuring station No.	Regional Construction Office in charge	Location		
	1	Hakodate	Route 37 at Asahihama, Oshamanbe-cho, Yamagoe-gun		
	2	Muroran	Route 37 at Takasago, Abuta-cho, Abuta-gun		
	3	Muroran	Route 235 at Komaba, Shizunai-cho, Shizunai-gun		
	4	Otaru	Route 5 at Kamiyama do, Niki-cho, Yoichi-gun		
	5	Asahikawa	Route 40 at Minami-cho, Shibetsu		
	6	Obihiro	Route 38 at Satsunai, Makubetsu-cho, Nakagawa-gun		
	7	Kushiro	Route 44 at Kaizuka-cho, Kushiro		
	8	Abashiri	Route 240 at Mitomi, Bihoro-cho Chinai, Abashiri-gun		
	9	Wakkanai	Route 238 at Kita 2-3, Hamatonbetsu-cho, Esashi-gun		
	10	Rumoi	Route 233 at Kaoruyama-cho, Rumoi		
	11	Otaru	Route 5 at Rankoshi-cho, Isoya-gun		
Akira Yoshimoto	Kyoto: at quarters of the Science and Technology Dept., Ritsumeikan University, on the southern foot of Mt. Kinugasa, Kyoto			May 1968 to Aug. 1969	A pavement mode (50 cm x 50 cm x 50 cm deep insulated with foam styrol on the sides and embedded into the ground
Kaxuhiro	Tokyo: within the premises of the Laboratory, Japan Highway Public Corporation			Jul. 1970 to Jan. 1971	Three pavements with different cross-sectional configurations constructed in a test field and embedded with Carlson type thermometers in respective courses

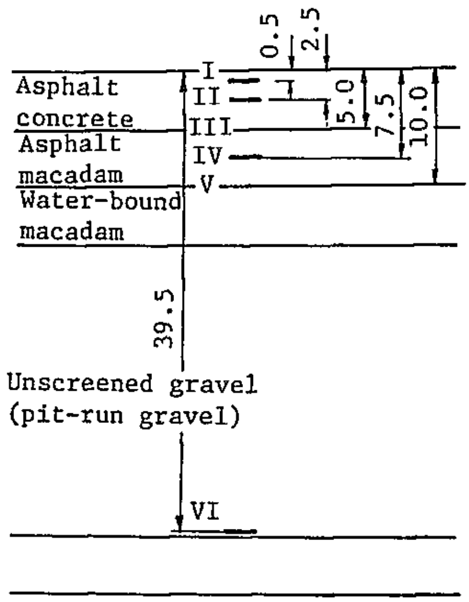

Fig. 3-6(a) Cross section and embedded positions of sensors (cm)

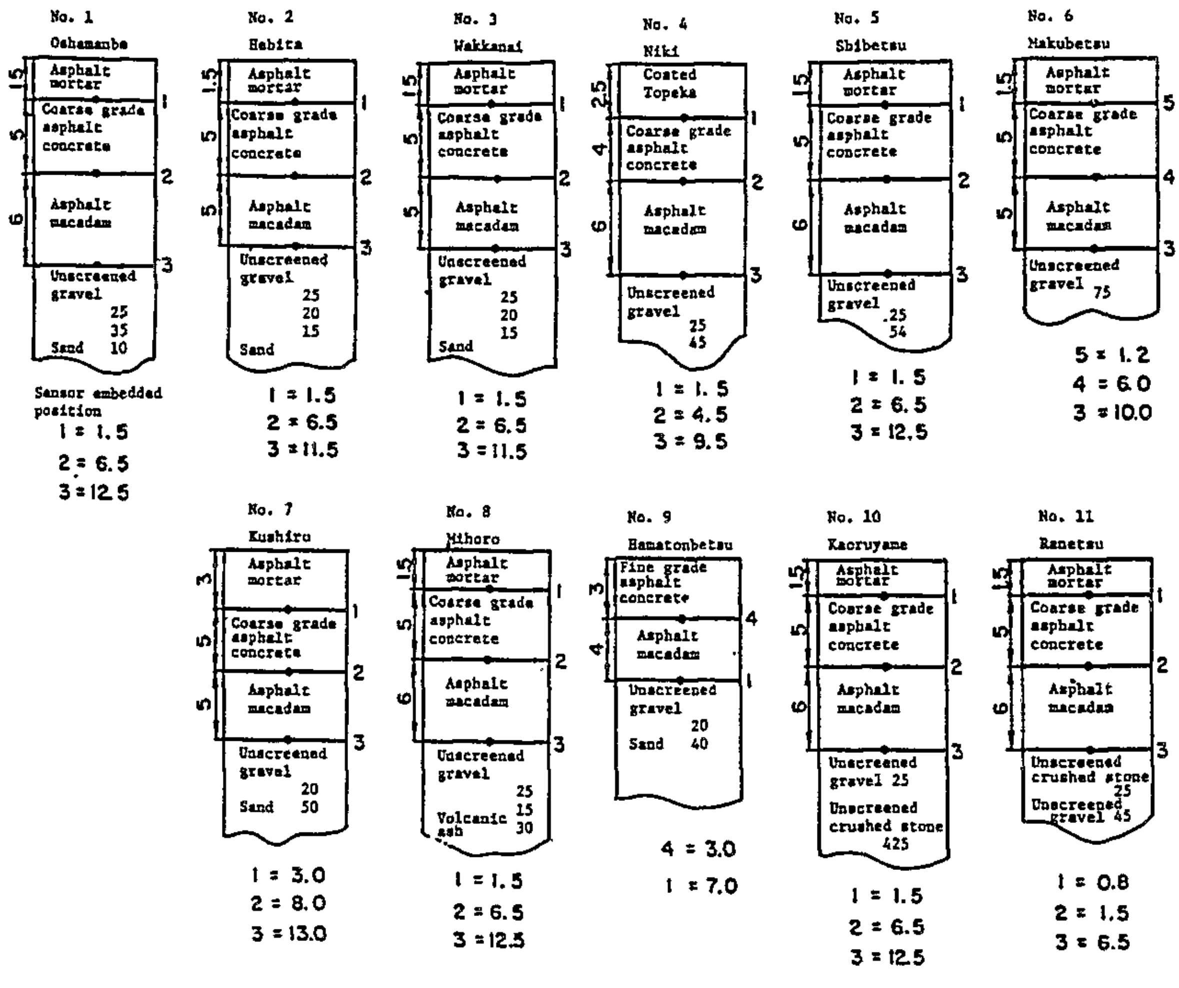

Fig. 3-6(b) Embedded positions of sensors (cm)

Table 3-2 Climatic conditions of measuring stations
(ambient temperature and rainfall)

Month		1	2	3	4	5	6	7	8	9	10	11	12	
Chiba		5.7°C	6.0	8.5	12.9	16.5	19.4	23.0	(24.9)	22.8	18.2	13.5	8.4	15.0
		84mm	113	129	139	138	167	119	131	196	(250)	159	91	1,715
Hokkaido		5.5°C	-4.7	-1.0	5.7	11.3	15.5	20.0	(21.7)	16.8	10.4	3.6	-2.6	7.6
		111mm	83	67	66	59	59	100	107	(145)	113	112	104	1,136

Over/Under: Monthly mean ambient temperature (°C)
Monthly rainfalls (mm)

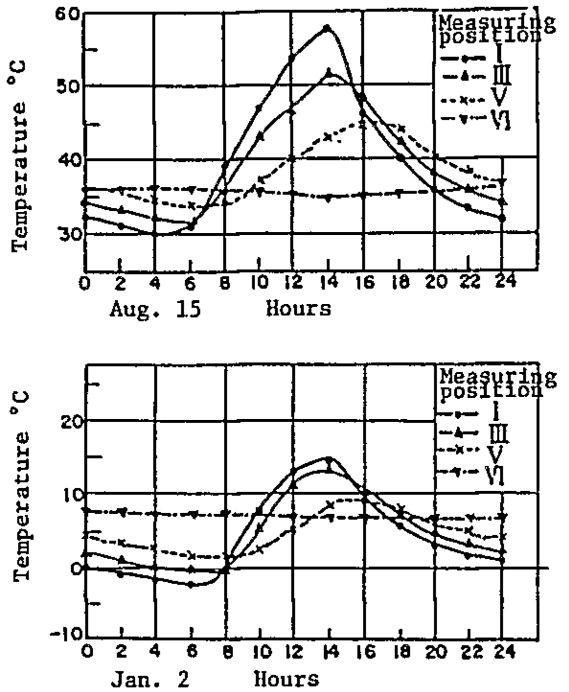

Fig. 3-7 Hourly temperature change (sunny day)

