A 15 YEAR PERFORMANCE AND EVALUATION OF CRCP EXPERIMENTAL ROAD

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INTRODUCTION

The Nakayama Bypass on National Highway Route 112 is a 3.4km long bypass carrying the heaviest traffic volume for this route. For this bypass, concrete pavements were adopted, taking into consideration durability and economy. For the first 1,383 m section within this bypass, a continuously reinforced concrete pavement (hereafter called "CRCP") experimental road was constructed and was put into service soon after completion in 1979.

This experimental road is located in a plane of alluvial soil, formed by a large river and small streams, with weak clayey soil deposits to a depth of approximately 20 meters. For this reason, the weak soil deposits were compacted and improved with fill soil in three phases starting in 1974, and the CRCP was placed to provide a pavement more durable than obtainable with traditional jointed concrete pavement.

The characteristics of this CRCP experimental road were;

A. Different diameter of longitudinal steel reinforcement (D13mm, D16mm) steel ratio is almost the same.
B. Two types of concrete these were traditional pavement quality concrete (hereafter TPQC in short) and dry-shrinkage compensated pavement quality concrete by adding expansion agents (hereafter CPQC in short)

The length are divided to 6 sections (Section A to F) according to the combination of steel diameter and the type of concrete.

This CRCP experimental road has received follow-up surveys 10 times during the last 15 year service period. Observed damage to road surface due to rutt-depth and cluster cracking has become significant recently, and it has been concluded that rehabilitation should be considered in the near future. For such reasons, a final survey was conducted during a two-year period from 1994 to 1995, performing the following tests to provide technical data for final selection of suitable rehabilitation methods.

a) Inspection of road surface conditions (crack width, crack spacing, and crack patterns, etc.)
b) Inspection of steel corrosion by core samples
c) Testing of load bearing capacity of pavement by FWD measurement
d) Testing of load transfer efficiency at occurred cracks by FWD measurement
e) Detection of erosion and voids existing under the CRC slab using road scope system (bore hole camera)

This paper describes the last ten field surveys and covers the following topics based on the results obtained from the above final surveys.

1) Behavior of CRCP for the last 15 years in a cold snowy region
2) Evaluation of this CRCP for serviceability
3) Proposed methods of rehabilitation

GENERAL DESCRIPTION OF EXPERIMENTAL ROAD

Place of Construction

This experimental road is located in Nakayama-chou, Higashi-Murayama-gun, Yamagata Prefecture, as shown in Figure 1. The test section of the road is straight at the start, has a curve (R = 500m) in the middle, and is straight at the end, as shown in Figure2. Longitudinal profile shows a down grade of 0.2% from Yamagata in the direction of Sagae.

This project area is located in a cold snowy region. Temperature varies significantly between summer and winter and between days, as peculiarly experienced in the inland area of Yamagata Prefecture. Maximum temperature recorded in the past is 38°C (1962) and minimum recorded temperature is -20.3 °C (1961).

Pavement Configuration

The pavement is configured of a 20 cm-thick granular subbase placed over sandy subgrade, 4 cm-thick asphalt interlayer, and a 25cm-thick CRC slab, as shown in Figure 3.

This experimental road has two types of longitudinal steel reinforcement (diameter D13 mm and D16 mm) and two types of concrete, TPQCand CPQC that uses an expansion agent to reduce drying shrinkage. Combinations of these two different types of concrete and reinforcement were applied in six sections like Figure 4.

Design of CRCP Slab

According to Cement Concrete Pavement Manual *(1)*, Japan Road Association, 1976th edition, at that time, since the design traffic category of this road was class C, the design thickness of CRCP was 20cm compared with 25cm of jointed concrete pavements. However based on the consideration of fierce rutting in winter by studded tires and other reasons, 25cm thickness was adopted.

Design layout of slab reinforcement having a significant influence on the behavior of CRCP is shown in Table 1. It should be noted that the steel ratio of both D13 bars using section and D16 bars using section were almost the same, 0.6%.

Concerning the reinforcement, prefabricated welded reinforcement mats were used. That was partly because especially in the case of D13, narrow steel spacing like 8cm would make it difficult to set supporting chairs manually and partly because there had been a successful experience *(2)* using welded reinforcement mats so far in Japan. The details of welded reinforcement mat are shown in Figure 5. From this figure, it can be seen that the steel laps was staggered though that is a little different from what other country's guideline *(3)* recommended.

Concerning the terminal treatments of CRCP, two anchor lug system at the Sagae side, and expansion joint system at Yamagata side, which consist of 2 expansion joints, were used in order to find out which system shows better performance.

Construction

Concrete casting was completed within 12days in September 1979. Typical machines like box-type spreader for spreading both lower and upper layer, concrete finisher for consolidation and rough finishing, and diagonal screed finisher for floating, were used for paving with foams. And it should be noted that paving width was 7.25m, and there was no longitudinal joints.

Concrete

Mix proportion of placed concrete and its strength are described in Table 2 and Table 3 Calcium Surfur Aluminate was used as the expansion agent

RESULTS OF FOLLOW-UP SURVEYS

Description of Cracks Occurring in CRC Slab

Transverse Cracks

Changes by time of the average spacing of cracks occurring during the 15-year period of service, from the time directly after (one week) the placement of CRC to the final survey (the average of the total length of surveyed section divided by the number of cracks occurring in the subject section) are shown in Figure 6.

It can be seen that the average crack spacing became smaller slowly in the first three years of service and there were not significant changes thereafter. The average crack spacing ranges from 0.5 to 0.9 m after 15 years(5500daysof service. Crack spacing of sections where reinforcement consisted of D16 mm bars and where an expansion agent was mixed with the concrete is wider than for other sections.

Apparently the average crack spacing measured after some sections shows bigger figure than that of previous measuring chance. Probably it might be affected by a little difference of survey lot among these 2 cases, as well as the increasing difficulty of finding cracks on wornout surface. Therefore it could be concluded that the real crack spacing is almost the same as that at after 8 years.

