STUDY REPORT ON LATERITIC SOIL FOR ROAD CONSTRUCTION TECHNICAL

STAGE II

MARCH, 1985

DEPARTMENT OF HIGHWAYS MINISTRY OF COMMUNICATIONS, THAILAND JAPAN INTERNATIONAL COOPERATION AGENCY

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PREFACE

Due to geological conditions in Northeast Thailand, limited suitable rock quarries have caused insufficient road construction materials for a long time. This problem can be solved by stabilizing available local material, lateritic soil, to substitute crushed rock base course.

Many research programmes have been done on lateritic soil-cement stabilization for several years. The first lateritic soil-cement test road was constructed by Siam Cement Company on Highway Rt. No. 24 in Ubonratchatani in Northeast region. Since then, about 1,400 kilometers of soil-cement roads have been constructed in this region. Research evaluation showed that most of the soil-cement roads performed well under low traffic conditions.

Research programme between' Thai Government through Department of Highways, Ministry of Communications and Japanese Government through Ministry of Construction, Japan International Cooperation Agency (JICA) and International Engineering Consultants Association (IECA) started from December, 1981 to March, 1984. A test road, with soil-cement base course, was constructed on National Highway Rt. No. 12 between the City of Khon Kaen and Chum Phae District, with an ADT of about 4,000 vpd, in Northeast Thailand. JICA and IECA have provided some necessary equipments for this project.

The study is aimed to obtain further information of the performance of lateritic soil-cement road under heavy traffic.

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It is divided into 3 stages:

Stage I : the soil-cement base construction and its performance evaluation

Stage II : the asphaltic concrete surface construction and its performance evaluation

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Stage III : the long term performance evaluation. The Stage I study was the joint research programme and the Report was completed in March, 1984.

Asphaltic concrete surface construction finished in November, 1983. Sensors had been installed on top and inside the surface course, respectively. The performance of surface course such as cracks, roughness and rut depth had been checked. Pavement evaluation had been done by Present Serviceability Index (PSI). Temperature, stresses and strains have been analyzed. The conclusion of Stage II Study is presented in this report. It is hoped that the results of this research project will be valuable for future research and application.

The report is accomplished under the cooperation of Japan International Cooperation Agency (JICA) and Department of Highways. Mr. 11. Yoshiaki Murao, the expert from JICA, provided some technical cooperation to the programme. Dr. Teeracharti Ruenkrairergsa, Department of Highways, provided some guidance and technical review throughout the project. The draft report was prepared by Mr. Wirote Panthawanggoon and reviewed by Dr. Pichit Jamnongpipatkul, Department of Highways. Their hard working are acknowledged.

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(Chamlong Saligupta) Director-General Department of Highways Ministry of Communications Thailand

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I. INTRODUCTION

I.1 General

Lateritic soil-cement has been used for road construction in Thailand for almost two decades. However, only in recent years that performance of lateritic soil-cement roads has been extensively studied (Ranananda et al., 1983, Ruenkrairergsa, 1982, 1983).

In order to have a better understanding of the behavior of soil-cement roads, a test section with base course, consisting of lateritic soil treated with varied cement content and two types of thickness, has been constructed. The test section is located on Highway Route No 12 between City of Khon Kaen and Chum Phae District (Fig. I-1). Construction of the lateric soil-cement base and the asphaltic concrete surface of the test section was completed in March and November, 1983, respectively.

During the base course construction, several sensors including strain gages, pressure cells and thermo-couples, were installed beneath the base and subbase course. After the base course construction, but before the surface construction, periodic measurements of stress-strain and deflection under a specified load as well as the temperature variation within the pavement structure were made. The collected data were analyzed and reported in our Stage I report.

During the surface construction in November, 1983, additional strain gages and thermo-couples were installed underneath the surface. The Stage II study, then, began after the

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surface construction.

Flow chart of the study is shown in Figure 1-2.

1.2 Purposes of the Study

The objectives of the study is outlined in the Stage I report and is reiterated here as follows: (1) To study the performance of lateritic soil-

.

stabilyzed with low and high ranges of cement content for being used as a base course of moderately

traffic roads.

- (2) To study the strength development of the soil-cement base while serving the actual traffic.
- (3) To study the induced stress distribution underneath the asphaltic concrete layer.
- (4) To study the deflection of surface and soil-cement base under loading.

(5) To study about the probable substitution ratio of

soil-cement base as compered to asphaltic concrete.

- (6) To check the design method as being employed (CPA method).
- (7) To study about the long term performance of the soil-cement road.

1.3 Outline of Stage II Study

Stage II study began after the surface construction in November, 1983, Additional strain gages and thermo-couples were installed for this study.

The Long Constant of the State Association (1997)



Fig. 1-2 Flow Chart of this Study

In this study, as well as, in the Stage I study, the stress-strain variation within the pavement structure under 1.1 different conditions (for examples, different base thickness, (a, a, b) > 0% cement content, loading conditions) was measured using Static and the second second second second Strain Indicator and Dynamic Amplifier with Pen Recorder. The temperature variation within the pavement structure was measured en de la compañía de e Bashar Sole and a using Pocket Thermometer. The Benkelman Beam deflection was recorded for the surface deflection. Description of the instruments was discussed in detail in our Stage I report.

In addition, Profilometer and Levelling method have been used to measure the roughness and cross-section (rutting depth) of the surface. Selected cored soil-cement samples from the test section have been tested to determine unconfined com-

pressive strength and stress-strain relationship.

1.4 Scope of This Report

In this report, Chapter II summarized the Stage I Report.

Chapter III presented the criteria of asphaltic concrete surface including design, construction and the results of material proand where the state has public the product of the perties. Details of sensors embedments beneath the surface course was mentioned in Chapter IV. Chapter V presented the results of analysis of the measuring data including the results of han setting the state of the set Deflection, Roughness and Cross-Section Form of Pavement, Crack approximate a state of the second second second Results, Temperature Measurements, Static and Dynamic Vertical 建晶体的 物质影响 医胸外部的 计分子 医子子的 化分子子 Stresses, Permanent Strain, Static and Dynamic Horizontal Stresses i (and the algebra is the state of the second and the second second second second second second second second s and Stress-Strain Relationship of Cored Samples. Chapter VI was

the conclusion of this report.

II. SUMMARY OF STAGE I REPORT

11.1 Test Road Detail

(1) The typical cross section of the test road is given in Fig. II-1. The pavement structure consists of the subgrade, selected material, subbase, soil-cement base and surface.

The roadway width is 6.5 m., road width is 11.0 m. The Standard crossfall is 2.5 %. The embankment's side slope is 2:1.

(2) The ADT between Khon Kaen and Chum Phae is estimated at 3,000 to 4,000 vehicles in two opposite directions. Large

vehicles are supposed to account for 30 percent of all the traffic.

The cement content and thickness varied in 8 test subsections. Earth pressure cells, embedment strain gages, and thermo-couples were installed to understand the pavement body behavior (Fig. II-2 and 3).

Lateritic soils were mixed in the field with cements and

water by road construction equipment. Grant and the construction

II.2 Conclusions in Construction and Test in Stage I

The second s (1) Material Properties. In the design stage, Good Soil 有效,在1971年,1991年,1991年,1996年,1996年,1996年,1996年,1996年,1996年,1996年,1996年,1996年,1996年,1996年,1996年,1996年,1996年,1996年 and Poor Soil were classified according to their P.I. To develop field strength of 17.6 ksc. (250 psi.), cement contents of 4.3 % and 12.8 % were needed for Good Soil and Poor Soil, respectively. t a thua tha the late and a stabilizer and that a stable stabilizer band and a first a stability and there a However, during the construction it was not practical to classify te level i in land, and not she can all its , differ that i the set field materials as Good Soil and Poor Soil as expected. As a the state of the second sec result, only one type of material was used instead of 2 types. The filed material possessed physical properties intermediate s is an one of the second to be adapting the fille and the second second second second second second second sec

-6- CARREN REPORT OF A CARRENT STREET



					Chum Phae		Crushed Rock Base	-8							
ŝ	Poor Soil	20 cm	320 2) 	T 40										
<u> </u>	cod Sail	20 cm	520	32	+350		20¢r	, }	14cm	×α) cm				
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4	Sood Soil G	l5 cm	1 20 200 7	32	t 200		Ceme				Materia			ction 4	Section
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condition No.	for Base Pr	cr Base	ength(Ib/f)	32	+ 000	Asphalt Coi	ished ik Bose				·· . · ·		•	<u> </u>	
Test C Subsection	Materia	Thickness	Jaraet Str		К Нол Х		<i>ड≵∕.</i> ∕			- - -					

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Chum Phoe Subbase Selected. Material < Unit: m Base 0.20 **90**0 00 0.20 Subgrade • 32+400 1 8. 83 |⊕ 00 • 35+320 0913 ω Goge (Longitudinal) Goge (Transverse) irre Cell to Couple ¥. Ŕ Ð * • 35+300 <u>55</u> 66 Ð Ø Pressure Thermo <u>@</u>.ß Strain Strain • 3St S20 Location ጄ ያ 8 4 a Sterio - 4 • S • 35+500 N \otimes ٥ Ϊ. Embedment 13 é പ് βm I Ð 4 ŵ • 35+120 3 . orfi jê fas Tî Sensor € I R Sensor 3 • 321100 Station Point Location of Point n in the second secon Second 8. **4** ĕ \overline{N}_{∞} legend > пъ •351050 1.091 • ⊕ Ŧ • 35+000 Subsection No. **GI**:0 610 52 0 MON Scorroge 1059 0011 bee boor 3.5... X thon

between Good Soil and Poor Soil.