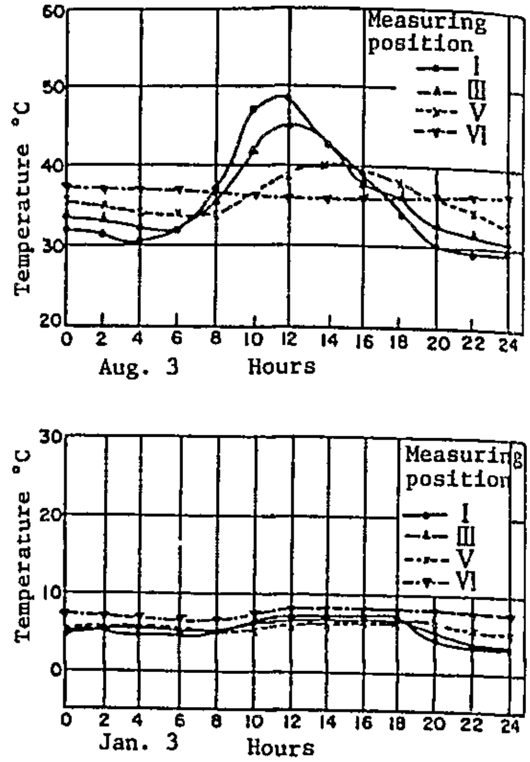

Fig. 3-8 Hourly temperature chang

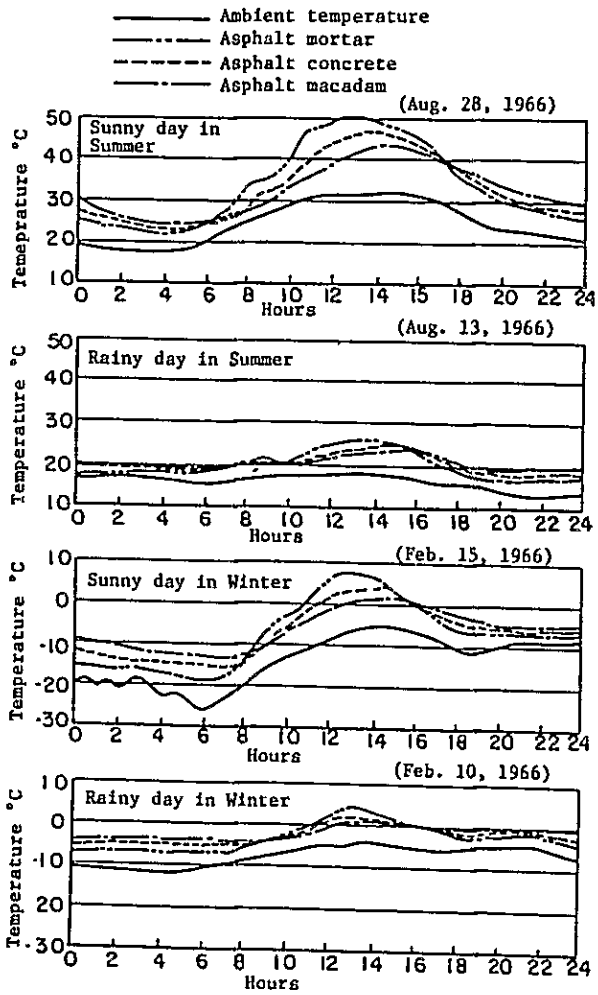

Fig. 3-9 Hourly changes of pavement temperature

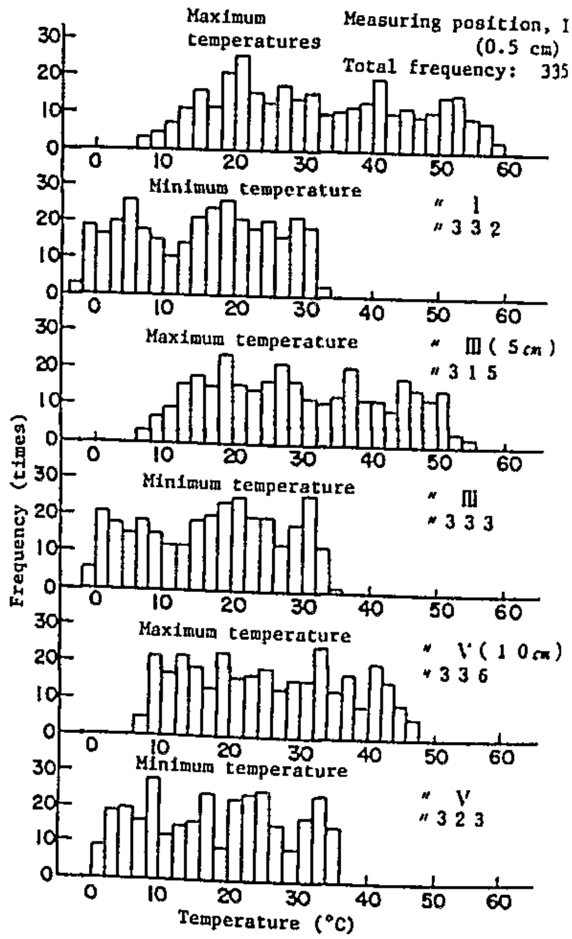

Fig. 3-10 Frequency of maximum and minimum temperature

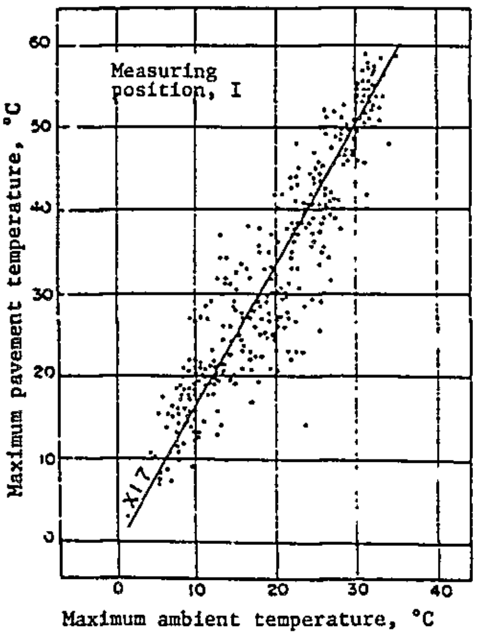

Fig. 3-11 Maximum and minimum temperatures on the pavement surface

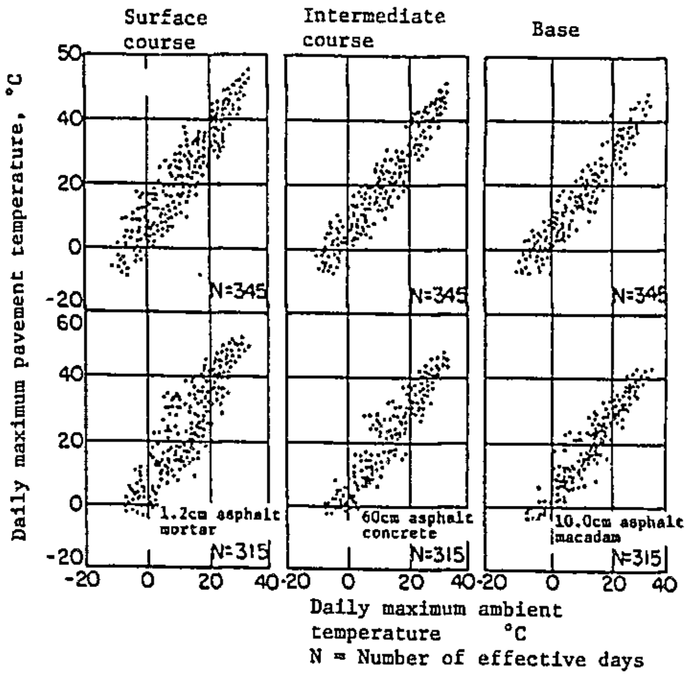

Fig. 3-13 Daily maximum ambient temperature vs. daily maximum pavement course temperatures (No. 6 Makubetsu)

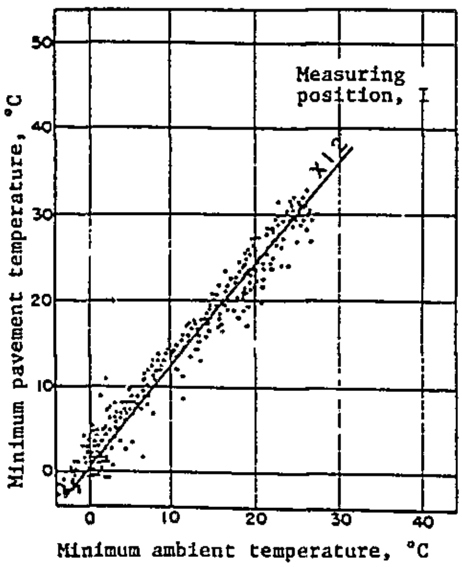

Fig. 3-12 Maximum and minimum temperatures on the pavement surface

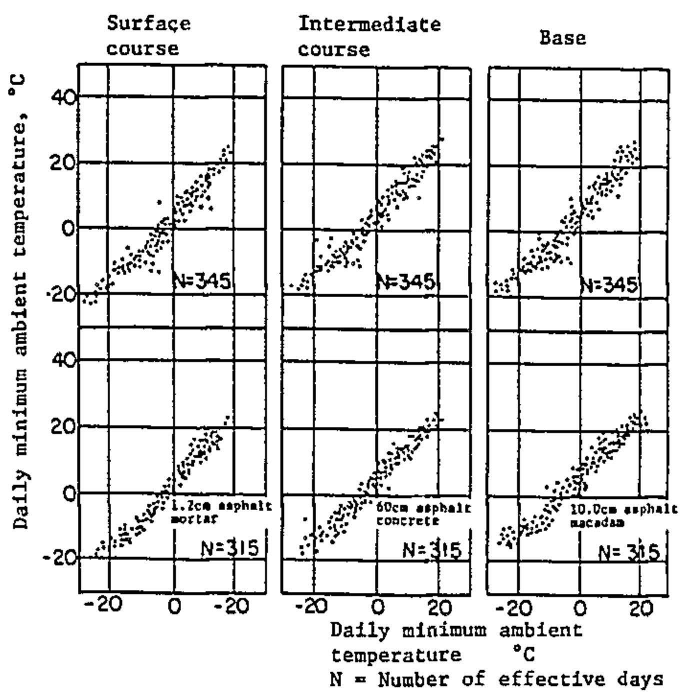

Fig. 3-14 Daily minimum ambient temperature vs. daily minimum pavement course temperature (No. 6 Makubetsu)