Figure 7 shows the relationship between the measured crack spacing and the calculated one by Vettter's formula. Excluding the data of after 15 years because of the reason above mentioned, Vetter's formula explains well the effects of steel arrangement and dry-shrinkage property of concrete on crack spacing, though the values of the measured in itself are much smaller than the calculated ones because of the traffic repetitions

Histogram of crack spacing distribution in the 15 years of service is shown in Figure 8. It can be seen from this figure that cracks occurred most often at a spacing of 0.2 to 0.3 meters. Crack spacing of 0.4 m or less reaches 75% or more and the existing CRCP shows generally cluster crack patterns. Changes in the average crack width (the average of the sum of crack widths of surveyed section, divided by the number of cracks) over time during the period of 15 years of service, starting directly after (one week) the placement of pavement to the final survey are shown in Figure 9.

It can be seen that crack widths generally increased after 180 days of service. (It can be presumed that main reason was due to shrinkage of concrete caused by temperate drop, because surveys were conducted in winter seasons.) The average crack width measured after 15 years of service ranges from 0.15 to 0.20 mm. Wide cracks, like more than 1.0mm,were not noted. It was also noted that there was no damage by punchout.

Longitudinal Cracking

It was noted that longitudinal cracking had started to occur about five years after commencement of service but such cracks were not significant, being only about 0.05 mm in width. At the survey conducted after 15 years of service, crack widths measured around 0.2 mm and their average length ranged from 5 to 11 meters like Photo 1.

Longitudinal cracking occurred at the inner wheel path, both for the up and down traffic lanes. It can be presumed that such cracks were mainly caused by bending stresses concentrated in the central area of slab due to accumulated traffic loads, as these CRC slabs were constructed monolithically in both the up and down traffic lanes and longitudinal joint was not provided .

Consideration about End Terminal

The horizontal movements of expansion joint width within y ears after concrete casting is shown in Figure 10Tha t shows the difference of the movement between anchor lug systems and expansion joint system is little. Therefore it could be concluded that the anchor lug system we used has little anchorage effects. In fact, the crack spacing of sectionF is larger than that of section D see Figure 6 T hat ineffectiveness of this system is due to the used deign of it, as well as the property of the soil that to anchored. Anyway, besides this construction work is troublesome, anchor lug system is not widely used at present in Japan for roads.

Measured Rutt-Depths of CRC Slab by Wearing

Rutt-depths caused by wearing were measured by transverse profile meters in all sections, and the results of measurement are shown in Table4. The average rutt-depth of each section due to wearing ranges from 11.0 to19.7 mm. The

CRC slab section paved with concrete containing an expansion agent, and with D13 mm reinforcement(Section B), shows a little larger value than those of other sections, but it seems that significant differences by different types of concrete are negligible.

The Road Rehabilitation Manual *(4)*, Japan Road Association, indicates a target level of 30 to 40 mm as a standard for maximum rutt-depth to determine necessity for rehabilitation. It can be concluded that this CRCP has reached the stage where it requires rehabilitation to assure the safety of vehicle traffic.

Inspection of Steel Corrosion by Core Sampling

From observations of reinforcement contained in core samples taken from various locations of different crack widths, it can be seen that rust had formed only on reinforcement taken from the areas of longitudinal cracks. Significant rust had formed on some parts of the reinforcement, but it was concluded that steel corrosion had not reached a level where the steel cross sectional area became less than that required.

Testing of Load Bearing Capacity of Pavement by FWD

Deflection, measured at load application points (D) set at various locations of different crack widths, using an FWD having a loading capacity of 5-ton, is shown in Figure 11.

Deflections measured in interior areas and at edges of slab increase as crack widths increase, but the deflection measured at interior areas amounted to 200 m or less in all sections.

Elastic modulus of all pavement layers (called "apparent elastic modulus") at the interior place, back-calculated by application of multi-layer elastic theory from the deflections based on the measurements taken by FWD, are shown in Figure 12. Models showing the cross section of pavement were developed for this analysis as shown in Figure 13 .

It can be seen from that the apparent elastic modulus at the interior place tends to become lower in all sections as transverse crack width increases. This suggests that an increase in crack width can result in the integration of reinforcement effect, namely the loss of load bearing capacity of CRC slab.

Tests indicated the subbase and subgrade had an apparent elastic modulus (geometric means) of nearly 470 MPa, respectively.

Testing of Load Transfer Efficiency by FWD Measurement

Load transfer efficiency, measured at various locations of different crack widths, using a deflection of FWD having a loading capacity of 5-ton, is shown in Figure 14. The Load transfer efficiency was obtained by equation 1.

$$Eff=2*(D_{300})/(D_{300}+D_0)*100 \qquad (1)$$

Where,

Eff	= load transfer efficiency (%),
D_0	= amount of deflection at center of load application point (m) , and
D_{300}	= amount of deflection at a point 300 mm away from the load application point (m) .

It can be seen that there are no significant differences in Load transfer efficiency by differences in crack width, both for normal concrete and concrete with an expansion agent. The indicated average load transfer efficiency measured in interior area and at edges of slab and at construction joints (center of slab) is high, as shown below.

Interior area of slab	94.7%
Edge of slab	95.5%
Construction joints	96.0%

Detection of Voids Under CRC Slab by Road Scope System

Surveys were conducted using a road scope system (bore hole camera), shown in Figure 15, to determine if erosion and voids had occurred under the CRC slab at locations where significant deflection was measured by FWD, where performance

of CRC slab was deemed low, and at locations where Load transfer efficiency and apparent elastic modulus were deficient. A sample of the survey data is shown in Photo 2.

It can be seen from the photographs taken by bore hole camera inside the pavement subbase that no voids nor erosion were found under the CRC slab in any of the sections. Subgrade was also found to be in a very dense condition.

EVALUATION OF SERVICEABILITY OF CRCP

Present condition of the CRCP pavement of the Nakayama Bypass is evaluated as described below, based on the results of follow-up surveys conducted in the last 10 times and those of the final survey.

This CRCP has been in use for approximately 15 years but cracking has almost ceased. Crack widths range from 0.15 to 0.20 mm at present and cluster cracking and longitudinal cracking can be seen at some areas, but punchouts or wide cracks are not occurring.

Longitudinal cracking can be defined as a precursor that can lead to structural damage. Since FWD measurements of Load transfer efficiency indicates high values, rusting of reinforcement has not significantly reduced the steel cross sectional area, and severe damage by cracking peculiar to CRCP was not noted, it can be presumed that the CRCP is still maintaining properly its structural function at present.