(2) Construction Controls. Construction controls indicated that most of the base course thickness were satisfied with the design thickness (15 and 20 cm.). The compaction was not satisfactory. Most of the field water content were on the dry side of Lab's O.M.C. and only half of the samples had maximum dry density of not less than 95 % of Modified Proctor Density.

(3) Unconfined Compressive Strength (UCS) of Soil Cement. The percent of cement contents by weight 4.3, 7.0, 10.0 and 12.8, were designed to use in subsections No. 425, 326, 227 and 128, respectively.

In general, unconfined compressive strength of soll-cement samples increased with the curing time. The strength also increased with percent cement up to 10 %. Beyond the 10 % cement content, the strength was unpredictable.

(4) Benkelman Beam Deflection. Benkelman Beam deflection of the surface of soil-cement base course under the axle load of about 12.3 (27,000 lb) varied from 0.10 to 0.30 cm. (0.04 to 0.12 in.). The duration of curing time did not have much effect on the deflection, but the percent cement content did. As the percent cement content increased to 10 %, the deflection and standard deviation of the deflection decreased.

(5) Crack Ratio. Crack survey was carried out after four and a half month curing. The crack ratio increased with cement

content.

(6) Temperature Distribution in Base Course. Temperature

distribution was measured on the same day of the crack survey. In general, the temperature decreased with depth from the surface.

(7) Static Vertical Stress. Static vertical stress at the bottom of the base tended to decrease with the increase in coment content, curing time, and base thickness. Load dispersion effects, the ratio of stress at the bottom of base to that at the bottom of subbase, were about five to ten.

(8) Dynamic Vertical Stress. In general, the dynamic vertical stresses decreased with the increase in curing time, cement content, and base thickness. However, there were some exceptions. The relationships with speed peaked within the range of

0 to 10 km/hr. and decreased and flattened as the speed increased.

(9) Accumulated Permanent Strain. Strain gages were embedded on trial both in longitudinal and transverse directions (parallel and perpendicular to center line, respectively). The

accumulated strain developed in the gages were recorded. The relation to curing time was divided into 6 stages as follows.

- (a) Strain occurring
- (b) levelling off
- (c) reducing slowly into a tensile condition
- (d) reducing rapidly
- (e) exceeding the measuring range of the gage
- (f) breaking of the gage

(10) Horizontal Stresses Under Static Loading. The horizontal stresses within and the bottom of the soil-cement base showed a tendency for compressive stresses to occur at the upper

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part and tensile stresses to occur at the lower part of the base, The transverse stress data were more scattered than the longitudinal 58.2 ones.

II.3 Conclusion in Supplementary Analysis in Stage I

W. CALE NO.

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(1) Comparison of Measured Values with Computed Value by Multi-Layer Elastic Theory. The vertical pressures measured by pressure cells showed higher values than computed values by the multi-layer elastic theory except for No. 4 pressure cell.

(二、) 相同的 (1) (1) (1) It was evident by the multi-layer elastic analysis that the thickness of soil-cement base course influences the vertical er mennen er allegen handet andere andere andere andere andere allegen andere andere andere andere andere ander pressure at the bottom of base course but it had little influence at the top of subgrade.

(2) Estimate of Elastic Modulus of Soil-Cement Base, It. was difficult to value the elastic modulus of soil-cement base course e der generale eine Patrice en der die het die der Patrice gebet der Berger Patrice in gebergen der Bergender B from the measured vertical pressures because the measurements were and in the provider port of the property property in the property in the property of the property in the property of the prope very scattered.

的复数形式 化过程 化可能加强 电影的复数形式 化分子的 化分子的

n is a thank of the warden, and the defend of the here However, the elastic modulus of soil-cement base course an de la compact de la morte de la compacta de la compacta de la del person was valued as E = 2,000 to 3,000 kgf./sq.cm. by Multi-Layer elastic ersenisionel trentpå (ognorpro analysis, using the adjusted deflections.

(3) Comparison of Measured Values with the Results of Finite Element Analysis. It was confirmed qualitatively that the measured stresses and strains agree with the results of Finite Element Analysis under the condition of cracks in the base course.

(4) Estimate of Durability of Test Road by Shell Design Criteria. The durability of the test road was investigated by the

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an Bhannath

Shell design criteria and it was considered that the test road is suitably durable.

(5) proposal to Reduce the Elastic Modulus of Soil-Cement. It was pointed out that even though the elastic modulus was reduced i.e. elastic modulus of soil-cement base was assumed 1,500 kgf./sq. cm., it was considered that the pavement was safe when the base course was 25 cm. thick and the surface was constructed by dense

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III. ASPHAL/TIC CONCRETE SURFACE

III.1 Types and Composition of Mix and Lange and Antiput terms

Asphaltic concrete could be classified according to the size of aggregates and amount of asphalt cement use. Each mix design/will be suitable for one particular job and objective of the work... Table III-1 shows types of asphaltic concrete and probable amount of asphalt cement (Thum-Umnanysuk, 1982).

III.2 Marshall Method of Mix Design

In the Marshall method, test specimen of 2-1/2 in. height x 4 in. diameter, prepared according to a specified procedure, are used. There are three principal features in the mix design (1) bulk specific gravity determination, (2) density, voids analysis, and (3) a stability-test of the compacted test. The stability is the maximum load resistance in pounds which the standard test specimen will develop at 140.F, when tested, and the flow value is the total movement, in unit of 1/100 in, occuring in the specimen between no load and maximum load during the stability test.

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To determine optimum asphalt content, the test will be varied by 1/2 % increment of asphalt content, with at least two asphalt contents above and two below optimum. Usually three to five test specimens are prepared for each asphalt content used. Asphalt may be expressed as a percentage by weight of total mix or as a percentage by weight of dry aggregate.

Table III-2 indicates the Marshall Design Criteria, presently adopted by DOH. The criteria were recommended by the

-14-

SE GRADE) GRADE C (OPEN GRADE)	8-4 8-5 C-1 C-2 6-3	BASE BUSE SURFACE SUFFACE	2 ⁻ 3.4 2.1 11.2		100 75-100		30-50 30-50 45-85 20-40 20-40	20-35 20-35 5 - 20 5 -	3-12 3-12	A A A A A A A A A A A A A A A A A A A	30.6.0
GRADE B (COAR	8 9 9 8 1	LEVENC EVENC SASE BASE	2/11 .2/1	PERCENT PASSING		00 100 100 100 100 100 100 100 100 100	2 35-55 30-50 30-50	0 1 20-35 20-35 20-35 0 1 40-22 10-22 5-20	310 6-16 6-16 312 5 4-2 4-12 2-8		3 D - 6
GRADE A (DENSE GRADE)	A-1 A-2 A 3 A.A	SURFICE SURFACE SURFACE BASE BASE	r.~ 2. 3.			e-07 001-08 001 001 01 001 001 001 001 001 001 00	55-75 50-70 40-65 45-6	35-50 35-50 35-50 35-50 18-29 18-29 19-30 19-34	13-23 13-23 13-23 13-2 8-16 8-16 7-15 7-15	A O I I I I I I I I I I I I I	3.5-7.0
DESIGNATION		USE	RECOMMENDED MINUMUM COMPACTED DEPTH FOR INDVIDUAL COARSE	SIEVE SIZE	11/2	5/4 1/2" 3/8"		8 ••• •••	3 .€	\$200	ASPHALT CEMENT CONTENT * BY NE TOTAL MIX

Min Max 2500 5000 5000 5000 Min Max Min
elfing 8 3 5 8 78 8 20 3 5 3 5 3 5 3 8 3 8
uling 3 5 5 3 5 8 3 5 8 3 5 9 5 9 5 9 5 9 5 9 5 9 5 9 5 9 5 9 5
3.88

Asphalt Institute in 1962. Figure III-l shows relationship between Minimum Voids in Mineral Aggregates (V.M.A.) and nominal maximum size of the compacted Dense-Graded Paving Mixture.