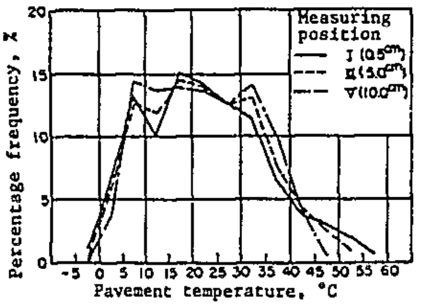

Fig. 3-15 Annual percentage frequency distribution of pavement temperatures measured at an interval of 2 hours

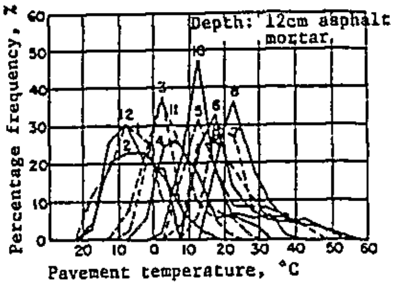

Fig. 3-18 Monthly frequency distribution of temperatures (No. 6 Makubetsu)

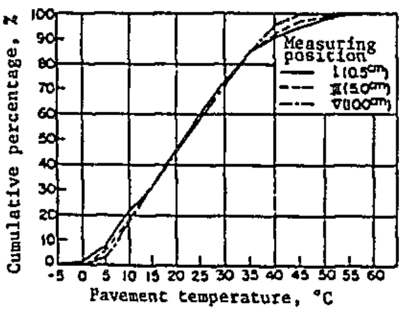

Fig. 3-16 Annual cumulative percentage frequency distribution of pavement temperatures measured at an interval of 2 hours

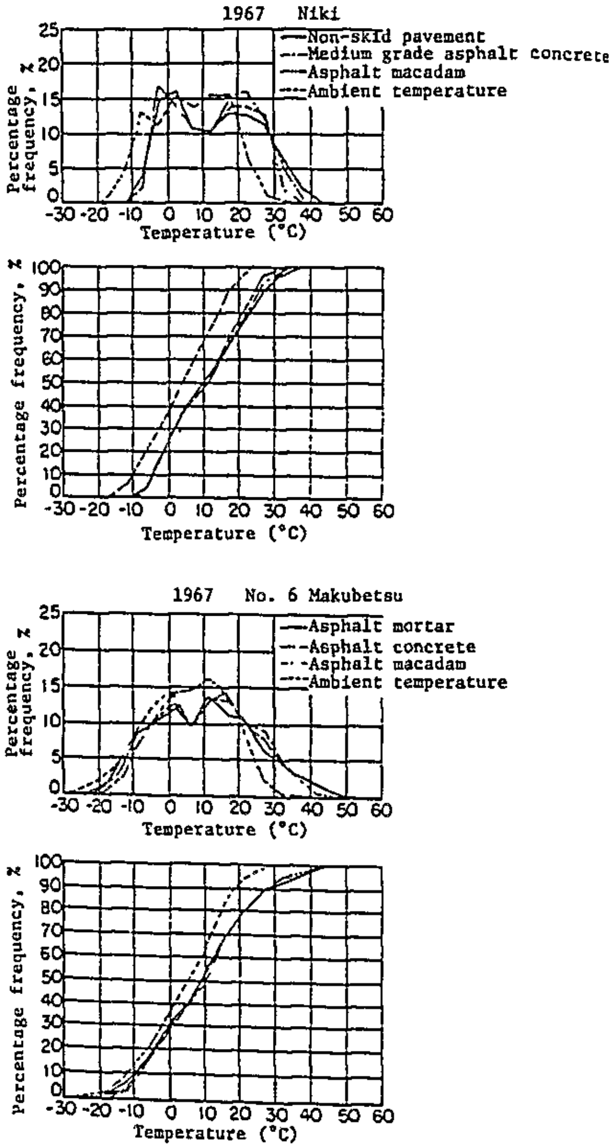

Fig. 3-19 Annual frequency of temperatures

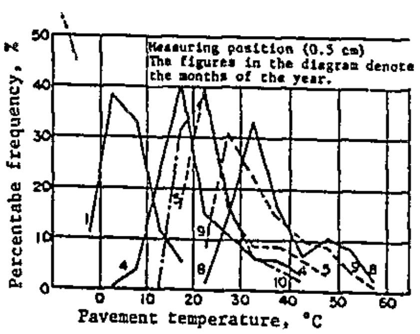

Fig. 3-17 Monthly percentage frequency distribution of pavement temperatures measured at an interval of 2 hours

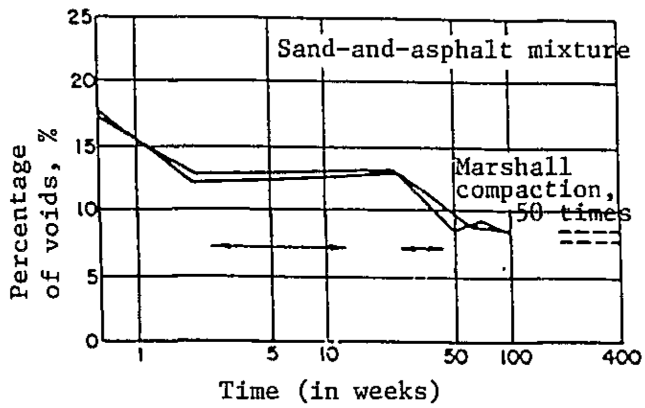

Fig. 3-20 Annual change in the percentage of voids in the surface course

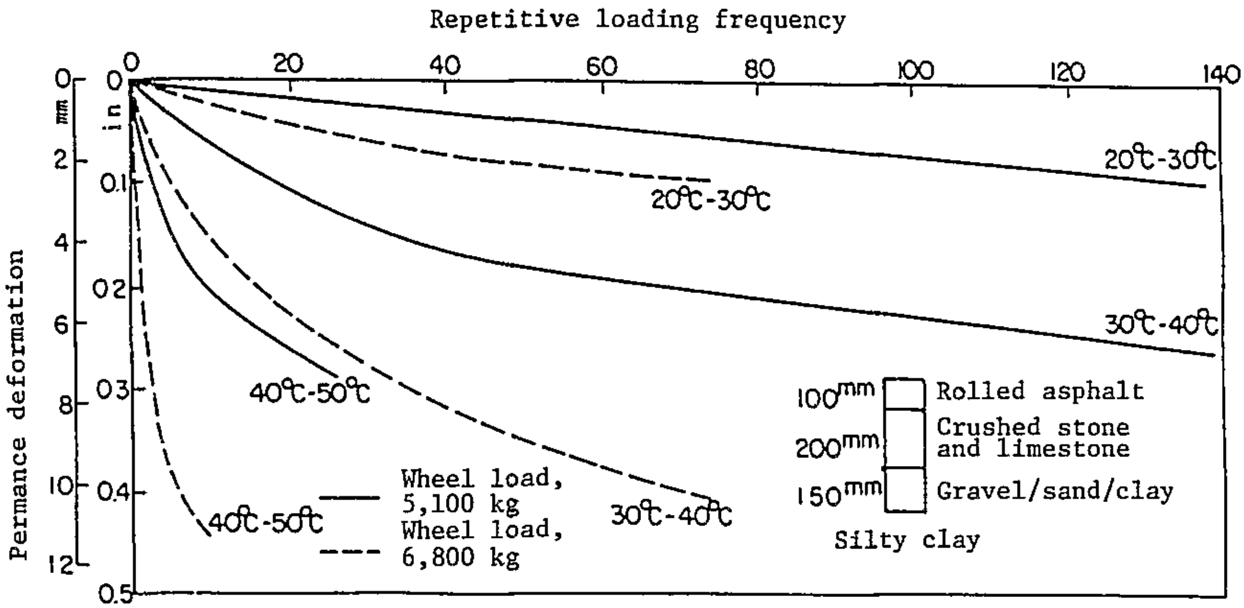

Fig. 3-21 Surface course permanent deformation vs. repetive loading frequency (TRRL)

Table 3-3 Specifications of road machine (TRRL)

Size of test track	Diameter: 67.2m (220 ft) Width : 3.0m (10 ft)	Test temperature	20 ∿ 50°C
Number and size of test wheels	—	Thickness of pavement	Rolled asphalt, 100 mm (4")
Wheel load	5,100 kg, 6,800 kg (pneumatically controlled)	Type of subbase	Concrete base + 150 mm gravel drainage course + 200 mm crushed stone course
Treading pressure	—	Others	Canopied for prevention of direct sunlight and heated by infrared lamps.
Wheel running speed	28 km/h		

Table 3-4 Effect of pavement temperature upon permanent deformation

Temperature distribution, °C	Distribution percentage, %	Effect on permanent deformation	Effect on permanent deformation of pavement ((2) x (3))
20 - 30	28	1	1
30 - 40	17	7	4.2
40 - 50	7	20	5.0
50 - 60	3	40	4.0

Table 3-5 Measurement of wheel load distribution

Wheel load, tons	Number of wheels (or number of axles)			Percentage, %		
	Route 4 (Mamada)	Route 17 (Fukiage)	Route 6 (Ushiku)	Route 4 (Mamada)	Route 17 (Fukiage)	Route 6 (Ushiku)
0 ∿ 1	7,190	12,520	7,315	52.10	50.9	64.4
1 ∿ 2	3,156	4,400	1,916	22.99	17.85	16.86
2 ∿ 3	1,379	3,140	953	10.00	12.75	8.38
3 ∿ 4	490	754	265	3.50	3.06	2.33
4 ∿ 5	616	592	208	4.50	2.40	1.83
5 ∿ 6	562	974	245	4.10	3.94	2.16
6 ∿ 7	305	1,008	244	2.18	4.09	2.15
7 ∿ 8	80	776	109	0.60	3.16	0.96
8 ∿ 9	18	351	52	0.10	1.42	0.46
9 ∿ 10	2	102	43	0.01	0.41	0.38
10 ∿ 11	2	5	9	0.01	0.02	0.08
11 ∿ 12	0	0	1	0	0	0.01
12 ∿	0	0	0	0	0	0
Total	13,800	24,622	11,360	100.0	100.0	100.0