The maximum rutt-depths caused by wearing of this pavement section range from 25.9 to 40.7 mm(See Table 4). This indicates that wearing has developed nearly to the target level (30 to 40 mm) for determining the necessity of rehabilitation. Immediate repairs are required to assure traffic safety.

With concrete pavement systems, if voids occur under the concrete slab the base course loses its support and excessive stresses can occur in concrete slab from repeated application of traffic loads, and damage to the concrete can develop rapidly.

According to the results of the exploration for voids using road scope system, voids or erosion that can be a basis for consideration of rehabilitation were not noted and it was concluded that subsurface layers and subgrade under the CRC slab were extremely dense in structure.

It can be presumed from the existing condition of the CRCP that it has reached the stage where rehabilitation is necessary to assure traffic safety, but that rehabilitation is required before disastrous structural damage is caused to CRC slab from repeated applications of traffic loads and weather factors (rainfall and snow fall).

PROPOSED METHODS OF REHABILITATION

As described previously, it has been determined that CRCP of the Nakayama Bypass still holds proper load bearing capacity as a CRC slab, and that damage to road surfaces consists mainly of rutting. rutt-depth caused by wearing have nearly reached the main reinforcement, which plays an important role in maintenance of proper CRCP function. Therefore, it is considered necessary to, as a minimum, restore the areas damaged by rutt with concrete, as a method of rehabilitation to sustain CRCP behavior continuously in the future.

That is, it is considered that a thin bonded concrete overlay, placed after a application of a bonding agent to the faces milled to the bottom of rutting in existing CRC slab, will be the most effective and economic method of treatment.

In the study of the bonded type concrete overlay, the accumulated fatigue (affect on strength) due to repeated traffic loads during the last 15 years of service of the CRC slab were taken into consideration in order to assure a design life of 20 years, in expectation of long service life even after rehabilitation.

Effective concrete slab thickness (Deff), from the standpoint of strength, considering accumulated fatigue, was 20 cm, based on elastic modulus measurements of the concrete slab by FWD, in accordance with the practical recommendations of AASHTO guide *(5)*.

As the required design slab thickness for this bypass is 25 cm based on traffic category, required thickness of concrete overlay becomes 5 cm. For this reason, an 8 cm bonded-type concrete overlay, placed on the existing CRC slab surfaces milled to a depth of 3 cm, was proposed as the rehabilitation method.

POSTSCRIPT

Because it is believed that CRCP is rarely affected by settlement and other events occurring in weak soils, compared with ordinary jointed concrete pavement, it was used for the Nakayama Bypass. This evaluation has proven that structural serviceability is still sustained, even after 15 years of service.

However, rutt-depth as deep as 40 mm caused by wearing from studded tires and tire chains have occurred in one section. For this reason, a thin bonded concrete overlay was proposed as the method of rehabilitation. Some sections were rehabilitated by this method in 1996.

We will keep checking the performance of this bonded type concrete overlay with follow-up surveys.

Table1 Design layout of slab reinforcement of CRC slab

main reinforcement	diameter,spacing (mm)		steel ratio of longitudinal steel	position from surface (cm)	lap length (cm)	Spec.of Steels
D13	L	D13, 83	0.61	8 *	40	SD30
	T	D10,300				
D16	L	D16,125	0.64			
	T	D13, 300				

note : L : longitudinal reinforcement
T : transversal reinforcement
*: : From the surface,1/3 thickness of slab

Table2 Mix Proportions of Concrete

Type	Design slump (cm)	Design Air Content (%)	water/Cement Ratio (%)	ratio of sand (%)	Unit weight (kg/m^3)					
					Cement	Water	Fine Agg.	Coarse Agg.	Additives	CSA
TPQC	2.5	4.0	43.0	38.8	330	142	703	1201	1.32	0
CPQC	2.5	4.0	39.4	37.9	322	142	677	1201	1.44	38

note : TPQC : traditional Pavement Quality Concrete
CPQC :dry-shrinkage compensated concrete
CSA :Calcium Sulpho aluminate Addictives

Table3 Strength of placed Concrete

Concrete Type	Split Strength at 28days (MPa)			Bending Strength at 28days (MPa)			Compressive Strength at 28days (MPa)		
	Max.	Min	Ave.	Max.	Min	Ave.	Max.	Min	Ave.
TPQC	4.21	2.11	3.35	5.70	4.46	5.12	43.0	32.1	38.4
CPQC	3.43	2.65	3.03	5.27	4.85	5.03	37.8	35.2	37.7

Table4 Rutt-Depths Due to Wearing, Measured in All Sections

Concrete Type	Section	Steel Type	Rutt-Depth (mm)	
			Max.	Ave.
TPQC	B	D13	40.7	19.7
	E	D16	25.9	12.9
CPQC	A	D13	32.6	16.6
	C	D13	30.3	16.3
	D	D16	28.6	14.7
	F	D16	25.1	11.0

