13.4 ESTABLISHMENT OF CONDITION EVALUATION EQUATIONS

13.4.1 Establishment of RRI and PSI Equations

(1) Choice of elements for RRI and PSI

For the following reasons, three variables, i.e., roughness, cracking and patching were selected as the elements employed in RRI and PSI formulas:

(1) The variables which have correlation with RRR and PSR are roughness, class 3 and class 4 cracks (cracking) all cracks, (cracking Index), faulting and pumping. Among them, all cracks, faulting and pumping are closely correlated with class 3 and 4 crack. Thus, the former three variables are represented by the latter and need not appearing RRI and PSI formulas whenever class 3 and class 4 crack is included. Another reason why faulting and pumping are neglected is that those defects should be repaired as a maintenance work prior to discussion as to when and how the particular road in question should be rehabilitated.

(2) Although little correlation exists between RRR and PSR and patching, the latter is included in RRI and PSI formulas considering the fact that it has same nature as cracking in terms of surface deterioration.

(2) Choice of Function of Element

The general form of RRI/PSI is assumed to be

$$RRI/PSI = Ao + A1 \times R' + A2 \times D'$$

where R' and D' are functions of roughness and surface deterioration, respectively. The coefficient Ao, A1 and A2 may then be determined by least squares regression analysis.

Based on the preliminary analysis, the following transformations were selected as functions of roughness and surface deterioration.

$$R' = \log(R)$$
$$D' = \sqrt{C + P}$$

Where: R: Roughness in cm per km
C: Cracking (Total of Class 3 and Class 4 cracks in m per 1,000 m^2)
P: Patching in m^2 per 1,000 m^2

(3) Proposed RRI and PSI Formulas

For several alternative forms of the RRI and PSI formulas, the coefficients were determined by least squares regression method as follows:

$RRI = 7.53 - 1.5 \log(R) - 0.11\sqrt{C+P}$	$(r = 0.756)$
$RRI = 7.97 - 1.7 \log(R) - 0.11\sqrt{C}$	$(r = 0.740)$
$RRI = 10.89 - 3.2 \log(R)$	$(r = 0.448)$
$RRI = 3.93 - 0.12\sqrt{C+P}$	$(r = 0.731)$
$RRI = 3.87 - 0.13\sqrt{C}$	$(r = 0.706)$
$PSI = 7.75 - 2.0 \log(R) - 0.06\sqrt{C+P}$	$(r = 0.745)$
$PSI = 8.05 - 2.2 \log(R) - 0.06\sqrt{C}$	$(r = 0.728)$
$PSI = 9.33 - 2.9 \log(R)$	$(r = 0.566)$
$PSI = 2.93 - 0.07\sqrt{C+P}$	$(r = 0.642)$
$PSI = 2.88 - 0.08\sqrt{C}$	$(r = 0.601)$

Where: R, C, P: as defined previously

r = coefficient of correlation between RRI/PSI and corresponding RRR/PSR.

Out of the above alternatives, the following formulas were selected for this study:

$$RRI = 7.53 - 1.5 \log(R) - 0.11\sqrt{C+P}$$
$$PSI = 5.75 - 2.0 \log(R) - 0.06\sqrt{C+P}$$

In case where roughness measuring equipment is not available, the RRI formula can be replaced with

$$RRI = 3.93 - 0.12\sqrt{C+P}$$

while, PSI cannot be estimated because of low accuracy.

13.4.2 Comparison between RRI/PSI and AASHO's PSI

(1) AASHO's PSI Equation

The following formula was established by AASHO based on the analysis of the AASHO Road Test results:

$$PSI = 5.41 - 1.8 \log(0.40\,R - 33) - 0.09\sqrt{C + P}$$

(For Rigid Pavement)

Where: R = roughometer reading in inches per mile

C = Cracking (total linear feet of Class 3 and Class 4 cracks per 1,000 sq. ft.)

P = patching in sq. ft. per 1,000 sq. ft.

In this equation, roughness was measured by the Bureau of Public Roads Roughometer, while roughness in this Study by the Bump Integrator. No relation between two measurements by both equipments are authorized. TRRL proposed very loose relation for the comparison between surface irregularity and riding quality.

Based on this very loose relation between two measurement of roughness, AASHO PSI formula was converted, as follows. Refer to Appendix 13-3.

$$PSI = 5.41 - 1.80 \log (0.20R - 30) - 0.05\sqrt{C + 3.3p}$$

(for rigid pavement)

R, C, P: as defined for the RRI/PSI formulas in this study

It is again noted that the above formula includes the estimation in the conversion of roughness value and the accuracy of the formula is therefore not warranted.

(2) Average Cracking and Roughness Conditions

Average cracking and roughness in AASHO's Road Test were estimated as discussed in Appendix 13-3. Based on the result of pavement surface condition survey, the average cracking and roughness in the Study Section were also assumed for each level of RRI and PSI.

Table 13.4-1 and Figure 13.4-1 summarize the comparison of RRI/PSI and AASHO's PSI.

These comparison shows:

1). In comparison with RRI and AASHO's PSI, the former indicates longer length of cracking and higher roughness where the index are lower than 3.0.

2). In comparison with PSI and AASHO's PSI, the former indicates shorter length of cracking and higher roughness where the index are lower than 3.0.

3). In comparison with RRI and PSI, the former always indicates longer length of cracking and higher roughness.

TABLE 13.4-1 AVERAGE CRACKING AND ROUGHNESS CONDITION

	ASSHTO Road Test		The Study in the Philippines					
PSI	Cracking ($m/1000\ m^2$)	Roughness (Cm/Km)	RRI	Cracking	Roughness	PSI	Cracking	Roughness
4.0	12	175	4.0	5	160	4.0	0	80
3.5	28	190	3.5	12	270	3.5	0	130
3.0	50	220	3.0	60	290	3.0	4	220
2.5	110	260	2.0	140	320	2.5	35	280
2.0	220	300	2.0	250	340	2.0	150	320
1.5	390	360	1.5	400	370	1.5	360	360

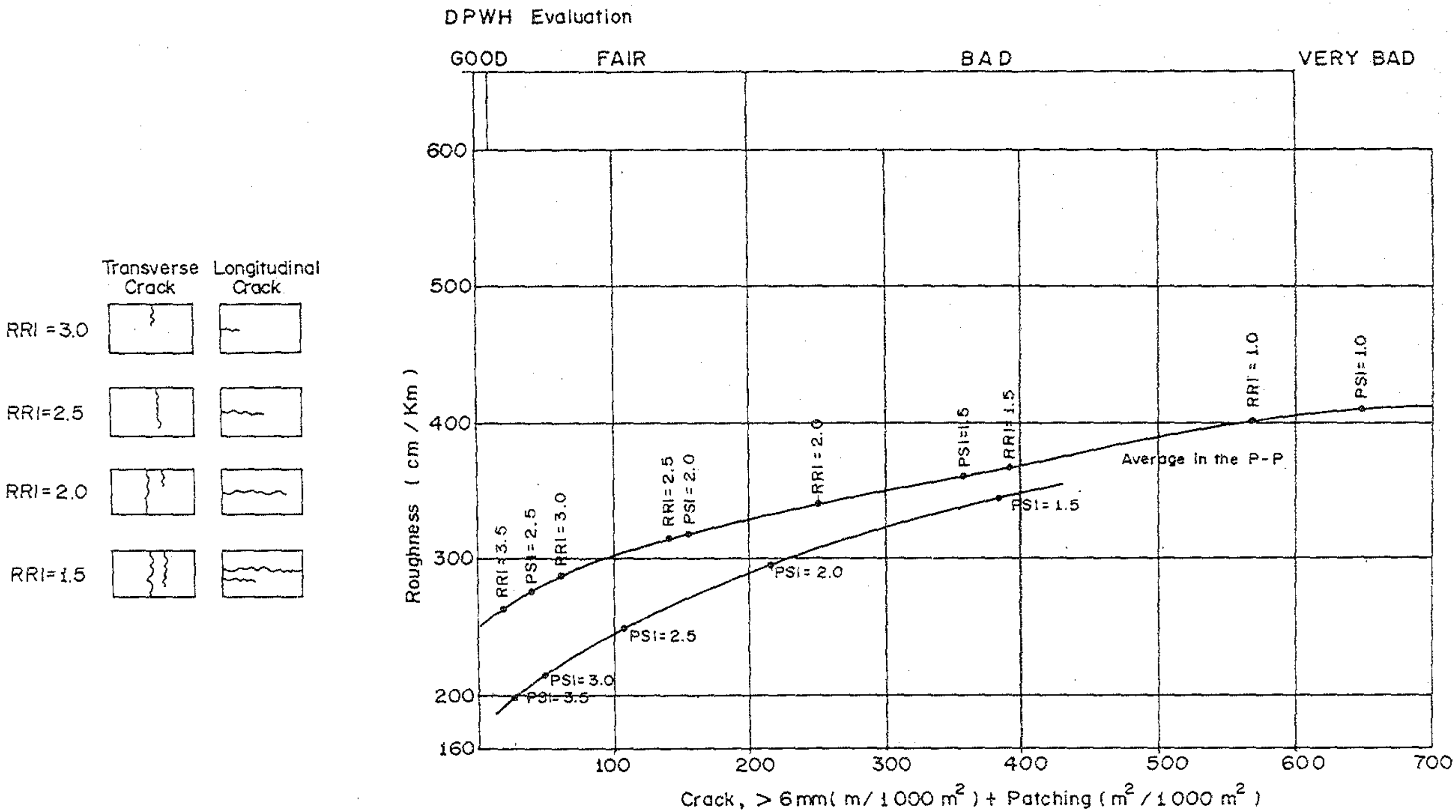


FIGURE 13.4-1 AVERAGE RELATIONSHIP BETWEEN RRI AND ROUGHNESS, CRACKING, CRACKING, PATCHING

13.4.3 Evaluation of RRI and PSI Equations

(1) Estimation of Initial RRI and PSI

Roughness values on Manila South Expressway were measured to identify average roughness of concrete pavement in the country. The survey results are:

Manila Bound (old PCC)	120 – 200	Average 160 cm/km
South Bound (new PCC)	200 – 350	Average 250 cm/km
South Bound (old PCC)	120 – 200	Average 160 cm/km

Roughness varies 120 to 200 cm/km except higher values found on the pavement of south bound inner lane (new PCC). Assuming that the pavement of Manila South Expressway represents the initial condition of the pavement in general, the initial RRI/PSI maybe estimated as follows:

Roughness	:	120 to 200, average 160 cm/km
Cracking	:	0
Patching	:	0
RRI	:	4.5 to 4.1, average 4.2
PSI	:	3.8 to 3.2, average 3.3
AASHTO's PSI	:	5.0 to 3.6, average 4.8

(2) Characteristics of RRI and PSI Equations

Correlation between Index (RRI, PSI and AASHO's PSI) and pavement conditions (cracking and patching) are graphically shown in Figure 13.4-2.

From this figure, the following were observed.

1.) RRI is closely related with cracking and patching, but no clean relation with roughness.

2.) PSI is loosely related with cracking and patching, and even roughness.

3.) AASHO's PSI has very loose relation with cracking and patching and a little loose with roughness.

4.) In any value of index, PSI is lower than RRI. It may indicate that road users in the country have high desire when evaluating the serviceability of roads (PSI), but conservative when planning the pavement rehabilitation (RRI).

(3) Evaluation of RRI and PSI Equations

Two indices, RRI and PSI have been discussed simultaneously comparing with AASHO's PSI. Among these index, RRI is recommended to identify the road sections where the pavement rehabilitation will be required based on the following observations.

1) *Structural performance of pavement is prefered to be given the precedence over functional performance.*

2) RRI is closely related to cracking which shows the structural performance, while PSI is loosely related with all deficiencies.

3) The initial RRI and PSI are estimated to be 4.2 and 3.3, respectively. RRI of 4.2 is reasonable but PSI of 3.3 is not practicable (AASHO's initial PSI is 4.5 for rigid pavement)

4) In comparison with AASHO's PSI, RRI indicates almost same length of cracking and the level of roughness, while PSI shows shorter length and lower roughness.

5) RRI shows the reasonable compatibility with AASHO's PSI, while PSI is not so comparable.

6) RRI may be, therefore, considered to show the same pavement condition without amendment as AASHO's PSI in the AASHTO Guide for Design of Pavement Structure 1986.

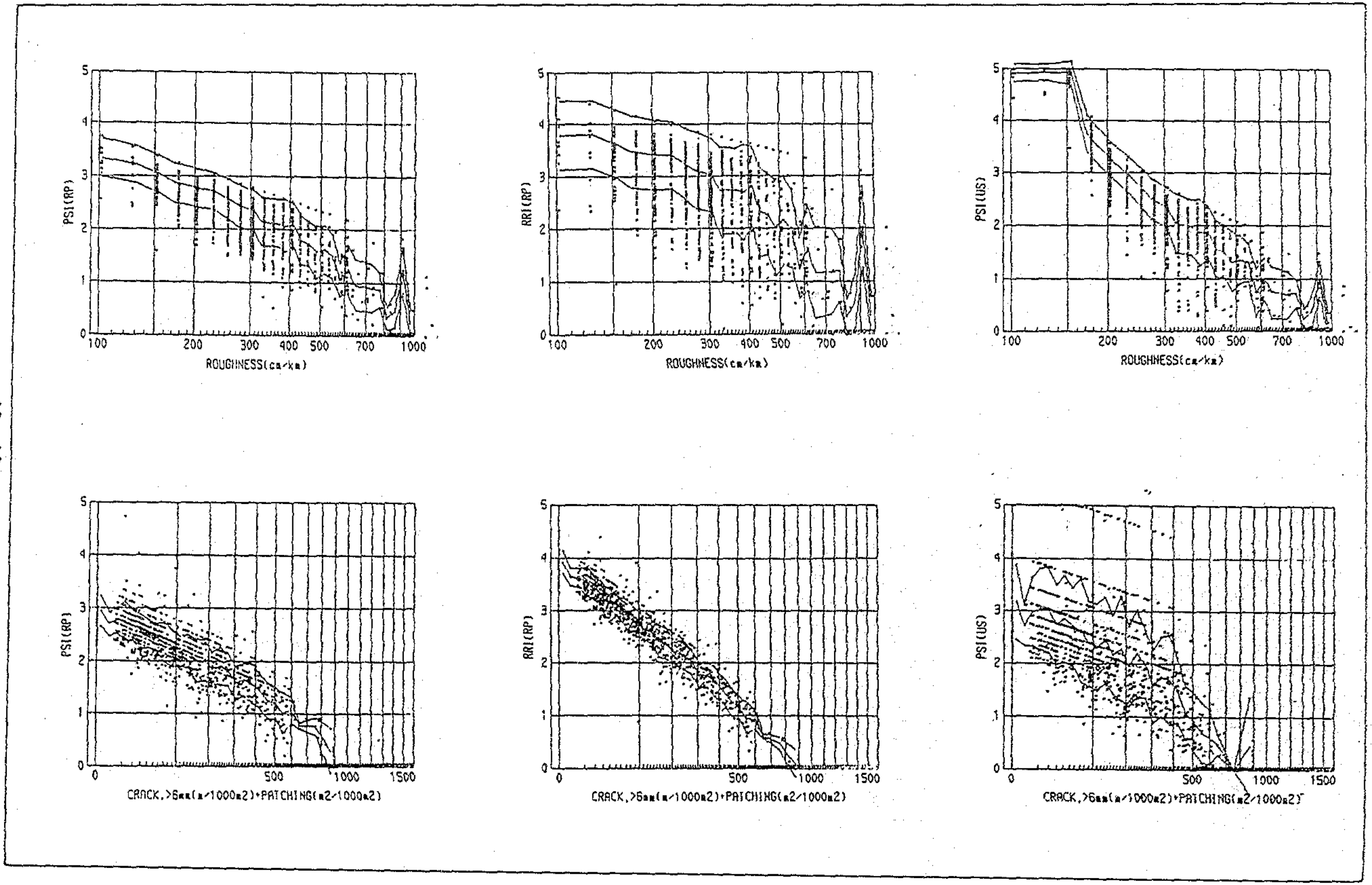


FIGURE 13.4-2 EVALUATION BY RRI AND PSI

13.5 EVALUATION OF PAVEMENT SURFACE CONDITION

13.5.1 Typical Conditions at Various Levels of RRI

Typical condition of a pavement at each level of RRI is shown on Table 13.5-1. See Figure 13.4-1.