III.3 Design of the Test Section Surface

The surface of the test section consists of 5-cm. thick asphaltic concrete layer. The asphaltic concrete is designed using Marshall Method Laboratory tests are performed on a densegraded asphaltic concrete mix to be used for the heavy traffic category. The aggregates used in the mix is shown in Fig.III-2, having the maximum nominal size 1/2 in. Figure III-3 shown the results of the tests following DOH'S practice, the asphalt content, as determined for the median of the limits for percent air voids, is used as the initial design value which is checked and adjusted to conform with other criteria.

From Figure III-3, the asphalt content providing 4 % air voids (median of 3-5 % range for the surface mix, Table III-2) is 5.2 %. At this asphalt content, other properties of the mix as determined from Fig. III-3 are as follow.

Density	99.7 % of optimum
Stability	2200 lbs.
Flow	11
Percent Voids in	15
Mineral Aggregates	

It will be noted that the stability value exceeds the minimum of 750 lbs. that the flow value is within the range of 8-16 and the percent voids in mineral aggregates exceed the



Aggregate for Compacted Dense-Graded Paring Mixtures





Sieve sizen a) Sta. 31+825-32+600, Right Lane



b) Sta. 31+700-32+675, Left Lane

Fig. 111-2 Gradation of Aggregate (After-construction)



minimum of 14, The percent voids in the total mix is within the limits. Thus, the mix design have all test properties within the allowable limits.

There are some tolerance in the plant-mix design as follows and the harder as the set of the

Aggregate passing = No. 4 and greater = 5 %

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No. 30 UT. = 3 %

and the state of the No. 200 11 F = 1.8

Bitumen : 0.3 %

a spirate has with the face of the second part to spin as As a result the range of asphalt content allowed for all ante addalation e enclas do tate the plant mix is between 4.9 and 5.5 %.

April 19 Contract and a contract of the

The properties of the mix for the design range are still den an t

within the allowable limits as indicated in Fig. III-3. erened i gelle blanet, og gad

III.4 Surface Construction

and the strength 3.3 Stor 5 121 III,4.1 Equipment

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Followings are the list of equipments used for

S. 195 P. the surface construction.

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- 1 continuous type asphaltic concrete plant

- 4 dump trucks 2.1.

- 1 paver

- 2 steel rollers

en lastatur (r. 8

- 1 self-propelled pneumatic tire roller

III.4.2 Plant Mix

In the plant mix, the amount of aggregates and

asphalt cement content are controlled within the design range to

specified temperature, 325 \pm 15°F, for the aggregates and 333 \pm 15'F, for the asphalt cement. The range of mixing temperture are 290-320°F, STATE STREETS

III.4.3 Construction Procedure

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The hot mix is hauled from the plant by dump trucks as spreaded at the test site by paver. Immediately after the spreading, compaction is performed. Followings are steps of compaction. 13. 164

than 250 F.

. f

(1) Initial or Breakdown Rolling. Steel rollers used to compact the mix. The compaction is required until the smoothness Annal to any provide the part of the spirit of the part of the and neatness of the surface are obtained. the production of stated of the spectrum of The smoothness and neatness of the surface Sec. 2. 200 1.000 are checked by using straightedge. Tempe-્યુ આ સ્વાર્થ કે છે. idoweki Kanada (1.1.1.2. rature during compaction must not be lower

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(2) Second Rolling. Rubber tire roller is 新生活 法公司

applied while the mix is still hot (tempe-经济 化特殊分配 网络印刷的复数印刷

rature is in the range of 170 ± 15 'F.). Calendry Bress (Freetrichter an Areis Chris 1419.6 This steps mill induce maximum density to gydd acerter Allebert the asphaltic concrete surface.

u wigo 2 kalend (3) Finished Rolling. The surface must be made

smooth using the steel rollers while the mix e . is still warm (temperature should be 140 \pm isa yya ge tiken i t 15'F.).

After the compaction, the surface is cured an and and the base

1.16 3.43

for 16 hours before open to traffic. er les beser é la level de Éscher desperar e dans prés

III.4.4 Properties of the Mix After Construction

The properties of Lab-compacted and fieldcompacted samples as compared with the properties in the design range are presented in Fig. III-4. It is indicated that field densities of both lanes are lower than the design range. However, they are greater than 98 % Maximum Density. The % Air Voids and V.M.A. of the field-compacted samples are greater and lower than the design range, respectively.

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IV. SENSOR INSTALLATION

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IV.1 Types of Sensors and Their Locations

Sensors, which were installed within the pavement structure of the test section, are pressure cells, strain gages and thermo-couples. Specification of these sensors are included in Appendix A.

Locations of Sensors are shown in Fig. 11-3.

IV.2 Steps of Sensor Installation

Sam Zeber at a most of the

IV.2.1 <u>Stage I Study</u> In the Stage I study, the sensors were installed within and under the soil-cement base course. Details of the installation was discussed in our Stage I report.

IV.2.2 <u>Stage II Study</u> Only strain gages and thermocouples were installed beneath the asphaltic concrete surface in the subsection nos. 1, 3, 4, 5 and 7.

The steps of installation are as follow.

(1) Before overlaying asphaltic concrete surface,

tacking coat was done by using asphaltic cement. Then, wooden form was placed at the embedment location, as shown in Fig. IV-1 (a). Then wooden form was firmly fixed on the base course.

(2) After overlaying asphaltic concrete surface, the wooden form was removed to provide the strain gage

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space. Fig. IV-1 (b) shows the section of asphaltic

while the asphaltic was still hot.

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(3) Within the providing space, thin layer of fine asphaltic concrete was used to level the surface of base course. Installed location was marked on the levelling surface. Fig. IV-1 (c) shows oot as turks on plan of embedment location. araan aharti giri

(4) Two strain gages were installed at the specified location. Asphaltic concrete was spreaded and com-General Contraction of the pacted in the space. Fig. IV-1 (d) shows this step. Williams in print

Steps: (2), (3) and (4) were done as quickly as possible to obtain the homogeneity of the asphaltic concrete surface course.

Cord embedment was done by digging a ditch along the side slope embankment and collecting the cord in the hole as shown in Fig. IV-1 (a).

an territe company and the second company IV.3 Measuring Instrument and Method

The measuring instrument and method were descriped in วาย และการสาทส์ไสสาว detail in our Stage I report and is reproduced in Appendix B of ele producer del é un l'assistive e la this report.

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V. RESULTS

V.1 DEFLECTION

V.1.1 <u>Recorded Deflection</u>

Benkelman Beam Deflection Method has been used to measure the surface deflection of the test section. The method was described by Ruenkrairergsa (1982). In our studies, the loads being used varied from the standard loads of 18,000 lbs. (8.036 tons) as shown in Table V.1-1. As a result, The recorded deflection was in error to some degrees. The recorded data is then multiplied by a correction factor which is simply the ratio of the standard load and the actual load.

Table V.1-2 shows the average surface deflection and standard deviation of each subsection measured at different period as compared with the soil-cement base deflection. Also, In the same table, presents the deflection of surface constructed on crushed rock base.

V.1.2 Deflection-Service Time

The surface course deflection is plotted against service time in Fig.V.1-1. It is indicated that the deflection tends to increase with the service time. However, from the period of 7 to 10 months, The deflection decreases. It is difficult to see the relationship at relatively short service time, since variation of climate particularly raining will exert large influence on the recorded deflection.

In the same figure, surface course deflection of crushed rock base is also plotted against service time. It can be seen

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WEIGHT		AUG. 7,84	3,637	8,697			
EAM TRUCK'S	DATE	MAY 7,84	3,138	8,659	1621		
I BENKELMAN BI		DEC.208.21,83	3,234	0,585	13,875		
TABLE VI-			FRONT WHEELS	REAR WHEELS	TOTAL		
	· · · · · · · · · · · · · · · · · · ·						

TABLE V.I-2 AVERAGE OF DEFLECTION AND STANDARD DEVIATION

(Unit, mm.