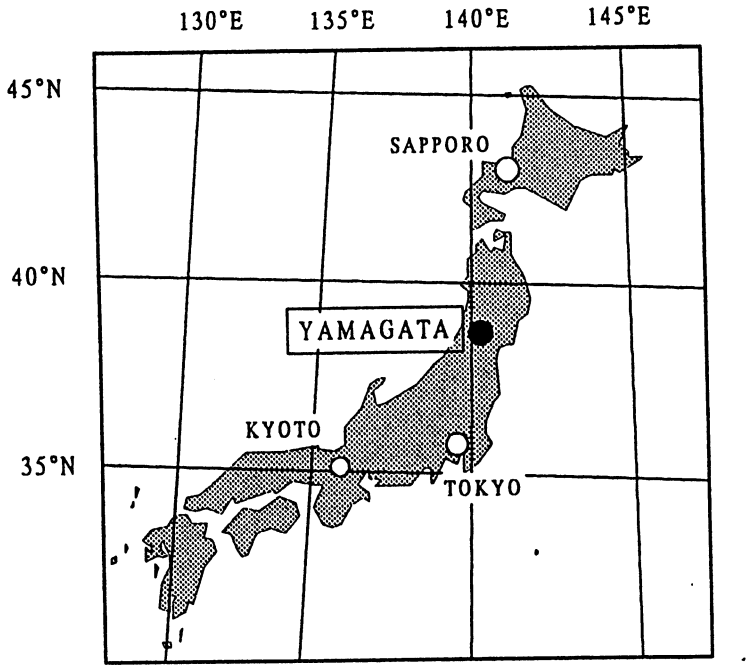


Fig 1. Location of CRCP Experimental Road

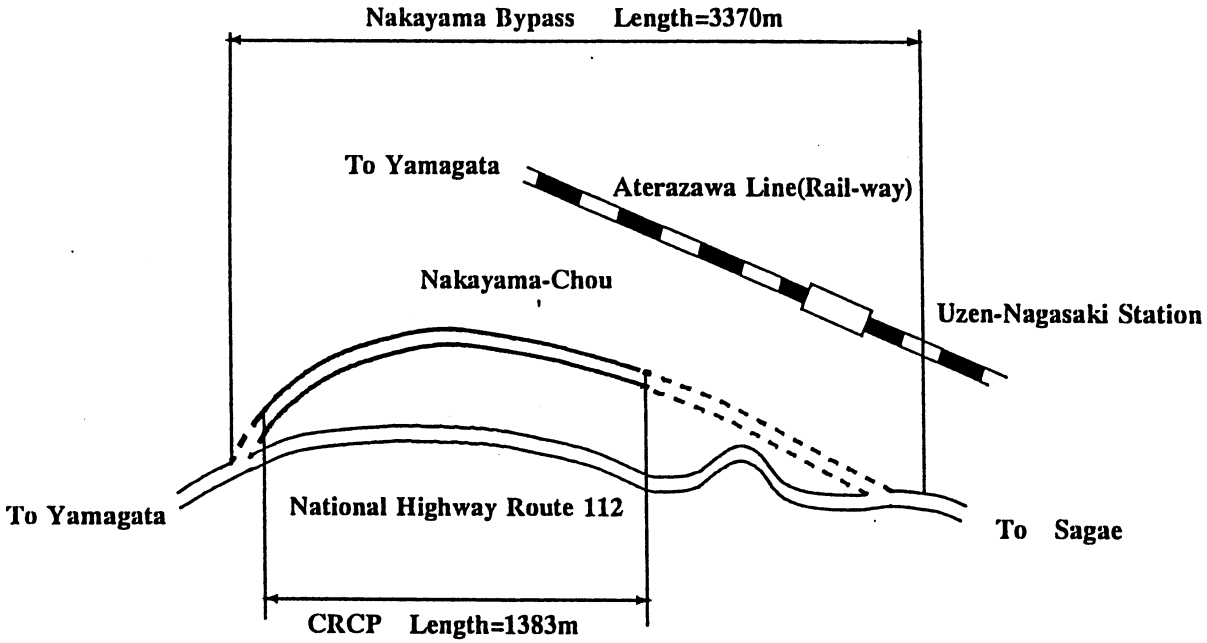


Figure 2 Location of Nakayama Bypass

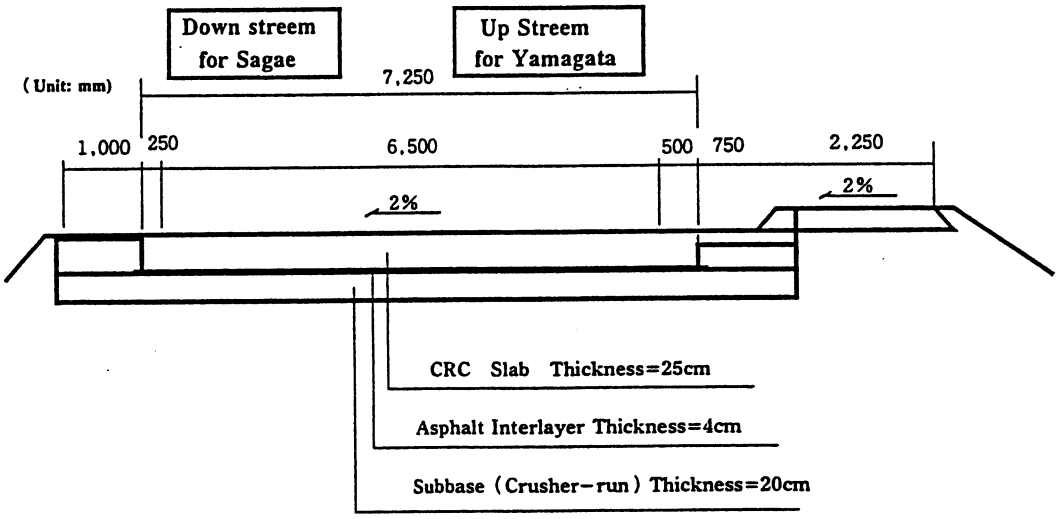


Figure 3 Standard Cross Section of Pavement

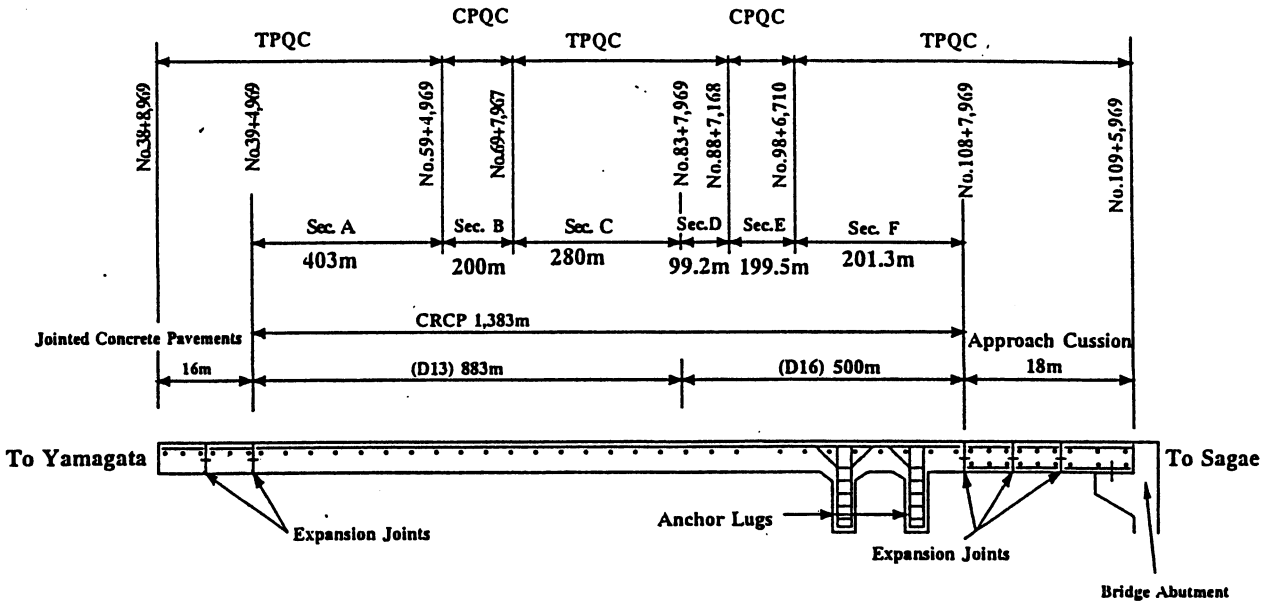


Figure 4 Typical Longitudinal Profile of CRCP xperimental Road

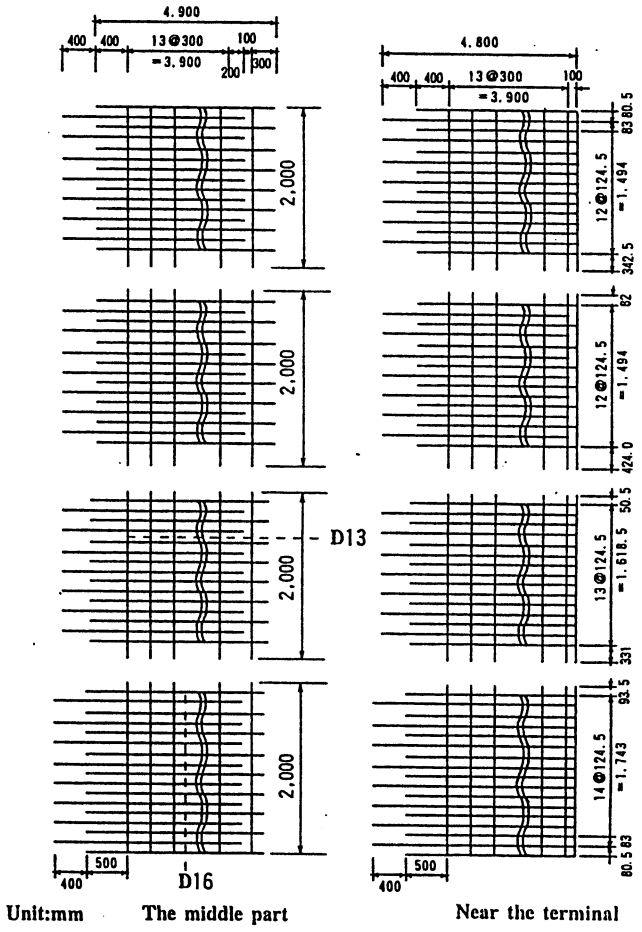


Figure 5 Details of the Prefabricated Welded Reinforcements(Example of D16)

