運輸省港湾技術研究所

(25th Anniversary Issue)

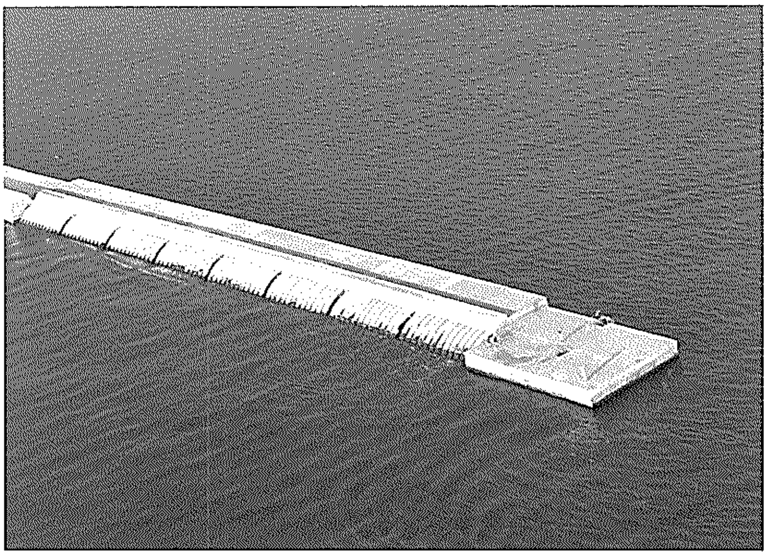
港湾技術研究所報告

REPORT OF THE PORT AND HARBOUR RESEARCH INSTITUTE

MINISTRY OF TRANSPORT

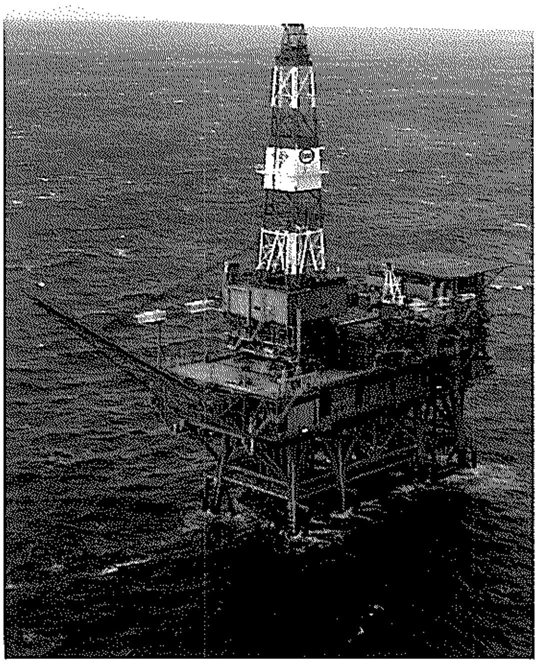
VOL. 26 NO. 5 DEC. 1987

NAGASE, YOKOSUKA, JAPAN



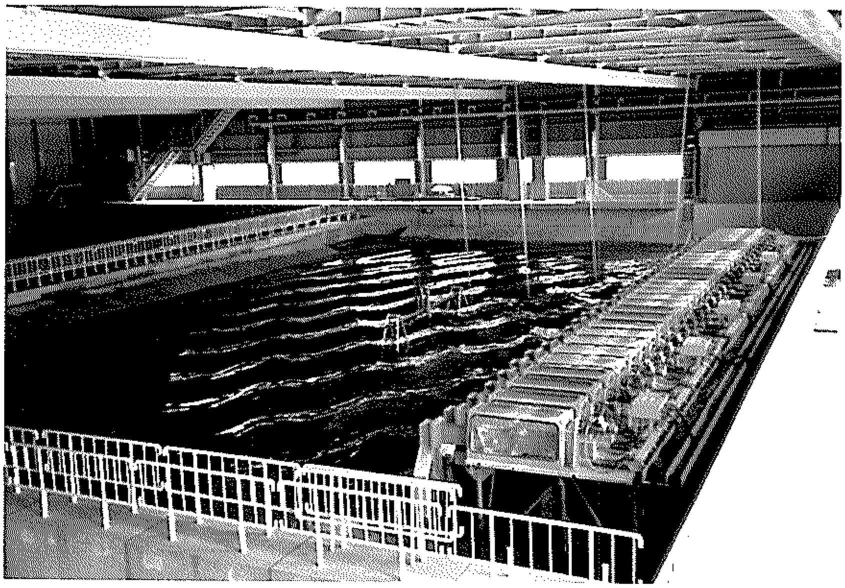
Curved Slit Caisson Breakwater

View of curved slit caisson breakwater completed in the construction at the port of Funakawa. (Courtesy of Akita Port Construction Office, the First District Port Construction Bureau, Ministry of Transport)



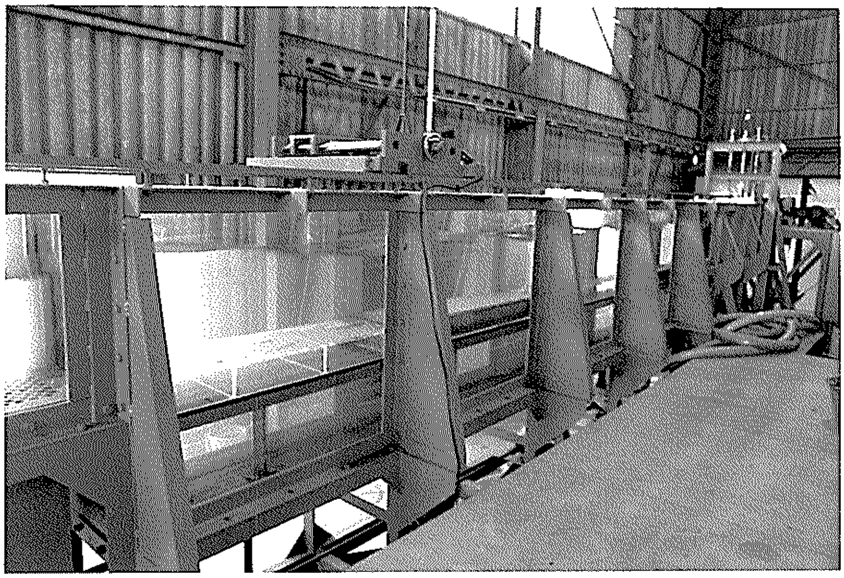
Facilities for Ocean Directional Wave Measurement

Four step type wave gauges and a two-axis directional current meter with a pressure sensor are installed on the legs of an offshore oil rig. They are operated simultaneously for detailed directional wave analysis.



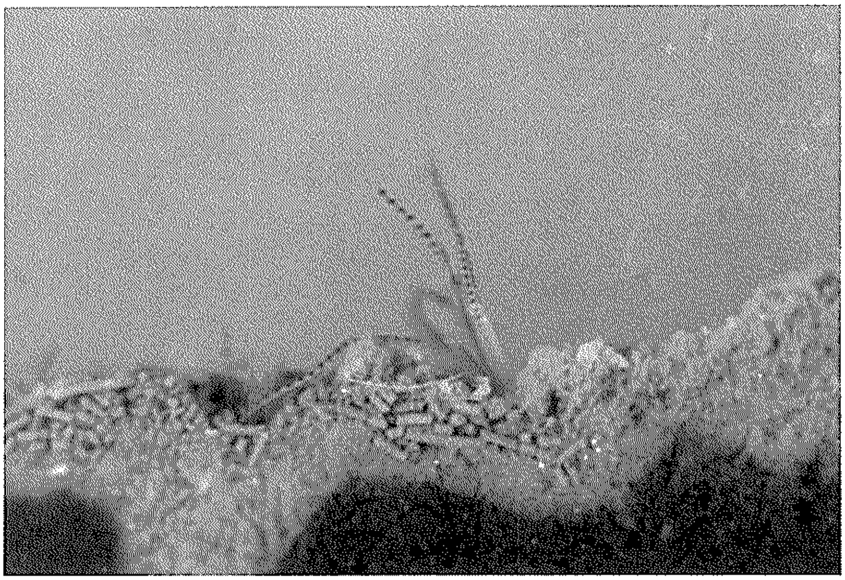
Serpent-type Wave Generator

The photograph shows the serpent-type wave generator in the short-crested wave basin and the superimposition state of two different oblique waves generated by the generator.



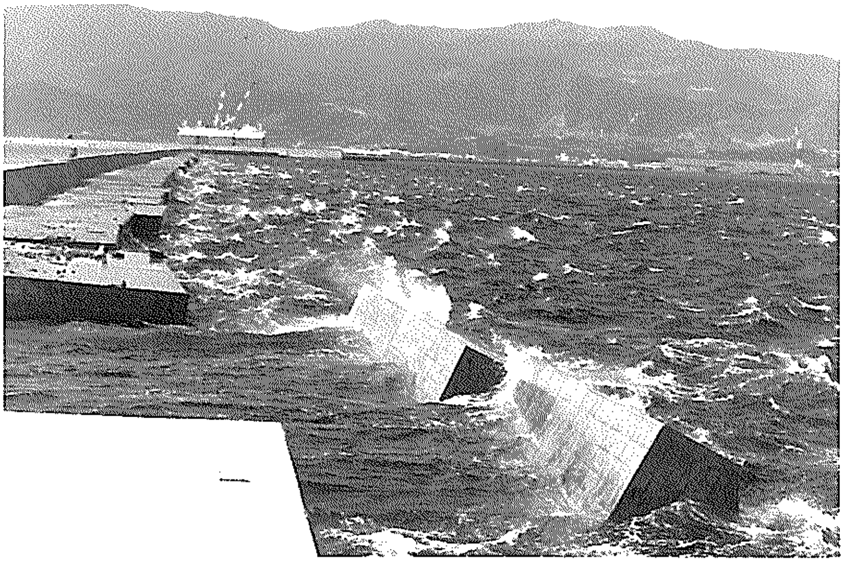
Wave-soil Tank

The experiments concerning the wave-soil interactions are conducted in this tank. The soil tank and the test section are located at the center of the tank. A movable floor is provided at the bottom of the test section and the level of the interface of mud layer and water can easily be adjusted to the level of the flume bottom.



Pararionospio Pinnata

The biomass of benthos is one of the most sensitive indices to know the effect of sea-bed sediment treatments on the marine environmental improvement. The picure shows a kind of benthos, *pararionospio pinnata,* which preferentially exists in the polluted sea-bed.