1 1× 4.5 000 8 Ŧ 5 6 Ñ Course to Surface Ratio of Base 80 9 4 30 36 õ ¥ 4 5 4 **9** 12 5 90 F) 3.8 2 4 6.7 80 9.6 6.2 0 4 6° N 4 26 6 E 3.7 Ž Ŷ N ß Q N ង Ŋ ŧ N Ŋ Aug. 6, 1984 0.189 0.0538 *∀ 0.0383 1690.0 6820 0.0614 0.168 0.0668 0.096 0203 0:0441 0345 00071 60010 2520 0.172 ¥ 0.161 33 ø Ŷ June.28, 1984 ø ø ø ø ŵ ω φ 0.249 0.0526 0334 0.0698 A 3 0.266 0.0556 0.190 0.0535 0.0853 0.1797 2610 6220 0.257 0.1083 0292 01144 0.201 0383 X Surface 12 • ¥ 4 4 4 4 4 4 4 Mar. 3, 1984 2660.0 0.0200 -6250.0 0.0494 026. 0.0609 00549 ۲ م 0.0297 0600 024 0.26 10 0.20 0.27 NX NX 017 50 0 <u>5</u> õ Q 2 Q 2 ē Q ð ŧ Dec.21, 1983 0.0434 0.0360 00743 1 Ż 0.0284 2620.0 0.0366 00221 0.1209 01205 0.18 0 7 0.22 0.28 0.28 0.15 220 41.0 222 ١Ÿ Mar.23 to May 16.1983 õ T ღ õ Ŧ ത Ð ≌ g Base Course 0.374 6850 4 م 0.465 0.366 0.214 0358 0.42 1950 0.87 1.03 4 8 ទ្ធ 126 8 õ ŝ ີ ເພີ Thicknee 900 00 ର Ω ଷ୍ପ 8 5 ିନ୍ଦ୍ର 8 £۵ <u>ی</u> Content Camont 8 ଧୁ 8 0.0 2 0 ちち ₩ ₩ 20 õ 12**8** Rock Base Crushed --ю ~ IQ φ æ 4 '9N Subsection 9502 Inemed ~ (los

Number of measuring point or time

Standord Deviation

Average Value

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indicates the measuring date

Note ;

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TIME

DEFLECTION VS. TRAFFIC SERVICE

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that in every case, except the surface course with 20 cm.-thick base having 4.3 % cement content, the surface course deflections of soll cement base are lower than that of crushed rock base.

Fig.V.1-2 shows the plots of left lane and right lane surface course deflections against servicing time for each subsection. Most of these results show that left lane deflection were smaller than the right lane deflections, except in subsection No.8. These effects would be due to the heavier traffic on the right lane.

V.1.3 Comparison Between Surface Course Deflection and

Base Course Deflection

In every case, surface course deflection is much lower than the base course deflection. The ratios of base course deflection to surface course deflection vary between 3.3 and 7.7. These ratios are plotted against % cement content in Fig. V.1-3. In case of 15 cm. base, as shown in Fig. V.1-3 (a), the variation is high. However the ratio tends to decrease as the % cement increases. On the other hand, in case of 20 cm. base, as shown in Fig. V.1-3 (b), the ratio tends to increase as the % cement increases.

V.1.4 Deflection VS. Cement Content

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The ratios of surface course deflections at various % cement content to the deflection at 4.3 % cement content are plotted against the % cement content in Fig.V.1-4. For comparison, the ratios of base course deflection at various % cement to the deflection at 4.3 % cement content are also plotted against

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FIG. V.I-4 RATIO OF DEFLECTION VS. %CEMENT CONTENT

% cement content in the same figure.

As seen from the figure, in case of 15 cm. base, the surface course deflection shows the variation as compared to the base course deflection. However, in case of 20 cm. base, the surface course deflection shows similar tendency with the base course deflection. The ratios are smaller.

V.1.5 Standard Deviation VS. Cement Content

Fig. V.1-5 Shows the raltionship between average deflection and standard deviation. In general, the standard deviation, δ_i tends to increase as the average deflection, \bar{x} , increases.

There is a board relationship between these two parameters, as

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shown below

 $= 2/3 \ddot{x} - 0.093$



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0.1 0.2 0.3 0.4

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FIG. VI- 5 STANDARD DEVIATION VS. AVERAGE DEFLECTION

(a) A set of the se

V.2 ROUGHNESS OF SURFACE COURSE

3-m. profilometer, of "Intergrating Irregularity Recorder, Type KKy-3T" manufactured by Tokyo Tanifuji Co.,Ltd. with the auto-readingdevice, was used to measure roughness (Fig.V.2-1). The roughness measurement was done in May and August, 1984.

An example of profile recorded by the 3-m. profilometer is shown in Fig. V.2-2.

V.2.1 Measuring Method and Recorded Roughness

Determination of roughness variation (0) is shown below.

- (1) Record roughness of the surface using 3-m.
 - profilometer for the whole appointed section;
- (2) Read elevations of recorded profile waves from the base line at 1.5 m. intervals in the recorded chart.
- (3) Divide all reading-values into groups from which each group has 6 reading values.
- (4) Find the differences (R) between maximum and miniumu values of individual groups.
- (5) Calculate their average values (\overline{R}) by dividing the total sum of R by the number of groups in the appointed section.

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 $\vec{R} = \sum R/n$ ------(1) where $\vec{R} =$ average difference in some section $\sum R =$ measured individual difference of each group

= number of group

(6) Compute the roughness deviation (\acute{O}) using the



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Station No.	эст m 110.5 1	12 13	.5 115	116	5 118	611	5 21	8	51 12	12	55	2 2	8.5
(mm)			•••••	- }		- 			 ,				
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Reading (mm)	ũ	12.5 1.	1 1 1	8	5 12	12	4	4		5		3	4
R-value (mm)	<u>R</u> i * [3.5-2=	1.5	0		æ	- 1 ∓1	5-11.5	= 3.5				
	Note)	Meosu	buur	Date	: May	1984							
		Meast	led	locatio	о, — С	Station al test	No: 32 Ine / 0	2+000 WP	~ 32+	8			
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Exemple of Profile Recorded by 3.m-Profilometer Fig. v. 2-2

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equation below.

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R/C Coefficient fixed in accordance with measurements involved in the group in case that the number of measurement

involved in a group is 6, C is 2.53 Table V.2-1 shows calculated roughness deviation for each

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subsection.

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V.2.2 Roughness Deviation

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The roughness deviations from OWP in both lanes are averaged for each subsection and plotted in Fig. V.2-3 in general, the roughness deviation for each subsection is relatively low particularly in subsections No.2,5,6 and 7. The deviations at 9-month old are also compared in the same figure. In all cases but subsection No.3. the roughness deviations at 9-month old are greater than those at 6-month old.

The roughness deviation can be used to indicate the serviceability. Table V.2-2 shows the relationship between the roughness index and service index as proposed by Koona (5).

V.2.3 Roughness Index

Roughness Index (RI) is calculated using the equation below.

	RI	a a	$\sum ai/L$ (cm./km.)(3)
where	Zai	1	Accumulated deviation of the measuring
:•	. .	2	wheel in one direction of either up or
			down by recording counter (cm.)

Measured distance (km.)

Table V.2-3 presents the calculated Roughness Index for the

whole soil-cement section. The Roughness Index can be used to

-41-

Measuring		Meosu	red.			Ans.	Section	No.				
Date	2 .	Locat	ion	-	5	3	4	5	9		8	Averac
		Right	owe.	1.66	1.II	1.82	1.54	1.34	16.0	1.26	42	88
fay 1984	6 months	Lane	IWP	0.55	0.71	111	1.15	0.67	0.63	0.83	0.71	080
алан 1 А.,	n lest Na	Left	1WP.	1.70	1.34	0.87	0.95	107	0.75	0.87	66.0	101
-		Lane	OWP	0.95	0.87	66.0	1.15	16.0	1.46	1.19	1.98	51 1
		Aver	0 0 0	8 1 1 1	101	1.20	1.20	1.00	0.94	104	1.28	
		Right	OW P	1.54	1.23	1,74	1.66	38 2	1.07	1.58	6]`]	4
g. 16 1984	g months	Lane	IWP	1.34	0.83	079	1.03	1.07	1.26	1.42	1.62	211
- - 		Left	WP	1.86	611	0.95	66.0	1.03	16.0	1.30	1.82	1.26
		Ę	OWP	0.82	1.26	1.03	4.15	0.67	1.03	103	1.36	Š.
• . -		Avero	æ.	1.39	1.13	<u>د اع</u>	121	8	20.1	1.33	1.50	Ĩ.
	Note :	Age : C	ount fro	in the S	urface o	ourse (construct	in N	ovember	1983		
· · ·	· · · ·	owe: C)ut When	el Path								
•	• •		 									

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Table V.2-1 Recorded Roughness Deviation

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		2			
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FIG. V.2-3 ROUGHNESS DEVIATION OF EACH SUBSECTION

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TABLE V.2 - 2 RELATIONSHIP BETWEEN ROUGHNESS DEVIATION AND SERVICE INDEX (KOONO)

					(UNIT; m.m.)
MEASURING INSTRUMENT		A	₿	C	D
STRAIGHT	EDGE	0 - I.IS	I.16 1.75	1.76 - 2.60	2.61 -
3 - m. PROFILO	METER	0 - 1.50	1.51 - 2.40	2.41 - 3.70	3.71 -

(UNIT; m.m.)