Table 3-6 Dump truck wheel load distribution

Breakdown \ Wheel load, tons	4∿5	5∿6	6∿7	7∿8	8∿9	9∿10	10∿11	Total
Number of dump trucks	0	28	70	80	10	17	0	230
Percentage, %		11.9	29.8	34.0	17.1	7.2	0	100
Breakdown \ Wheel load, tons	4 tons up	5 tons up	6 tons up	7 tons up	8 tons up	9 tons up	10 tons up	Total
Number of dump trucks	0	235	207	137	17	17	0	-
Percentage, %	-	100.0	88.1	58.3	24.3	7.2	0	-

Note: Measured along the Fukiage Section of Route 17 from 13:40 to 15:10. (Public Works Research Institute)

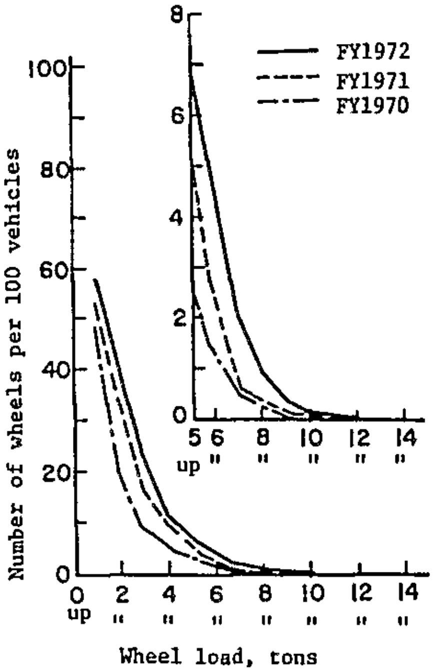

Fig. 3-22 National total of wheel load measurement (39 measuring stations over an aggregate 77 days)

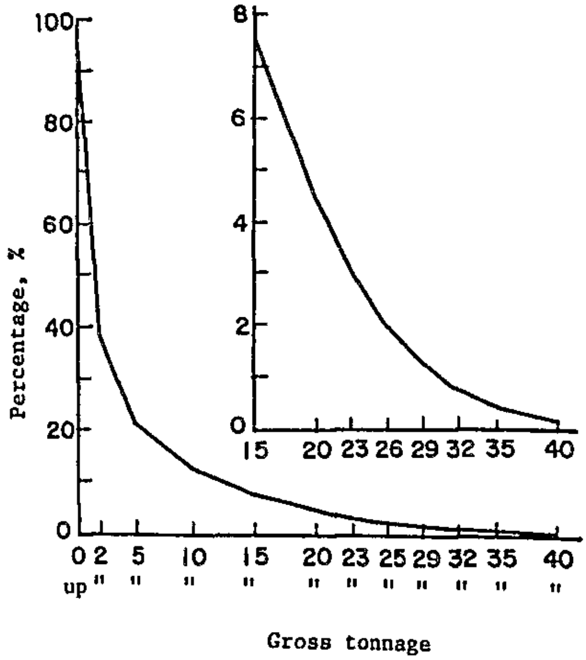

Fig. 3-23 National total of vehicle gross tonnage (measured at 12 measuring station over an aggregate 35 days)

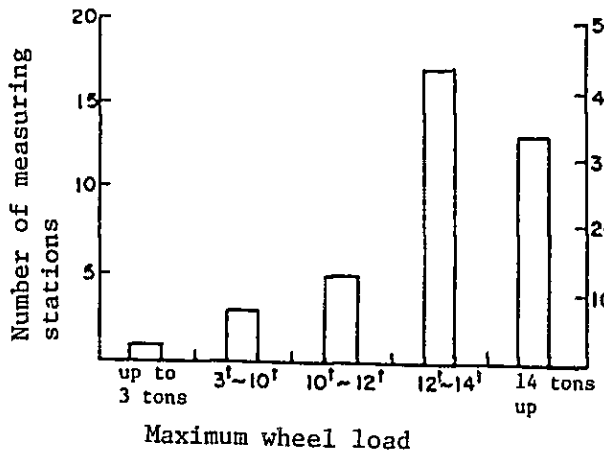

Fig. 3-24 Enumeration of maximum wheel loads (at 39 measuring stations)

Table 3-7 Distribution of mean wheel loads

Classification	Year measured	Sample size: Hokkaido	Tohoku	Kanto	Hokuriku	Chubu	Kinki	Chugoku	Shikoku	Kyushu	Total	Mean value of total traffic volume (vehicles/day direction)	Mean value of large vehicle traffic (vehicles/day direction)	Mean ratio of large vehicles (%)
A traffic	1965-70	0	0	0	0	0	0	0	0	2	2	933	141	15.0
	1966-74	0	0	0	0	0	0	0	0	1	1	1,263	183	14.5
	1975-78	0	0	0	0	0	0	0	0	2	2	2,469	193	7.8
	Mean value	0	0	0	0	0	0	0	0	5	5	1,600	170	10.5
B traffic	1965-70	1	0	0	0	0	0	0	0	23	33	5,068	762	15.0
	1966-74	5	0	0	0	0	13	4	7	10	39	5,052	746	14.8
	1975-78	15	4	0	2	0	6	0	20	15	62	4,904	744	15.2
	Mean value	21	4	0	2	0	28	4	27	48	134	4,986	749	15.0
C traffic	1965-70	10	0	0	0	0	13	0	0	81	103	8,924	1,727	19.4
	1966-74	9	7	2	21	5	47	6	21	37	155	9,550	1,881	19.7
	1975-78	38	6	13	40	18	20	6	17	76	234	8,523	1,959	22.0
	Mean value	57	13	15	61	23	79	12	38	194	492	8,915	1,889	21.2
D traffic	1966-70	0	0	0	0	0	9	0	0	16	25	13,265	3,780	28.5
	1966-74	0	1	4	1	8	8	21	0	11	54	12,771	4,627	36.2
	1975-78	4	0	16	0	29	3	28	0	14	94	12,558	4,980	39.7
	Mean value	4	1	20	1	37	20	49	0	41	173	12,704	4,735	37.3

Classification	Year measured	Number of wheels: 0∿1	1∿2	2∿3	3∿4	4∿5	5∿6	6∿7	7∿8	8∿9	9∿10	10∿12	12∿14	4 tons up	Total
A traffic	1965-70	1,573 (84)	187 (10)	58 (3.1)	23 (1.2)	10 (0.54)	13 (0.67)	2.5 (0.13)	1 (0.054)	0 (0)	0 (0)	0 (0)	0 (0)	0 (0)	1,886 (100%)
	1966-74	2,108 (83)	193 (7.6)	74 (3.0)	59 (2.3)	22 (0.87)	23 (0.91)	15 (0.59)	10 (0.39)	2 (0.079)	10 (0.39)	9 (0.36)	1 (0.039)	0 (0)	2,526 (100%)
	1975-78	4,457 (89)	251 (5.0)	127 (2.5)	63 (1.3)	40 (0.8)	21 (0.42)	21 (0.41)	16 (0.32)	10 (0.20)	5 (0.10)	3.5 (0.070)	0 (0)	0 (0)	5,013 (100%)
	Mean value	2,833 (87)	214 (6.6)	89 (2.7)	46 (1.4)	24 (0.75)	18 (0.55)	12 (0.37)	8.8 (0.27)	4.4 (0.14)	4 (0.12)	3.2 (0.098)	0.2 (0.006)	0 (0)	3,257 (100%)
B traffic	1965-70	8,273 (82)	1,001 (9.9)	401 (4.0)	204 (2.01)	104 (1.03)	69 (0.68)	40 (0.40)	23 (0.23)	17 (0.18)	6.6 (0.065)	4.7 (0.047)	0.78 (0.007)	0.09 (0.009)	10,145 (100%)
	1966-74	8,561 (84)	736 (7.2)	378 (3.7)	228 (2.2)	129 (1.3)	88 (0.86)	52 (0.50)	36 (0.35)	18 (0.17)	8.2 (0.081)	4.9 (0.049)	0.61 (0.006)	0.38 (0 004)	10,239 (100%)
	1975-78	8,305 (83)	757 (7.5)	455 (4.5)	241 (2.4)	119 (1.2)	76 (0.76)	43 (0.43)	23 (0.23)	13 (0.13)	5.9 (0.06)	3.8 (0.038)	0.43 (0.004)	0 (0)	10,043 (100%)
	Mean value	8,371 (83)	810 (8)	420 (4.1)	229 (2.3)	119 (1.2)	79 (0.77)	45 (0.44)	27 (0.27)	15 (0.15)	6.7 (0.067)	4.3 (0.043)	0.57 (0.006)	0.13 (0.001)	10,124 (100%)
C traffic	1965-70	13,940 (78)	1,872 (10)	955 (5.3)	450 (2.5)	246 (1.4)	193 (1.02)	105 (0.59)	55 (0.31)	35 (0.20)	13 (0.073)	7.7 (0.044)	1.5 (0.008)	0.16 (0.009)	17,862 (100%)
	1966-74	15,146 (77)	1,762 (9.0)	1,176 (6.0)	669 (3.4)	357 (1.8)	217 (1.1)	134 (0.68)	77 (0.39)	42 (0.22)	22 (0.11)	16 (0.080)	1.9 (0.01)	0.61 (0.003)	19,622 (100%)
	1975-78	13,187 (74)	1,904 (11)	1,331 (7.5)	657 (3.7)	356 (2.0)	212 (1.2)	113 (0.64)	49 (0.28)	20 (0.11)	8.3 (0.047)	4.5 (0.025)	1.4 (0.008)	0 (0)	17,845 (100%)
	Mean value	13,934 (76)	1,855 (10)	1,210 (6.6)	620 (3.4)	335 (1.8)	208 (1.1)	118 (0.64)	59 (0.32)	30 (0.16)	13 (0.072)	8.5 (0.046)	1.6 (0.008)	0.21 (0.001)	18,391 (100%)
D traffic	1966-70	19,493 (73)	2,967 (11)	1,685 (6.3)	1,049 (3.9)	555 (2.1)	425 (1.6)	327 (1.2)	167 (0.62)	75 (0.28)	25 (0.093)	17 (0.064)	2.8 (0.011)	0.55 (0.002)	26,790 (100%)
	1966-74	17,017 (61)	3,388 (12)	2,846 (10)	2,122 (7.7)	1,015 (3.7)	628 (2.3)	392 (1.4)	191 (0.69)	88 (0.32)	36 (0.13)	15 (0.055)	2.3 (0.008)	0.5 (0.002)	27,740 (100%)
	1975-78	15,824 (57)	3,863 (14)	3,501 (13)	2,070 (7.5)	1,085 (3.9)	657 (2.4)	379 (1.4)	168 (0.61)	87 (0.31)	38 (0.14)	22 (0.080)	1.3 (0.008)	0.04 (0.0001)	27,697 (100%)
	Mean value	16,610 (60)	3,611 (13)	3.091 (11)	1,978 (7.2)	1,005 (3.6)	623 (2.3)	378 (1.4)	176 (0.64)	86 (0.31)	36 (0.13)	19 (0.070)	1.8 (0.007)	0.24 (0.0009)	27,614 (100%)