Breakwater Damaged by Storm

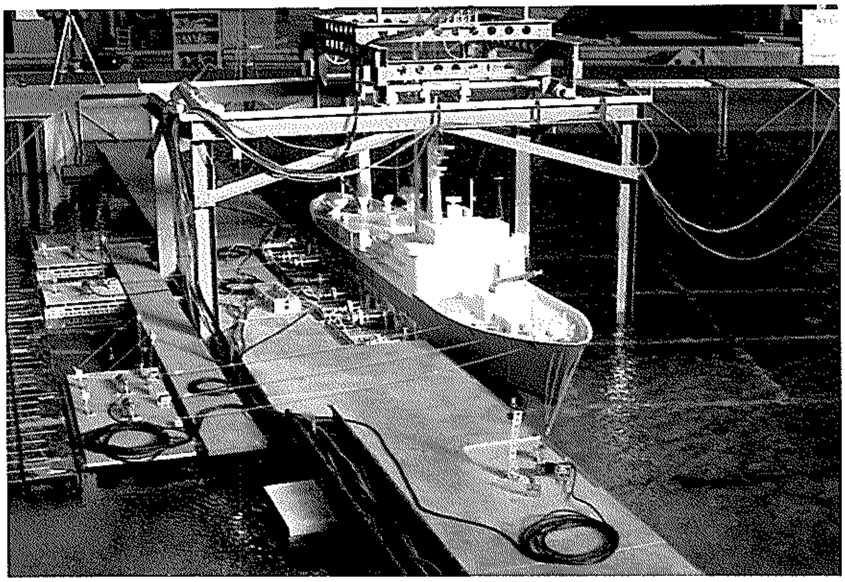
This photograph shows a breakwater damage by a storm.
The breakwater is of the composite type with concrete caisson on a rubble mound. Two caissons were severly damaged due to the instability of a rubble mound.

Nondestructive Evaluation of Pavement

Nondestructive methods for evaluating the load carrying capacity of airport concrete pavements have been developed by using Falling Weight Deflectometer(FWD).

Seismic Damage to Gravity Quaywall

The 1983 Nipponkai-Chubu earthquake(Magnitude : 7.7)caused serious damage to port facilities in northern part of Japan. This photo shows the damage to gravity quaywall. The concrete cellular block walls were collapsed and completely submerged.

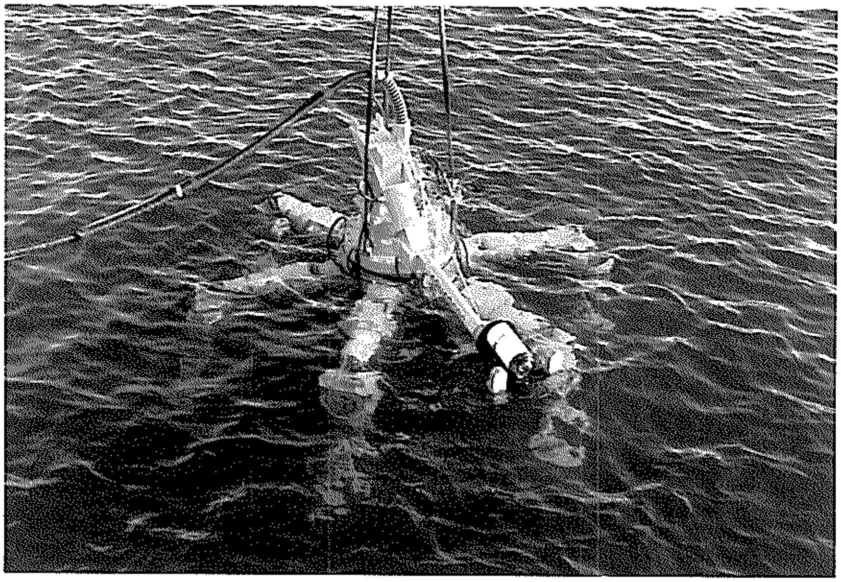


Model Experiment of Mooring Ship

Model ship is moored at a quay wall with fenders and mooring ropes subjected to gusty wind and/or irregular waves.

Vessel Congestion in Japan

As Japan is surrounded by the sea, there are many crowded water areas with various sizes and types of vessels. Arround there, many construction works were planned such as ports and harbours, off-shore airports, huge bridges and so on, so that many marine traffic observations and marine traffic simulations have been carried out.



Underwater Inspection Robot

This is the six-legged articulated underwater inspection robot named "AQUAROBOT". The robot controlled by a computer can walk on uneven sea bed without making water muddy.

Foreword

The Port and Harbour Research Institute is a national laboratory under the Ministry of Transport, Japan. It is responsible for solving various engineering problems related to port and harbour projects so that governmental agencies in charge of port development can execute the projects smoothly and rationally. Its research activities also cover the studies on civil engineering facilities of air ports.

Last April we have celebrated the 25th anniversary of our institute because the present organization was established in 1962, though systematic research works on ports and harbours under the Ministry of Transport began in 1946. As an event for the celebration, we decided to publish a special edition of the Report of the Port and Harbour Research Institute, which contains full English papers only. These papers are so selected to introduce the versatility of our activities and engineering practices in Japan to overseas engineers and scientists. It is also intended to remedy to a certain extent the information gap between overseas colleagues and us.

The reader will find that our research fields cover physical oceanography, coastal and ocean engineering, geotechnical engineering, earthquake engineering, materials engineering, dredging technology and mechanical engineering, planning and systems analysis, and structural analysis. Such an expansion of the scope of research fields has been inevitable, because we are trying to cover every aspect of technical problems of ports and harbours as an integrated body.

The present volume contains eleven papers representing six research divisions of the institute. The materials introduced in these papers are not necessarily original in strict sense, as some parts have been published in Japanese in the Reports or the Technical Notes of the Port and Harbour Research Institute. Nevertheless they are all original papers in English and are given the full format accordingly. We expect that they will be referred to as usual where they deserve so.

It is my sincere wish that this special edition of the Report of the Port and Harbour Research Institute will bring overseas engineers and scientists more acquainted with our research activities and enhance the mutual cooperation for technology development related to ports and harbours.

December 1987
Yoshimi Goda
Director General

港湾技術研究所報告 (REPORT OF P. H. R. I.)

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REPORT OF THE PORT AND
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7. Development of New Evaluation Methods and New Design Methods of Rehabilitation Works for Airport Pavements

Katsuhisa SATO*
Yoshitaka HACHIYA**

Synopsis

When airport pavements are damaged by traffic and environmental effects so that their serviceability or load carrying capacity is degraded, we need to consider maintenance or rehabilitation works for them. In Japan, these maintenance or rehabilitation works used to be conducted on the basis of the subjective judgements of the airport administrators. However, recently, we have developed several new methods to make decisions regarding the necessity for airport pavement maintenance or rehabilitation works on the basis of a quantitative evaluation of the surface conditions and the load carrying capacity.

For this, Pavement Rehabilitation Index (PRI) equations have been developed to correlate the surface conditions with the opinions of pavement engineers and each of the criteria of the PRI for the necessity of rehabilitation works for runways, taxiways and aprons has been decided.

Non-destructive methods for evaluating the load carrying capacity of airport pavements have been developed to take the place of the conventional destructive evaluation methods. Dynaflect deflections are used for asphalt pavements and Falling Weight Deflectometer (FWD) deflections for concrete pavements.

Many studies have been made for design methods for the maintenance or rehabilitation works. Some new overlay thickness design methods have been proposed. One is a thickness design method for an asphalt overlay on an existing asphalt pavement using Dynaflect deflections and another is a thickness design method for an asphalt overlay on an existing concrete pavement using three-layer elastic theory. New bonding techniques have also been developed for bonded concrete overlays and a prestressed concrete precast slab pavement has been developed to replace normal concrete pavements in which concrete slabs are placed in-situ.

Finally, a new maintenance and rehabilitation system has been proposed.

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港湾技術研究所報告
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7. 空港舗装の新しい評価および補修方法の開発

佐藤勝久*・八谷好高**

要　旨

空港舗装が荷重作用や環境作用により破損し，供用性が低下したり荷重支持力が不十分となると，何等かの維持・補修が実施される。

従来この維持・補修は，空港の管理者の主観的判断により行われてきたが，今回路面性状や舗装の荷重支持力等に基づき客観的に維持・補修の必要性を判断して実施する方法を開発した。

まず舗装の路面性状の実測値を用いて舗装の供用性を表わす評価式を作成し，この式より求まる指数 PRI (Pavement Rehabilitation Index) に関し，滑走路，誘導路，エプロンのそれぞれに対し維持・補修の必要性の基準値を求めた。

次に舗装構造の荷重支持力の評価に関し，従来から行われている解体調査に基づく方法に代わるものとして，アスファルト舗装に対し，ダイナフレクトを用いた非破壊評価法を，コンクリート舗装に対し，フォーリングウエイトデフレクトメーターを用いた非破壊評価法を提案した。

一方維持・補修工法の設計についても多くの研究がなされ，ダイナフレクトの評価に基づくアスファルト舗装上のアスファルトコンクリートによるオーバーレイ厚の算定法，多層弾性理論に基づくコンクリート舗装上のアスファルトコンクリートによるオーバーレイ厚の算定法，コンクリート舗装上の付着コンクリートオーバーレイのための新しい付着工法，工場製作のプレキャスト版による舗装工法等を開発した。

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**土質部　主任研究官（舗装補修担当）

Contents

1. Introduction

In airport pavements, maintenance and rehabilitation works keep the proper functions of the pavements after the pavements are opened to traffic, and also extend their service life. Maintenance and rehabilitation strategies are therefore quite important. However, the maintenance or rehabilitation works of airport pavements in Japan have conventionally been carried out only on the basis of the empirical and subjective judgement of airport administrators, and the execution has not necessarily been systematic. The development of a systematic method for maintenance and rehabilitation works has been eagerly awaited in order to make the best possible use of available funds.

We have carried out various studies concerning airport pavements, including those for the surface condition evaluation, structural evaluation, prediction of distress and performance, and maintenance and rehabilitation strategies.

Through these studies, we have developed several new methods for the functional and structural evaluation of airport pavements, and for the design of rehabilitation works. The Pavement Rehabilitation Index (PRI) has been introduced for the evaluation of the pavement surface condition and PRI criteria for the judgement of necessity of rehabilitation works to runway, taxiway and apron pavements have been decided. Non-destructive evaluation methods for load carrying capacity of airport pavements have also been proposed instead of conventional destructive evaluation procedures. Moreover, some new overlay thickness design methods have also been proposed and a new design method of a prestressed concrete precast slab pavement has been developed.

On the basis of such new information, we have proposed a new maintenance and rehabilitation system for airport pavements and main results of this series of studies are introduced in this report.

2. Pavement Rehabilitation Index for Evaluation of Pavement Serviceability

It is very important to quantitatively evaluate the pavement serviceability because it helps engineers to recognize the degree of pavement distress and the necessity of maintenance of rehabilitation works. An equation for evaluating the pavement serviceability was developed through comparing pavement engineers' opinions with objective values for the pavement surface conditions.

In the case of asphalt pavements, 42 distressed sections (one section being 20 m×30 m) were selected and pavement engineers' opinions were taken for each section in the questionaire concerning the items shown in Table 1. In the case of concrete pavements, 25 sections (one section being 20 m×20 m) were selected, and an opinion survey was performed for each section, using a questionaire concerning the items shown in Table 2. Quantitative measurements were also performed at each section.

The results of the questionaire were quantified by the Quantification Theory II[1].