NOTE A. B ; IT IS DIFFICULT TO DISTINGUISH BETWEEN A AND B

, C) -

1

IN DAYTIME , HOWEVER SPOTTING HEAD - LIGHT OF A CAR IN NIGHT TIME, B LOOKS LIKE A LITTLE WORSE THAN A C ; COMPARING WITH A AND B , C LOOKS LIKE WORSE IN ROUGHNESS, PASSING BY A CAR, VIBRATION CAUSED BY UNEVENNESS (ROUGHNESS) IS SLIGHTLY FELT, D & ROUGHNESS IS CLEARLY BAD AT SIGHT, PASSING BY A

CAR , VIBRATION CAUSED BY UNEVENNESS (ROUGHNESS)

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 $= \sum_{i=1}^{n} \sum_{j=1}^{n} \sum_{j=1}^{n} \sum_{i=1}^{n} \sum_{j=1}^{n} \sum_{i=1}^{n} \sum_{j=1}^{n} \sum_{i=1}^{n} \sum_{j=1}^{n} \sum_{j=1}^{n} \sum_{i=1}^{n} \sum$

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Table V.2 - 3 Recorded Roughness Index

evaluate the surface condition as shown in Table V.2-4.

Date: 1

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Fig. V.2-4 shows the relationship between the roughness deviation ((d) and roughness index (RI). The broad relation is derived from these data, is shown below.

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		0F RI	EVALUATION	Excellent	Vary Good	Good	Bod		
		EVALUATION	n/km.)					•	
		sble V. 2-4	RI (G	0.~30	20~100	100~200	0ver 200		·
		F				I I	r i		• • • •



V. 3 CROSS SECTIONAL FORM

V.3.1 Measuring Method

The cross sections of all test subsections were surveyed in August, 1984 by levelling method. The line of measurement was 10 m. apart. The measuring interval was 20 cm.

-49-

Fig. V.3-1 shows the average cross sections for each subsection. In the same figure, the designed crossfall of 2.5 % is also shown.

V.3.2 Deviation from the Design

The deviation from the design on each surveyed line is calculated using the equation below.

$\bar{\mathbf{Y}} = \mathbf{\Sigma} \mathbf{Y} \mathbf{i} / \mathbf{n}$

 \hat{Y} = the deviation on each surveyed line $\sum \hat{Y}$ = vertical deviation in absolute, mm.

n = number of measured point

The deviation for the test section are calculated and plotted in Fig. V.3-2. The average deviation in the left lane of subsection 1 is the lowest, about 3.6, and in the right lane of subsection 2 is the highest, about 18.9 mm.

V.3.3 Rut Depth

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Rut depth for each subsection is determined from the cross-section plot presented in Fig. V.3-1. The results are summarized in Table V.3-1. The measured rut depth of the test v ited. (4) is section ranged from 0 to 3 mm, which is relatively low,

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FIG. V.3-1 CROSS-SECTION

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Table V. 3-1 Rut Depth

5 000-050 050-100 100 -150 150 -200 200-250 250 -300 300-350 350-400 52 N -Subsection No. and Station No. 32+___ 2 ø N Ň -P ø -بليه ŝ **8**1 2 9 10 *) 2 ю Ю Į, 0 and the second 1.25 N ••••• 0 **si** Ņ -OWP WP **OWD** dMI Medeurad Average Loothon Hoge -÷. throm the Ş Medeuring -Aug 16 Deta **198**

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Note; Age count from the eurface constriction in November, 1983

OWP Outer wheel path

EWP times wheel poth

(Unit , m.m.)

V.4 CRACK RESULTS

V.4.1 Recorded Cracks

Crack survey was carried out on May 8, 1984, 6 months after surface construction. It was observed that most of the cracks were longitudinal. Lane joint crack was remarkable in subsections 2 and 4. Edge crack close to shoulder was remarkable in subsections 3,7 and 8.

The investigated area was divided into 1.0 x 1.0 m.square meshes. The meshes in which cracks developed were recorded according to the classification in Table V.4-1. The classified cracks of all subsections are presented in Fig. V.4-1. The crack area and crack ratio were then determined. The crack ratio was calculated using the equation below.

alculated using the equation motor.

Crack ratio (%) = $\frac{\text{Crack Area } (\text{m}^2) \times 100}{\text{Investigated area } (\text{m}^2)}$ -----(1)

Result of the calculation are presented in Table IV.4-2.

V.4.2 Crack Ratio VS. Cement Content

The relationship between crack ratio and cement content is plotted in Fig. V.4-2. The crack ratio in every subsection is less than 7% and tends to increase as the cement content. However, as the cement content over 10 %, the crack ratio decreases.

In the Table V.4-2 is shown the plot between the crack ratio of all subsections including the results of before and after surface construction. In all cases, the crack ratio of base course is higher than that of the surface course, ranging from 3 to 73.

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Fig, V4-1 Classified Crack.

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		Crock	Ratio	к. (%)	0.7	6.1	40	2.6	2.4	1.7	5.3	2.6	e Construct Course
		Totai	Ared	(m ²)	\$6.1	18.3	12.05	7.85	7.3	5.2	16.0	7.8	er the Bos
			~₽ ₽	Areo (m)	0.75	1.2	L.35	0.75	0.6	0.3	0.6	1.2	d Matter Su
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V.4.3 Crack Ratio and Roughness Deviation

The relationship between the crack ratio and roughness deviation are used to evaluate the surface condition. Ministry of Construction of Japan (6) has divided the surface condition into 7 different classes according to the crack ratio and roughness deviation. For each class, the maintenance alternative is recommended as shown in Fig. V.4-3.

The data obtained from the test section are plotted in the same figure. It indicates that the surface is in good condition although subsection No.2 might need surface treatment work.

V.4.4 Present Serviceability Index

The concept of Serviceability Rating was introduced to evaluate the smoothness and riding ability of pavement in the AASHO Test Road (7). Based on this concept, Japan Road Association (6) has proposed the Present Serviceability Index (PSI) equation below.

PSI = $4.53-0.518\log 6 - 0.371 \sqrt{C} - 0.174 D^2 - (2)$

where PSI = Present Serviceability Index

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C = Crack ratio (%)

D = Average rut depth (cm.)

Japan Road Association has also recommended the maintenance for the PSI calculated from the above equation as shown in Table

V.4-3.

PSI of all test subsections are calculated and presented in Table V.4-3 and Fig. V.4-4. The PSI can be used to evaluate



- II Surface treatment or do nothing.
 III Surface treatment.
 IV Overlay or surface: treatment.
 - V Overlay

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- VI Whole reconstruction or overlay with
 - partial reconstruction or overlay
- 3211 Whole reconstruction or overlay with.
 - portial reconstruction.
- Note; () Subsection No. 1

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Fig V 4 - 3 Maintenance Alternetive for Asphalt Pavement with Crack Ratio and Roughness Deviation.
	Roughness	Crack		Calcula	tion Process		
Sub Section	Deviation	Ratio	Rut Depth				
CZ Z	7	ç	Ċ	0.518	0.371	0.174	PSI 1
	(mm.)	(%)	(m.)	۲og ک	٩	D ^S	
	1.39	α 1	0.150	0.074	0.310	0.00 4	4 142
N	1,13	9	0.125	0.027	916.0	0.003	3.584
ра 19 2 в 19/3 ст 2 в 19/3 ст	£1.1	0 4	0.23	0.027	0.742	0.008	3.753
4	1.21	26	0.150	0.043	0.598	0.004	3.885
2	1.04	2 4	0.175	• •	0.575	0.005	3.941
g	1.07		09120	0.015	0.484	0.004	4.027
7		23	0.125	0.064	0.854	0,003	3.609.
8	05.1	2.6	00100	160.0	0.598	0.002	3.839
Note;	ъ В С С С С С С С С С С С С С С С С С С	3 - 0.518 Log	Y -0.371 S	<u>c</u> - 0.174 D			
M 	Crack surv	ey was done in	May, 1984				

Data of Roughness and Rut Depth were collected in August , 1984

- 3 Present Serviceability Index (PSI)

Table V.4-3



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V.5 TEMPERATURE MEASUREMENT

V.5.1 Temperature Distribution

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Temperature distribution in the pavement structure, form the surface course to the bottom of base course, was measured in May, 1984. The measurements were performed from 7 a.m. to 5 p.m. Results of the measurements are shown in Fig. V.5-1. The surface temperature varies from 29°C to 55°C., although the air temperature showed much smaller variation from 27°C, to 39°C. At bottom of the surface, the temperature was the highest. Below this, the range of temperature variation was narrow. The distribution is best seen from subsections 4 and 5 from which several thermocouples were embedded in many levels.