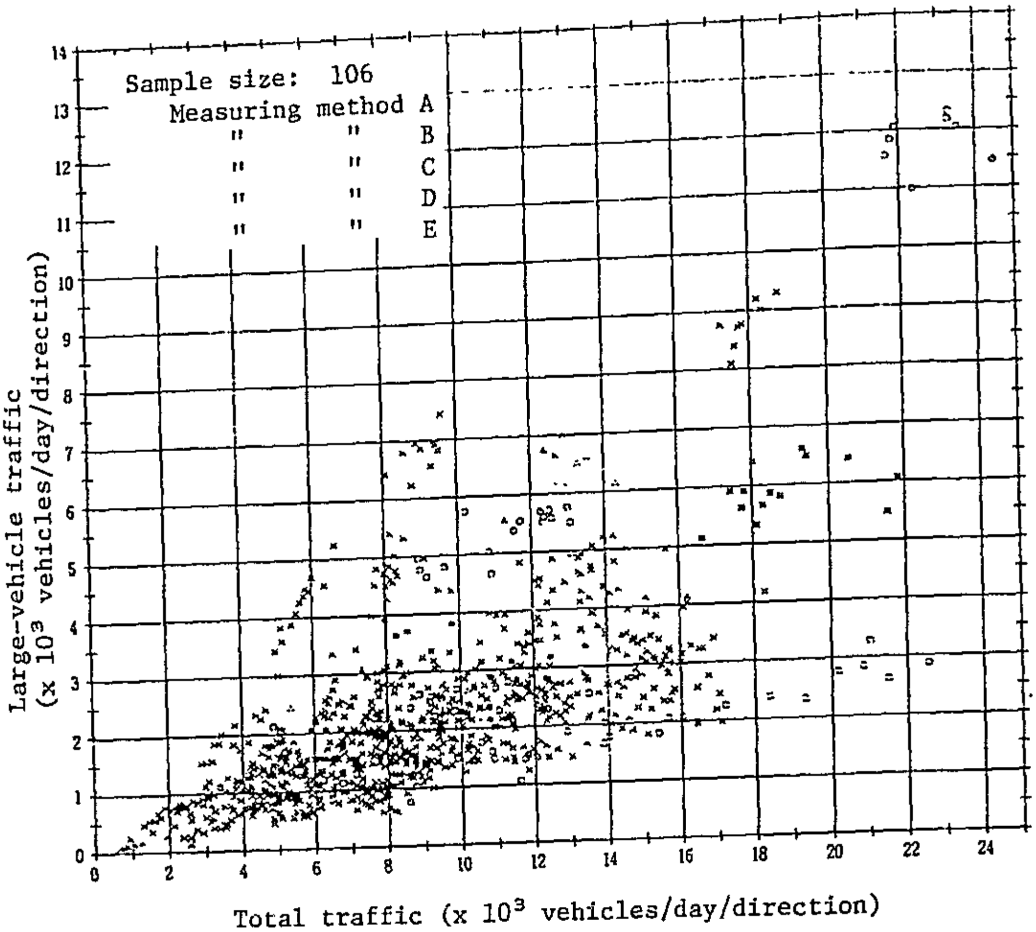

Fig. 3-25 Total traffic vs. large-vehicle traffic

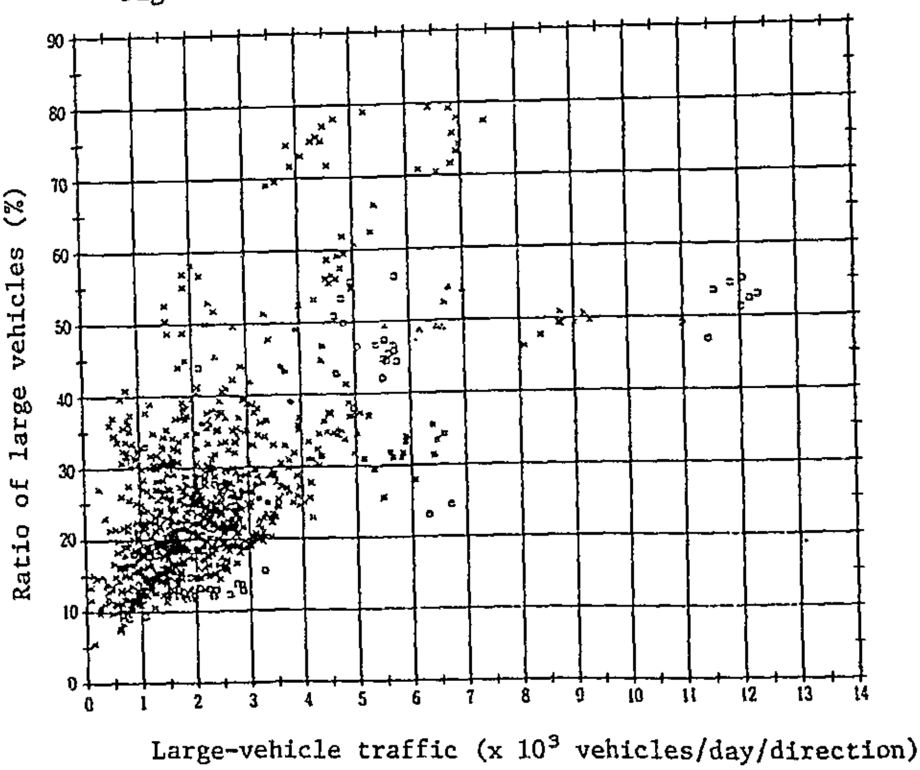

Fig. 3-26 Large-vehicles traffic vs. ratio of large vehicles

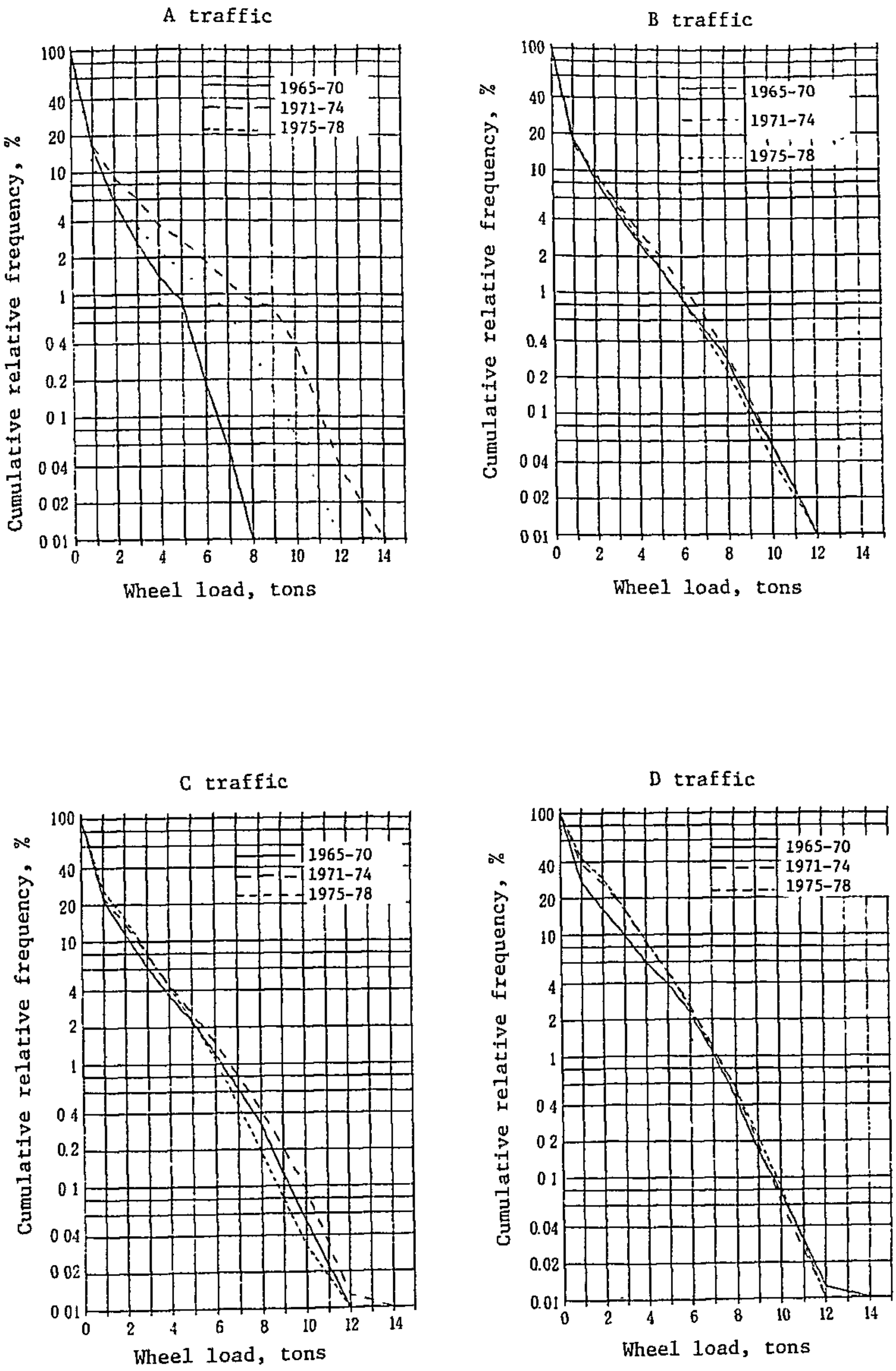

Fig. 3-27 Cumulative relative frequency distribution of wheel loads

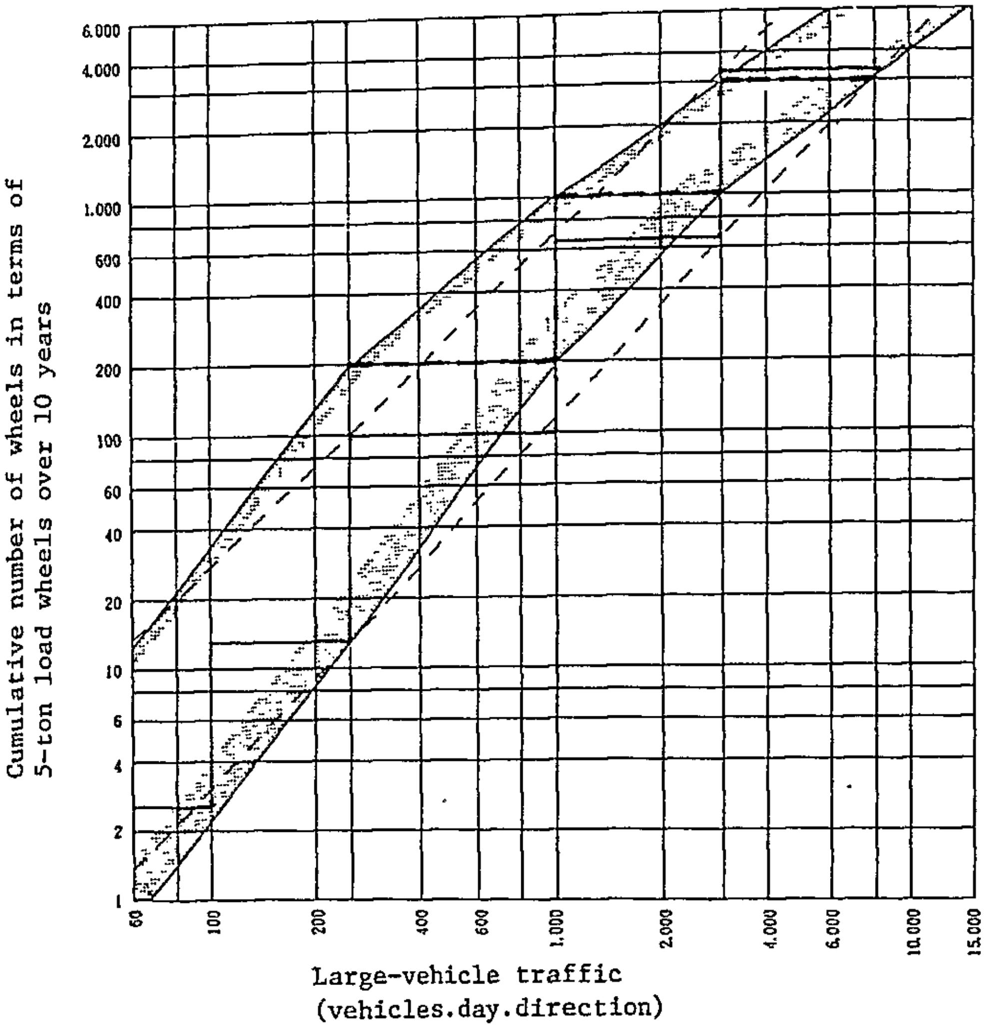

Fig. 3-28 Traffic load for pavement design

Table 3-8 Location of wheel path survey stations, and survey results

Regional Construction Bureau	Location of survey station	Route No.	Number of lanes	Lane width, m	Mean value of wheel path (a) as measured from the righthand end of lane, m		Standard deviation of a, %		(Mean value of a)/(Lane, width), %		(Standard deviation of a)/(Lane width), %	
Hokkaido Development Bureau	Furano-cho, Furano	38	2	3.3	2.50		27.1		75.8		8.20	
	Nohoro, Ebetsu	12	4	3.6		2.59		47.8		72.0		13.3
Tohoku Regional Construction Bureau	Sanno 3-chome, Akita	7	2	4.2	3.32		46.6		79.1		11.1	
	Atago-cho, Morioka	4	4	3.25		2.26		39.3		69.6		12.1
	Koriyama, Sendai	4	4	3.25		2.23		38.5		68.6		11.8
Kanto Regional Construction Bureau	Fukiage-cho, Ohsato-gun, Saitama	17	2	4.5	2.97		31.8		66.1		7.06	
	Ohsenba, Kawagoe	16	4	3.25		2.62		28.8		80.6		8.87
Hokuriku Regional Construction Bureau	Shibatake, Niigata	7	2	3.3	2.61		23.1		79.1		7.00	
	Dankuro, Sekiya, Niigata	8	4	3.6		2.47		12.7		68.7		11.8
Chubu Regional Construction Bureau	Kanaguchi, Kasugai	19	2	3.8	2.75		35.1		72.3		9.23	