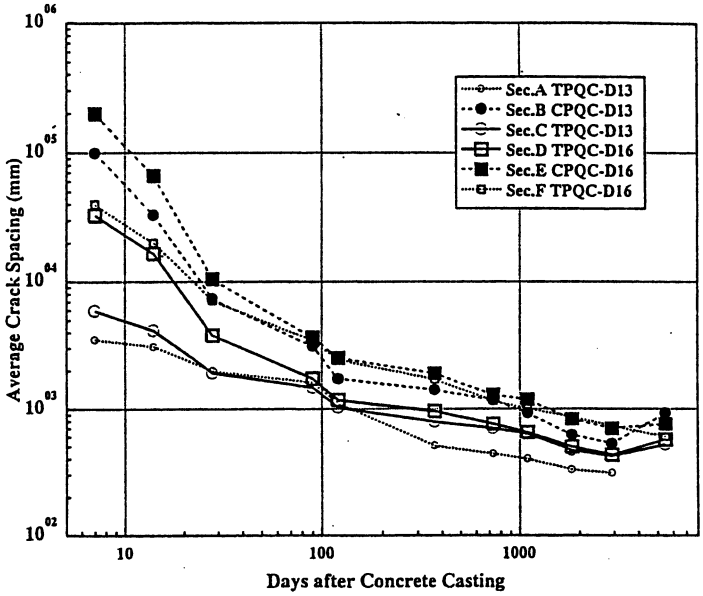


Figure 6 Average Crack Spacing Over a Period of Time

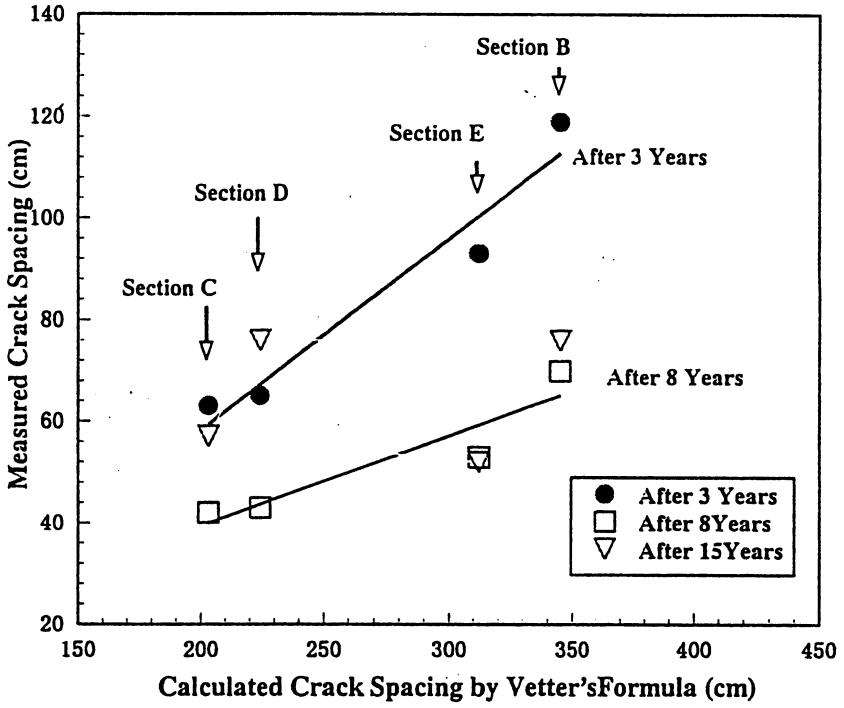


Figure7 Comparison of Calculated and Measured Crack Spacing