For comparison of the temperature distribution at various period of the year, hourly temperature measurements was performed on subsection 4 in November, 1983, May, June and August, 1984. The results are shown in Fig. V.5-2. All the measurements showed similarities in the distribution. The temperature variation in November was the lowest. This was due to cool weather at that period.

V.5.2 Temperature VS. Time of the day

In general, the temperature increased with time since starting the measurement 7.00 a.m. until the peak was obtained at about 3.00 p.m. At the bottom of the surface, the temperature was the highest. Selected temperature variation in the day was shown in Fig. V.5-3.



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V.6 Static Vertical Stress

Pressure cells have been used to measure vertical stress. Cell no. 1,2,4 and 5 and 3 have been embedded at the bottom of Base and Subbase Courses, respectively.

V.6.1 Vertical Stress Vs.Age

Fig. V.6-1 has shown this relationship in subsection nos. 3,4,5 and 6. Stress range had varied from 0.5 to 8 KSC. Tendency of stress had been decreased upon the increasing age. Especially, after surface course construction, Vertical stresses have reduced about 2-3 times of the early age. Load condition A had been tested by Benkelman Beam Truck.

The relationship of vertical stress at the Bottom of Base Course VS. Age has shown in Fig. V.6-2. Vertical Stress in 15-cm. base thickness; for 4.3 % cement content had bigger stress than 7 % cement content. But in 20-cm. base thickness, before surface construction, Stress of 4.3 % cement content had smaller than the other type. Stresses in 15-cm. base thickness had bigger than those in the other thickness.

V.6.2 Load Dispersion Ratio VS. Age

Before surface course construction, there was big difference between vertical stress at the bottom of base and subbase. Fig. V.6-3 has shown the relationship of Load Dispersion Ratio and Age. This effect tended to reduce as the increasing age. The ratios were ranged from five to ten.







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V.7 Dynamic Vertical Stress

Four and one earth pressure cells were embedded at the Bottom of soil-cement base course and subbase course, respectively. Cell No. 3 was at the bottom of subbase course. Various weights of Benkelman Beam Trucks had been used in the tests. Dynamic vertical strains were recorded by means of Dynamic Strain Meter and Amplifier.

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V.7.1 Dynamic Vertical Stress VS. Speed

With various ages, these relationships had plotted in case of Load Condition A and B as shown in Fig. V.7-1 (a) to (c) and Fig. V.7-1 (d) to (f), respectively. Their trends were peaked within the range of 0-10 K.P.H., after surface construction. Both types of Load conditions, stresses of Cell No.3 were the smallest value,

V.7.2 Effect of Different Ages.

The stresses of each pressure cell, of both Load Condition, had shown some doubtful results. Their stresses ranged from 0.5 to 3.7 KSC. Cell No. 3 had shown small range of stress, but another did big range, as shown in Fig. V.7-2 (a) to (j).

V.7.3 Vertical Stress VS. Age

The relationship of verticle stress and age of Cell No.1,2,4 and 5 at speeds of 10, 20, 30 and 40 K.P.H. were used Fig. V.7-1 as references. Only Load Condition A has showed in Fig. V.7-3. Age has been counted after surface course construction. Vertical stresses of Cell No. 4 and 5 had decreased at 6 months age and they increased again, except Cell no.2. For Cell no.1, Vertical Stresses increased as the increased age.







Fig. V.7-2 Effect of Different AGES

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Hg. V.7-2 (cont.)



Fig. V.7-2 (cont.)





Fig. V.7-2 (cont.) 1 (1000 - 1000)







V.7.4 Vertical Stress VS. Cement Content

Their relationships at various ages have presented in Fig.V.7-4. Both types of thickness, the more the cement content increased, the less the vertical stress was, at age 1.5 months, as shown in Fig. V.7-4 (a) and (d), And also in Fig. V.7-4 (f), vertical stresses had shown the same performance. But in Fig. V.7-4 (b), (c) and (e), stresses had shown the opposite performance.

V.7.5 Vertical Stress VS. Thickness

Fig. V.7-5 has shown these relationships. Both types of cement content had performed that the deeper the thickness was, the less the vertical stress was, except in Fig. V.7-5 (c). Most of all ages, stress trends had shown the same performance.

V.7.6 Load Dispersion Effect

The relationship of the ratio of stress at the bottom of base to that at subbase and speed of Load Condition A and B had shown in Fig. V.7-6. These ratios tended to decrease as the increasing speed and age. Load Condition A, the ratios had bigger range than the other one.

V.7.7 Effect of Stress Ratio of Loading Condition A/B

Fig. V.7-7 has shown the relationship of stress ratio of Load Condition A/B and speed, at various ages. All ratios were positive values. Their boundaries tended to decrease as the speed increased. At age 9 months, the boundary ratio was the smallest one, and each ratio value closed to be 1.00.



SERVICE HARD DEPENDENCY ...









V.8 Accumulated Permanent Strain

V.8.1 Durability of Strain Gage in Soil Cement Base

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The strain gages being used in this study, according to the manual, have tensile strain limit about 0.2 % (about 2,000 micron). From our experience, strain values up to-10,000 micron could be measured. In this report, the upper strain Limit is specified at -8,000 micron. The durability of a strain gage was then determined for three specified strain Limits = 2,000, over -4,000 and over -8,000 micron. The age at which a strain reading over the Limits was recorded and presented in Table V.8-1 from which the survival ratio at various time was determined. Survival ratio of gages was calculated from the equation below.

Survival Ratio of gages, %

1.1

Number of gages within Strain Reading Limit x 100

Total Number of Embedded Strain gages

Table V.8-1 shows the survival ratios for the specified strain limits. Fig. V.8-1 presents the survival ratio against age. The survival ratio at 50 % for strain limits of -8,000, -4,000 and \pm 2,000 micron is obtained at 10, 7.5 and 4 months, respectively. Thus, all the strain measurement should be completed between 5-7 months after construction to have the survival ratio over 75 %.

V.8-2 Accumulated Permanent Strain in Soil Cement

Less than half of the embedded strain gages are survived. the measurement of the survived strain gages shows the general

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Toble V.8-1 Survival Ratio of Gage with Passed Monts



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Fig. V.B.- I Survival Ratio of Gage vs AGE

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Fig V8-2 Permonent Stroin VS AGE

tendency as discussed in our Stage I Report. Fig.V.8-2 shows, for example, the permanent strain of gage No. 46 plotted against age.

V.8.3 Accumulated Permanent Strain in Asphaltic Concrete

The strain reading strated on November 9, 1983, The measurement was performed several times during the first few days. The results indicated that the strain developed during first few days was fluctuated, This is probably due to the temperature variation.

Strains of Gage Nos.51 and 52 are plotted against the temperature in Fig.V.8-3. There appears to be some linear relationship between these two parameters. Using linear regression analysis, the relationship between developed strain and temperature of Gage Nos. 41,42,51 and 52 are shown in Table V.8-2.

Since Gage No.41 shows poor correlation. The result has been discarded. It appeared that the temperature correction factor would be about 70/°C. This correction factor will be applied to determine the accumulated permanent strain developed in the asphaltic concrete.

Fig. V.8-4 shows the relationship between the accumulated permanent strain and age of Gage Nos.41, 42, 51 and 52. The adjusted temperature is 40°C. The strain was compressive at early ages. However, there is tendency that as the age is older, the strain will change to tensile. This is shown clearly by Gage Nos. 51 and 52.





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V.9 Static Horizontal Strain

As mentioned before, there are three types of loading conditions. Those are Load Condition A,B and C. And also, Longitudinal and Transverse Gage have been embedded in the test sections. Benkelman Beam trucks have been used in the tests.

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V.9.1 Static Strain VS. Age

Only the sample of Load Condition A and embedded strain gages on the top of soil-cement base course had shown in Fig. V.9-1. The tensile strain of Longitudinal Gages increased as the age increased. And Transverse Gage's strains reduced as the increasing age.

V.9.2 Distribution of Static Strain

Because of small number of survival strain gages in all sections, only Longitudinal Gages in subsection No.5 and 7 could show strain distribution in the pavement. Fig. V.9-2 has presented the distribution of static strain with various ages and load conditions. Generally, the strains at the top of base course have tended to be greater than those at the bottom of base course. Most cases, tensile strains had increased at the upper gages and compressive strain had increased at the lower gages as the increasing age, except in case of Load Condition A&B of strain gages in subsection No.1. (picking up Transverse Gage No.12 and 14).