The following is a summary of the Quantification Theory II. When one evaluation item (ex. rutting) is rated at the A-category (or B- or C-category), the numerical score x_{11} is given (x_{12}, x_{13} respectively). The total score for each section is,

Table 1 Example of Questionaire (Asphalt Pavement)

Item		Section / Category	T-26 Judgement	T-26 Remarks	T-27 Judgement	T-27 Remarks	T-28 Judgement	T-28 Remarks
(1) Rutting		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
(2) Roughness		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
(3) Opening of construction joint or reflection crack		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
(4) Longitudinal and transverse crack		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
(5) Alligator crack		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
(6) Scaling or pot hole		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
(7) Surface texture		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
(8) Bleeding		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
Necessity of rehabilitation	for runway	unnecessary	A		A		A	
		desirable in the near future	B		B		B	
		necessary immediately	C		C		C	
	for taxiway	unnecessary	A		A		A	
		desirable in the near future	B		B		B	
		necessary immediately	C		C		C	
	for apron	unnecessary	A		A		A	
		desirable in the near future	B		B		B	
		necessary immediately	C		C		C	

Table 2 Example of Questionaire (Concrete Pavement)

Item		Section / Category	E-11 Judgement	E-11 Remarks	E-12 Judgement	E-12 Remarks	E-13 Judgement	E-13 Remarks
(1) Longitudinal and transverse crack		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
(2) Alligator crack		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
(3) Diagonal crack		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
(4) Joint spall		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
(5) Level difference		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
(6) Scaling or pot hole		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
(7) Surface texture		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
(8) Patching		no or minor influence	A		A		A	
		appreciable influence	B		B		B	
		pronounced influence	C		C		C	
Necessity of rehabilitation	for runway	unnecessary	A		A		A	
		desirable in the near future	B		B		B	
		necessary immediately	C		C		C	
	for taxiway	unnecessary	A		A		A	
		desirable in the near future	B		B		B	
		necessary immediately	C		C		C	
	for apron	unnecessary	A		A		A	
		desirable in the near future	B		B		B	
		necessary immediately	C		C		C	

$$\alpha_i = \sum_{j=1}^{R} \sum_{k=1}^{k_j} \delta_i(jk) x_{jk} \tag{1}$$

where, x_{jk}: numerical value given to the k-th category of the j-th item
$\delta_i(jk)$: =1, if i-sample checks in the k-th category in the j-th item
=0, otherwise
k_j: =number of the categories in the j-th item
R: total number of items

Figure 1 shows the schematic patterns of α with respect to each group. If the variance of each group ($\sigma_W{}^2$) can be minimized and the variance between the groups ($\sigma_B{}^2$) maximized at the same time, then the visual evaluation item (A, B and C) can be precisely predicted and classified by the use of α obtained from Equation (1). Because the correlation ratio (η^2) will be expressed as $\sigma_B{}^2/(\sigma_W+\sigma_B{}^2)$, the procedure described above is equivalent to finding out the score of each category so as to maximize η^2.

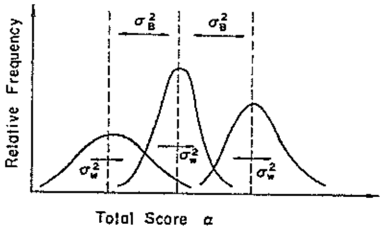


Fig. 1 Distribution on Total Scores

Table 3 shows the scores which were obtained through the application of the Quantification Theory II to the results of the questionaire of each category in each item in the case of asphalt pavements. Table 4 shows the scores in the case of concrete pavements. As the form of Equation (1) shows, the item which indicates the greater differences of the score given to A, B and C makes a greater contribution to the total score. Tables 3 and 4 also show the difference of the score given to A and C as a range, and Tables 5 and 6 show the three items ranked in the range.

The measured values regarding the items which make a greater contribution to the total score were used for the development of equations for evaluating the pavement serviceability. The following three items were finally selected for each pavement type, in consideration of the ease of measurement.

(A) Asphalt pavement
① Cracking ratio, CR (%): cracking area/area of a section × 100
② Rut depth, RD (mm): maximum rut depth in a section
③ Roughness, SV (mm): standard deviation of roughness measured by a 3 m profilometer

(B) Concrete pavement
① Cracking index, CR (cm/m²): length of cracks/area of a section
② Failure ratio of joints, JC (%): summation of the length of failed joints/total length of joints × 100

Table 3 Scores Given to Each Category (Asphalt Pavement)

Item	Category		Necessity of rehabilitaton					
			Runway		Taxiway		Apron	
			Score	Range	Score	Range	Score	Range
(1) Rutting	no or minor influence	A	−0. 279	0. 732	−0. 243	0. 653	−0. 200	0. 604
	appreciable influence	B	0. 700		0. 051		0. 003	
	pronounced influence	C	0. 453		0. 410		0. 404	
(2) Roughness	no or minor influence	A	−0. 494	1. 202	−0. 397	1. 093	−0. 416	1. 044
	appreciable influence	B	0. 244		0. 116		0. 186	
	pronounced influence	C	0. 708		0. 696		0. 628	
(3) Opening of construction joint or reflection crack	no or minor influence	A	−0. 165	0. 499	−0. 174	0. 525	−0. 180	0. 507
	appreciable influence	B	0. 040		−0. 041		−0. 023	
	pronouned influence	C	0. 334		0. 351		0. 327	
(4) Longitudinal and transverse crack	no or minor influence	A	−0. 193	0. 374	−0. 150	0. 262	−0. 137	0. 609
	appreciable influence	B	0. 145		0. 027		0. 015	
	pronounced influence	C	0. 181		0. 476		0. 472	
(5) Alligator crack	no or minor influence	A	−0. 079	0. 400	−0. 082	0. 358	−0. 145	0. 558
	appreciable influence	B	0. 076		0. 091		0. 177	
	pronounced influence	C	0. 321		0. 276		0. 413	
(6) Scaling or pot hole	no or minor influence	A	−0. 022	0. 151	−0. 068	0. 633	−0. 050	0. 512
	appreciable influence	B	0. 033		0. 044		0. 012	
	pronounced influence	C	0. 129		0. 565		0. 462	
(7) Surface texture	no or minor influence	A						
	appreciable influence	B						
	pronounced influence	C						
(8) Bleeding	no or minor influence	A	−0. 022	0. 156	−0. 021	0. 360	−0. 032	0. 434
	appreciable influence	B	0. 040		0. 000		0. 022	
	pronounced influence	C	0. 134		0. 339		0. 402	

Table 4 Scores Given to Each Category (Concrete Pavement)

Item	Category		Necessity of rehabilitation					
			Runway		Taxiway		Apron	
			Score	Range	Score	Range	Score	Range
(1) Longitudinal and transverse crack	no or minor influence	A	−0.328	0.728	−0.292	0.956	−0.272	1.019
	appreciable influence	B	0.162		−0.063		−0.147	
	pronounced influence	C	0.400		0.664		0.747	
(2) Alligator crack	no or minor influence	A						
	appreciable influence	B						
	pronounced influence	C						
(3) Diagonal crack	no or minor influence	A	−0.081	0.224	−0.072	0.391	−0.043	0.401
	appreciable influence	B	0.103		0.061		0.009	
	pronounced influence	C	0.143		0.319		0.358	
(4) Joint spall	no or minor influence	A	−1.039	1.533	−0.814	1.597	−0.668	1.359
	appreciable influence	B	0.216		−0.007		−0.029	
	pronounced influence	C	0.494		0.783		0.691	
(5) Level differennce	no or minor influence	A	−0.162	0.343	−0.066	0.335	−0.061	0.650
	appreciable influence	B	0.114		0.007		0.589	
	pronounced influence	C	0.181		0.269			
(6) Scaling or pot hole	no or minor influence	A	−0.168	0.440	−0.154	0.610	−0.139	0.612
	appreciable influence	B	0.119		0.018		−0.011	
	pronounced influence	C	0.272		0.456		0.473	
(7) Surface texture	no or minor influence	A						
	appreciable influence	B						
	pronounced influence	C						
(8) Patching	no or minor influence	A	−0.025	0.115	−0.022	0.010	−0.046	0.211
	appreciable influence	B	0.090		0.079		0.165	
	pronounced influence	C						

Table 5 Items Having a Greater Range (Asphalt Pavement)

		Order	Item
Necessity of rehabilitation	Runway	1	roughness
		2	rutting
		3	opening of construction joint or reflection crack
	Taxiway	1	roughness
		2	rutting
		3	scaling or pot hole
	Apron	1	roughness
		2	longitudinal and transverse crack
		3	rutting

Table 6 Items Having a Greater Range (Concrete Pavement)

		Order	Item
Necessity of rehabilitation	Runway	1	joint spall
		2	longitudinal and transverse crack
		3	scaling or pot hole
	Taxiway	1	joint spall
		2	longitudinal and transverse crack
		3	scaling or pot hole
	Apron	1	joint spall
		2	longitudinal and transverse crack
		3	lenel difference

③ Level Difference, SV (mm): maximum difference in a section

The total score for each observer was then calculated by substituting the scores for each category in each item shown in Table 1, into Equation (1), and the mean value of the total scores was calculated for each section. Subsequently, multiple regression analysis on the assumption of linear form was carried out between the mean values of the total scores as a dependent variable and the measured values of the above-mentioned items as independent variables. If the independent variables are converted to exponential form, then the coefficient of correlation becomes a little greater. However, because the difference was very small in this case, the following linear form was applied here.

$$y=a_0+a_1x_1+a_2x_2+a_3x_3 \qquad (2)$$

where, y : total score
$x_1 \sim x_3$: measured values
$a_0 \sim a_3$: coefficients

The coefficients were determined from a set of input data, and the resulting equations were as follows.

(A) Asphalt pavement

$$y=a_0+0.0776CR+0.00881RD+0.113SV \tag{3}$$

where, $a_0=-0.845$ for runways
-0.836 for taxiways
-0.847 for aprons

(B) Concrete pavement

$$y=a_0+0.0520CR+0.0531JC+0.0958SV \tag{4}$$

where, $a_0=-1.07$ for runways
-1.06 for taxiways
-1.06 for aprons

Using a linear transformation which expresses a score 0 as the minimum value and a score 10 as the maximum value, Equations (3) and (4) were then converted into the Equations (5) and (6).

(A) Asphalt pavement

$$y=10-0.450CR-0.511RD-0.655SV \tag{5}$$

(B) Concrete pavement

$$y=10-0.290CR-0.296JC-0.535SV \tag{6}$$

The y values obtained from these equations were named the PRI (Pavement Rehabilitation Index). Tables 7 and 8 show the threshold values of the PRI with respect to several levels of the necessity of rehabilitation. Therefore, when airport pavements are evaluated by the surface condition, the PRI is calculated by the use of Equation (5) with measured values for the cracking ratio, rut depth and roughness, and by the use of Equation (6) with measured values for the cracking index, failure ratio of joints and level difference, and the necessity of the pavement rehabilitation is then judged on the basis of the threshold values shown in Tables 7 or 8.