TABLE 13.5-1 TYPICAL CONDITION OF PAVEMENT AT EACH LEVEL OF RRI

	Roughness	Surface Deterioration in terms of Class 3 and Class 4 Crack 1/	PSI (The Study)	PSI (AASHO)
RRI = 3.0	290 cm/km	60 m/1000 m^2 Transverse crack; 1/4 width of slab, longitudinal crack; 1/5 length of slab. Fatigue failure is appearing.	2.4	2.9
RRI = 2.5	320 cm/km	140 m/1000 m^2 Transverse crack; 2/3 width of slab, longitudinal crack; 1/2 length of slab. Fatigue failure is about to occur.	2.1	2.3
RRI = 2.0	340 cm/km	250 m/1000 m^2 Transverse crack; 1.2 width of slab, longitudinal crack; 0.8 length of slab. Fatique failure has occured	1.7	1.9
RRI = 1.5	370 cm/km	400 m/1000 m^2 Transverse crack; 1.8 width of slab, Longitudinal crack; 1.3 length of slab. Slab has been destroyed	1.5	1.5

1/ Patching is converted to equivalent length of crack

13.5.2 Overall Evaluation

Measured roughness, cracking and patching and RRR/PSR values were plotted in succession incorporated with corresponding RRI/PSI values in Appendix 13-4 for the North Study Section and the South Study Section.

Figure 13.5-1 shows the summary of pavement length by level of RRI.

As for the North Study Section, (Sta. Rita-Aritao), the average RRI is 2.47 and 3.17 for Manila and Cagayan Bounds, respectively. RRI of this section varies rather equally from 1 to 4.0.

It is noted the RRI of the Manila Bound lane is 2.47, very low comparing with Cagayan Bound. This value is the indication that the rehabilitation is urgently required.

While, the average RRI of the South Study Section (Calamba-Calauag) is 3.23 and 3.33 for Manila and Bicol Bounds, respectively. RRI of this section does not scatter equally, rather concentrates on RRI of near 3.9. It may mean that majority of the sections are in acceptable condition. There are no remarkable difference in RRI between both directions.

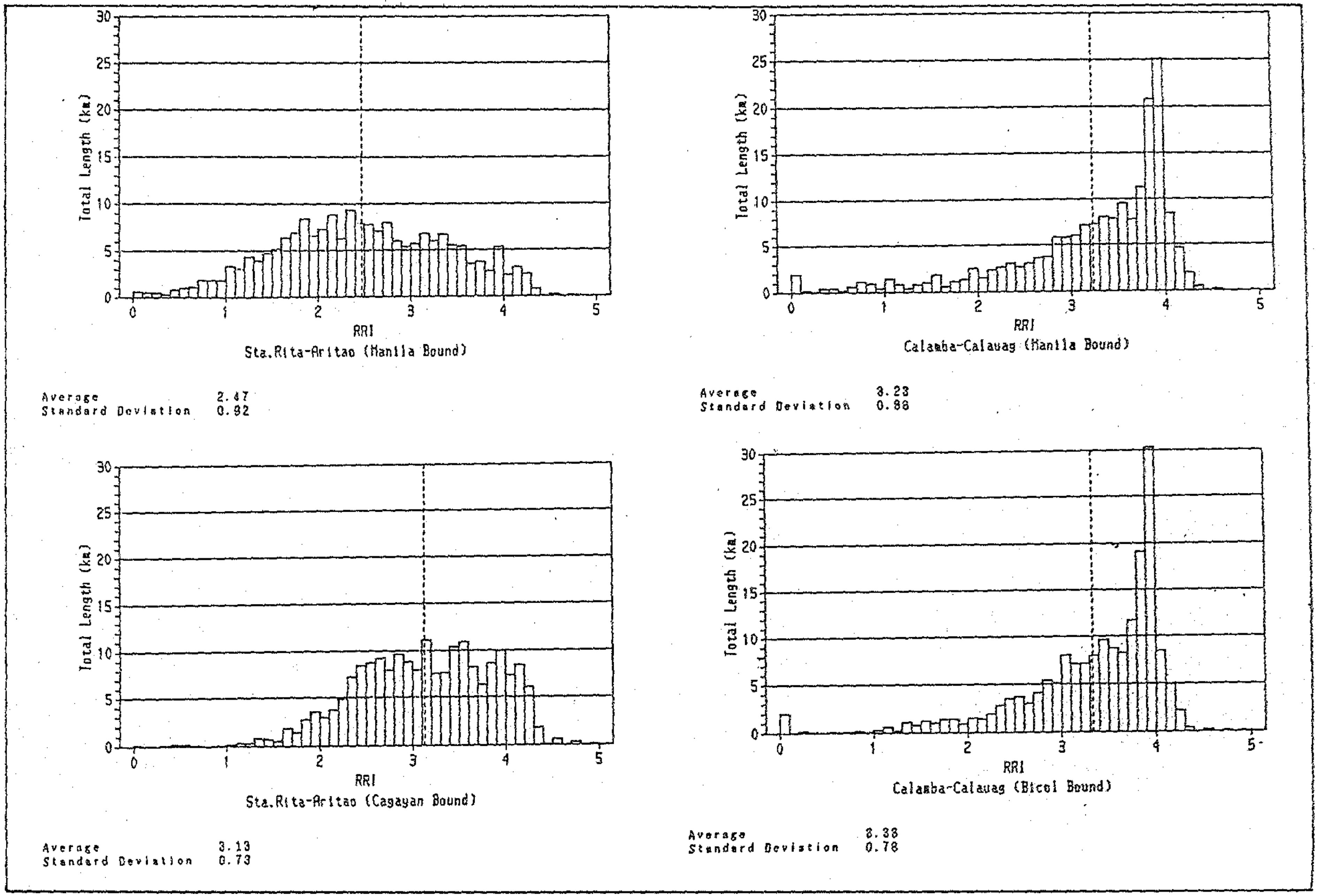


FIGURE 13.5-1 SUMMARY OF PAVEMENT LENGTHS BY LEVEL OF RRI

CHAPTER 14
PAVEMENT DETERIORATION SURVEY AND ANALYSIS

14.1 APPROACH

Pavement deterioration survey was conducted to obtain and assess the engineering informations necessary for analysis of probable causes of pavement deterioration. Those informations includes the bearing strength of subgrade and subbase, pavement material properties, traffic volume and loading, drainage condition and past construction history, among others.

In Analyzing and evaluating probable causes of pavement deteriorations, types of distress, its extent and severity should also be identified.

The survey was conducted for the selected fifteen (15) slabs, nine (9) for the North Study Section and six (6) for the South.

(1) Traffic Loading

Data and information on traffic loading were obtained from the output of IBRD assisted Pavement and Axle Load Study, October 1985, Department of Public Works and Highways.

(2) Analysis Methods

The pavement technology is still in the state of the art, though gradually but steadily transitioning from an art to a science. In developing the pavement design, many empirical design procedure have been proposed. Most of these have been based on evidence gained from the observation of existing road.

Concurrently with the development of these empirical design method, theoretical approaches have been based on analytical and mechanistic methods to estimate the stress-strain or deflection data of pavement. The main value of these theoretical approach at present lies in the interpretation and extrapolation of evidence obtained from field observation.

The representative example of the former is AASHTO design method based on serviceability-performance concept developed from the AASHO Road Test. The latter includes the theoritical analysis of Westergaard and the method of the Portland Cement Association (PCA) for fatigue cracking of portland cement concrete pavement.

In the Study, therefore, the following six (6) design/analysis methods were applied to analyze the causes of pavement deterioration.

- MPWH Method (AASHTO Interim Guide, 1972)
- AASHTO Method 1986 (AASHTO Guide, 1986)
- PCA Method
- TRRL Method
- JRA Method
- Westergaard Method

(3) Study Flow

The study flow on pavement deterioration survey and analysis is presented in Figure 14.1-1.

The study is divided into four (4) steps:

Step 1: Survey

- Collection of data on Traffic Loading
- Pavement Distress Survey
- Engineering Survey

Step 2: Analysis on Basic Factors

- Analysis on Traffic Loading
- Analysis on Pavement Material Properties

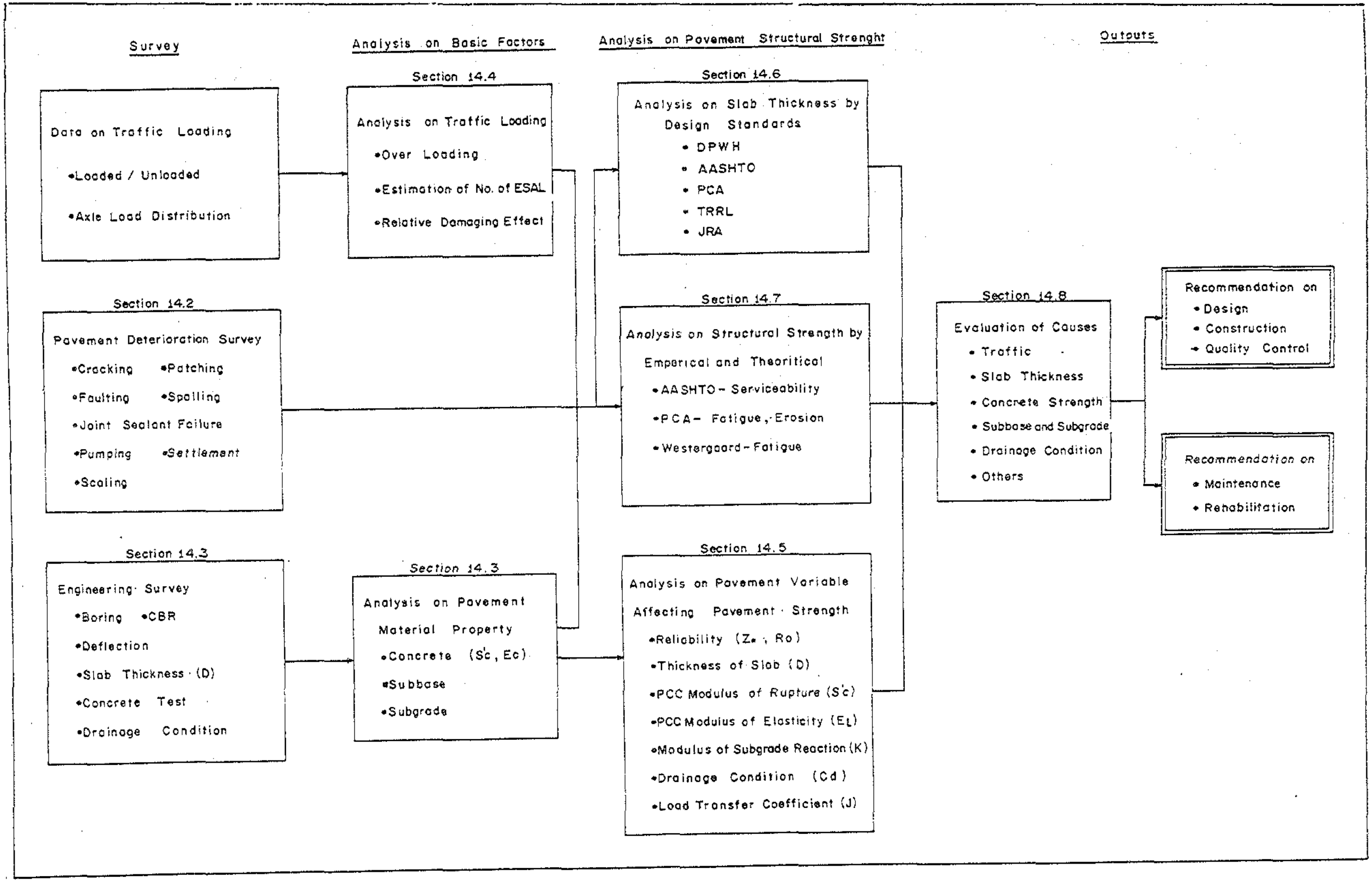
Step 3: Analysis on Pavement Structural Strength

- Analysis on Slab Thickness by Design Standard
- Analysis on Structural Strength by Empirical and Theoretical Methods
- Analysis on Pavement Variable affecting Pavement Strength.

Step 4: Evaluation

- Evaluation of Causes

TABLE 14.1-1 STUDY FLOW ON PAVEMENT DETERIORATION SURVEY AND ANALYSIS



14.2 PAVEMENT DETERIORATION SURVEY

14.2.1 Selection of Slabs for Survey

One of the major objective of the Study is to assess the causes of pavement deterioration. For this purpose, the detailed deterioration survey on the pavement was carried out for the five (5) spots, three (3) from the north study section and two (2) from the south.

The selection of spots was done in such a way that each spot should represent the topographical and ecological conditions, roadbed types, type of deterioration, the conditions of slabs and traffic characteristics through the entire length of the study section. Main consideration in selection were given to the following factors.

Geology

Along the north study section, the soft ground area seems to exist within Baliuag, Bulacan (km 50-60). The section where the pavements are seriously deteriorated are from San Ildefonso to San Miguel (Km 70-80) and from San Jose to Sta. Fe (Km 160-220).

As to south study section, there is a fault zone of the Philippine Fault between Pagbilao and Atimonan (Km 140-190), where the pavements are deteriorated in a particular shape (cave-in shape). Refer to Figure 14.2-1, the Geological Map of the Pan-Philippine Highway (Lucena-Daet). On the other hand, the transverse cracks were very clearly observed in the section from Alaminos to Tiaong (Km 70-90).

Basically, the section for the deterioration survey were selected from the section mentioned above. And then, the particular spots were further chosen taken into consideration the following factors.