V.9.3 <u>Strain Ratio or Stress Ratio at the Bottom</u> To at the Top of Base Course

It could say the strain ratio or stress ratio because the Young's Modulus of Eleaticity (E) of all strain gages have




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the same value (E=28,000 KSC.). These stress ratios and cement content relationship has presented in Fig. V.9-3. Only three types of cement contents (4,3,10 and 12.8 %) could be figured out the stress ratios in subsection No.5,7 and 1, respectively. Subsection No.5 and 7 had calculated from Longitudinal Gage No. 51 & 55 and 71 & 75; respectively. Only Transverse Gage No.12 & 14 could be represented the results of subsection No.1. Fig. V.9-3 has shown the comparison of Unload and three kinds of Load Conditions, with ages 1.5 and 9.5 months after surface course construction. The ratios in cases of 4.3 and 10 % cement content were less or equal to one at age 1.5 months. And older age, the ratios of 4.3 and 7% cement content had been constant and bigger results, respectively. But 12.8 % cement content, stress ratios had been big and small range in the early and elder ages, respectively.

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Fig. V. 9-3 Stress Rotio at the Bottom to at the Top of Base Course

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V.10 Dynamic Horizontal Stress

After surface course construction, subsection No.3,4 and 5 were selected only the results of strain gages at the bottom of asphaltic concrete surface course. Those Longitudinal and Transverse strain gages are Gage No.31,41 & 5, and 32, 42 &52, respectively. Both Load Conditions were shown the results with various ages. The sign convention : tensile and compressive stresses are negative and positive, respectively.

V.10.1 Horizontal Stress VS. Speed

Fig.V.10-1 (a) to (c) and (d) to (f) were represented these results in case of Load Condition A and B, Respectively. Most of their relationship were tensile stresses, except Gage No. 41 and age 1.5 months. Most of the stresses had small ranges of stresses. Only static stresses (atO.K.P.H.) had wide ranges of stresses. Their peak's trends were within speed's range of 10 to 20 K.P.H., except age 6 months of both load conditions.

V,10.2 Effect of Different Ages

Comparison of the relationship of the Horizontal Stress and Speed at various ages of each strain gage had shown fluctuation results. The more speed increased, the less the tensile stresses were. There were no big stress difference at various age. Gage No.42 had been broken since July 11, 1984., as shown in Fig. V.10-2.

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Fig, V. 10-2 Effect Of Different Ages





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Fig. V. IO-2 (Cont.) 200 10 10 10





Fig. V. 10-2 (Cont.)







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V.11 Stress-Strain Relationship of Cored Samples

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Coring Machine had used to cored the soil-cement base before surface course construction. Less cored samples had obtained because soll-cement base course probably had less homogeneous There were four cored samples from subsection No.2,4 and 6, as in Table V.11-1. Fig.V.11-1 had represented the stress-strain relationship of all samples.

-108-

The biggest density of the sample was from subsection No.6 at 3.50 m. Lt. of Center Line of Sta. 32+268(density = 2.363 gm./cc.) and this sample had the biggest E of soil-cement (E = 4663 KSC.). Those E results of cored samples could not find the average value of E because each sample had different cement content.

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Fig VII-1 Results of Stress - Strain Relationship of Core Samples.

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The study in Stage II started from the completion of the surface. Most of the data collection in this study were within 10 months after the surface construction.

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 Asphaltic concrete surface of the test section was constructed 7 months after the completion of soil-cement base.
 The asphaltic concrete was designed following the Marshall method.

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2. Results of material testing after construction indicated that the surface construction was satisfactory. Properties of the mix were in general conformed with the design.

3. Benkelman beam deflection of surface course with soilcement base was lower than that with crushed rock base, verying from 0.01 to 0.04 cm. The variation of the deflection of the surface course with 15-cm. base was high. On the other hand, the deflection of the surface course with 20-cm.base showed tendency that it decreased with % cement content.

4. Roughness of the surface course was measured using 3-m. profilometer. Roughness deviation and roughness index were then determined. The results indicated that the surface course was in very good condition.

5. Cross sections along the test section were surveyed by levelling method. The crossfall was deviated from the designed crossfall of 2.5 %. The deviation of the crossfall from the design along the surveyed line varied from 3.6 to 18.9 mm.

> The rut depth was also measured varying from 0 to 3 mm. 6. The crack ratio in every subsection was less than 7 %

and tended to increase as the cement content increased. However, as the cement content over 10 %, the crack ratio tended to decrease.

7. The crack ratio and roughness deviation were plotted together to determine surface condition. It was shown that all subsections were in good condition.

The crack ratio was also used with the roughness deviation and rut depth to determine PSI (Present Serviceability Index) of the test section. The calculated PSI varied from 3.50 to 4.14 which was relatively high.

8. Temperature measurement within the pavement throughout the day showed that the surface Temperature varied from 23°C, to 55° C. and was peaked at about 3 p.m. At the bottom of the surface, the Temperature was the highest.

9. The vertical stress at the bottom of the base was much reduced than the stress developed at the early age. This was due to the addition of surface layer and the increasing in age of soil-cement.

The stress at the bottom of base with 4.3% was smaller than that of 7.0 %, in both thickness.

10. The vertical stress under dynamic load showed tendency as without the surface course, except that the magnitude of the stress was reduced. The stress was peaked within the range of

0-10 km./hr. and decreased and flattened as the speed increased. 11. Less that half of the embedment strain gages in the soil-cement base were survived. The strain developed in similar fashion as without the surface course.

Five strain gages were installed beneath the surface. The

results of measurements showed that the strain was compressive at early age and change to tensile as the age increased.

12. For static Horizontal Strain, only the sample of Load Condition A, as the increasing age the tensile strain increased and reduced on Longitudinal and Transverse Gages, respectively. Generally, distribution of strains showed that tensile strains had increased at the upper gages and compressive strains had increased at the lower gages.

Strain Ratio at the Bottom to at the Top of Base Course could figure out from the gages in subsection nos. 5,7 and 1 with 4.3, 10 and 12.8 % cement content, respectively. At early age the ratios of 4.3 and 7 % cement content had been constant and bigger results, respectively.

13. Dynamic Horizontal Strain showed only the strains at the bottom of asphaltic concrete surface course of strain gages in subsection nos. 3,4 and 5. The relationship of stress and speed, their peak's trends were within speed's range of 10-20 K.P.H., for Load Condition A and B. These relationships at various ages of each strain gage had shown fluctuation results. The more speed increased, the less the tensile stresses decreased.

14. Less cored samples of soil-cement base could be done only in subsection nos. 2,4 and 6 before surface course construction. Stress-Strain Relationship of Cored Samples had been done to find the Young's Modulus, E, of each sample. The densities of samples were 1.977 to 2.363 gm./c.c. (123.36-147.45 lb./ft³.) and E values were 2700-4663 kg./cm.²(38,400-66,300psi.). The average of E value could not find because each sample had different cement

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APPENDIX A

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ltem	Description
i Model	BE-5kA
2 Capacity	5 kg/cm ² (71 psi)
3. Diometer of Pressure	Ø 88 mm
Sensitive Surface	
4 Disc Thickness	20 mm (08in.)
5. Disc Diameter	Ø 100 mm (4 in.)
6. Length of Cable	IOm (33 ft.)
7 Manufacturer	Kyowa Electronic
	Instrument Co., Ltd.
	Japan

Table A-1 Specification of Earth Pressure Cell

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14 (* 4 1	ltem	Description
o e ho	Model	KM ~ 120 • H7 - IL LIOO • 3
2	Resistance	120±1%
3.	Dimensions	120 x 15 x 4,5 (mm)
4.	Modulus of Elasticity	28,000 kg / cm²(4x10 psi.)
5	Length of Cord	10 m. (32.8 ft)
6.	Monufacturer	Kyowa Electronic
		Instrument Co, Ltd.
		Japan

Table A-2 Specification of Strain Gage.

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APPENDIX B

B.1 Measuring Instrument and Method

B.1.1 Permanent Strain

B.1.1.1 Instrument

1. Bridge Box

2. Static Strain Indicator (Type SM-60D,

Kyowa)

10 B.1.1.2 Connection (see Fig.B-1)

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m OO}(\eta_{\rm OO})$). I. First connect terminal of strain gage

embedded with Bridge Box.

2. Second; connect the bridge box cable

with Static Strain Indicator;

B.1.1.3 Method

Permanent Strain is measured under no vibration description loading condition above the strain gage.

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B.1.2 Static Strain and Vertical Static Stress

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B.1.2.1 Instrument

The same as B.1.1.1. In case of vertical static stress, only Static Strain Indicator.

B.1.2.2 Connection

The same as B.1.1.2. In case of vertical static stress, connect earth pressure cell cable with Static Strain Indicator directly.

B.1.2.3 Method

Static strain and vertical static stress are measured with various weights of Benkelman Beam Truck as shown in Table V.1-1 and under 3 different loading condition as shown in Fig.B-2

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Table. B-1	,≗. M	ean	ing	of	Ind	lica	lion	+	, 7	for	Str	ain	Read	ting.	