	Kohari Shinden, Komaki	41	4	3.5		2.32		44.1		66.3		12.6
Kinki Regional Construction Bureau	Ryugaoka, Ohtsu	1	2	4.2	2.89		41.1		68.8		9.80	
	Nakajima, Fushimi-ku, Kyoto	1	4	3.6		2.40		48.1		66.7		13.4
Chugoku Regional Construction Bureau	Kushiro, Hamada	9	2	3.3	2.49		44.1		75.5		13.4	
	Daimon-cho, Fukuyama	2	4	3.25		2.46		41.8		75.7		12.9
Shikoku Regional Construction Bureau	Fukuonji, Matsuyama	11	2	3.6	2.47		27.6		68.6		76.7	
	Yashima Nishi-machi, Takamatsu	11	4	3.25		2.16		46.8		66.5		14.4
Kyushu Regional Construction Bureau	Hinode-cho, Hayami-gun, Ohita	10	2	3.75	2.72		29.1		72.6		7.75	
	Ujuku-machi, Kagoshima	225	4	3.25		1.90		61.8		58.5		19.0
Mean value	Two-lane road						34.0		73.1		9.02	
	Four-lane road							44.0		69.3		13.02

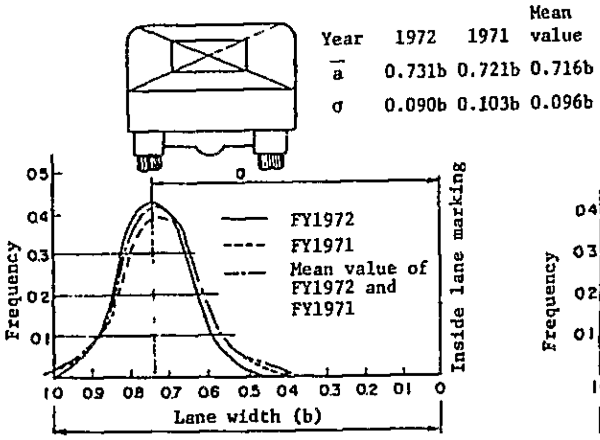

Fig. 3-29 Distribution of value (a) in the two-lane load

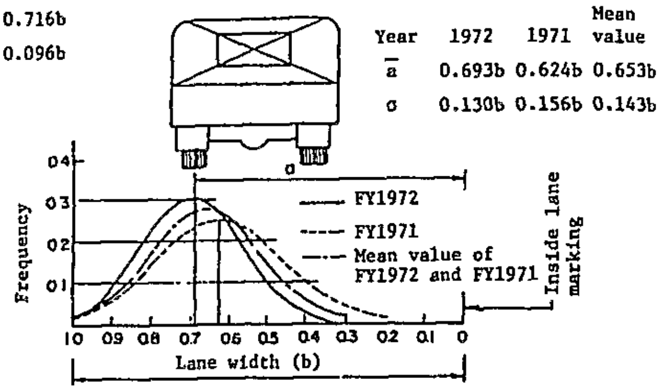

Fig. 3-30 Distribution of value (a) in the four-lane road

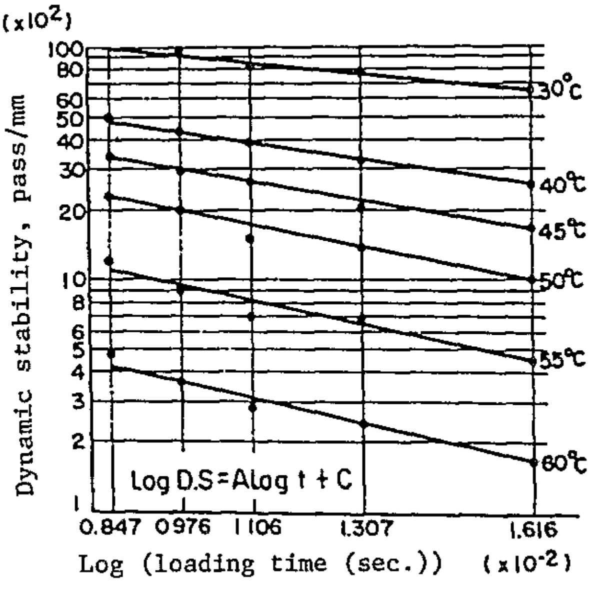

Fig. 3-31 Dynamic stability vs. loading time (log D.S.-log t) by Dr. Sugawara

Table 3-9 Specifications for crushed stone

Class	Specific gravity	Moisture absorption, %	Wear loss, %
1	more than 2.45	less than 3.0	less than 3.5
2			less than 40

Table 3-10 Tolerance of contaminants

(wt. %)

Classification	Maximum limit
Clay lump	0.25
Soft stone	5.0
	1.0*
	1.0**

* The maximum value may be 1.5% if what is lost from crushed stone by washing is crushed stone dust.