Roadbed Type

For the types of roadbeds, the following two types of sections were selected:

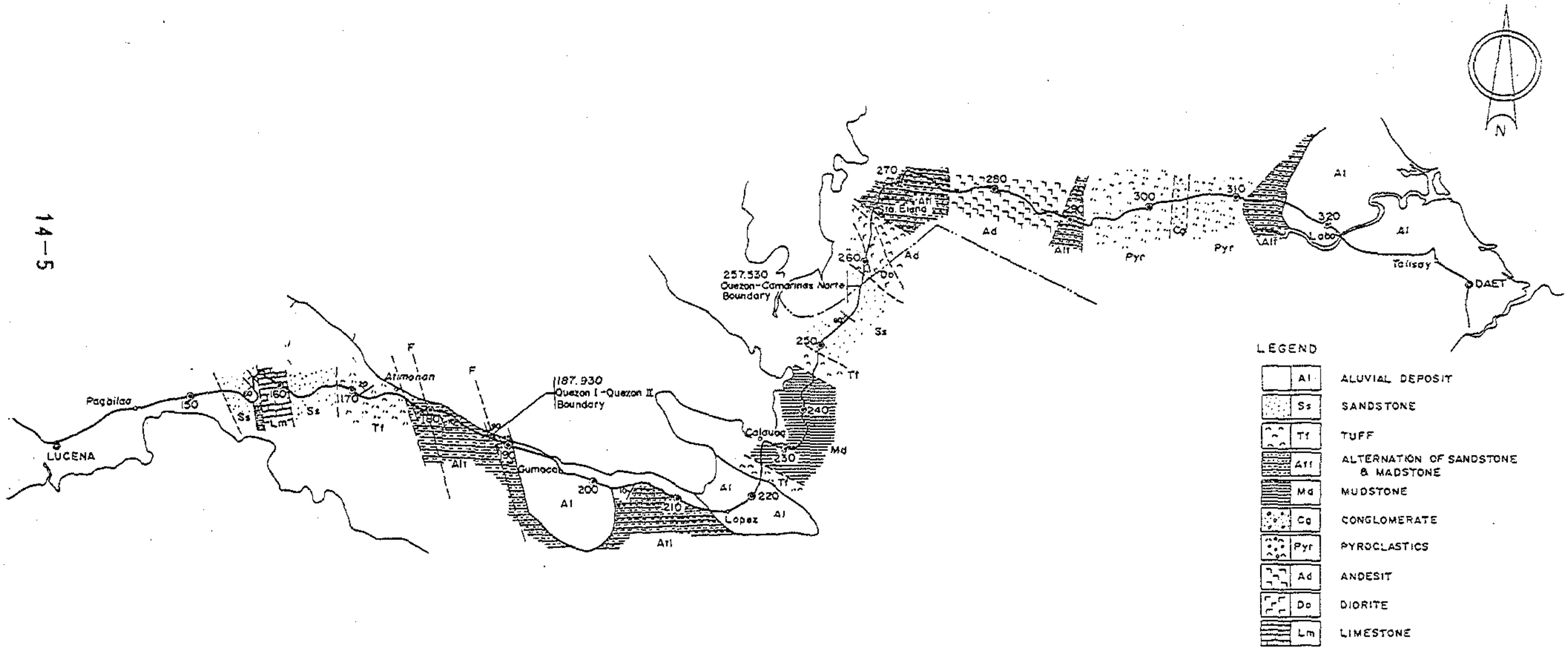
- Representative embankment section
- Representative cut section

Types of Cracks

Main deficiency of the concrete pavement is a crack. Three types of cracks, namely: longitudinal, transverse and random cracks, were prominently observed in the study section and were selected as a typical one:

- Section predominant to longitudinal crack
- Section predominant to transverse crack
- Section predominant to block/random crack

FIGURE 14.2-1 GEOLOGICAL MAP OF PAN-PHILIPPINES HIGHWAY (Lucena – Daet)



Slab Conditions

Even in a section with no variations in the past traffic volume and their load distribution, there are slabs varying in severity of the deterioration. Some have been completely destroyed while others are still good. Causes of deteriorations may be revealed by assessing these differences of slab condition. Three slabs in good, fair and bad surface condition were picked out as a unit of slabs to be surveyed from typical sections selected in advance.

Construction History

- Simultaneous construction year in a section
- Sections not recently rehabilitated

Finally, a total of 5 typical spots with 3 survey slabs in each spot were selected for survey. Thus, the total number of slabs surveyed are 15.

14.2.2 Condition of Slabs Selected for Survey

The condition of slabs selected for deterioration survey are summarized in Table 14.2-1 with brief descriptions. Refer to Appendix 14-1.

The conditions of slabs surveyed were precisely evaluated with RRI Equation established in the Study based on roughness, cracking and patching. RRI of each slab are shown in Figure 14.2-2.

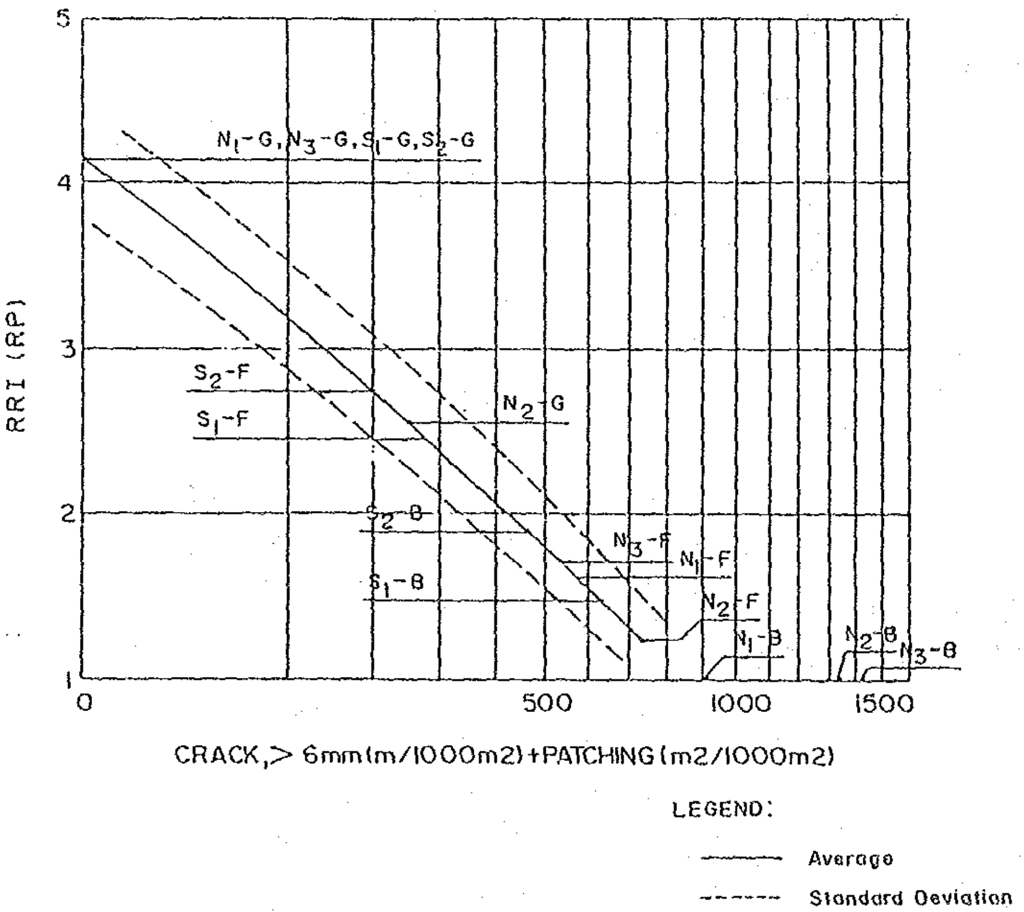


FIGURE 14.2-2 RRI VS. CRACKING SLABS FOR DETERIORATION SURVEY

TABLE 14.2-1 SLABS SELECTED FOR DETERIORATION SURVEY

Section Construction Year	Geological Conditions and Road Bed Type	Main Deficiency	No. of Survey Slab	Cracks (m) Wide	Cracks (m) Fine	Patching	Pumping	Sealant Failure	Settlement	Others
50+000-50+500 Sta. Rita-Gapan 1975	- Soft Ground - Embankment	Transverse Crack	N_1-G	none	a little	none	none	broken sealant	none	
			N_1-F	8.60 m.		none	none	broken sealant	none	Spalling
			N_1-B	13.80 m.	1.9 m.	none	none	broken sealant	Yes	Spalling
76+600-76+800 Sta. Rita-Gapan 1976	- Embankment	Longitudinal Crack	N_2-G	3.60 m.		none	none	broken sealant	none	
			N_2-F	10.90 m.		none	none	broken sealant	none	Spalling
			N_2-B	20.30 m.	7.30 m.	none	none	yes	none	Spalling
168+300-168+400 San Jose-Aritao 1978	- Cut	Block/Random Crack	N_3-G	none	a little	none	none	lack of sealant	none	
			N_3-F	8.10 m.	3.40 m.	none	none	broken sealant	none	Spalling
			N_3-B	22.10 m.	3.30 m.	none	none	no sealant	none	Joint Shattering
79+200-79+300 Sto. Tomas-Tiaong 1972	- Embankment	Transverse Crack	S_1-G	none	a little	none	none	broken sealant	none	
			S_1-F	4.10 m.	4.30 m.	none	minor	broken sealant	none	
			S_1-B	9.50 m.	5.30 m.	none	minor	broken sealant	Yes	
160+200-160+300 Lucena-Gumaca 1978	- Fault Zone - Cut	Block/Random Crack	S_2-G		4.60 m.	none	yes	no sealant	Yes	Spalling
			S_2-F	3.0 m.		none	none	broken sealant	none	
			S_2-B	6.70 m.	7.20 m.	none	none	broken sealant	Yes	

NOTE: All slabs selected were in Manila bound lanes.

14.3 ENGINEERING SURVEY

In order to obtain the data necessary to analyze causes of the pavement deteriorations, the following engineering surveys were conducted at the selected deterioration surveys slabs.

—	Geo-technical survey	:	BH-1 and 2
—	CBR test	:	TP-1 to TP-16
—	Concrete Strength test	:	3 points x 15 slabs
—	Deflection survey	:	6 points x 15 slabs
—	Drainage survey		

The survey results are presented in Appendix 14-2.

14.3.1 Geo-Technical Survey

Sta. Rita to San Jose Section (Km. 39 + 000 to Kms. 159+000), approximately 120 km. in length, passes through flat alluvial terrain in the Luzon Central Plain. Especially, topographical external apperances in the area of Bulacan Province (kms. 39+000 to kms. 84 + 500) about 45.5 km long section, seem to manifest existence of a soft ground.

In this area, there are many section whose pavement are severely deteriorated. In order to investigate if the settlement due to the soft ground affects the pavement deterioration in this area, a boring survey was conducted at two sites, shoulders of the N_1-G (Km. 50 + 020) and the N_1-B slabs (Km. 50 + 100).

The boring was composed of drilling a borehole with standard penetration test (SPT) at the sites and collecting soil samples for laboratory tests.

Laboratory tests on the following items were carried out for representative samples of each stratum:

- Natural moisture content
- Atterberg limit
- Sieve analysis
- Unconfined compression test
- Consolidation test

Boring logs for N_1-G and N_1-B are shown in Appendix 14-2. The stratification is almost the same in borings in two sites and is characterized by typical marine deposits consisting of alternate layers of sand, gravel and clayey silt. Dense sand and gravel layers with more than 30 in SPT value continue from approximately 6.0 m in depth. The layer subjected to analyze consolidation is only a very soft layer of silt found between 4.0 m and 6.0 m in depth.

The laboratory test results are summarized in Appendix 14-2.

Based on data obtained, settlement analysis were carried out at N_1-G and N_1-B. Settlement heights were analyzed to be 2.0 cm and settlements were estimated to be finished within 37 or 45 days for 90% consolidation. It is, therefore, assumed that consolidation settlement may have been completed before pavement was constructed, considering the time after the completion of embankment.

14.3.2 CBR Test

Sampling for the CBR test was carried out at the shoulder shutting on free edge of the concrete slab in each selected deterioration survey slab, for a total of 15. In addition, a sample was collected at a completely destroyed part near a longitudinal joint of S_2-B slab to check uniformity of the supporting capacity on the slab. Sampling of each spot was done for 2 layers under the concrete slab, subbase course and subgrade.

For the sampled materials, the following tests were carried out:

- Natural moisture content
- Consistency
- Particle size distribution
- Specific gravity
- Compaction under condition of natural moisture content
- CBR under condition of natural moisture content

The CBR test mentioned above was performed in accordance with a test method stipulated in Cement Concrete Pavement Manual, 1980, Japan Road Association, (JRA method). This method is to estimate approximate CBR values of the existing subbase and subgrade under in-situ density and moisutre content in laboratory. The procedure is outlined below.

- Specimens passing the 37.5 mm sieve with natural moisture content is put into mold with an internal diameter of 180 mm in 3 approximately equal layers.

- Each layer is compacted by 67 uniformly distributed blow from the hammer weighing 4.5 kg dropping free from a height of 45 cm.
- Penetration test is carried out for a compacted specimen: after soaking for days.

The result of soil and CBR tests are shown in Appendix 14-2.

For the subbases, the lowest volume of CBR is 3 (N_1-B) and the highest is 53 (N_2-B). As to the subgrade, the lowest is 1.0 (S_2-B) and the highest 46 (N_3-B). On the average, the North Study Section shows the higher value of CBR for both subbase and subgrade comparing with values in the South Study Section.

Estimated CBR and K value combined for subgrade and subbase are shown in Table 14.3-1. The estimation method is explained in Chapter 16.

14.3.3 Concrete Strength Test

Concrete coring was carried out at 3 points for each deterioration survey slab in order to estimate concrete strength and thickness of these slabs.

TABLE 14.3-1 ESTIMATED COMBINED CBR AND K VALUE

Station	Location	Slab Condition	CBR_m Value	K-Value (Psi)
North Study Section				
Km 50	TP – 2	G	6	160
	TP – 3	F	16	230
	TP – 1	B	5	140
Km 76	TP – 4	G	38	400
	TP – 6	F	27	310
	TP – 5	B	46	470
Km 168	TP – 7	G	12	210
	TP – 9	F	43	440
	TP – 8	B	47	480
South Study Section				
Km 79	TP – 10	G	2	80
	TP – 12	F	6	160
	TP – 11	B	7	170
Km 160	TP – 13	G	23	280
	TP – 14	F	3	100
	TP – 15	B	2	80
	TP – 16	B	2	80
Average			18	250

Core samples were carefully measured both the length and the diameter and then compression tests were made.

The results are shown in Appendix 14-2.

Core samples yielded various compressive strengths ranging from 233.15 kg/cm^2 to as high as 359.92 kg/cm^2. There are also differences in the length of core (thickness of slab). The thinnest is 8.125 in (20.7 cm) and the thickest 11.5625 in (29.4 cm).

The estimated compressive strength, modulus of rupture and elestic modulus are shown in Table 14.3-2. The estimation methods are discussed in Chapter 16.

14.3.4 Deflection Survey

Rebound deflection test was undertaken at several points (6 in average) of each deterioration survey slab using a Benkelman Beam apparatus.

Rebound deflection was measured in accordance with the procedure described in the AASHTO Road Test: Report 5, as follows:

- Wheel Road of truck used for measurement was 4080 kg.
- The support of Benkelman Beam apparatus was placed on the shoulder to avoid the movement of the apparatus.
- Measurements were done at free edges, joints, cracks and deteriorated pavement areas directed by the Study Team to determine load-transfer capability.

The results are shown in Appendix 14-2.

The highest deflection is 1.8 mm (S_1-B) and the smallest 0 (N_3-B). The average deflection is 0.28 mm.