Table 7 Criteria of PRI (Asphalt Pavement)

Section	A	B	C
Runway	more than 8.0	8.0 to 3.8	less than 3.8
Taxiway	more than 6.9	6.9 to 3.0	less than 3.0
Apron	more than 5.9	5.9 to 0	—

A: Rehabilitation not required
B: Rehabilitation desirable in the near future
C: Rehabilitation required immediately

In addition to evaluation using PRI, it is also necessary to evaluate each individual item. The threshold values shown in Tables 9 and 10 were introduced for this purpose.

Table 8 Criteria of PRI (Concrete Pavement)

Section	A	B	C
Runway	more than 7.0	7.0 to 3.7	less than 3.7
Taxiway	more than 6.4	6.4 to 2.3	less than 2.3
Apron	more than 5.7	5.7 to 0	—

A: Rehabilitation not required
B: Rehabilitation desirable in the near future
C: Rehabilitation required immediately

Table 9 Criteria for Each Category (Asphalt Pavement)

Item	Section	A	B	C
Cracking ratio *CR* (%)	Runway	less than 0.1	0.1 to 6.5	more than 6.5
	Taxiway	less than 0.9	0.9 to 12.7	more than 12.7
	Apron	less than 1.9	1.9 to 17.0	more than 17.0
Rut depth *RD* (mm)	Runway	less than 10	10 to 38	more than 38
	Taxiway	less than 17	17 to 57	more than 57
	Apron	less than 22	22 to 70	more than 70
Roughness *SV* (mm)	Runway	less than 0.26	0.26 to 3.64	more than 3.64
	Taxiway	less than 0.91	0.91 to 6.57	more than 6.57
	Apron	less than 1.50	1.50 to 8.63	more than 8.63

A: Rehabilitation not required
B: Rehabilitation desirable in the near future
C: Rehabilitation required immediately

Table 10 Criteria for Each Category (Concrete Pavement)

Item	Section	A	B	C
Cracking index *CR* (%)	Runway	less than 0.2	0.2 to 5.6	more tnan 5.6
	Taxiway	less than 0.6	0.6 to 7.6	more than 7.6
	Apron	less than 1.1	1.1 to 11.1	more than 11.1
Faliure ratio of joints *JC* (%)	Runway	less than 0.1	0.1 to 1.3	more than 1.3
	Taxiway	less than 0.1	0.1 to 3.2	more than 3.2
	Apron	less thae 0.1	0.1 to 5.7	more than 5.7
Level difference *SV* (mm)	Runway	less than 5	5 to 10	more than 10
	Taxiway	less than 5	5 to 12	more than 12
	Apron	less than 6	6 to 14	more than 14

A: Rehabilitation not required
B: Rehabilitation desirable in the near future
C: Rehabilitation required immediately

3. Non-destructive Structural Evaluation Methods

3.1 Evaluation Method by Dynaflect for Asphalt Pavement

Until recently, the structural evaluation of airport pavements has been conducted on the basis of destructive test procedures. However, because these destructive tests are time-consuming and costly, they are not suitable for airport pavements having many constraints including limited time for investigation or repair.

Over the past ten years, researchers have developed non-destructive test procedures for evaluating pavement structures. The use of a non-destructive procedure has the advantage of rapid evaluation with the minimum disturbance to normal traffic operation. Dynaflect is a non-destructive test device which was developed in the United States and widely used for measurement of pavement surface deflections. It was chosen for our study. Figure 2 shows the Dynaflect device, while Fig. 3 shows the location of the load and geophones.

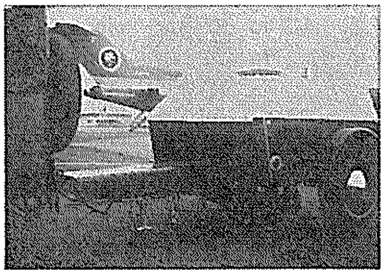


Fig. 2 Dynaflect

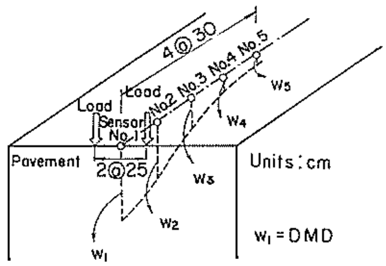


Fig. 3 Locations of Loads and Geophones

Deflection in asphalt pavement is affected by both the load and the pavement temperature. In Dynaflect measurements, because the load is fixed, adjustment of deflections for load is unnecessary. The influence of temperature must be corrected, however, in order to compare the data taken at various tempeartures.

In order to adjust the measured deflections to their values at the standard surface temperature of 20°C, the temperaure adjustment factors were derived as shown in Fig. 4. The factors are applicable to only the Dynaflect Maximum Deflection (DMD), which is a deflection of No. 1 geophone (refer to Fig. 3). Some examples of the application of the temperature adjustment factors are given in Fig. 5. The DMD measurements were conducted at two temperatures for each pavement. The estimated value at one temperature derived from the value at the other temperature using the temperature adjustment factor was compared with the value measured for the pavement. The result shows that more than 95% of the estimated values are between 100 ± 15% of the measured values.

The thickness of a pavement is designed as a function of the type of aircraft and the traffic volume for given subgrade conditions. This means that surface deflections of the pavement are also influenced by the design load and the design coverage of the pavement. The DMD values measured for some airport asphalt

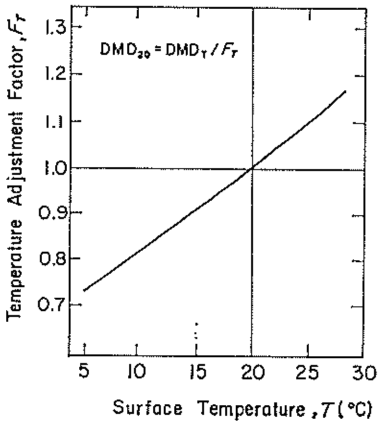


Fig. 4 Temperatufe Adjustment Factors

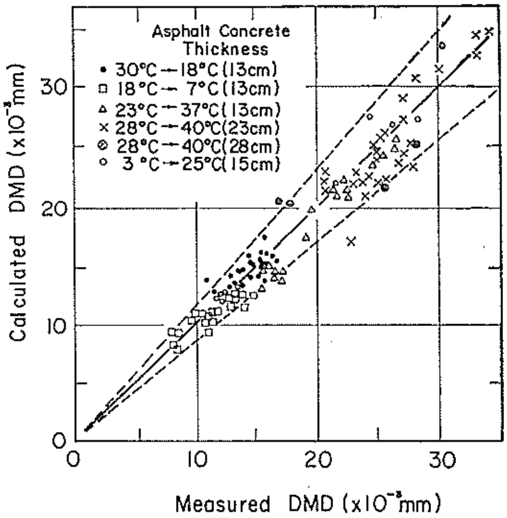


Fig. 5 Relationship between Measured and Calculated DMDs

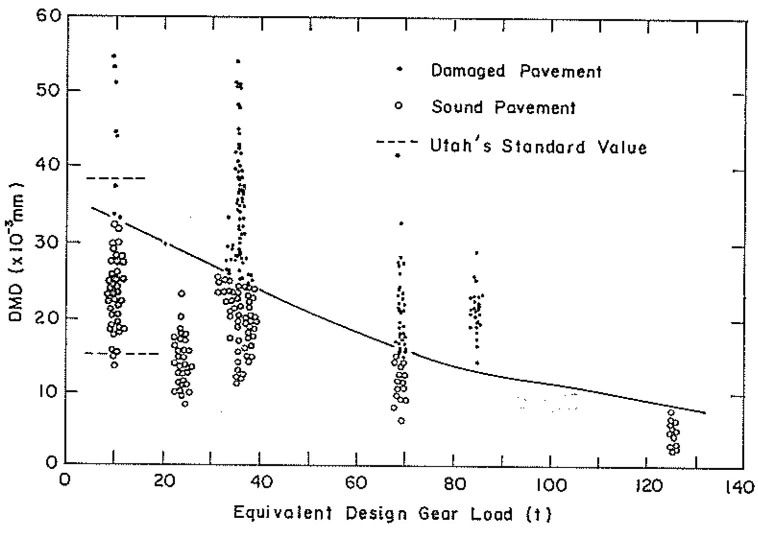


Fig. 6 Relationship between DMD and Equivalent Design Gear Load

pavements are plotted in Fig. 6 with their converted values (at 20°C) against the Equivalent Design Gear Load (EDGL) of the pavement for which the DMD values have been measured.

The EDGL is defined as the allowable gear load for a coverage of 5,000 which the pavement being considered can sustain for the design CBR value and the pavement thickness. For example, according to the CBR design method, a pavement thickness for a coverage of 20,000 for a Boeing-747-200B (gear load: 82.5 tf) and for the design CBR value of subgrade of 5% is 170 cm. This pavement thickness is equal to the required thickness for a coverage of 5,000 for a gear load of 115 tf. This gear load of 115 tf is the EDGL of a pavement of 170 cm thickness.

From Fig. 6 in which the deflection data on both damaged or sound pavements are plotted, it can be seen that the DMD values measured on structurally damaged pavements are generally higher than those for sound pavements, although these data are scattered because of the difference of the degree of distress. The solid line in the figure is drawn to separate these data into two parts. A pavement with a DMD beyond this line might be regarded as being structurally damaged and requiring some maintenance or rehabilitation works. The standard values for highway pavements presented by the Utah State Department of Highways in the United States of America are also shown in the figure for comparison[2]. A DMD value indicated by this line is defined as a standard DMD value for the corresponding EDGL and can be used as a critical deflection to judge the structural adequacy of asphalt pavements. From these results, non-destructive structural evaluation of airport asphalt pavements is therefore possible.

3.2 Evaluation Method by Falling Weight Deflectometer for Concrete Pavement

Since Dynaflect is not suitable to concrete pavements because of its small weight, the Falling Weight Deflectometer (FWD, Fig. 7), which can apply a load about ten times as that of Dynaflect, was chosen, and its applicability to the structural evaluation of airport concrete pavements was studied[3].

Fig. 7 Falling Weight Deflectometer

The thickness of a concrete slab is determined on the basis of flexural stresses at the bottom of the slab caused by aircraft loads. The stresses are calculated by the use of Westergaard's formula for a concrete slab resting on a Winkler foundation[4]. In addition to loading condition, the elastic modulus and Poisson's ratio for a concrete slab and the base support value are necessary for this calculation. The stresses are also influenced by the joint efficiency. Moreover, voids under concrete slabs sometimes occur at joints from repeated loadings and/or the seepage of water. These items should be checked from the standpoint of a structural evaluation for concrete pavements. We then tried to evaluate them by using FWD measurements.

First, the elastic modulus E_c of a concrete slab and the base support value K were calculated from the FWD deflections measured for the interior portion of the slab using Westergaad's deflection equation[5]. In this calculation, Poisson's ratio of the concrete slab was assumed to be constant because its influence was considered to be small. Figures 8 and 9 shows typical examples. The modulus E_c and the K value calculated from the FWD deflections were considerated appropriate, judging from the results of laboratory tests and/or other field tests. However, the K value greatly varies with time. The reason for this is the warping of the concrete slab due to temperature differentials within the slab.