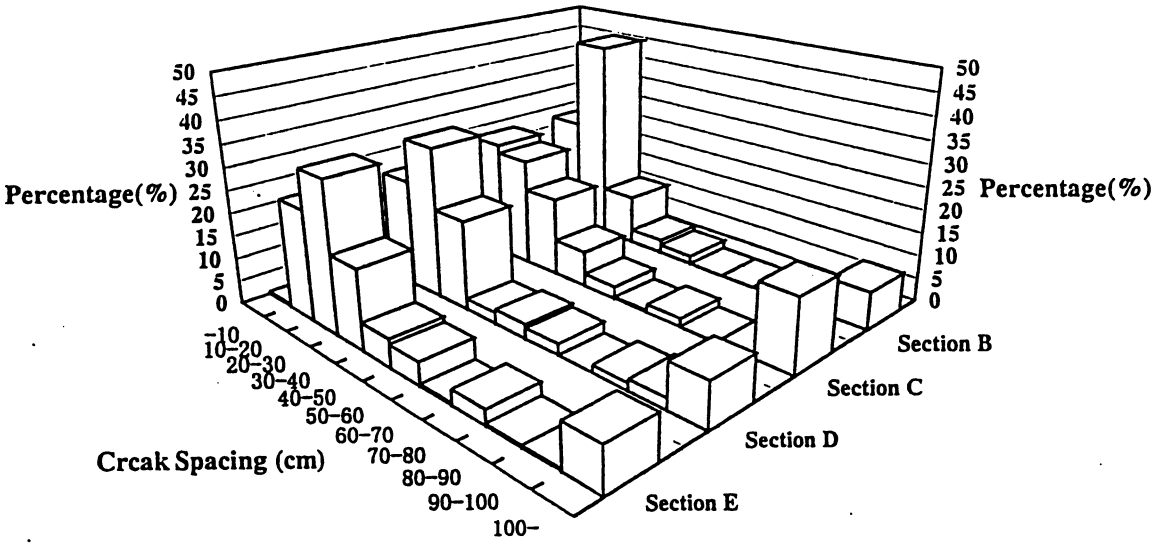


Figure8 Distribution of Crack Spacing

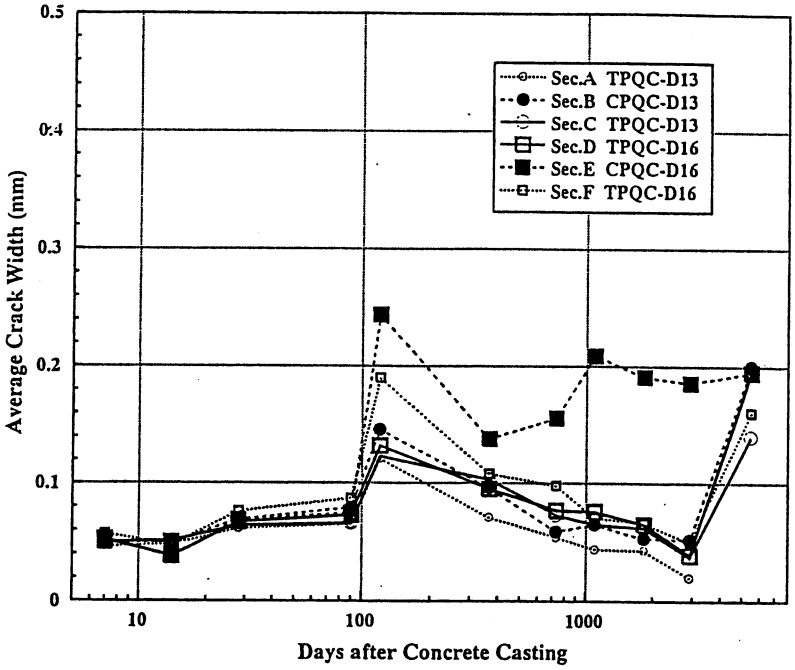
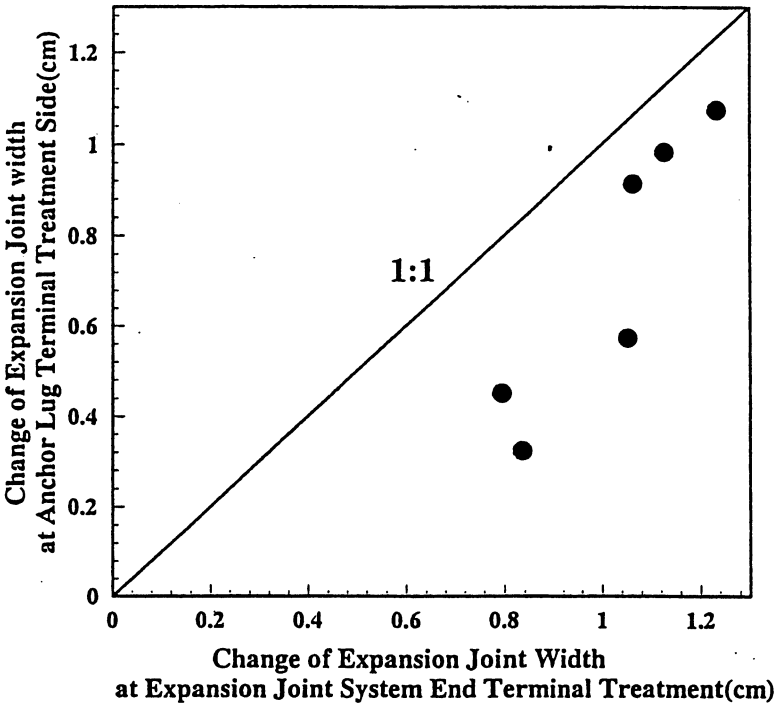
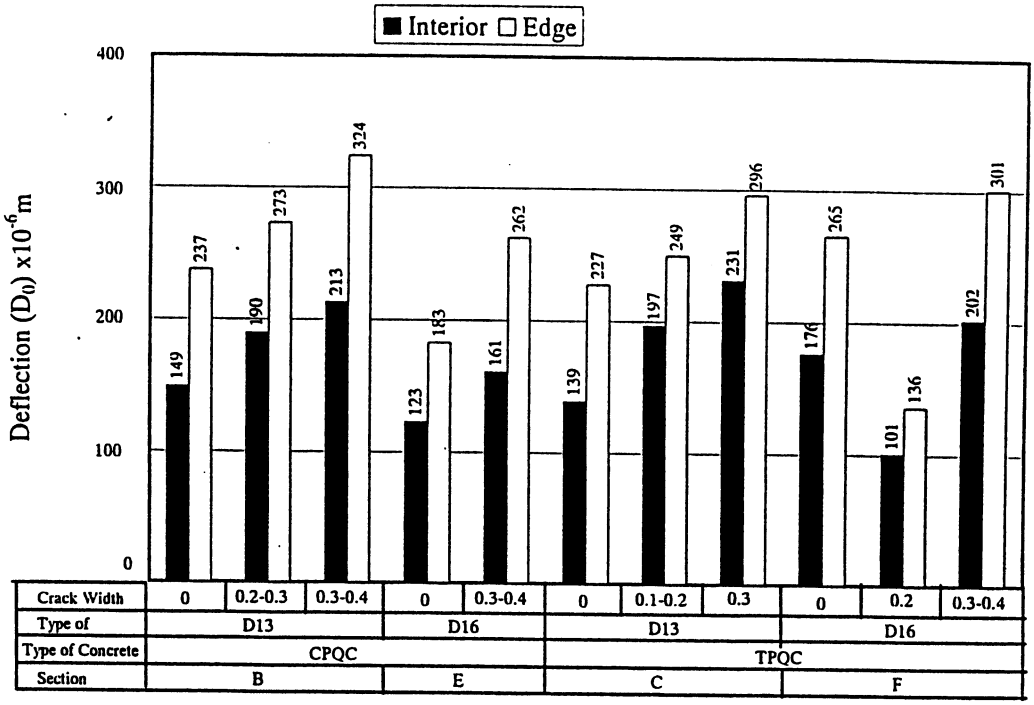