TABLE 14.3-2 ESTIMATED STRENGTH OF CONCRETE

Survey Slab	Thickness of Slab	Estimated Compressive Strength psi	(kg/cm^2)	Estimated Modulus of Rupture psi	(kg/cm^2)	Estimated Elastic Modulus psi x 10^6	(kg/cm^2)
N_1-G	9.98	2,999	(211.3)	525	(36.9)	3.12	(0.22)
N_1-F	9.40	3,456	(243.5)	605	(42.6)	3.35	(0.24)
N_1-B	8.81	3,598	(253.5)	630	(44.4)	3.42	(0.24)
N_2-G	10.02	2,564	(180.6)	449	(31.6)	2.89	(0.20)
N_2-F	9.96	2,600	(183.2)	455	(32.1)	2.91	(0.21)
N_2-B	9.44	2,531	(178.32)	443	(31.2)	2.87	(0.20)
N_3-G	9.38	2,912	(205.2)	510	(35.9)	3.08	(0.22)
N_3-F	9.23	3,132	(220.7)	548	(38.6)	3.19	(0.22)
N_3-B	8.60	2,451	(172.7)	429	(30.2)	2.82	(0.19)
S_1-G'	11.25	2,734	(192.6)	478	(33.7)	2.98	(0.21)
S_1-F	8.63	2,576	(181.5)	451	(31.8)	2.89	(0.20)
S_1-B	8.71	2,557	(180.2)	447	(31.5)	2.88	(0.20)
S_2-G	9.77	3,163	(222.8)	554	(39.0)	3.21	(0.23)
S_2-F	9.60	2,978	(209.8)	521	(36.7)	3.11	(0.22)
S_2-B	9.27	3,293	(232.0)	576	(40.6)	3.27	(0.23)
Average	9.47	2,903	(204.5)	508	(35.8)	3.07	(0.22)

14.3.5 Drainage Condition Survey

The drainage conditions of the pavements were investigated simultaneously with the ocular survey of pavement deterioration.

The drainage conditions along the Study Road were classified into three categories, Good (G), Fair (F), and Bad (B).

Along the entire Study Section, no surface drainage facilities including side ditches were observed except few section in the urban area. It was, therefore, feared that not only road surface water but also water from the mountain slopes run onto pavements and possibly penetrate into pavement structures.

The subsurface drainage such as cross pipes were sometimes not observed even at the places where they should be installed because of a concentration of water at mountain sides.

14.4 ANALYSIS ON TRAFFIC LOADING

A vehicle may be fully loaded, unloaded or in an intermediate condition. The axle loads which it imposes on the road, and the structural damages which it will do depend on the degree of loading. To assess the severity of a given traffic flow in terms of the damage it may do to the pavement, it is necessary to know the composition of the axle loads in terms of their weight. This is often termed the axle-load spectrum (distribution) of the traffic.

The AASHO Road Test established the relative damaging effect of a nine range of axle loads. The number of passenger of each axle load required to cause the same damage as one passage of a standard axle of 18,000 lb (1960 kg) as an equivalent factor (18 kip ESAL).

14.4.1 Past Traffic Volume

The past traffic volume that the road served after its completion to date was estimated based on the average daily traffic (ADT) in 1968, 1980 and 1986 which were obtained from nine (9) representative section, four (4) for the north study section and five (5) for the south.

The average daily traffic volume and the total traffic volume to date are shown in Figure 14.4-1. Refer to Appendix 14-3.

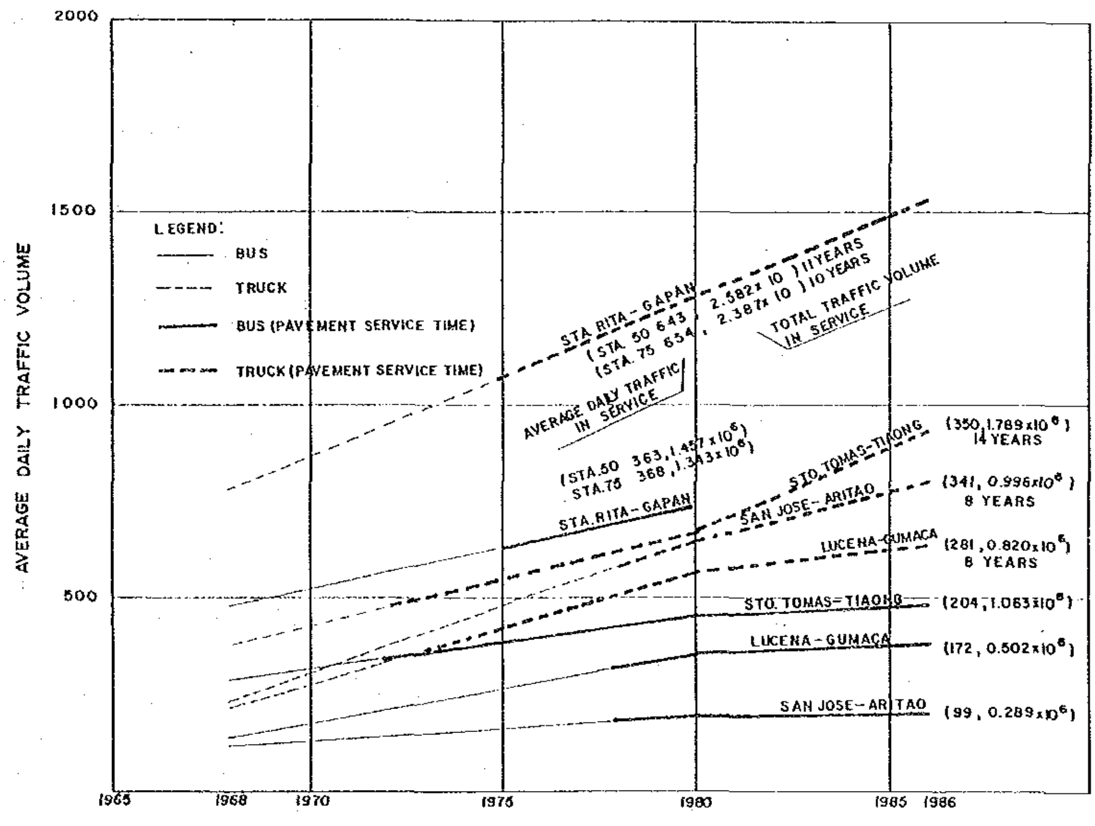


FIGURE 14.4-1 AVERAGE DAILY TRAFFIC VOLUME OF TRUCKS AND BUSES (BOTH DIRECTION)

14.4.2 Axle Load Distribution

The Department of Public Works and Highways, with the assistance from IBRD, conducted a nationwide survey on axle load and presented the report entitled "Pavement and Axle Load Study" in 1985.

In that report, the load distributions and axle loads were surveyed at many sections, among others which are included in this Study are the following stations:

- Station 01: Sta. Fe – San Jose Location
- Station 02: Gapan – Plaridel Location
- Station 07: Calamba – Sto. Tomas Location
- Station 29: Lucena – Lopez Location

The load distribution to be applied to the sections under this Study were determined based on the data obtained from the load distribution of the four (4) stations mentioned above as shown in Appendix 14-3.

Since light vehicles does little damage to the pavements, it is generally accepted that only heavy trucks and buses are taken into consideration for pavement design, especially so for the analysis of the causes of the pavement deterioration.

The Pavement and Axle Load Study, however, manifested only the load distributions of loaded trucks. In this Study, therefore, axle loads of the empty trucks were analyzed by empty ratio and the vehicle weight of each type of trucks. The axle load distribution of buses was also estimated using data of all survey stations in Luzon Island covered by the Axle Load Study because no available data was obtained from the survey results of the above four (4) stations.

Among the design methods which will be applied in the analysis of the causes of pavement deteriorations require the axle load distributions to be separated into single and tandem axles. Therefore, besides the distribution that all axles were treated as a single axle, the distributions for single and tandem axles were also derived from the basic data of the Pavement and Axle Load Study.

The axle load distributions of heavy trucks and buses by both bounds of each section are shown in Appendix 14-3.

14.4.3 Estimated Numbers of ESAL

The cummulative numbers of 18 kip equivalent single axle loads (ESAL) from the completion of the road to date were estimated based on traffic volumes shown in Figure 14.4-1 and axle load distribution presented in Appendix 14-3. The estimates were only done for trucks and buses. For the analysis of the slabs thickness required by the design standards to assess the causes of the pavement deterioration, the ESAL for 20 years after the completion of the road was also estimated.

The outputs of the estimates are presented in Table 14.4-1 for trucks, Table 14.4-2 for buses and Table 14.4-3 for the summary.

The axle load equivalent factors used are for slab thickness of 9 inches (23 cm) and P_t (Terminal Serviceability Index) of 2.0 and are presented in Table 14.4-4.

TABLE 14.4-1 NUMBER OF AXLES AND ESAL OF HEAVY TRUCKS

	Sta. Rita-Gapan		San Jose-Aritao		Sto. Tomas-Tiaong		Lucena-Gumaca	
	Manila Bound	Cagayan Bound	Manila Bound	Cagayan Bound	Manila Bound	Bicol Bound	Manila Bound	Bicol Bound
Average Truck Volume/day 1/	643	643	341	341	350	350	281	281
Single Axle 2/								
1. Max Load (tons)	19	16	18	16	15	15	16	16
2. No. of Axles/day	1,200	1,400	600	766	680	706	522	554
a. within load limit	945	1,320	514	728	598	660	444	510
b. overloaded	255	80	86	38	82	46	78	44
3. No. of ESAL/day	2,432	510	812	241	608	276	568	259
a. within load limit	135	136	86	70	72	64	60	54
b. overloaded	2,997	374	726	171	536	212	508	205
Tandem Axles 3/								
1. Max. Load (tons)	38	32	34	30	28	26	28	26
2. No. of Axles/day	162	100	158	72	80	66	81	68
a. within load limit	6	14	8	28	8	12	8	12
b. overloaded	156	86	150	44	72	54	73	56
3. No. of ESAL/day	2,885	1,221	2,600	656	794	456	819	468
a. within load limit	4	10	2	32	8	4	8	2
b. overloaded	2,881	1,211	2,598	624	786	452	811	466
Total No. of ESAL/day								
1. Total	5,317	1,731	3,412	897	1,402	732	1,387	727
a. axles within load limit	139	146	88	102	80	68	68	56
b. axles overloaded	5,178	1,585	3,324	795	1,322	664	1,319	671

NOTES: 1/Average daily truck traffic since completion of construction to date
2/Loaded and empty trucks
3/Loaded trucks only (empty trucks are induced in single axle class)

TABLE 14.4-2 NUMBER OF AXLES AND ESAL OF BUSES

	Sta. Rita-Gapan		San Jose-Aritao		Sto. Tomas-Tiaong		Lucena-Gumaca	
	Manila Bound	Cagayan Bound	Manila Bound	Cagayan Bound	Manila Bound	Bicol Bound	Manila Bound	Bicol Bound
Average Bus Volume/day 1/	363	363	99	99	204	204	172	172
Maximum Loads (tons)	11	11	11	11	11	11	11	11
Number of Axles/day	726	726	198	198	408	408	344	344
a. within load limit	568	568	154	156	320	320	270	270
b. overloaded	158	158	44	42	88	88	74	74
Number of ESAL/day	475	475	131	129	268	268	226	226
a. within load limit	178	178	50	48	100	100	84	84
b. overloaded	297	297	81	81	168	168	142	142

NOTES: 1/ Average daily bus traffic since completion of construction to date. All buses are single axles.

TABLE 14.4-3 SUMMARY OF TRAFFIC VOLUME

Section Bound			Sta. Rita-Gapan Station 50 Manila Bound	Sta. Rita-Gapan Station 50 Cagayan Bound	Sta. Rita-Gapan Station 75 Manila Bound	Sta. Rita-Gapan Station 75 Cagayan Bound	San Jose-Aritao Station 168 Manila Bound	San Jose-Aritao Station 168 Cagayan Bound	Sto. Tomas-Tiaong Station 79 Manila Bound	Sto. Tomas-Tiaong Station 79 Bicol Bound	Lucena-Gumaca Station 160 Manila Bound	Lucena-Gumaca Station 160 Bicol Bound
Avarage Daily Traffic to Date		T	643	643	654	654	341	341	350	350	281	281
		B	363	363	368	368	99	99	204	204	172	172
		TL	1,006	1,006	1,022	1,022	440	440	554	554	453	453
Average Daily Traffic for 20 yrs		T	761	761	788	788	512	512	407	407	401	401
		B	416	416	429	429	120	120	222	222	240	240
		TL	1,177	1,177	1,217	1,217	632	632	629	629	641	641
Cummulative Traffic to Date	Traffic Volume	T	2.582×10^{6}	2.582×10^{6}	2.387×10^{6}	2.387×10^{6}	0.996×10^{6}	0.996×10^{6}	1.789×10^{6}	1.789×10^{6}	0.821×10^{6}	0.821×10^{6}
		B	1.457×10^{6}	1.457×10^{6}	1.343×10^{6}	1.343×10^{6}	0.289×10^{6}	0.289×10^{6}	1.042×10^{6}	1.042×10^{6}	0.502×10^{6}	0.502×10^{6}
		TL	4.039×10^{6}	4.039×10^{6}	3.730×10^{6}	3.730×10^{6}	1.285×10^{6}	1.285×10^{6}	2.831×10^{6}	2.831×10^{6}	1.323×10^{6}	1.323×10^{6}
	E S A L	T	2.134×10^{7}	0.695×10^{7}	1.959×10^{7}	0.545×10^{7}	0.996×10^{7}	0.261×10^{7}	0.716×10^{7}	0.374×10^{7}	0.405×10^{7}	0.212×10^{7}
		B	0.191×10^{7}	0.191×10^{7}	0.016×10^{7}	0.041×10^{7}	0.038×10^{7}	0.037×10^{7}	0.137×10^{7}	0.137×10^{7}	0.066×10^{7}	0.066×10^{7}
		TL	2.325×10^{7}	0.88[illegible]x10^{7}	1.975×10^{7}	0.586×10^{7}	1.034×10^{7}	0.298×10^{7}	0.853×10^{7}	0.511×10^{7}	0.471×10^{7}	0.278×10^{7}
Cummulative Traffic for 20 years	Traffic Volume	T	5.555×10^{6}	5.555×10^{6}	5.752×10^{6}	5.752×10^{6}	3.738×10^{6}	3.738×10^{6}	2.971×10^{6}	2.971×10^{6}	2.927×10^{6}	2.927×10^{6}
		B	3.037×10^{6}	3.037×10^{6}	3.132×10^{6}	3.132×10^{6}	0.876×10^{6}	0.876×10^{6}	1.621×10^{6}	1.621×10^{6}	1.752×10^{6}	1.752×10^{6}
		TL	8.592×10^{6}	8.592×10^{6}	8.884×10^{6}	8.884×10^{6}	4.614×10^{6}	4.614×10^{6}	4.592×10^{6}	4.592×10^{6}	4.679×10^{6}	4.679×10^{6}
	E S A L	T	4.591×10^{7}	1.495×10^{7}	4.721×10^{7}	1.313×10^{7}	3.738×10^{7}	0.980×10^{7}	1.190×10^{7}	0.621×10^{7}	1.444×10^{7}	0.756×10^{7}
		B	0.398×10^{7}	0.398×10^{7}	0.037×10^{7}	0.096×10^{7}	0.115×10^{7}	0.112×10^{7}	0.213×10^{7}	0.213×10^{7}	0.230×10^{7}	0.230×10^{7}
		TL	4.98[illegible]x10^{7}	1.893×10^{7}	4.758×10^{7}	1.409×10^{7}	3.853×10^{7}	1.092×10^{7}	1.403×10^{7}	0.834×10^{7}	1.674×10^{7}	0.986×10^{7}