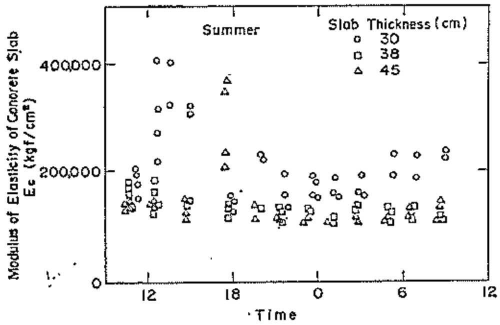


Fig. 8 Modulus of Elasticity of Concrete Slab Calculated from FWD Deflections

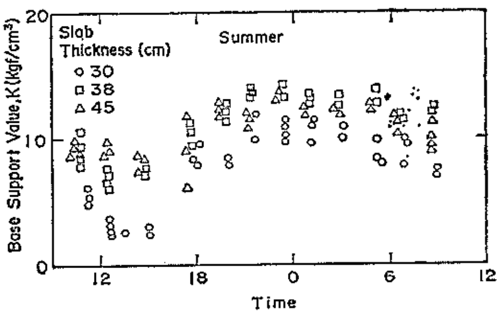


Fig. 9 Base Suoport Value Calculated from FWD Deflections

The joint efficiency was then investigated. FWD measurements at joints were carried out as in Fig. 10 and the load transfer effectiveness E_{ff}' was calculated by Equation (7).

$$E_{ff}'=\frac{d_2}{(d_1+d_2)/2}\times 100 \qquad (7)$$

where, d_1: deflection of a loaded slab
d_2: deflection of an unloaded slab

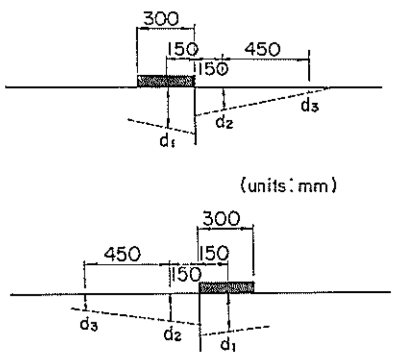


Fig 10 Measurement of FWD at Joint for Load Transfer Effectiveness

Figure 11 shows the changes in E_{ff}' with joint openings for several joint structures. The E_{ff}' decreases with the joint openings in keyed joint (A-1), but is rather constant in dowel bar joint (A-2). In the case of hybrid joints using key and dowel bar together (A-3, A-4), it locates between those of A-1 and A-2. Since these tendencies are similar to those observed in the load transfer effectiveness E_{ff} for actual aircraft, E_{ff}' seems to be a useful indicator to represent the joint efficiency.

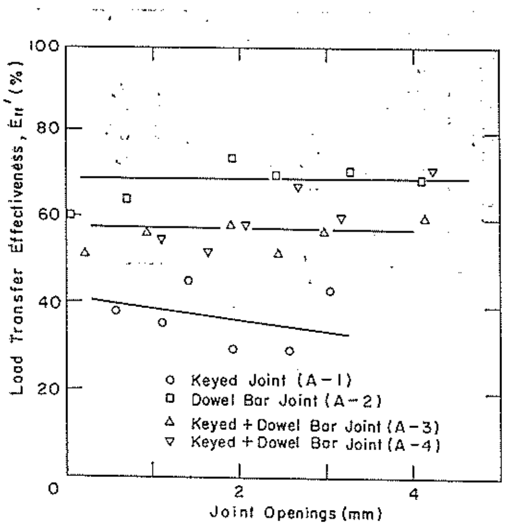


Fig. 11 Changes in E_{ff}' with Joint Openings

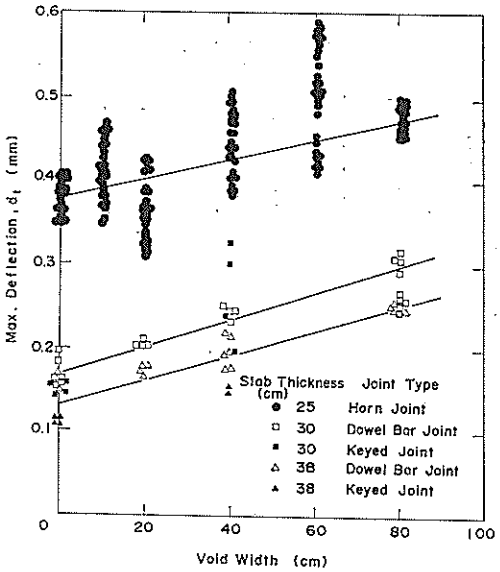


Fig. 12 Maximum Deflection at Joint of Concrete Pavement with Void

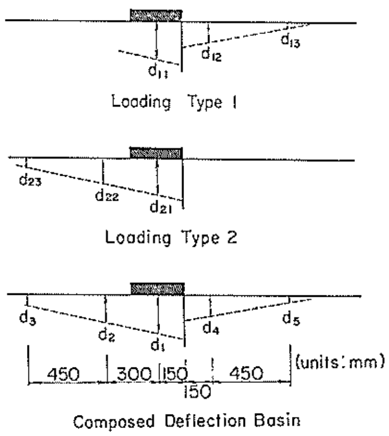


Fig. 13 Measurement of FWD at Joint for Detecting Void under Concrete Slab

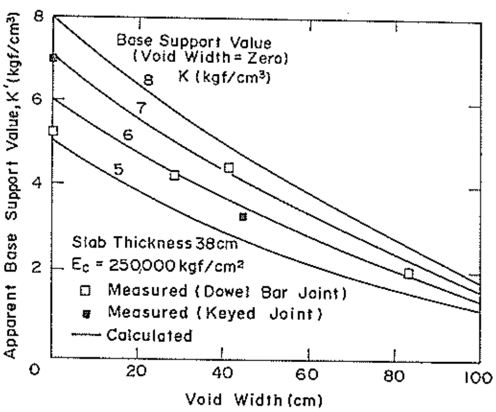


Fig. 14 Comparison between Measured and Calculated Apparent Base Support Values

In addition to the structural condition of joints, voids under concrete slabs must also be evaluated. Voids under concrete slabs at joints are probably produced by the pumping of the base and/or subgrade fines. Figure 12 shows the relationship between the size of the voids and the FWD maximum deflection at joints. FWD maximum deflection increases with the size of the voids.

Because the increase of the FWD maximum deflection in a concrete pavement with voids under the concrete slab is similar to that for insufficient base support,

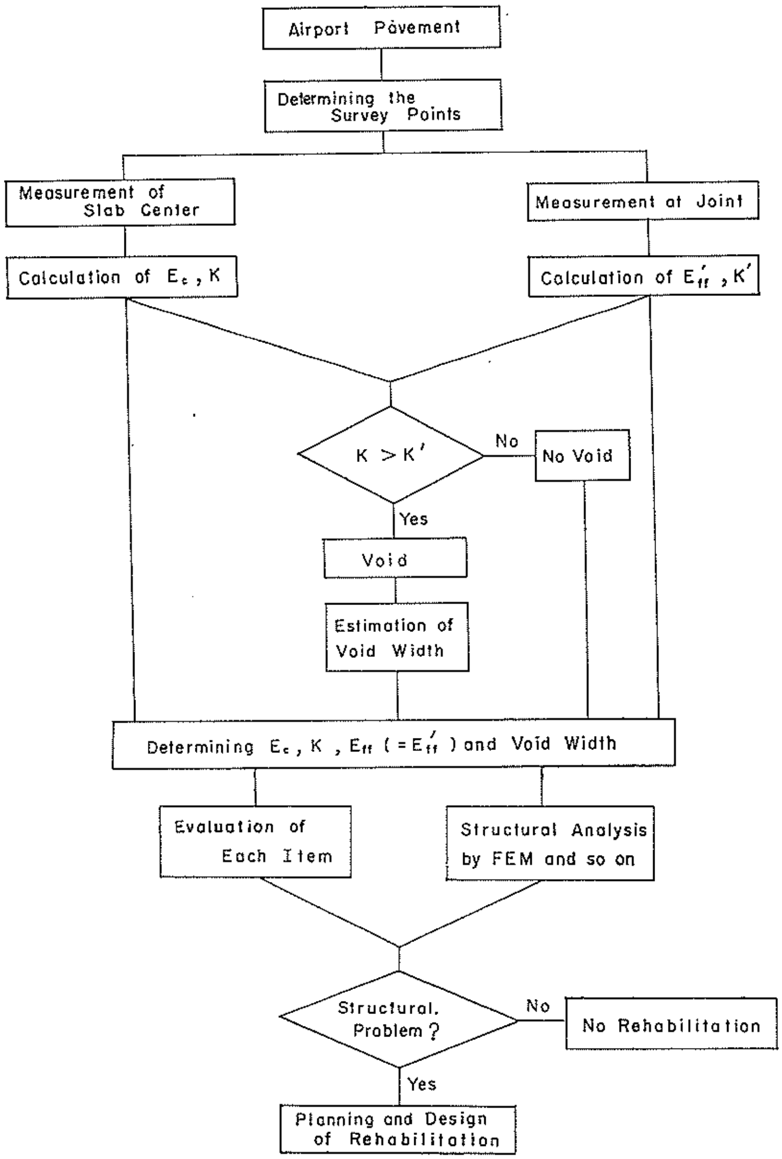


Fig. 15 Flow Chart for Structural Evaluation of Concrete Pavements

the FWD deflection seems to be a suitable index to evaluate concrete pavements with voids under the concrete slabs. We tried to detect existence of voids and their sizes from FWD deflections.

FWD measurements as shown in Fig. 13 were conducted and the deflections at five points used for the calculation of the apparent base support value K' which includes the effect of voids under a concrete slab, assuming that E_c and E_{ff} ($=E_{ff}'$) are constant. Figure 14 shows the relationship between the void size and the K' value calculated from the FWD measurements. The theoretical results from finite element analysis are also shown in Fig. 14. Both are in good agreement. It may be possible to detect existence of voids and their sizes by comparing the K' value calculated from FWD measurements, to the theoretical relationship between the void size and K' value.

Since the factors necessary for the structural evaluation of concrete pavements can be estimated from FWD measurements, the structural evaluation of concrete pavements can be performed in accordance with the flow shown in Fig. 15.

4. New Design Methods of Rehabilitation Works

4.1 Asphalt Overlay on Existing Asphalt Pavement

Until recently, the thickness of an asphalt overlay was designed through destructive tests on the basis of the evaluation for subgrade and each layer of an existing pavement. However, because this design procedure must be based on time-consuming and costly destructive evaluation, we have developed a quite simple and convenient procedure for overlay thickness design based on the non-destructive evaluation of existing pavements.

Figure 16 shows the flow chart of this new overlay thickness design procedure.

① Prepare the input data such as a set of Dynaflect measurements, the thickness of the asphaltic layer of the existing pavement (h_1), the surface temperature (T°C), and the allowable value of DMD (DMD_0). The selection of DMD_0 is very important. The standard DMD value shown in Fig. 6 can be used for this purpose.

② Adjust the DMD value to its equivalent value at 20°C using Fig. 4.