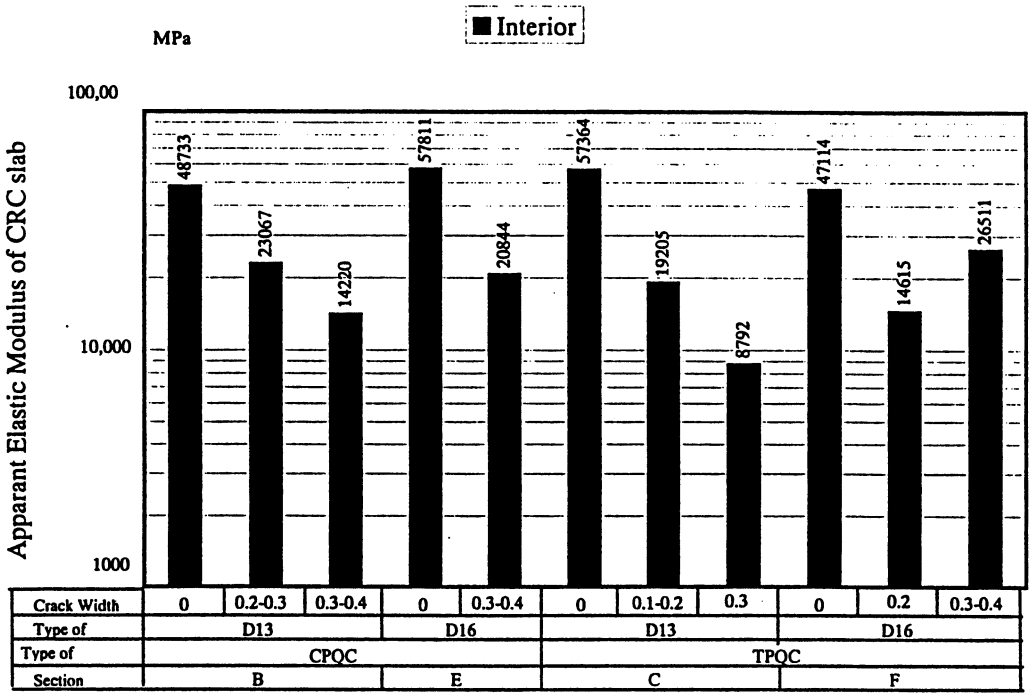
Figure9 Average Crack Width Change with Time



Figue10 Comparison of Slab End Movement



Figue 11 Deflections of Load Application at the Various Crack Width



Figue 12 Backcalculated Apparent Elastic Modulus of CRC slab at Various Crack Width