- Average daily traffic were projected based on annual growth rate from 1968-1986
- T: Trucks (Single and tandem axle); B: Buses

TABLE 14.4-4 AXLE LOAD EQUIVALENCY FACTOR

Slab Thickness = 9 in; Pt = 2.0

Axle Load			Axle Load Equivalency Factor	
Kips	Tons	KN	Single	Tandem
2	0.907	8.889	.002	.0001
4	1.814	17.778	.002	.0005
6	2.721	26.667	.010	.002
8	3.628	35.556	.032	.005
10	4.535	44.444	.081	.012
12	5.442	43.333	.175	.025
14	6.349	62.222	.338	.047
16	7.256	71.111	.601	.081
18	8.163	80.000	1.00	.132
20	9.070	88.889	1.58	.204
22	9.977	97.778	2.38	.305
24	10.884	106.667	3.47	.441
26	11.791	115.556	4.88	.620
28	12.698	124.444	6.70	.850
30	13.605	133.333	8.98	1.14
32	14.512	142.222	11.8	1.50
34	15.420	151.111	15.3	1.95
36	16.327	160.000	19.5	2.49
38	17.234	168.889	24.6	3.13
40	18.141	177.778	30.7	3.89
42	19.048	186.667	38.0	4.78
44	19.955	195.556	46.6	5.82
46	20.862	204.444	56.7	7.02
48	21.769	213.333	68.4	8.40
50	22.676	222.222	82.0	9.98
52	23.583	231.111		11.8
54	24.490	240.000		13.8
56	25.397	248.889		16.1
58	26.304	257.778		18.7
60	27.211	266.667		21.6
62	28.118	275.556		24.9
64	29.025	284.444		28.5
66	29.932	293.333		32.6
68	30.839	302.222		37.1
70	31.746	311.111		42.1
72	32.653	320.000		47.6
74	33.560	328.889		53.6
76	34.467	337.778		60.3
78	35.374	346.667		67.7
80	36.281	355.556		75.7
82	37.188	364.444		84.4
84	38.095	373.333		94.0
86	39.002	382.222		104.0
88	39.909	391.111		116.0
90	40.816	400.000		128.0

TABLE 14.4-5 RELATIVE DAMAGING EFFECTS (PER DAY)

	Section Bound	Sta. Rita-Gapan		San Jose-Aritao		Sto. Tomas-Tiaong		Lucena-Gumaca	
		Manila Bound	Cagayan Bound	Manila Bound	Cagayan Bound	Manila Bound	Bicol Bound	Manila Bound	Bicol Bound
Actual Traffic Volume	Truck	643	643	341	341	350	350	281	281
	Bus	363	363	99	99	204	204	172	172
	Total (B)	1,006	1,006	440	440	554	554	453	453
No. of Axle	Truck (Single)	1,200	1,400	600	776	680	706	522	544
	Truck (Tandem)	162	100	158	72	80	66	81	68
	Bus	726	726	198	198	408	408	344	344
	Total (C)	2,088	2,226	956	1,036	1,168	1,180	947	966
No. of ESAL	Truck (Single)	2,432	510	812	241	608	276	568	259
	Truck (Tandem)	2,885	1,221	2,600	656	794	456	819	468
	Bus	475	475	131	129	268	268	226	226
	Total (A)	5,792	2,206	3,543	1,026	1,670	1,000	1,613	953
No. of ESAL/Actual Volume (A/B)		5.76	2.19	8.05	2.33	3.01	1.81	3.56	2.10
No. of ESAL/No. of Axle (A/C)		2.77	0.99	3.71	0.99	1.43	0.85	1.70	0.99

NOTE: Relative Damaging Effects = A/B

Figure 14.4-2 (a)-(b) and 14.4-3 (a)-(b) show the distribution (%) of each axle load of heavy trucks (single and tandem axles) for Sta. Rita-Gapan Section and Lucena-Calauag Section, respectively. Those two figures are shown as the representatives of the Study Sections, while figures for all sections are presented in Appendix 14-4.

Figure (a) shows distribution (%) of each load by number of axles, while Figure (b) present the same by number of the equivalent single axle loads (ESAL) when converted to ESAL.

When comparing Figures (a) and (b), caution should be given to the great difference in the distribution (%) of each axle load between the number of axles and the number of ESAL.

14.4.4 Relative Damaging Effects

The damaging effects to the pavement imposed by each axle load are expressed by the axle load equivalent factor.

Republic Act 4136 promulgated in 1964, regulated the weights of goods vehicles in the Philippines as 8 tons for most heavily loaded wheel and 14.5 tons for most heavily loaded axle group. The damaging effects of these vehicles are compared with those of vehicles, the weight of which were actually surveyed.

	Weight	Damaging Effects ($t = 9$ in., $P_t = 2.0$)
Regulation		
Single Axle	8 t (18 kip)	1.000
Tandem Axle	14.5 t (32 kip)	1.500
Actual Maximum		
Single Axle	19 t (42 kip)	38.0
Tandem Axle	38 t (84 kip)	94.0

Relative damaging effects (RDE) are computed for each direction of the sections, using the following formula. Table 14.4-5 summarizes the analysis of the damaging effects.

$$RDE = \frac{\text{Number of 18 kip Equivalent Single Axle Load (ESAL)}}{\text{Number of Actual Traffic Volume}}$$

As shown in Table 14.4-5 the highest RDE of 8.0, was found at Manila Bound of San Jose-Aritao Section and the lowest RDE of 1.8, at Bicol Bound Sto. Tomas-Tiaong Section. It simply means that the number of vehicles should be considered by 8.0 times and 1.8 times when the pavement design is executed.