Com	ection		Macolog
Sensor	Meter	Ngu	Wannuð
	Static Strain		Compression
Strain Gage	Indicator		Tension
Pressure	Static Strain		Tension
Cell	Indicator	-	Compression
	Dynamic Strain	+	Tension
Smain Gage	Indicator		Compression
Pressure	Dynamic Strain	÷	Compression
Celi	Indicator		Tension

-121-Reor Tyres or Transverse Longitudinal Gage Goge (a) Condition A. (Understhe Regrestyres) in the state of the sector of the sector of the many selected the profession because addition of Rear Tyres 1: St. C. Artes de 1115 ides, charlenes. Pales (F) (Frankes) and a 영화 문제하 Alexa St. n sanco delas And frank in a start 计通过 法相关 (b) Condition B (between the Rear Tyres) ergenerige investoren die opgenig geboord geboorde is A state of the second second second second Front Tyre 14.1 Sec. 1 计分词 计算机 (c) Condition C (under the Tyre } Front

Flg. 8-2. Three-kind of Loading Conditions

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- B.1.3.1 Instrument
 - 1. Bridge Box
 - 2. Dynamic Strain Amplifiers (Model DPM-311 A, Kyowa)
 - 3. Pen Recorder
 - (Model Multicorder, MC-6620, Watanabe)

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1. 1. 1.

- B.1.3.2 Connection (See Fig. B-3)
 - 1. First, connect terminal of strain gage enbedded with Bridge Box.
 - 2. Second, connect the Bridge Box cable with Dynamic Strain Amplifiers.
 - 3. In case of earth pressure cell, connect
 - the cable with Dynamic Strain Amplifiers

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- directly.
- 4. Connect each channel of Dynamic Strain
 - Amplifiers with Pen Recorder.

B.1.3.3 Method

 Mark the survey point (the place where gage and cell are embedded) on the pavement surface.

lanas

- 2. Use the same vehicle as static strain measurement as shown in Table V.1-1.
- 3. Measure the weight of front and rear axle.
- 4. Prepare the measuring instruments and connect them.
- 5. Make the vehicle run as if the center of rear tires passes right on the survey

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point. The loading condition is the same as condition B. in Fig. B-2.
6. Change vehicle speed from 5 km/hr. to 50 km/hr, at intervals of 5 km/hr.
7. Make the vehicle run 2 times under the same speed.

B.1.3 Temperature

B.1.3.1 Instrument

Pocket Thermometer (Model 2542 Tokogawa Electric Works, Japan)

B.1.3.2 Connection

Connect terminal of thermo-couple installed with Pocket Thermometer.

B.1.3.3 Measuring Date

Temperature in base has been measured at the same day as permanent and static strain measurement. Particularly on Aug, 13, 1983, the temperature in base were measured from 7.00 am to 5.00 pm. every hour in order to understand the variation in the day time.

B.2 Interpretation of Reading

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B.2.1 Reading of permanent Strain

B.2.1.1 Definition of Curing Time

The soil-cement base construction in every subsection was finished in the evening. In the end of the construction, strain gage, earth pressure cell, thermo-couple were installed. Initial reading (Initial Measured Value) was measured at about 9 a.m. in the day after the construction day. Therefore, curing time is defined to start from the day after the construction day as the initial day (0 day curing time)

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B.2.1.2 Permanent Strain

Permanent strain is calculated as follows $E_p = E_m - E_i$ (1) where $E_P =$ permanent strain

 E_m = measured value concerned

 $E_i = initial$ value on 0 day curing time

Permanent strain value expresses the comparison (difference) with initial value.

B.2.2 Reading of Static Strain

Static strain is calculated as follow.

 $E_s = E_m - E_0$ (2) where $E_s =$ static strain $E_m =$ measured strain under the loading condition $E_0 =$ measured strain under no

- measured struin under n

loading condition

Static strain value expresses the strain amount

caused by some load.

B.2.3 Reading of Dynamic Strain (See Fig.B-4)

Dynamic strain is calculated as follows.

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Fig. B-4 Example of the Recorded Dynamic

Phenomenal

Strain wave form

er a transfer a trans Where Ed = dynamic strainthe second state = measured length of the where the fact was required dynamic strain on that the chart paper (mm) and definitions on the $i \neq j$ length of calibration on the G . chart paper (mm)

c = calibration strain value

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If one wants to know more detail, refer to "Operation Manual, DPM-300 Series, Dynamic Strain Amplifiers".

B,2.4 Reading of Perssure Cell

B,2.4.1 Static Stress

(a) and a follows. Static stress is calculated as follows.

 $\mathbf{p}_{\mathbf{s}} = (\mathbf{c}_{\mathbf{s}}^{\mathsf{S}} \mathbf{X} (\mathbf{c}_{\mathbf{n}}^{\mathsf{S}} \mathbf{c}_{\mathbf{s}}))^{\mathsf{S}} \dots \dots (4)$ (where $P_s = static stress (kg/cm.²)$ $C_{c} = calibration constant (kg/cm.²)$ n de la composition d En la composition de l measured strain under the (pré delle mer pris loading condition. $\epsilon_0 = measured strain under no$ loading condition.

san kang an ana ang ang banga B.2.4.2 Dynamic Stress

Dynamic stress is calculated as follows

 $P_{o} = C_{c} X \frac{a}{b} X \infty$ (5) where $P_{0} = dynamic stress (kg/cm.²)$

 $C_{c} = calibration constant (kg/cm.²)$

measured length of the required dynamic stress ta na sanga 到了了你们们就是你们的正确是你的! on the chart paper (mm) the life area of stangering proteins b length of calibration on and and the state of a the chart paper (mm)

ind a no high and the date of a social strain value of calibration (see Fig: B-5)

B.2.5 Reading of Vehicle Speed,

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When we make the vehicle run above the earth pressure cell installed in the dynamic load test, we receive the recording as shown in Fig. B-6 (a)

Vehicle speed is calculated as follows.

in wugʻilar et rati daa oo birrataya V in amilai <u>o «Xod</u>urga p. (6) vehicle speed (cm/sec) chart paper speed (cm/sec) ć distance between the front wheel and the rear wheel of the vehicle (cm) (See Fig. B-6 (b)) distance between the front wheel phenomenon and the rear A REAL PARTY OF wheel phenomenon on the chart n hinde de paper (cm)

B.2.6 Meaning of Indication + and - in Instrument

Owing to the instrument characteristics, that is, kind and connection of instruments, the meaning of





indication + and - changes.

Table $B \rightarrow 1$ shows the meaning of indication + and -.

B.3 Discussion of Stress and Strain Measurement

B.3.1 Stress Measurement

Pressure cell is used to detect strain through a minute change in the pressure-sensitive surface and converts it to an out put voltage proportional to the soil pressure.

strain gages are set at 4 places in the outer ring and strain gage has a strain under the external

force:

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14.2.5

Calculation curve is made using oil pressure as shown in Fig. B-7(b)

Non-linearity is 2 % of Full Scale, 5 kg/cm.²

(71 psi.)

Therefore, we consider that we measure approximately correct pressure with this cell.

B.3.2 Strain Measurement

Strain gage is used for measurement of internal stress of mortar or concrete and embedded into these test materials for its application.

A special surface treatment is provided to secure good bonding with mortar, and due considerations are given for the water proofing property and

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as shown in Fig. 8-8

In general, the elastic modulus of concrete is about 140,000 to 500,000 kg/cm² $(2x10^{6} - 71x10^{6}$ psi.), while E of this kind of gage is 28,000 kg/cm² (40x10⁴ psi.). So, the case of embedding it in concrete is no problem.

However, in this test, we have an attempt to embed it in the soil cement base. According to the results of Report No. MR. 85 the elastic modulus of soil-cement is about 5,000 to $15,000 \text{ lb/in}^2$ (352 to 1,054 kg/cm²).

The Elastic modulus of the gage is bigger than that of soil cement. In this case, there is passibility that gage does not work or the slipping occurs around the gage. But there is not better kind of gage except this type of gage for embedding in soil cement in today's technology.

Let us emphasize that gage installation in soil-cement is trial.

In Analysis, if the deformation of soil cement is the same as that of gage material, stress can be calculated as follows.

In this test, the elastic modulus of the gage is bigger

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Ball (2011 All Antiple 144)

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than that soil cement, so, there is high possibility that the deformation where the gage's installation is smaller than that where no installation of the gage.

In calculating stress, the following idea is introduced. The stress in the gage is the same as the stress in soil-cement, so the stress in calculated as follows.

> $\delta' = \varepsilon E_G \dots (8)$ where $\delta' = \text{stress } (\text{kg/cm.}^2)$ $\varepsilon = \text{strain}$ $E_G = \text{Elastic modulus of the gage}$

> > $(kg/cm.^2)$

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