③ Compare the DMD value at 20°C with DMD_0. If the former is greater than the latter, go to the next step.

④ Divide the existing pavement into an upper layer (asphaltic material) and a lower layer (non-asphaltic material), and calculate each modulus of elasticity using two-layer elastic theory. In this step, the moduli of the asphaltic material at T°C and the non-asphaltic material of the existing pavement are obtained.

⑤ Calculate the modulus of deformation of the asphalt concrete for overlay at T°C by Equation (8).

$$\log E = 5.237 - 0.02688\,T \tag{8}$$

where, E: modulus of deformation of asphalt concrete (kgf/cm²)
T: surface temperature (°C)

⑥ Assume the overlay thickness, h_0.

⑦ Considering the overlay pavement structure as the three-layer elastic system shown in Fig. 17, estimate the DMD values after overlay using three-layer elastic theory. The DMD value calculated in this step corresponds to the DMD value

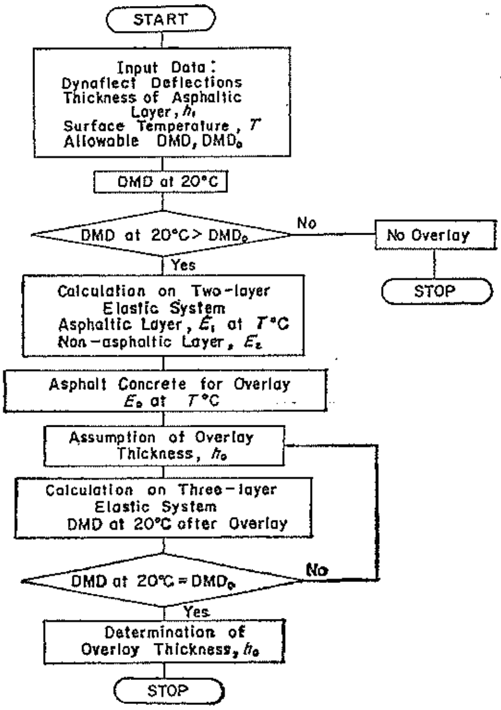


Fig. 16 Flow Chart of Proposed Scheme for Overlay Thickness Design

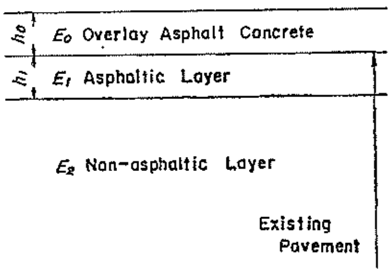


Fig. 17 Pavement Structure after Overlay

after overlay at T°C.

⑧ Adjust the DMD value obtained in step ⑦ to the value at 20°C.

⑨ Repeat step ⑥ through ⑧ until calculated DMD value in step ⑧ sufficiently approaches DMD_0.

Confirmation of the accuracy of the above overlay thickness design procedure is made by comparing the DMD value predicted from deflections on the existing pavement with the DMD value actually measured after overlay. Figure 18 shows a close agreement between the predicted and measured DMD values. The overlay thicknesses employed herein are between 3.6 and 19 cm. From Fig. 18, it can be seen

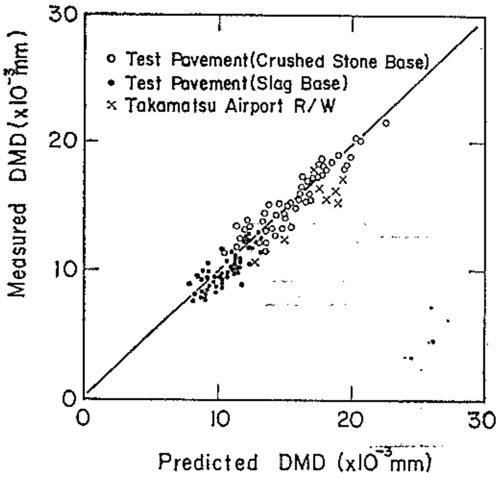


Fig. 18 Relationship between Predicted and Measured DMDs

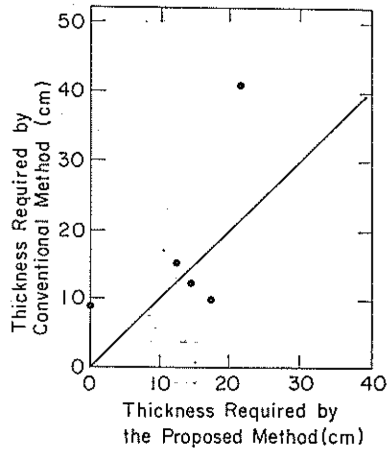


Fig. 19 Comparison of Thicknesses Required between by Conventional Method and by Proposed Method

that three-layer elastic theory can predict the DMD value after overlay with high accuracy.

Furthermore, the overlay thicknesses designed by this proposed method and by the conventional method being used in Japan are compared in Fig. 19. The conventional method is to make use of the effective thickness of an existing pavement by using equivalency factors of each component layer. The overlay thicknesses obtained by the newly proposed method agree approximately with those by the conventional method.

Judging from the facts shown in Figs. 18 and 19, the procedure proposed in this paper is recommended for the overlay thickness design of airport asphalt pavements.

4.2 Asphalt Overlay on Existing Concrete Pavement

Until now, a design method used for asphalt overlays on existing asphalt pavements has also been used for asphalt overlays on existing concrete pavements. Using the design CBR value of existing subgrade and the new design load, the required thickness of the asphalt pavement where the base courses are composed of granular materials is determined. The existing concrete slab is assumed as the base course and the thickness is converted to the equivalent thickness of a granular base using an equivalency factor (*E. F.*) shown in Table 11 (equivalent thickness$=E.F.\times$ concrete slab thickness). The other existing base courses are also evaluated using a method similar to that for concrete slabs. The required asphalt overlay thickness can then be calculated by subtracting the total equivalent thickness of the existing pavement from the required pavement thickness[4].

However, it has been recognized that this method gives too thick an overlay when the existing slab can act as a satisfactory slab, and improvement of the design method has been awaited.

Table 11 Equivalency Factor of Existing Concrete Slab

Condition of existing slab	Equivalency factor
Reinforced slab without pumping and cracks	2.5
Plain slab without pumping and cracks	2.0
Initially cracked slab without pumping (crack spacing is greater than 1 m)	1.5
Others	1.0
Mechanically stable granular base course (used as calculation basis)	1.0

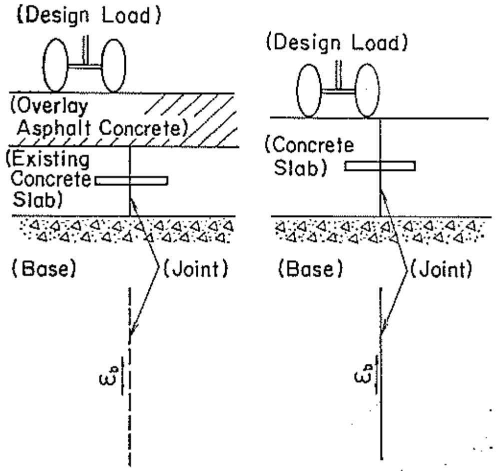


Fig. 20 Schematic Represention of Structural Analysis of Asphalt Overlay

When reflection cracks are not expected to appear in the asphalt overlay layer, it becomes clear that the three-layer elastic theory where the pavement is assumed to be composed of an asphalt overlay, a concrete slab and subgrade, can describe the behavior of an asphalt overlay on an existing concrete pavement. In this case, it was found that the critical design factor is the strain at the bottom of the existing concrete slab as shown in Fig. 20[6]. Therefore, the effect of the asphalt overlay to strengthen the existing pavement can be estimated in terms of the strain at the bottom of the existing concrete slab.

From the various trials, it was found that the elastic properties of the pavement component layers can be estimated as follows, for use in the calculations.

① existing subrade

The modulus of elasticity (E modulus) is estimated using Equation (9).

$$K_{75}=\frac{1}{h}\sqrt[3]{\frac{E_2^4}{E_1}} \tag{9}$$

where, K_{75}: base support value (kgf/cm³)

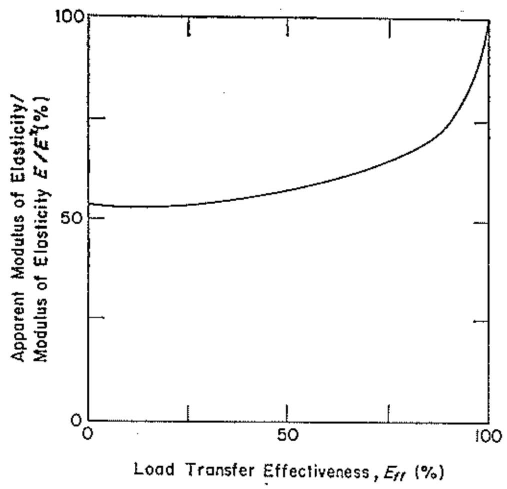


Fig. 21 Relationship between E_{ff} and E^*

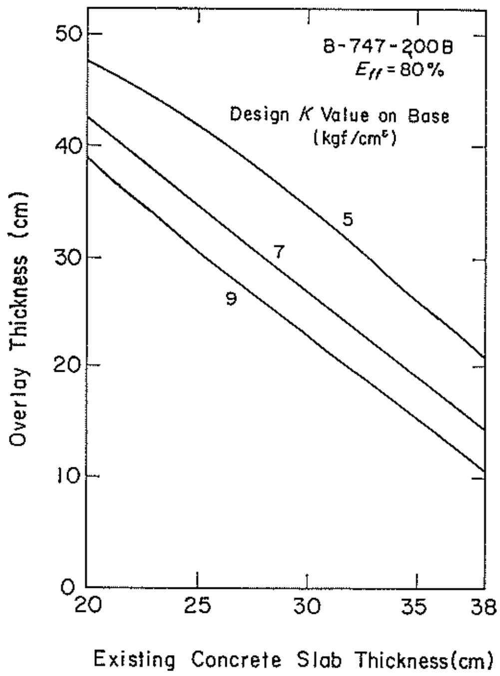


Fig. 22 Overlay Thickness Design Curves for B-747-200B (E_{ff}=80%)

h : existing concrete slab thickness (cm)

E_1 : E modulus of existing concrete slab which can be estimated in

② (kgf/cm²)
E_2 : E modulus of existing subgrade (kgf/cm²)

Poisson's ratio is assumed to be 0.3.

② existing concrete slab

The structural performance of a concrete slab varies with the load transfer effectiveness at an existing joint (E_{ff}). The apparent E modulus (E^*) should be used as the E modulus of a concrete slab. It was found that E^* changes with E_{ff} as shown in Fig. 21. E_{ff} is defined in Equation (7) and can be obtained by the use of the FWD deflections measured on the joint portion of a concrete slab.

Poisson's ratio is 0.15.

③ asphalt overlay layer

E modulus and Poisson's ratio are 7,000 kgf/cm² and 0.3, respectively.