FIGURE 14.4-2 (a) HEAVY TRUCKS
AXLE LOAD DISTRIBUTION (NO. OF AXLES)
STA. RITA – GAPAN SECTION

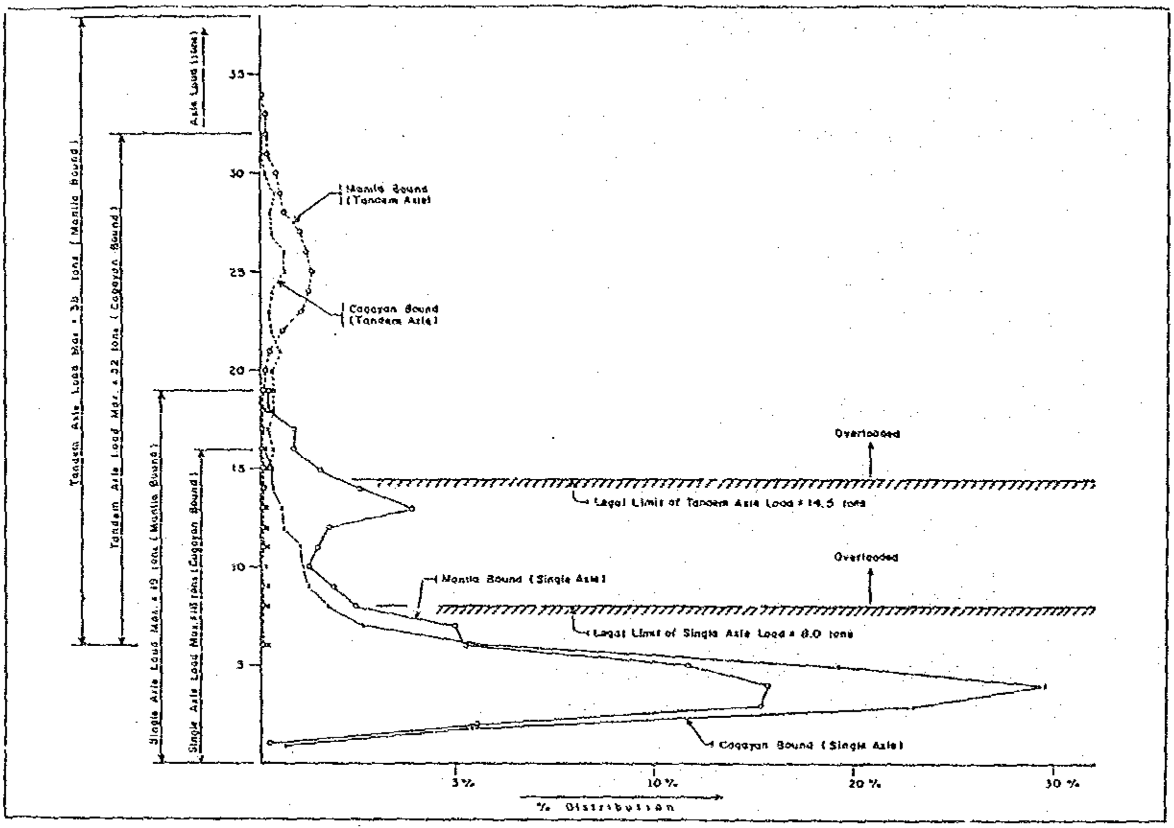


FIGURE 14.4-2 (b) HEAVY TRUCKS
AXLE LOAD DISTRIBUTION (ESAL)
STA. RITA – GAPAN SECTION

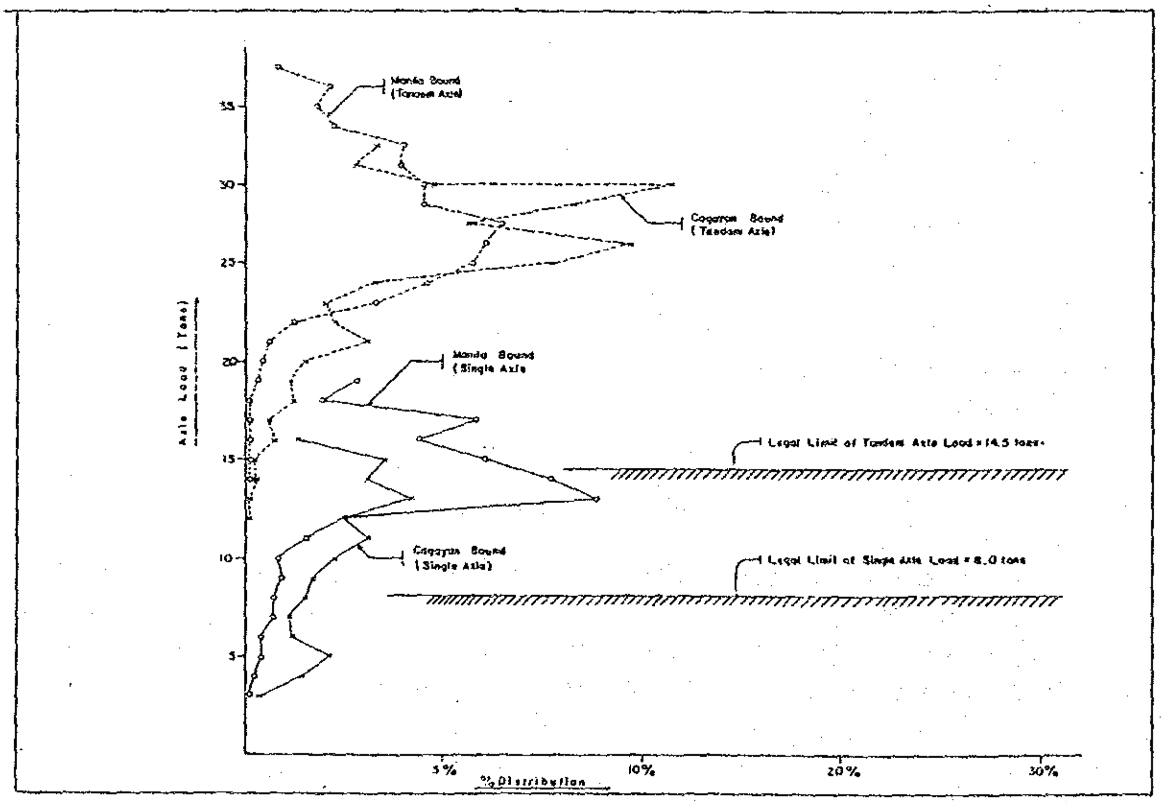


FIGURE 14.4-3 (a) HEAVY TRUCKS
AXLE LOAD DISTRIBUTION (NO. OF AXLES)
LUCENA – CALAUAG SECTION

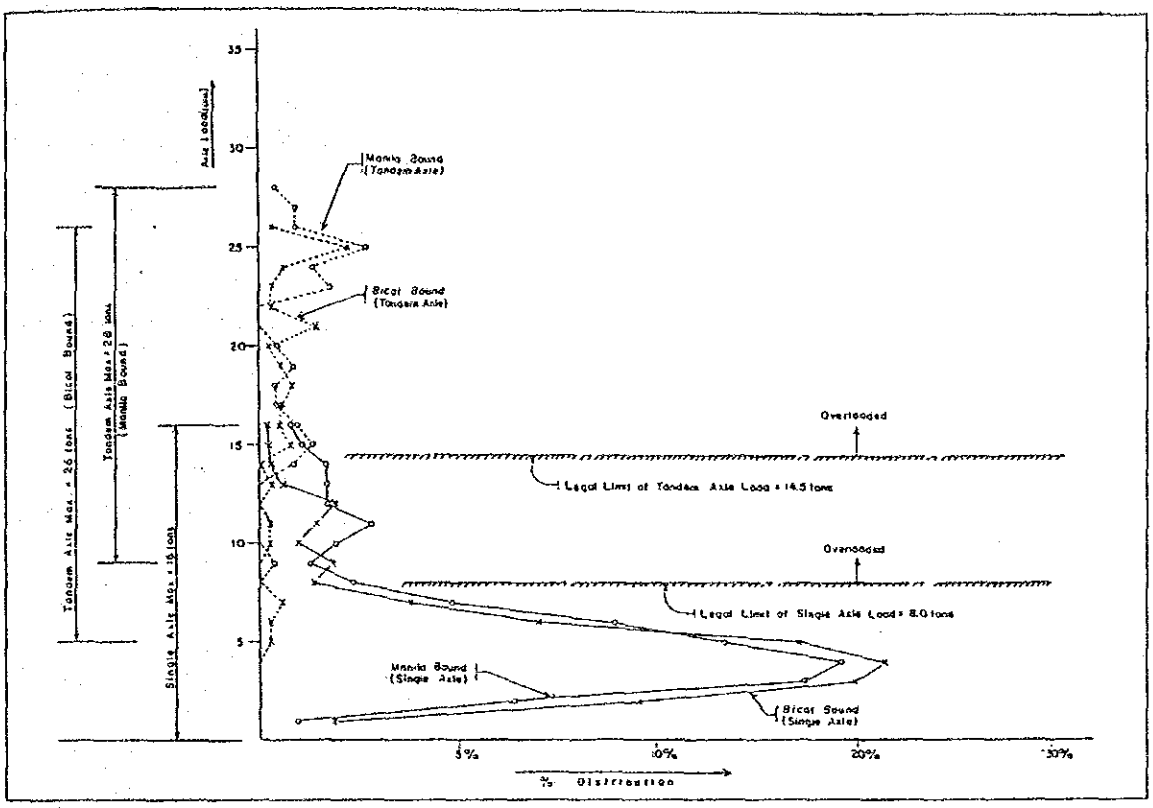
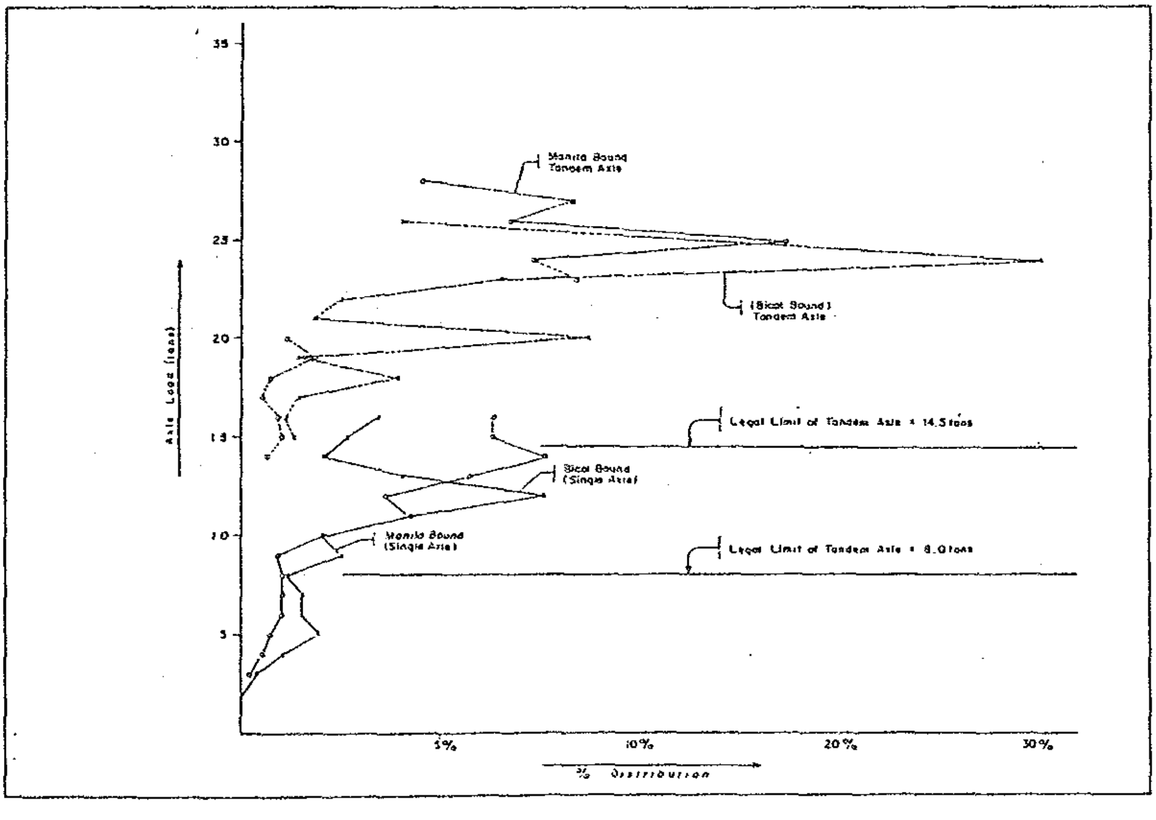


FIGURE 14.4-3 (b) HEAVY TRUCKS
AXLE LOAD DISTRIBUTION (ESAL)
LUCENA – CALAUAG SECTION



14.5 ANALYSIS ON VARIABLES AFFECTING PAVEMENT STRENGTH

14.5.1 Average and Variation Ranges of Pavement Variables

There are several pavement variables affecting the performance and the service life of the pavement, some are severely and others not so much. In evaluating the probable causes of the pavement deterioration, it is quite meaningful to estimate the effects of these variables against pavement strength.

In estimating the degree/severity of these effects, AASHTO Design Equation was used as a model because the equation involves most number of pavement variables than others do.

The average values of variables were set at the mathematical averages that surveyed slabs showed. The variation ranges of pavement variables were also given taking into consideration the maximum and minimum values of survey slabs. See Table 14.5-1.

TABLE 14.5-1 AVERAGE AND VARIATION RANGE OF PAVEMENT VARIABLES

			Average	Range of Variation			Remarks
				Poor		Good	
Reliability	ZR	Standard Normal Deviation	0	90% (-1.282)	～	95% (-1.645)	
	So	Standard Error of Prediction	0	0.3			Constant
Initial Seviceability Index	Po(R)	Initial Psi Measured by Roughness	4.5(150)	4.1 (250)	～	4.5(150)	( ) Roughness
Concrete	D	Thickness of Slab (Inch)	9	8	～	12	
	S'c	Modulus of Rupture (Psi)	500	450 (2600)	～	650(3700)	( )Compressive Strength
	Ec	Modulus of Elasticity (Psi)	3.05×10^6	2.87×10^6	～	3.47x10	
Subgrade on Subbase	K	Modulus of Subgrade (Pci) Reaction	250	80(3)	～	550(56)	( ) CBR
Transverse Joint	J	Load Transfer Coefficient	4.0	4.4	～	3.6	
Drainage System	Cd	Drainage Coefficient	0.9	0.7	～	1.1	

Note: 1.) Average values is the mathematics average obtained from slabs surveyed.
2.) When analyzing the affecting ratio of one variable, other variables were kept constant at average values.

14.5.2 Analysis on Effects of Pavement Variables

The analysis to evaluate the effects of pavement variables was done by a computer with a program for AASHO Design Equation. When calculating the effect of one pavement variable, other variables were kept in the average values.

Figure 14.5-1 illustrates the degree of the effects. The effects are converted to the slab thickness to obtain the equivalent slab thickness due to change in the values of pavement variables. Refer to Appendix 14-5.

Within the practical ranges of pavement variables in this country, the order of effects are summarized as shown in Table 14.5-2.

TABLE 14.5-2 EFFECTS OF PAVEMENT VARIABLES

	Range	In Case of D = 9 in.	In Case of D = 12 in.
Drainage Coefficient	0.7 – 1.1	2.36 in.	2.90 in.
Concrete	450 – 650 psi	1.86 in.	2.34 in.
Reliability	0 – 95%	1.67 in.	2.00 in.
Subgrade	80 – 550	1.24 in.	1.12 in.
Load Transfer	4.4 – 3.8	0.76 in.	0.94 in.
Initial RRI	4.1 – 4.5	0.32 in.	0.41 in.

Among pavement variables, drainage coefficients show the greatest effect that thickness of slab may differ by 2.36 in. (6.0 cm) if drainage factor changes from 0.7 (very poor) to 1.1 (excellent).

The concrete strength and reliability have almost same effects, about 1.8 in. (4.6 cm) in the range of 450 psi (32 kg/cm^2) and 650 psi (46 kg/cm^2).

The strength of subgrade gives rather small effect, about 1.3 (3.3 cm) in the range of K = 80 (CBR = 3) and K = 550 (CBR = 56).

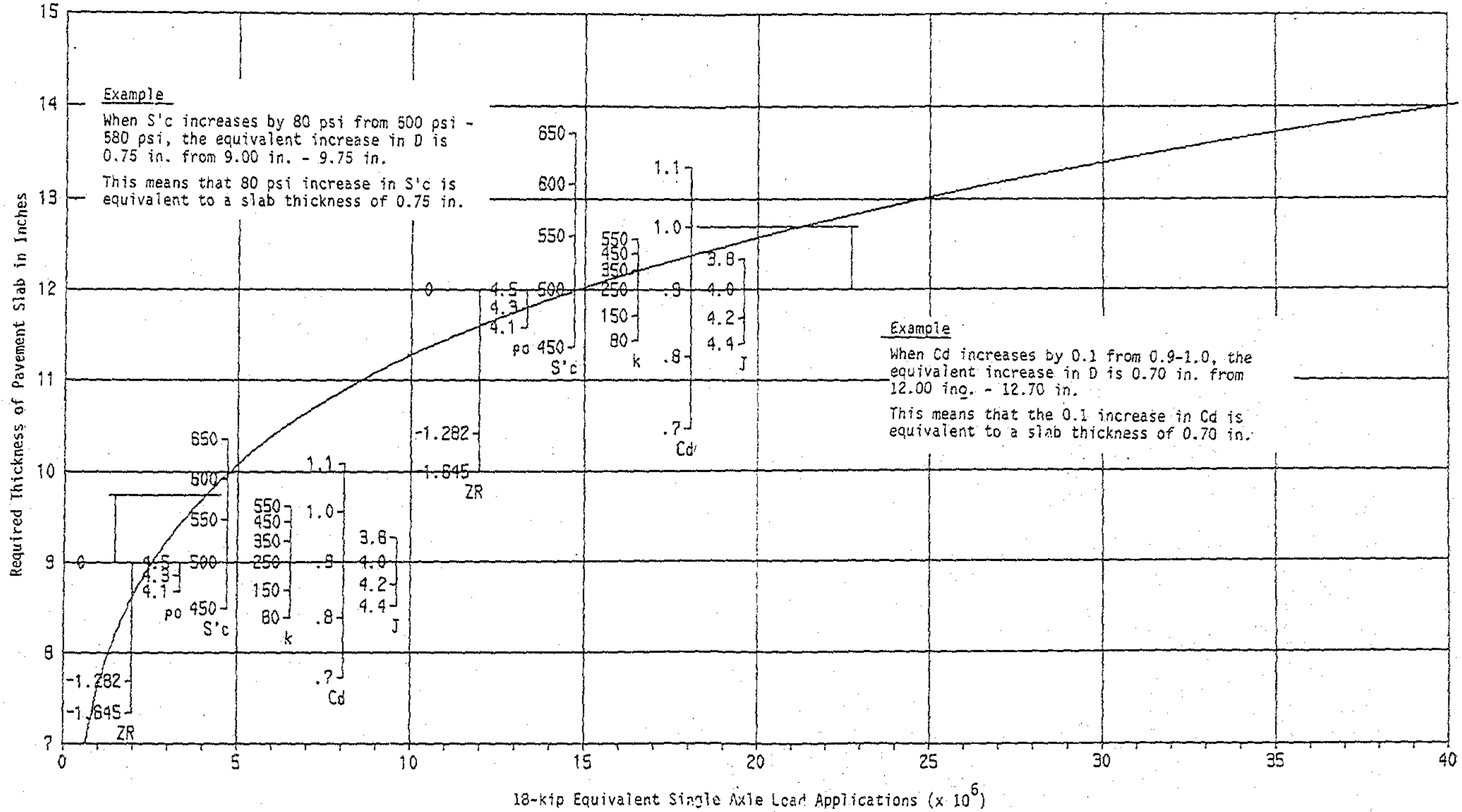


FIGURE 14.5-1 EQUIVALENT SLAB THICKNESS OF PAVEMENT VARIABLES

14.6 ANALYSIS ON SLAB THICKNESS BY DESIGN STANDARDS

The thickness of slab at the location of each slab surveyed was analyzed for two cases, 1) from the completion of section to date and 2) for 20 years. The 20 year period was selected as the average of performance period of pavement.

14.6.1 Design Methods

The following design methods were used for the analysis. The outline of design methods are described in Volume VI of the Study.

- MPWH Method: AASHTO Interim Guide for Design of Pavement Structures, 1972: American Association of State Highway and Transportation Officials.
- AASHTO Method: AASHTO Guide for Design of Pavement Structure, 1986: American Association of State Highway and Transportation Official.
- PCA Method: Thickness Design for Concrete Highway and Street Pavement, 1984: Portland Cement Association.
- TRRL Method: Road Note 29. A Guide to the Structural Design Pavement for New Roads, Third Edition, 1970: Department of the Environment, Road Research Laboratory.
- JRA Method (Method 1) : Manual for Cement Concrete Pavement, 1984; Japan Road Association.
- JRA Method (Method 2) : Modified Westergaard Method.

Table 14.6-1 summarizes design requirements for rigid pavement of these methods.

14.6.2 Analysis on Slab Thickness Required

Table 14.6-2 summarizes the values used for required thickness, e.g. the estimated ESAL, K value of subbase, and strength of concrete used for computation and the required thickness of concrete slabs analysis are also shown.

The outputs of analysis are presented in Appendix 14-6.

According to the analysis, the thickness of existing concrete slabs are thinner, about 1 to 2 inches (2.5 to 5.0 cm) for the cummulative traffic load to date and about 3 to 4 inches (7.5 to 10 cm) thinner for 20 years traffic loads, comparing with the thickness required by the design standards.

Figure 14.6-1 shows the ratio between the lack of concrete slab thickness and occurence of cracking.

TABLE 14.6-1 SUMMARY OF DESIGN REQUIREMENT FOR RIGID PAVEMENT

	General			Materials							Factors in determining slab thickness
	Performance Period (1)	Reliability (2)	Traffic (3)	Subgrade (4)	Subbase (5)	Concrete (6)	Provision of Reinforcing Bar (7)	Environment Impact (8)	Drainage (9)	Load Transfer (10)	
MPWH Method (AASHTO 1972)	. Recommend 20 years performance period . P_o = 4.5 . P_t = 2.5 or 2.0	-	. 18 kip ESAL application for single and tandem	. $D_{15}/D_{85} < 5$. Effective Modulus Subgrade (K value)	. 6 type of subbase with no thickness requirement	. $E_c = 4.2 \times 10^6$ (psi) . Working Stress (f_t)	. Plain concrete slab . Reinforced concrete slab	-	-	3.2	(1), (3), (4), (6)
AASHTO Method (1986)	. Suggest 20 years of performance period 30-50 years for analysis period . P_o = 4.5 . P_t = 2.5 or 2.0	. Error of traffic and performance prediction	. 18 kip ESAL application for single, tandem and triple axles	. No specific quality of requirement . Effective modulus of subgrade reaction (K value) combined with subbase modulus (ESB) . Adjusted by seasonal factor, rigid foundation, relative damage of slab and loss of support of subbase.		. No specific quality requirement . PCC Elastic Modulus (EC) . PCC Modulus of Rupture (S'c)	. Plain concrete slab . Reinforced concrete slab	. Loss of PSI due to Roadbed Swelling and frost heave	. Drainage coefficient 0.7 for very poor 1.25 Excellent	. Load Transfer coefficient by joint types and shoulder 2.3 ~ 4.4	(1), (2), (3), (4), (6), (9), (10),
PCA Method (1984)	. Recommend 20 years performance period	-	. Truck Traffic only . Load Safety Factor (LSF) Equivalent Stress	. K value 50 ~300 pci K value of subgrade and subbase combination	. Thickness 4~6 in. . Stress Ratio Factor . Erosion Factor	. No Specific quality requirement . Modulus of Rupture for concrete (MR)	. Plain concrete slab . Rainforced concrete slab	-	-	. Type of shoulder affect stress ratio factor . Types of shoulder and joint affect erosion factor	. Fatigue Analysis (3), stress ratio factor . Erosion analysis (3) Erosion factor
TRRL Method (1970)	. Recommend 20 years performance period	-	. Commercial vehicle only . Simplified from 18 kip ESAL	. Classification weak normal stable	. Mini. thickness 0 ~ 150 mm	. 28 MN/m^2	. Unreinforced concrete slab . Reinforced concrete slab	-	-	-	(3) (4)
JRA Method	. 20 years specified	-	. Truck traffic . Traffic class L, A, B, C and D	. CBR	. Mini. thickness 150 ~ 600 mm	. 45 kg/m^2	. Required D6 mm - 3 kg/m^2	. Thermal stress	-	. Load transfer coefficient	(3)