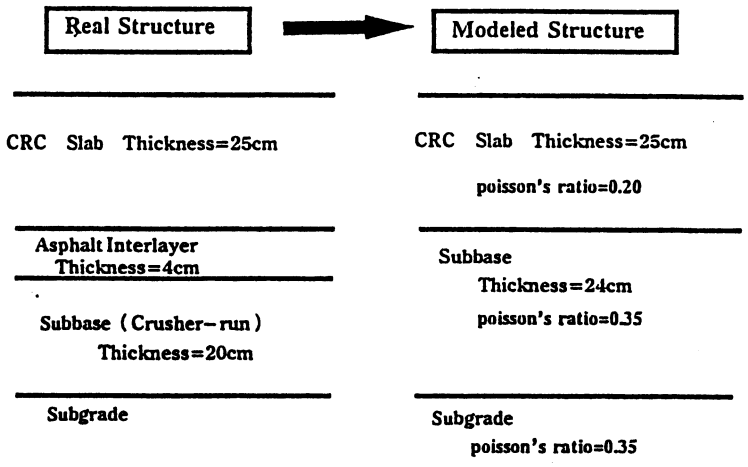
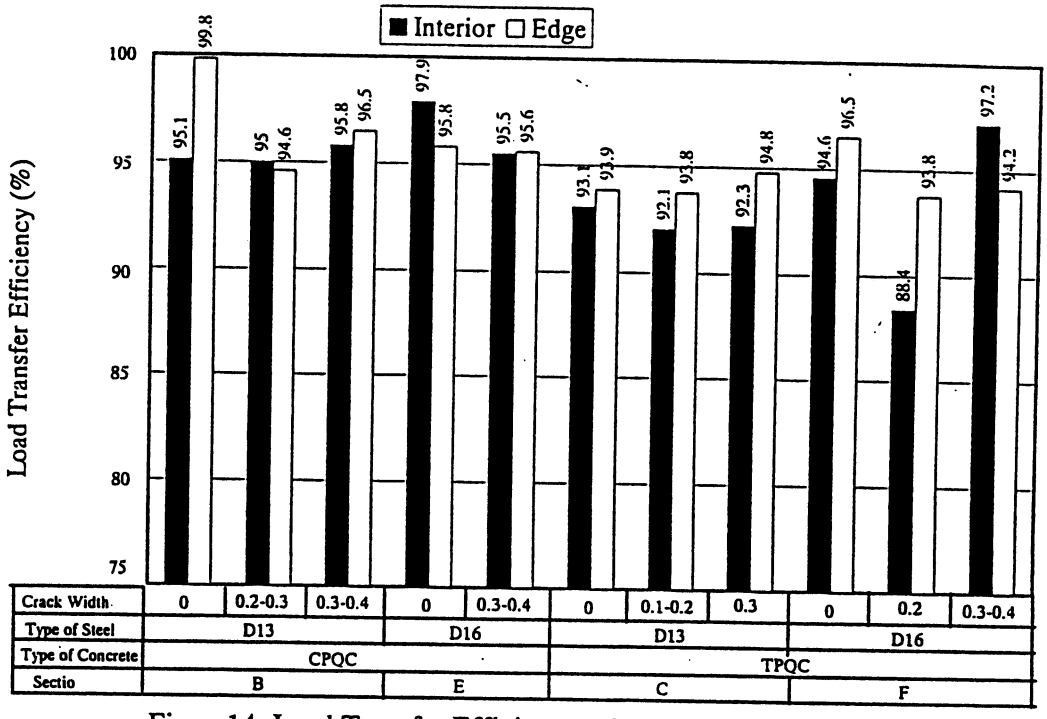


Figure 13. Modeling of Pavement Cross Section



Figue 14 Load Transfer Efficincy at the Various Crack Width

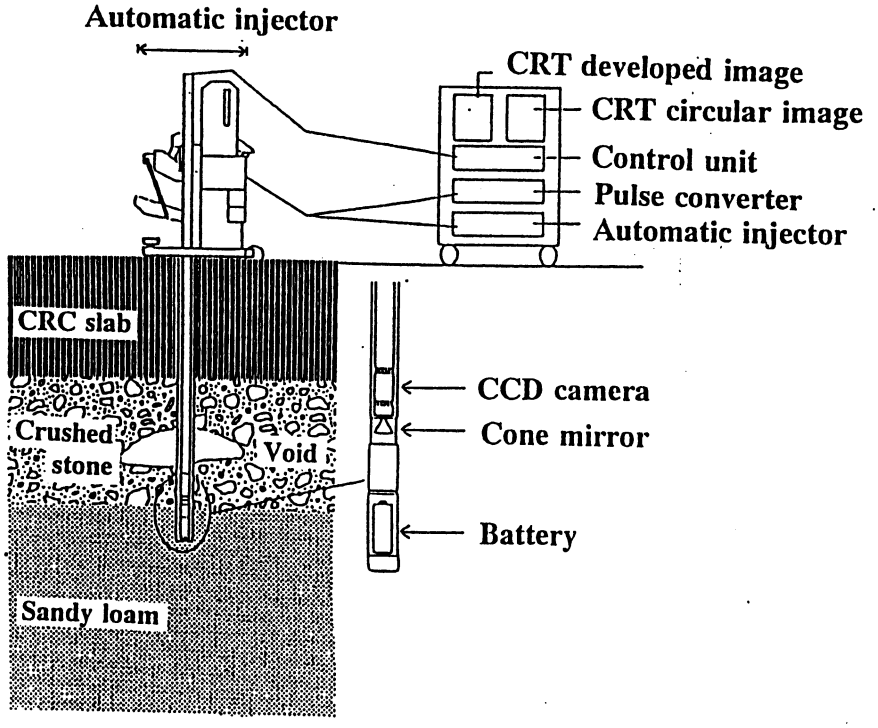


Figure 15 Structural View of Road Scope System



PHOTO 1: View of Longitudinal Cracking

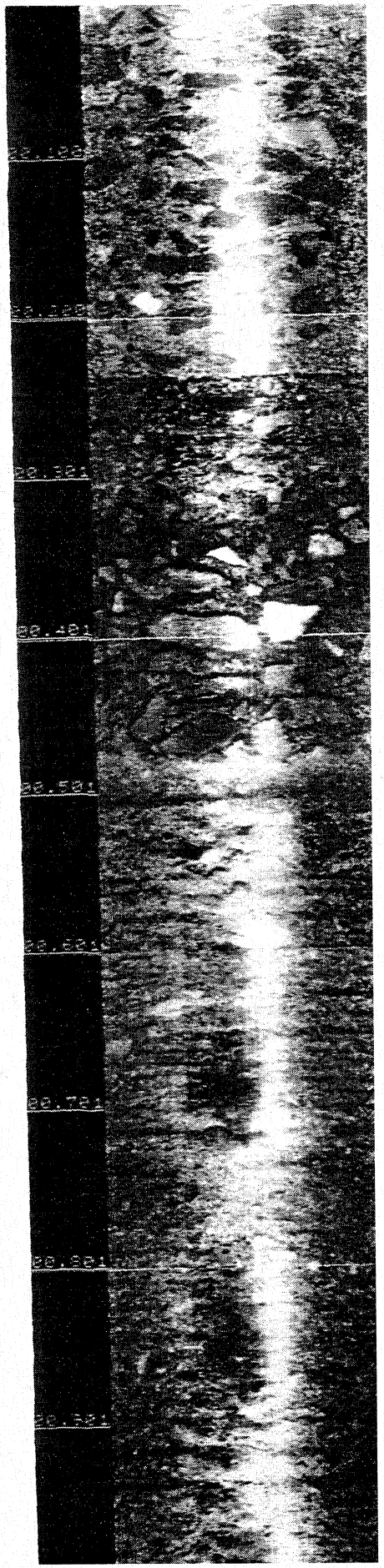


PHOTO 2: Subsurface View of Pavement Base Course Taken by Bore Hole Camera

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