** Not applicable to blast furnance slag.

Table 3-11 Durability limits

Application	Surface course, base course	Upper subbase course
Loss, %	12	20
Note: Repeat five times the sodium sulfate method specified in JIS A 1122.		

Table 3-12 An example of contaminants

Contaminant	Content (wt.% to total sample)
Clay, clay lump	0.25
Flaky stone[1)]	5.0
Slender or flat stone[2)]	10.0

1) Testing in accordance with JIS A 1126.

2) Grain, the ratio of the major to minor length of the smallest rectangular parallelopiped enclosing which is larger than 5.

Table 3-13 Grading of crushed stone (JIS A 5001 - 1973)

Nominal size		Nominal size of sieve, mm / Grade range, mm	Gravimetric percentage of undersize, %													
			100	80	60	50	40	30	25	20	13	5	2.5	1.2	0.4	0.074
One-sized crushed stone	S-80 (Grade 1)	80∿60	100	85∿100	0∿15											
	S-60 (Grade 2)	60∿40		100	85∿100	-	0∿15									
	S-40 (Grade 3)	40∿30				100	85∿100	0∿15								
	S-30 (Grade 4)	30∿20					100	85∿100	-	0∿15						
	S-20 (Grade 5)	20∿13							100	85∿100	0∿15					
	S-13 (Grade 6)	13∿5								100	85∿100	0∿15				
	S-5 (Grade 7)	5∿2.5									100	85∿100	0∿25	0∿5		
Graded crushed stone	M-40	40∿0				100	95∿100	-	-	60∿90	-	30∿65	20∿50	-	10∿30	2∿10
	M-30	30∿0					100	95∿100		60∿90		30∿65	20∿50	-	10∿30	2∿10
	M-25	25∿0						100	95∿100	-	55∿85	30∿65	20∿50	-	10∿30	2∿10
Crusher-run	C-40	40∿0				100	95∿100	-	-	50∿80	-	15∿40	5∿25			
	C-30	30 0					100	95∿100	-	55∿85	-	15∿45	5∿30			
	C-20	20∿0							100	95∿100	60∿90	20∿50	10∿35			

Note: These sieves correspond to the standard sieves, 101.6 mm, 76.2 mm, 63.5 mm, 50.8 mm, 38.1 mm, 31.7 mm, 25.4 mm, 19.1 mm, 12.7 mm, 4,760 μ, 2,380 μ, 1,190 μ, 420 μ and 74 μ, specified in JIS Z 8801 "STANDARD SIEVES."

Table 3-14 Specifications for stone dust

Mesh, mm	Gravimetric percentage of undersize, %
0.5	100
0.15	90 ∿ 100
0.074	70 ∿ 100

Table 3-15 Specifications for stone dust from igneous rock

Item	Specification
PI	up to 6
Thermal denaturalization	not permitted
Immersion swelling	up to 50%
Separation test	up to grade

Notes (1) Heat up to 200°C, and check the appearance.

(2) Add water to the stone dust and make paste. Determine that gravimetric ratio of water to stone dust at which the paste shows a flow of 200 mm on the cement mortar flow table at the 15th drops.

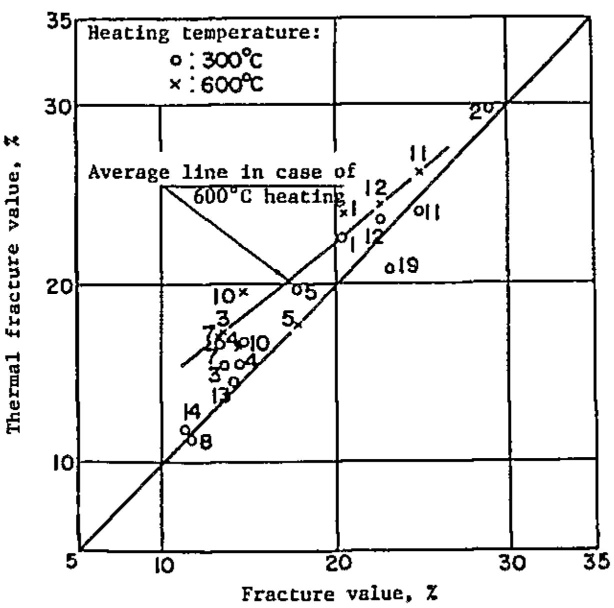

Fig. 3-32 BS fracture value vs. termal fracture value

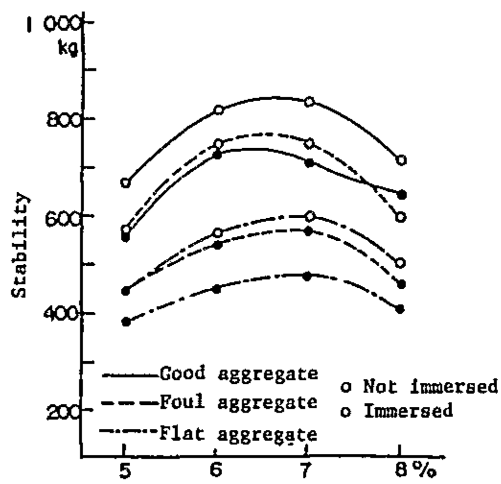

Fig. 3-33 Aggregate state and Marshall stability

Table 3-16 Grain shapes and tolerance

Standard	Specification for grain shapes	Tolerance, %	Remarks
Asphalt Pavement Design Manual (Japan Road Association)	Grain called flat or slender if ratio of the major to minor sides of the smallest rectangular parallelopiped enclosing it is larger than 5.	up to 10	Applied to the asphalt mixtures for surface course. Applied to 5 mm sieve oversizes.
Japan Highway Public Corporation's Common Specifications (Japan Highway Public Corporation)	Grain called slender if the ratio of its length to width exceeds 3. Grain called flat if the ratio of its width to thickness exceeds 3.	up to 25	Applied to 5 mm sieve oversizes.

Table 3-17 Types of mixtures, and selection standards

		Ordinary districts	Snowy districts
Base course		① Coarse grade asphalt concrete (20)	
Surface course		② Dense grade asphalt concrete (20, 13) ③ Fine grade asphalt concrete (13) ④ Dense grade gap asphalt concrete (13)	⑤ Dense grade asphalt concrete (20F, 13F) ⑥ Fine grade gap asphalt concrete (13F) ⑦ Fine grade asphalt concrete (13F) ⑧ Dense grade gap asphalt concrete (13F)
Wearing course	Anti-wear use		⑥ Fine grade gap asphalt concrete (13F) ⑦ Fine grade asphalt concrete (13F)
	Non-skid use	⑨ Open grade asphalt concrete (13)	

Note 1: The mixtures 4 and 8 are used for the surface course for a non-skid effect.

Note 2: The mixtures 6 and 7 are also used for a wearing course for improved anti-wear effect.

Note 3: The number in the circle denotes the identification number. The number in parentheses denotes the maximum grain size. The symbol F means that the filler is used much.

Note 4: The general properties of the mixtures are as follows.

① Coarse grade asphalt concrete (20) usually is used for the base course.

② The dense grade asphalt concrete (20, 13) is excellent in flow resistance and skid resistance. The mixture with a maximum grain size of 20 mm is particularly excellent in flow resistance.

③ Fine grade asphalt concrete (13) is excellent in durability.

④ Dense grade gap asphalt concrete (13) is excellent in durability and skid resitance.

⑤ Dense grade asphalt concrete (20F, 13F) are excellent in wear resistance. The mixture with a maximum grain size of 20 mm is excellent in flow resistance as well.

⑥ Fine grade gap asphalt concrete (13F) is excellent in wear resistance and durability.

⑦ Fine grade asphalt concrete (13) is particularly excellent in wear resistance and durability, but is lacking in flow resistance.

⑧ Dense grade gap asphalt concrete (13F) is excellent in skid resistance and wear resistance.

⑨ Open grade asphalt concrete (13) is excellent in skid resistance, but is lacking in durability.