TABLE 14.6-2 REQUIRED THICKNESS OF CONCRETE SLAB

SECTION CONST. YEAR	NO. OF SURVEY SLAB	VEH. TYPE	TRAFFIC TO DATE ADT (1)	TRAFFIC TO DATE ESAL (10^6)	TRAFFIC 20 YEARS ADT (2)	TRAFFIC 20 YEARS ESAL (10^6)	EXISTING PAVEMENT CONDITION SUBGRADE AND BASE CBR	SUBGRADE AND BASE K (pci)	CONCRETE SLAB D (inches)	CONCRETE SLAB S'c	CONCRETE SLAB Ec (10^6)	REQUIRED THICKNESS OF CONCRETE SLAB (inches) FROM CONSTRUCTION TO DATE MPWH	AASHTO	PCA	TRRL	JRA	FOR 20 YEARS MPWH	AASHTO	PCA	TRRL	JRA
STA. RITA	N_1 – G	T	643	21.34	761	45.91	12	210	9.98	525	3.12	11.98	11.98	12.00	9.45	11.22	13.48	13.15	13.00	10.31	11.59
– GAPAN 50+0–50+500	N_1 – F	B	363	1.91	416	3.98	16	230	9.40	605	3.35	11.09	11.73	11.00	8.46	10.15	12.49	12.88	12.00	9.33	10.46
1975	N_1 – B	TL	1006	23.25	1177	49.89	5	140	8.81	630	3.42	11.11	12.49	12.00	9.45	10.31	12.49	13.71	12.50	10.31	10.61
STA. RITA	N_2 – G	T	654	19.59	788	47.21	38	400	10.02	449	2.89	12.22	12.95	12.50	8.23	11.70	14.06	14.59	13.00	9.29	12.08
– GAPAN 75+6–76+8	N_2 – F	B	368	0.16	429	0.37	27	310	9.96	455	2.91	12.32	13.89	12.50	8.23	11.83	14.14	15.65	13.00	9.29	12.36
1976	N_2 – B	TL	1002	19.75	1217	47.58	46	470	9.44	443	2.87	12.19	13.78	12.00	8.23	11.63	14.04	15.57	12.50	9.29	12.09
SAN JOSE	N_3 – G	T	341	9.96	512	37.38	12	210	9.38	510	3.08	10.70	10.16	11.00	8.42	10.98	13.14	12.17	12.00	9.88	11.63
– ARITAO 168+3–168+4	N_3 – F	B	99	0.38	120	1.15	43	440	9.23	548	3.19	9.80	11.15	10.00	7.44	9.80	12.20	13.48	10.50	8.90	10.34
1978	N_3 – B	TL	440	10.34	632	38.53	47	480	8.60	429	2.82	11.12	12.61	11.50	7.44	11.43	13.79	15.31	12.50	8.90	12.11
STO. TOMAS	S_1 – G	T	350	7.16	407	11.90	2	80	11.25	478	2.98	11.20	12.56	12.00	9.25	11.93	12.07	13.33	13.50	9.45	12.12
– TIAONG 79+2–79+3	S_1 – F	B	204	1.37	222	2.13	6	160	8.63	451	2.89	11.21	12.61	12.00	8.27	11.80	12.11	13.41	13.50	8.74	11.95
1972	S_1 – B	TL	554	8.53	629	14.03	7	170	8.71	447	2.88	11.23	12.64	12.00	8.27	11.73	12.13	13.44	13.00	8.74	12.05
LUCENA	S_2 – G	T	281	4.05	401	14.44	23	280	9.77	554	3.21	8.83	10.04	10.00	6.66	9.64	10.90	12.04	11.00	7.99	10.12
– GUMACA 160+2–160+3	S_2 – F	B	172	0.66	240	2.30	3	100	9.60	521	3.11	9.69	10.91	11.00	7.64	10.89	11.79	13.00	12.00	8.98	11.41
1978	S_2 – B	TL	453	4.71	641	16.74	2	80	9.27	576	3.27	9.30	10.46	10.50	8.62	10.39	11.30	12.43	12.00	9.96	10.91

N: North Study Section
S: South Study Section
G: Good Slab Condition
F: Fair Slab Condition
B: Bad Slab Condition
T: Trucks (Single and Tandem)
B: Bus
TL: Total

ADT (1): Average Daily Traffic (Trucks and Buses) After Completion To Date
ADT (2): Average Daily Traffic (Trucks and Buses) For 20 Years
ESAL: Equivalent Single Axle Load Applications

CBR: California Bearing Ratio (%) (Natural Moisture Content)
K: Effective Modulus Of Subgrade (pci) (Subgrade and Subbase Combination)
D: Thickness Of Concrete Slab (inches)
Sc: Modulus Of Rupture Of PCC (psi)
Ec: Elastic Modulus Of PCC (psi)

AASHTO: AASHTO 1986
MPWH: AASHTO 1972
PCA: Portland Cement Association
TRRL: Road Note 29
JRA: Japan Road Association

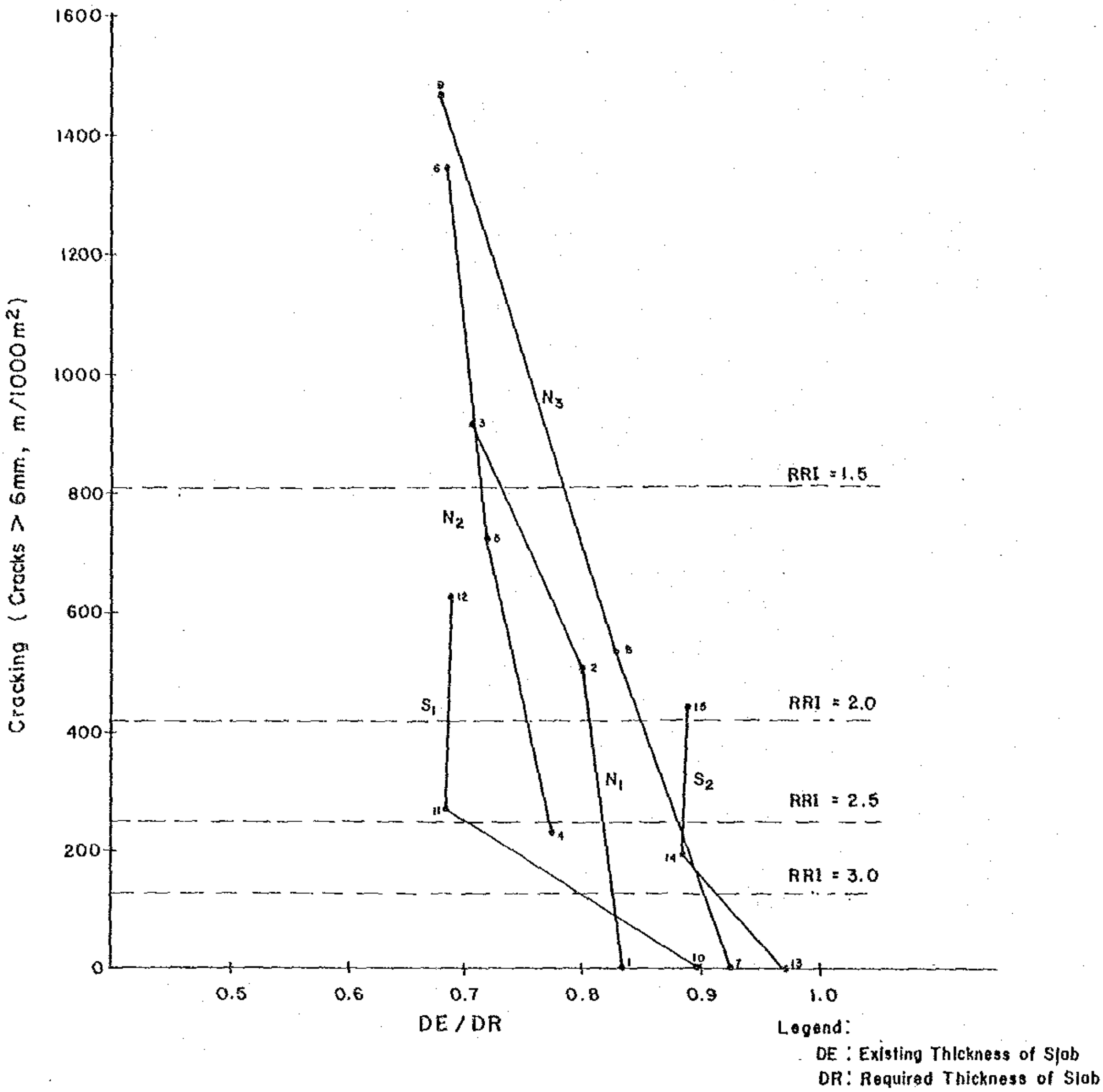


FIGURE 14.6-1 CRACKING THICKNESS OF SLAB

14.7 ANALYSIS ON STRUCTURAL STRENGTH BY EMPIRICAL/THEORETICAL METHODS

14.7.1 Analysis by AASHTO Method

The serviceability-performance curves for the existing conditions of surveyed slabs were drawn by a computer using the AASHTO Equation Program, as shown in Appendix 14-7. Figure 14.7-1 shows an example of the performance for Sta. Rita-Gapan, N_1, slab.

(1) Evaluation by Serviceability-Performance Curves

With reference to these curves, the following observations may be made.

<u>For slabs with RRI less than 2.0 (8 slabs)</u>

N_1-B, N_1-F, N_2-B, N_2-F, N_3-B, N_3-F, S_1-B, S_2-B

The serviceability-performance curves justify the existing condition of slabs which were either destroyed or severely deteriorated.

<u>For slabs with RRI of 2.0 to 3.0 (3 slabs)</u>

N_2-G,S_1-F, S_2-F

The existing conditions of slabs may or may not be explained by the curves. Because the curves of these slabs indicates the deterioration, but RRI's are near 2.5. This value means that 55 percent of people states unacceptable.

<u>For slabs with RRI higher than 4.0 (4 slabs)</u>

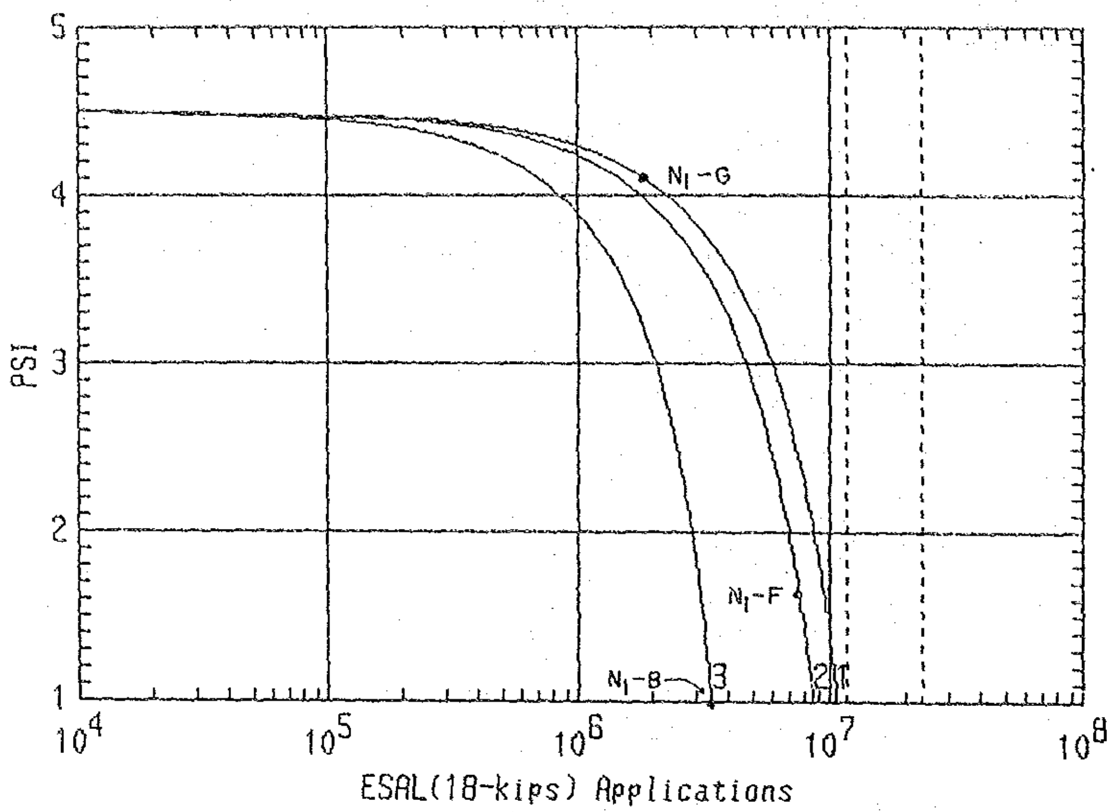
N_1-G, N_3-G, S_1-G, S_2-G

The curves cannot justify the existing condition which are assessed having the high RRI, but the curves forecasted the deterioration.

The evaluation by this method are summarized in Table 14.8-2 with the indications of the probable causes assessed.

(2) *Correlation between Cracking and ESAL*

Figure 14.7-2 show the correlation between cracking and the ratio between actual number of ESAL applications to date against the expected number of ESAL to P_t = 2.5. The relatively proportional relation is observed.