Figure 22 is an example of a design chart for an asphalt overlay on an existing concrete pavement, using the proposed design procedure. It shows the required thickness of the asphalt overlay when the design aircraft is a Boeing-747-200B and when the load transfer effectiveness at the joints is 80%.

4.3 Bonded Concrete Overlay on Existing Concrete Pavement

Concrete overlays are classified into bonded, partially bonded and unbonded overlays according to the degree of bonding between the existing and overlay concrete slabs (Fig. 23). A bonded overlay has the thinnest overlay thickness of these three types of concrete overlays. The advantages of the thinner overlay are lower cost and fewer problems in matching with the adjacent facilities. A bonded overlay, however, is not widely used, because bond characteristics in concrete overlays have not been revealed and because insufficient bond in conventional techniques has caused certain problems such as separation at the corners and/or the edges of slabs.

Some studies were carried out to investigate the factors that affect the bonding between existing and overlay concrete slabs in bonded overlays, and to propose an effective means to ensure higher bonding[7].

Bond strengths in conventional bonding techniques (Fig. 24) were examined by the use of bending, tensile and shear strength tests. Figures 25 to 27 are the test results. They show that the shear strength, which is the most important in bonded overlays, is quite small. This is considered to be a cause of above-mentioned defects

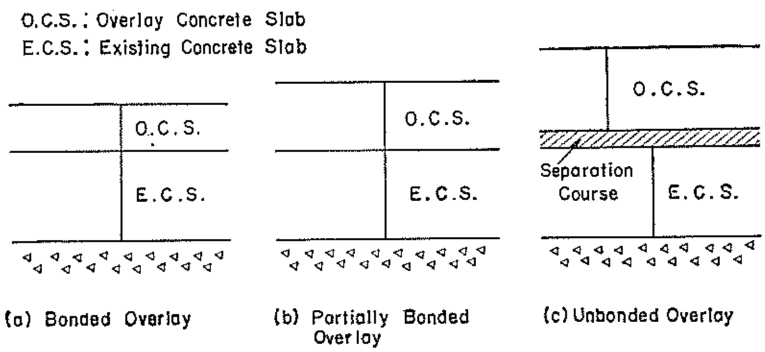


Fig. 23 Types of Concrete Overlays

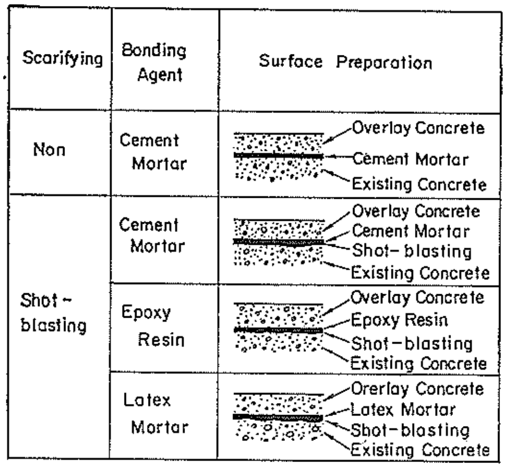


Fig. 24 Conventional Bonding Techniques

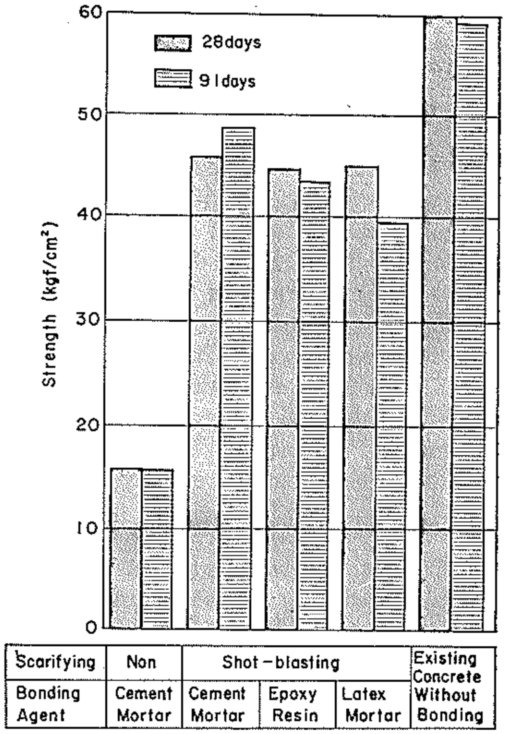


Fig. 25 Bending Strength

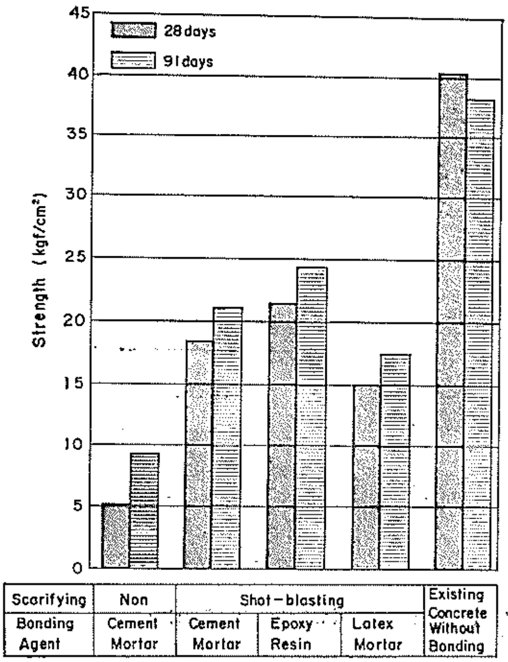


Fig. 26 Tensile Strength

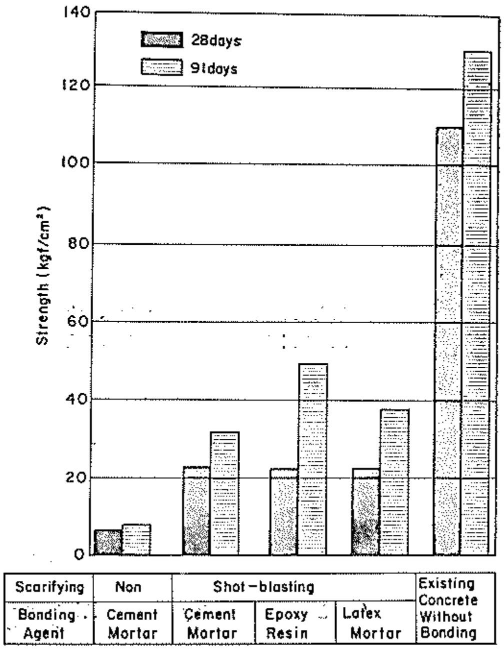


Fig. 27 Shear Strength

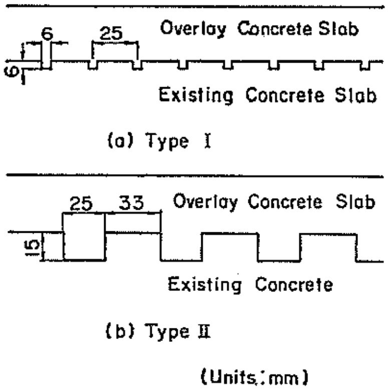
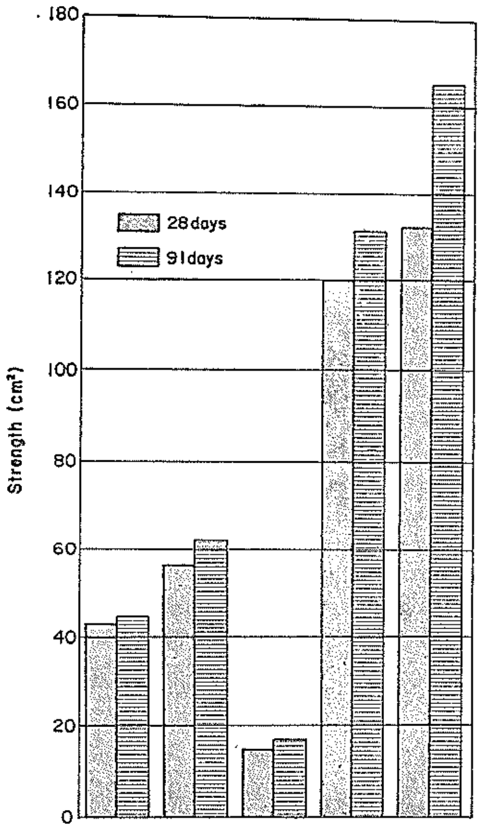


Fig. 28 New Bonding Techniques



Scarifying	Shot-blasting		Non	Shot blasting	Existing Concrete Without Bonding
Bonding Agent	Cement Mortar	Cement Mortar	Cement Mortar	Cement Mortar	
Type	——	Type I		Type II	

Fig. 29 Shear Strength by New Bonding Techniques

in bonded overlays.

New bonding techniques were therefore attempted to improve bond shear strength. One of the proposed techniques is to make small grooves on the existing concrete slabs (Type I), while another proposed technique is to make large grooves (Type II). In the latter case, the coarse aggregates of the overlay concrete can enter the grooves.

The bond strengths in these proposed techniques were tested and Fig. 29 shows the test results of shear strengths. The bond shear strength of Type I is 1.3~1.4 times as large as that of the specimens without grooving. Type II also has good efficiency in improving the bond shear strength, and the improvement of Type II is much larger than that of Type I. The bond strength of Type II is closer to that of the existing concrete without bonding. It was observed that the coarse aggregates of the overlay concrete were cracked along the bonding surface. From this result, we can understand why Type II has a quite large efficiency in improving the bond shear strength.

It can be concluded that the new bonding techniques proposed here, are quite effective in improving bond strengths. Therefore, if suitable grooves are provided at the corners and/or edges of the existing concrete slabs, the problems related to bonded overlays will be reduced and the reliability increased. The size of the grooves should be varied according to the design conditions.

4.4 Prestressed Concrete Precast Slab Pavement

In concrete pavements, instead of placing concrete slabs in-situ which requires a long curing time, a pavement using prestressed concrete (PC) precast slabs which are manufactured in a factory has long been considered.

However, because the conventional method of connecting laid PC precast slabs is by prestressing tendons or screws, it has always been subject to problems such as being inefficient, difficult to lay exactly level and impossible to remove and reset the portions where some slabs have been damaged.

To overcome these problems, we have invented the horn-joint shown in Fig. 30[8].

This joint is designed to connect the slabs by inserting arc-shaped steel bars from the surface of the slabs into arc-shaped plastic tubes cast into the concrete and grouting the gap between the tube and the bar. This joint also makes it possible to replace and rejoin slabs simply by cutting the reinforcing bars and knocking the old bars and grout out of the tubes.

Laboratory tests were conducted to evaluate the function of the horn-joint and to design the optimum material and radius of tube, and the quality of the filling

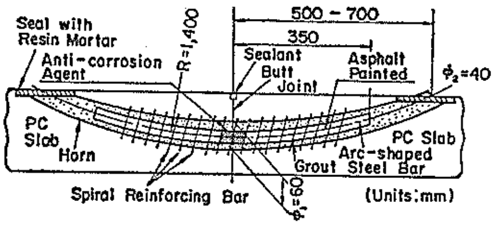


Fig. 30 Horn-joint

grout.