Table 3-18 Standard mix proportions of mixtures

Type of mixture		① Coarse grade asphalt concrete (20)	② Dense grade asphalt concrete (20)	(13)	③ Fine grade asphalt concrete (13)	④ Dense grade gap asphalt concrete (13)	⑤ Dense grade asphalt concrete (20F)	(13F)	⑥ Fine grade gap asphalt concrete (13F)	⑦ Fine grade asphalt concrete (13F)	⑧ Dense grade gap asphalt concrete (13F)	⑨ Open grade asphalt concrete (13)
Finish thickness, cm		4~6	3~5		3~5	3~5	3~5		3~5	3~4	3~5	3~4
Maximum grain size, mm		20	20	13	13	13	20		13	13	13	13
Gravimentric percentage of undersizes, %	25 mm	100	100				100		100	100	100	100
	20	95~100	95~100	100	100	100	95~100					
	13	70~90	75~90	95~100	95~100	95~100	75~95	95~100	95~100	95~100	95~100	95~100
	5	35~55	45~65	55~70	65~80	35~55	52~72		60~80	75~90	45~65	23~45
	2.5	20~35	35~50		50~65	30~45	40~60		45~65	65~80	30~45	15~30
	0.6	11~23	18~30		25~40	20~40	25~45		40~60	40~65	25~40	8~20
	0.3	5~16	10~21		12~27	15~30	16~33		20~45	20~45	20~40	4~15
	0.15	4~12	5~16		8~20	5~15	8~21		10~25	15~30	10~25	4~10
	0.074	2~7	4~8		4~10	4~10	6~11		8~13	8~15	8~12	2~7
Asphalt quantity, %		4.5~6	5~7		6~8	4.5~6.5	6~8		6~8	7.5~9.5	5.5~7.5	3.5~5.5
Asphalt penetration		40~60 60~80 80~100										

Table 3-19 Specification for petroleum asphalts
(Japan Road Association) (1961)

Classification	Penetration (25°C, 100g 5 sec.)	Softening point, °C	Ductility			Evaporation, %	Postevaporation penetration (as against before-evaporation penetration), %	Carbon tetrachloride solubles, %	Flash point, °C
			Type A	Type B	Type B				
40～ 60	40～ 60	more than 40	25°C more than 100 15°C more than 100 10°C more than 100	25°C more than 100 15°C more than 100 10°C more than 30	25°C more than 70 15°C more than 70 10°C more than 5	less than 0.4	more than 70	more than 9.5	more than 230
60～ 80 80～100	60～ 80 80～100		25°C more than 100 15°C more than 100 10°C more than 100	25°C more than 100 15°C more than 100 10°C more than 100	25°C more than 70 15°C more than 70 10°C more than 10				
100～120 120～150	100～120 120～150	more than 35	10°C more than 100 5°C more than 100	10°C more than 100 5°C more than 50	10°C more than 70 5°C more than 5				more than 210
150～200	150～200		10°C more than 100	10°C more than 100	Type C not present in these classes				
200～300	200～300	more than 30	5°C more than 100	5°C more than 50					more than 200

Remarks: Type A usually is produced from asphalt-base crude oil. Just as was available in the pre-war days, to solid paraffin content is about 1% or less. The emulsification is well. The ductility is more than 100 even at low temperatures, but plunges to zero when the temperatures falls a bit from a critical point. The thermal sensitivity is high.

Type B generally is produced from mixed base crude oil. The solid paraffin content is about 3%. The emulsification is hard. The change in ductility and the thermal sensitivity fall between Types A and C.

Type C is produced from mixed base crude oil containing a comparatively high content of paraffin. Usually subjected to a slight aeration during production. Usually contains more solid paraffin than Type B. Very difficult to emulsify. The ductility is less than 100 at around 15°C, but levels off to a certain value above 0 even with decline in temperature.

The solid paraffin content referred to above is measured according to Marcusson method.

Table 3-20 Specifications for paving grade petroleum asphalt

(Japan Road Association) (1967)

<table>
<tr><th>Classification</th><th>Penetration (25°C, 100g, 5 sec.)</th><th>Softening point, °C</th><th>Ductility (15°C)</th><th>Evaporation, %</th><th>Post-evaporation penetration index (as against before-evaporation index), %</th><th>Carbon tetrachloride solubles, %</th><th>Flash point, °C</th></tr>
<tr><td>60 ∿ 80</td><td>60 ∿ 80</td><td>43.0∿53.0</td><td rowspan="4">more than 100</td><td rowspan="2">less than 0.3</td><td rowspan="3">more than 75*</td><td rowspan="4">more than 99.5</td><td rowspan="2">more than 240</td></tr>
<tr><td>80 ∿ 100</td><td>80 ∿ 100</td><td>41.0∿51.0</td></tr>
<tr><td>100 ∿ 120</td><td>100 ∿ 120</td><td>40.0∿50.0</td><td rowspan="2">less than 0.5</td><td rowspan="2">more than 210</td></tr>
<tr><td>120 ∿ 150</td><td>120 ∿ 150</td><td>38.0∿48.0</td><td>more than 70</td></tr>
</table>

* If the softening point is above 17.5°C, the post-evaporation penetration should preferable be more than 80%.

Remarks: The specific gravity, viscosity and temperature should preferably be commented upon. The measurement and the viscosity-temperature relationship should be made in accordance with the methods agreed upon between the supplier and purchaser.

Table 3-21 Specifications for paving grade petroleum asphalt (1975)

The paving grade petroleum asphalt shall be homogenous and free of water, shall not foam when heated up to 180°C, and shall meet the following requirements.

Classification	50 ∿ 80	80 ∿ 100	100 ∿ 120	120 ∿ 150
Penetration (25°C, 100g, 5 sec.)	60 to 80	80 to 100	100 to 120	120 to 150
Softening point, °C	44.0 ∿ 52.0	42.0 ∿ 50.0	40.0 ∿ 50.0	38.0 ∿ 48.0
Ductility (15°C) cm	more than 100	more than 100	more than 100	more than 100
Post-evaporation penetration, %	more than 80	more than 80	more than 75 (3)	more than 70
Post-evaporation penetration ratio, (as against pre-evaporation penetration), % (2)	less than 110	less than 110		-
Weight loss of this film oven test, % (1)	less than 0.6	less than 0.6	-	-
Penetration ratio after thin film oven test, %	more than 55	more than 50	-	-
Carbon tetrachloride solubles, %	more than 99.5	more than 99.5	more than 99.5	more than 99.5
Flash point, °C	higher than 260	higher than 260	higher than 210	higher than 210
Specific gravity (25/25°C)	more than 1.000	more than 1.000	-	-

Notes: ① The loss here may happen to be an increase.

② Post-evaporation penetration ratio

= (Post-evaporation penetration (sample just after evaporation test) ÷ (pre-evaporation penetration (according to JIS K 2207) × 100 (%)

③ If the softening point in higher than 47.5°C, the post-evaporation penetration should preferably be more than 80%. As regards the classes 60 to 80 and 80 to 100, the kinematic viscosity measured at 120°C, 140°C, 160°C and 180°C shall be expresses in the CGS system. It the viscosity is measured by a non-CGS system instrument, the type of the instrument and the conversion formula shall be stated.
As regards the classes 100 to 120 and 120 to 150, the specific gravity and viscosity vs. temperature relationship should preferably be stated. The specific gravity and the viscosity vs. temperature relationship shall be measured in accordance with the methods agreed upon between the vendor concerned.

Note 1: In the Japan Road Association's Specifications, the asphalts coming under classes 40 to 60, 150 to 200 and 200 to 300 are not specified because they are rarely used. For these, refer to JIS K 2207.

Note 2: As regards the classes 100 to 120 and 120 to 150, if the softening point is higher than 46.0lC, the asphalt may be denaturalized seriously at the time of mixing or may be turned into a mixture defying compaction. Thus, the temperature control and rolling work should be carried out with utmost care.

Note 3: The stripping resistance varies largely depending on the combination of asphalt and aggregate. Prior to purchase, therefore, the stripping tests should be carried out. The materials testing methods used include the static stripping test (see Appendix) for asphalt and the immersion Marshall test for mixtures.

Table 3-22 Specifications for paving grade petroleum asphalt (JIS K 2207 - 1980)

Item \ Classification	40 ∿ 60	60 ∿ 80	80 ∿ 100	100 ∿ 120
Petetration (25°C), 1/10 mm	40 to 60	60 to 80	80 to 100	100 to 120
Softening point, °C	47.0 ∿ 55.0	44.0 ∿ 52.0	42.0 ∿ 50.0	40.0 ∿ 50.0
Ductility (15°C), cm	more than 10	more than 100	more than 100	more than 100
Trichloro-ethane solubles, %	more than 99.0	more than 99.0	more than 99.0	more than 99.0
Flash point, °C	higher than 260	higher than 260	higher than 260	higher than 260
Change in mass of thin film oven test, %	less than 0.6	less than 0.6	less than 0.6	-
Change in penetration of thin film oven test, %	more than 58	more than 55	more than 50	-
Change in mass by evaporation, %	-	-	-	less than 0.5
Ratio of post-evaporation to pre-evaporation penetration, %	less than 100	less than 100	less than 100	-
Specific gravity (25/25°C)	more than 1.000	more than 1.000	more than 1.000	more than 1.000
Remarks: As regards the straight asphalt classes 40 to 60, 60 to 80 and 80 and 100, a test report of kinematic viscosity measured at 120°C, 140°C, 160°C and 180°C shall be furnished.				

Note 1: As regards the class 100 to 120, the viscosity vs. temperature relationship should preferably be added to the test report. The viscosity vs. temperature relationship should be measured according to the method agreed upon between the vendor and vendee concerned.

Note 2: For the classes 120 to 150, 150 to 200, and 200 to 300, refer to JIS K 2207.

Note 3: Usually, the classes 60 to 80 and 80 to 100 are employed. The class 40 to 60 may be applied where the traffic is heavy and the temperature is high. The class 100 to 120 may be used for penetration macadam.

Note 4: The table above is a classification of asphalt by penetration. Asphalt is also classified by 60°C viscosity. For details, refer to the Asphalt Pavement Design Manual.

Note 5: The stripping resistance varies largely depending on the combination of asphalt and aggregate, and other factors. Thus, a stripping resistance test should preferably be carried out prior to purchase. There are practiced materials testing methods such as asphalt film stripping test, and immersion Marshall test and immersion wheel tracking test for mixture, etc. For details, refer to the Asphalt Pavement Design Manual.

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