It was confirmed that the horn joint has almost the same load transfer effectiveness as that of a conventional straight dowel bar joint. It was also made clear from the tests that the bars and grout in tubes can be easily knocked out because of their tapered design giving them a "horn-like" shape.

A new type of PC precast slab pavement in which slabs are capable of being connected at their adjacent edges by the use of horn-joints, was made and various tests conducted to investigate the construction properties and structural strength.

The PC precast slabs in the test pavement were designed for DC-8-63 aircraft. The slab thickness was 20 cm and the amount of prestressing was 29 kgf/cm^2. The length of each slab was 10 m and the width 2.3 m in consideration of ease of transportation and handling on site. The weight of each slab was 11.5 tf. In the test pavement, six precast slabs were arranged on a levelling course of fine aggregate (sand 7: cement 1 by volume, 2 cm thick in average) over the base course of crushed stone. The measured K values on the base course were 10~15 kgf/cm^3. Figure 31 shows the test pavements for PC precast slabs.

Static loading tests in which the load was increased step by step (in four stages up to the maximum gear load of 73.6 tf) were conducted before and after the connection of the precast slabs. After the static loading tests, traffic tests (gear load: 73.6 tf) on the lane shown in Fig. 31 were also conducted repeatedly for over 5,000

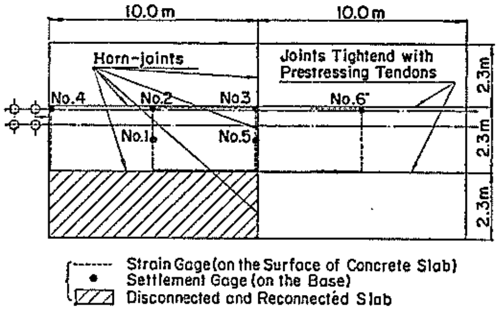


Fig. 31 PC Precast Slab Test Pavement

Fig. 32 Loading Vehicle

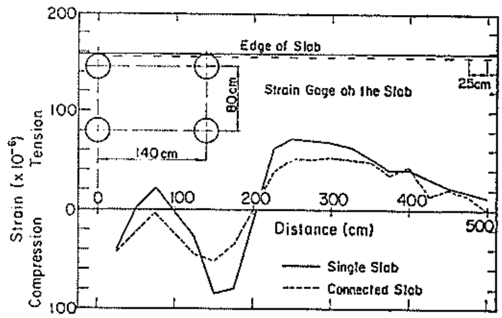


Fig. 33 Comparison of Slab Edge Strains of a Single Slab and Connected Slabs

times. The loading vehicle shown as in Fig. 32 was used for these tests.

Figure 33 shows the measured strain at the edge of a slab under a 50.8 tf gear load placed at the edge of the slab. The measurements were carried out both before and after the connection of the slabs. The former is called "a single slab". From the comparison of these results, the strains of connected slabs seem to be considerably less than those for a single slab. Also, the strains of connected slabs are almost similar to, but slightly less than those from the finite element analysis. From this result, it is clear that the horn-joint has a sufficient load transfer effectiveness.

Figure 34 shows the change of the measured strains at the longitudinal joint of the connected slabs in the traffic tests. The maximum strain increases slightly with repeated loadings. However, since the measured strain after 5,000 repetitions (number of wheel passes: 10,000) is not over that at the center of a slab from the finite element analysis, and because the cracking strain of a slab is considered to be about 300×10^{-6} based on the results of laboratory bending tests on concrete samples, the possibility of cracking in the connected slabs is negligible and the durability of the slabs connected with horn-joints is judged to be sufficient for normal traffic.

Since PC precast slab pavements interconnected with horn-joints have been confirmed to have a sufficiently smooth surface and good structural performance, and to also facilitate the exchange of damaged slabs, this new type of pavement is suitable for application to airport pavements, and especially for maintenance and rehabilitation works. It has been successfully applied to the rehabilitation work of the runway concrete pavements at Chitose Airport[9].

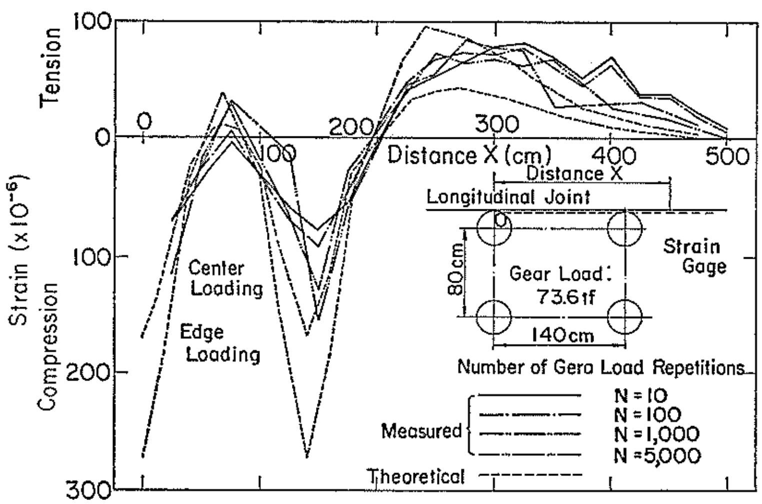


Fig. 34 Changes of Joint Edge Strain Distribution with Repeated Loadings

5. Proposal of a New Maintenance and Rehabilitation System

Airport pavements are maintained or rehabilitated when the necessary functions cannot be fulfilled due to degradation of the pavements due to aircraft traffic and environmental effects. Figure 35 shows the flow chart for the maintenance and rehabilitation system for airport pavements that have been thought out on the basis of the results contained in Chapter 2, 3 and 4.

The fundamental procedure of the proposed maintenance and rehabilitation system consists of ① judgement of necessity of maintenance or rehabilitation based on the evaluation of pavement surface condition, ② structural evaluation for the pavements in which maintenance or rehabilitation works are judged to be necessary,

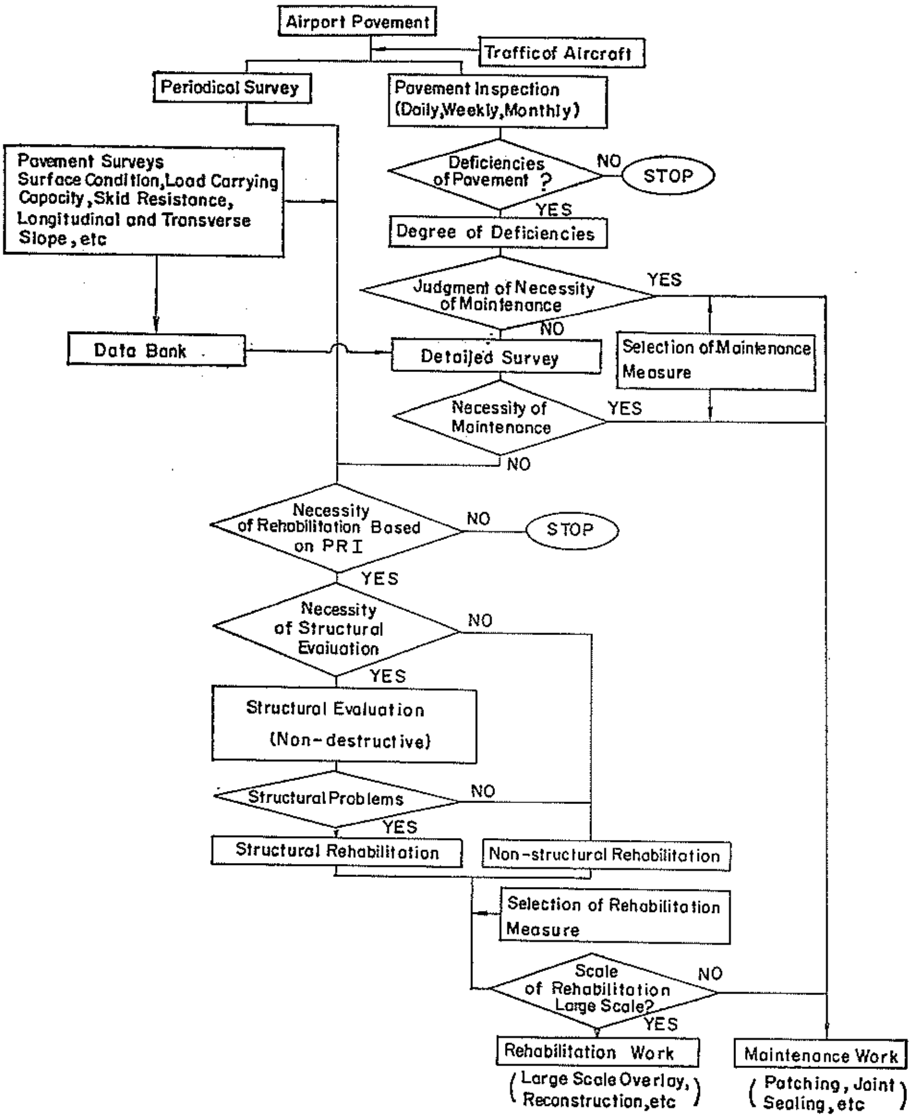


Fig. 35 Proposed New Maintenance and Rehabilitation System for Airport Pavements

③ selection of appropriate maintenance or rehabilitation work, and ④ design of the selected maintenance or rehabilitation works.

6. Concluding Remarks

Through these studies, several new evaluation methods and several new design methods for maintenance or rehabilition works for airport pavements have been developed.

Pavement serviceability has been able to be objectively evaluated on the basis of the qualitative value PRI and non-destructive evaluation of the structural adequacy and load-carrying capacity of existing pavements have been achieved by the use of Dynaflect deflections for asphalt pavements and by the use of FWD deflections for concrete pavements.

The following results have been obtained for maintenance or rehabilitation works. A new overlay thickness design method based on the non-destructive evaluation of an existing pavement has been developed for an asphalt overlay on an existing asphalt pavement. For an asphalt overlay on an existing concrete pavement, a new design method by the use of three-layer elastic theory has been developed for cases where reflection cracks are not expected to appear in the asphalt overlay layers. New techniques for bonding to grooves on the surface of existing concrete slabs are recommended in order to increase the reliability of bonded concrete overlays on existing concrete pavements. A prestressed concrete precast slab pavement interconnected with the newly invented horn joints has been developed for an easy construction method in the reconstruction with concrete pavements.

Based on these results, a new maintenance and rehabilitation system for airport pavements has been proposed including ① pavement evaluation, ② selection of appropriate maintenance or rehabilitation works, ③ design of the selected maintenance or rehabilitation works.

Even though many such advances have been achieved in the evaluation methods and maintenance or rehabilitation works of airport pavements, the study on the pavement management system of airport pavements in Japan has just begun. For example, with respect to predicting the distress and performance of airport pavements as the most important item for the determination of maintenance or rehabilitation policy, we have just developed a data bank for airport pavements and are now collecting actual data. We intend to continue efforts to study the many related subjects involved in the development of a pavement management system.

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