	Curve 1	Curve 2	Curve 3
ZR	0	0	0
So	0	0	0
D	9.98in	9.4in	8.81in
S'c	525psi	605psi	630psi
J	4	4	4
Cd	1	.9	.8
Ec	3120ksi	3350ksi	3420ksi
k	210pci	230pci	140pci
Po	4.5	4.5	4.5

FIGURE 14.7-1 STA. RITA – GAPAN; N_1

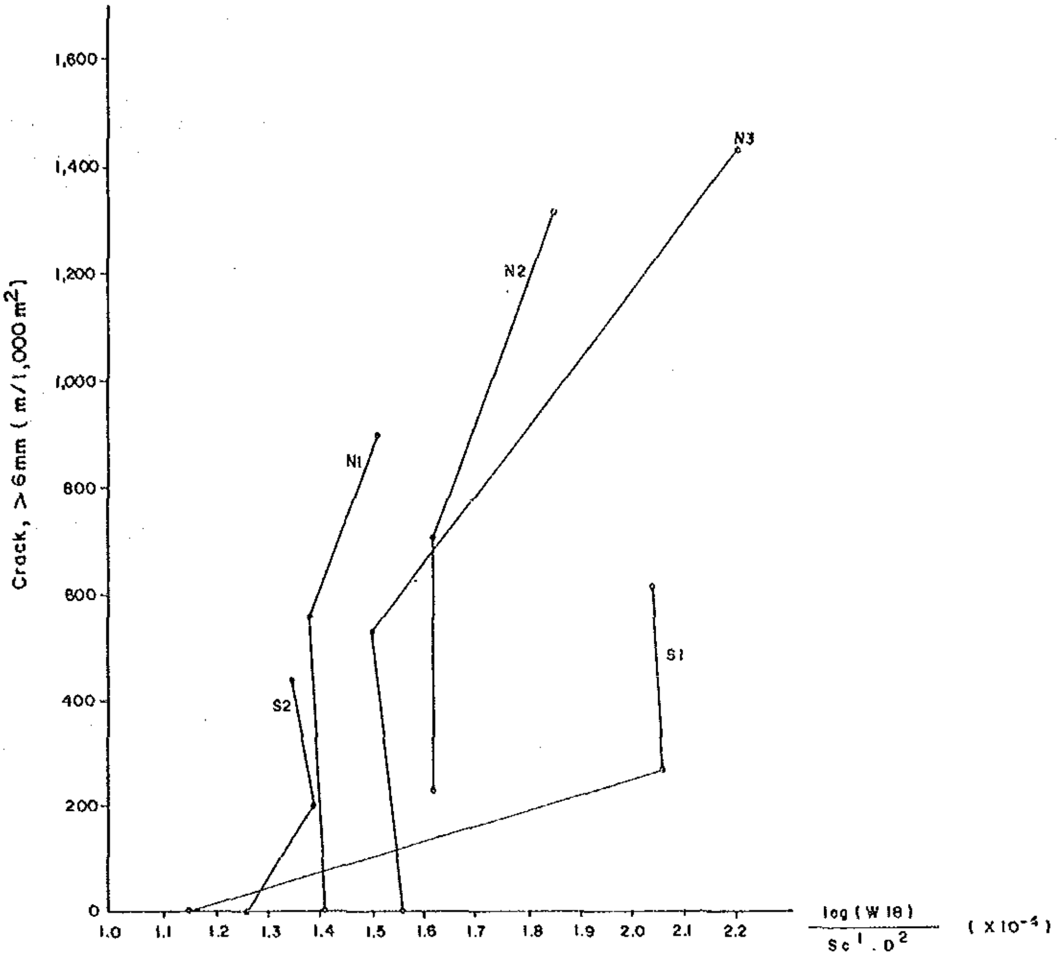


FIGURE 14.7-2 CRACKING VS. STRESS

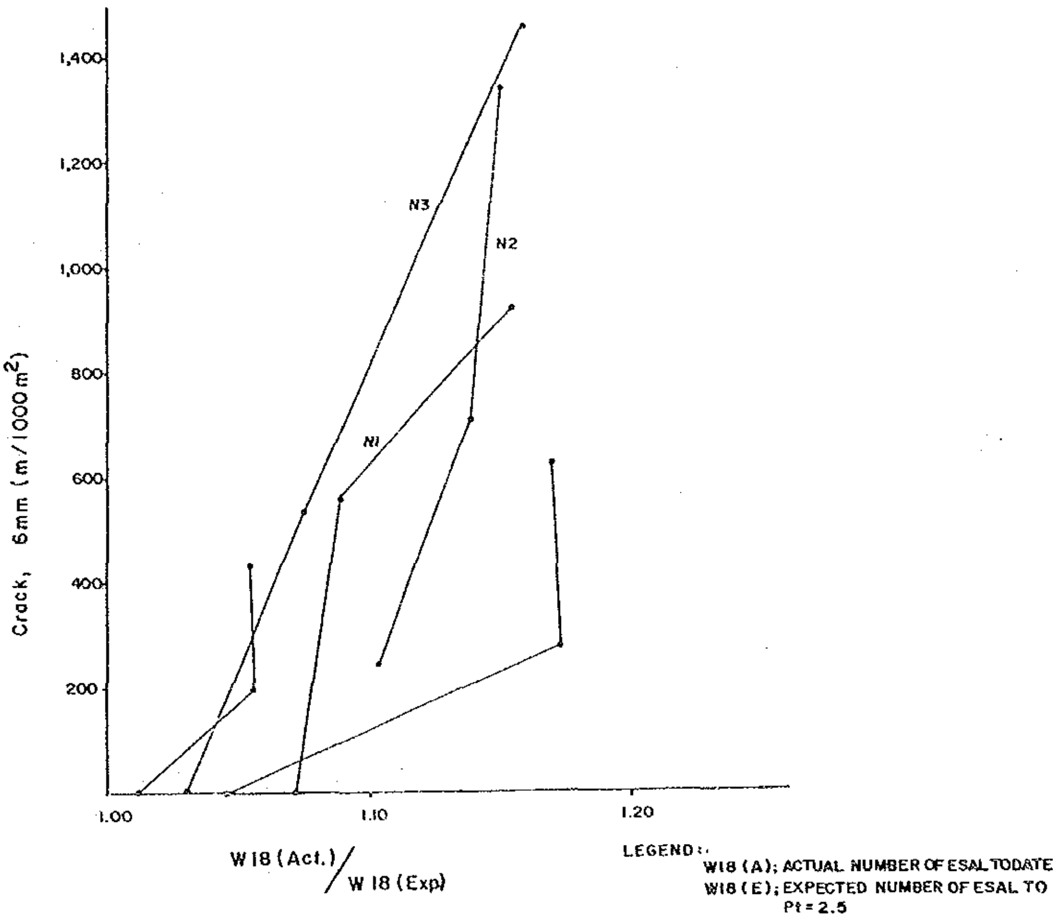


FIGURE 14.7-3 CRACKING VS. ESAL APPLICATIONS

Based on Spangler Equation described below, correlation between cracking and stress was analyzed as shown in Figure 14.7-3, from which only loose correlation is observed.

Spangler Equation

$$\sigma = \frac{J.P.}{D^2}\left(1 - \frac{al}{l}\right)$$

Where:

σ	=	maximum tensile stress in concrete, psi (Multiply σ psi, by 47.88 to obtain pascals Pa).
p	=	wheel load, lb
D	=	slab thickness, inches
al	=	distance from corner of slab to center of load, inches, (equals $a\sqrt{2}$, where a is the radius of a circle equivalent to the tire contact area)
J	=	load transfer coefficient (equals 3.2 for protected corner)
l	=	$\left[\frac{ZD^3}{22\,(1-\mu^2)}\right]^{0.25}$
Z	=	E/k.
E	=	Young's modulus of elasticity of concrete, psi.
k	=	Modulus of subgrade reaction, psi/in.
μ	=	Poisson's ratio for concrete, 0.2.

14.7.2 Analysis by PCA Method

In accordance with PCA design method, the analysis on both fatigue and erosion were made. The results are shown in Appendix 14-7 and plotted in Figure 14.7-4.

It is observed that the fatigue ratio are remarkably higher than the erosion ratio. The ratio are expressed in terms of percent which should be less than 100 percent if the pavement is not deteriorated.

14.7.3 Analysis by Modified Westergaard Method

Following modified Westergaard Method, stresses due to wheel and temperature and fatigue ratio were analyzed by an electric computer. The result are shown in Appendix 14-7 and plotted in Figure 14.7-4.

By both methods, PCA and Westergaard, the relatively proportional correlation between cracking and fatigue ratio is observed.

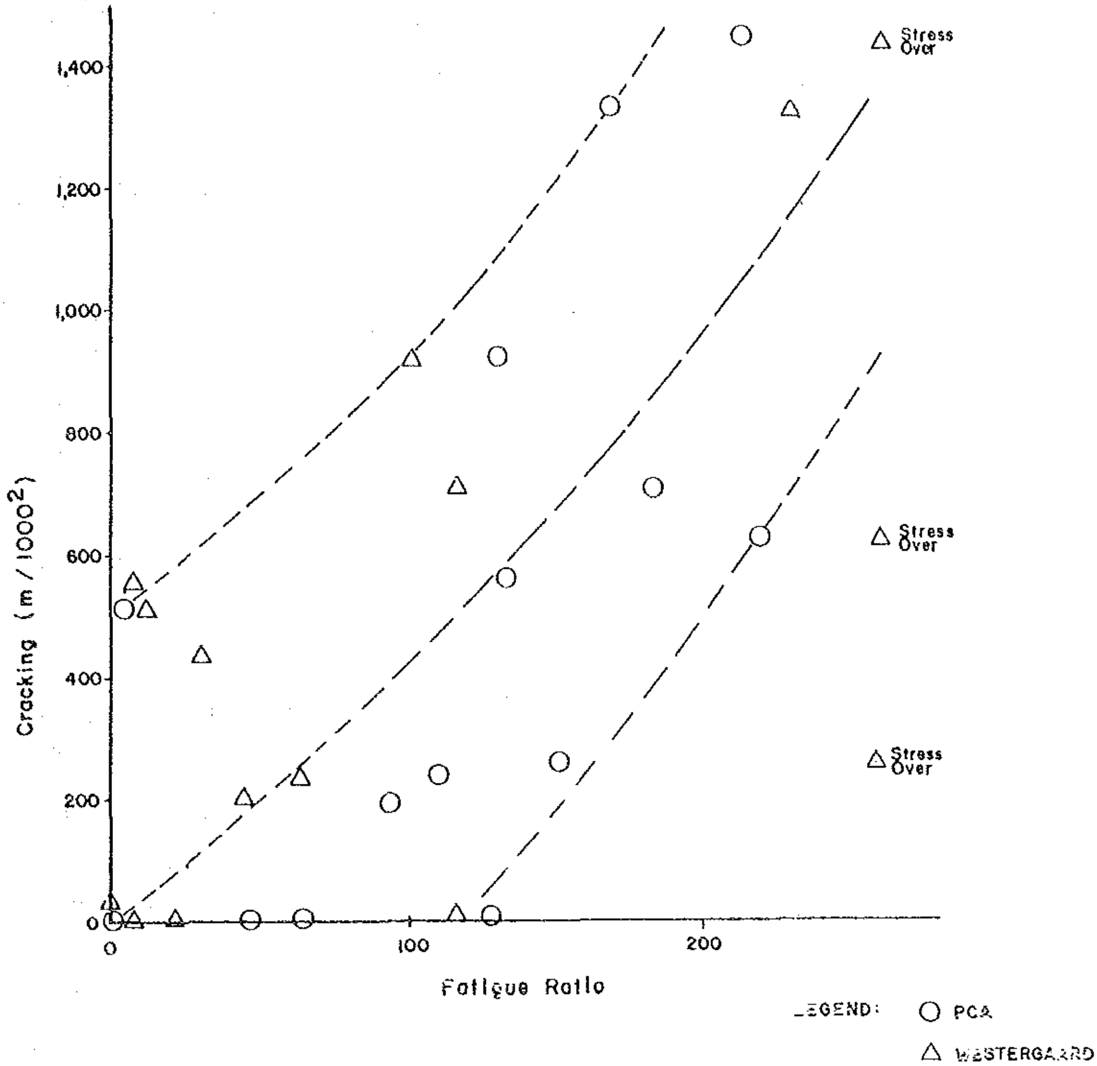


FIGURE 14.7-4 CRACKING VS. FATIGUE RATIO

14.8 EVALUATION ON CAUSES OF PAVEMENT DETERIORATION

The pavement technology is still in state of the art, though gradually but steadily transitioning from an art to a science. The assessment of the probable causes of pavement deterioration is, therefore, the inference rather than the analysis. Realizing this fact, the probable causes of pavement deterioration were evaluated based on field observations and analysis discussed in the previous sections.

The existing conditions of slabs surveyed for deterioration analysis are summarized in Table 14.8-1 including the results of the engineering surveys.

The outputs of deterioration analysis are presented in Table 14.8-2. In the same table, the assessment and observation on the probable causes are summarized. The results of analysis based on AASHTO Guide 1986 was given the preference in the evaluation on the causes of pavement deterioration because of many concepts involved such as reliability, drainage condition, loss of support (void beneath concrete slab) as well as load transfer factor.

The assessment of the probable causes of pavement deterioration are summarized as follows:

1) Traffic

The primary causes might be over-weighed traffic. The basis of this inference are:

- Fatigue ratio is considerably high comparing with erosion ratio (PCA and Westergaard)
- Transverse cracks are predominant.
- Relative damaging effect of traffic are high.
(8.1 for Manila bound along the North Study Section)
(3.6 for Manila bound along the South Study Section)

2) Concrete Slab

The qualities of concrete vary in wide ranges. The modulus of rupture vary from 430 psi to 630 psi. The required value is 525 psi (at 14 days BPH Memo Circular No. 48, 580 psi assumed at 28 days).

The thicknesses of concrete slab are thin (8.60 – 11.25 inches) in general and vary in wide ranges. It did not tally with the number of ESAL estimated.

3) Subbase and Subgrade

In some sections, the CBR values of subbase are lower (the lowest is 3) than those of subgrades. This might be due to saturation/erosion of subbases. Although the resilient modulus of subbase and subgrade has relatively small effects to the structural strength of pavement in case of concrete pavement, the localized loss of support and void beneath concrete slab may have significantly involved the pavement performance.

4) Drainage System

The drainage system involves the most remarkable effects to the pavement performance among those pavement variables. Moreover, the drainage factor is changeable according to the condition of sections/spots.

In the Study sections, the highest drainage factor is estimated as 1.1 (good, San Jose-Aritao) and the lowest 0.8 (poor, many sections).

TABLE 14.8-1 SUMMARY OF EXISTING CONDITION OF SURVEYED SLABS

Section (Station) Const. Yrs./Service Yrs.	Geological Condition, Road Bed Type	Daily Traffic Total ESAL (ESAL/Yrs)	Survey Slab	Cracks: Type Condition	Cracks: Wide / Fine / Total	Cracks: (Cracking) Cracking Index	Concrete Slab: $D_{(in)}$	Concrete Slab: $S'c_{(psi)}$	Concrete Slab: $Ec_{(10^6)}$	Subbase and Subgrade: Classification of Subbase / Subgrade	Subbase and Subgrade: CBR	Subbase and Subgrade: $k_{(pci)}$	Drainage Factor: Cd	Load Transfer: J
Sta. Rita – Gapon	Soft Ground	T: 643 B: 363	N_1 – G	Transverse Fine	0 / 2.0 / 2.0 m	(0) (132.4)	9.98	525	3120	Stone fragments Sand and gravel / Silty soils	34.31 / 2.62 / 12.00	210	1.0	4.0
(50+0 — 50+500)		1006	N_1 – F	Trans./Longl. Severely Deteriorated	8.6 / 0.5 / 9.1 m	(569.3) (602.4)	9.40	605	3350	Stone fragments Sand and gravel / Stone fragments Sand and gravel	53.06 / 9.61 / 15.00	230	0.9	4.0
1975 / 11 Yrs	Embankment	23.25 X 10^6 (2.11 X 10^6)	N_1 – B	Trans./Longl. Severely Deteriorated	13.8 / 1.9 / 15.7 m	(913.6) (1039.3)	8.81	630	3420	Silty or clayey Gravel and sand / Silty or clayey Gravel and sand	3.17 / 5.29 / 5.00	140	0.8	4.0
Sta. Rita – Gapon		T: 654 B: 368	N_2 – G	Transverse Deteriorated	3.6 / 1.0 / 4.6 m	(238.2) (304.5)	10.02	449	2890	Silty or clayey Gravel and sand / Stone fragments Sand and gravel	6.21 / 57.14 / 38.00	400	0.9	4.0
(75+6 — 76+8)		1122	N_2 – F	Trans./Longl. Considerably Deteriorated	10.9 / 1.0 / 11.9 m	(721.6) (787.8)	9.96	455	2910	Stone fragments Sand and gravel / Stone fragments Sand and gravel	3.45 / 42.99 / 27.00	310	0.8	4.0
1976 / 10 Yrs	Embankment	19.75X10^6 (1.98X10^6)	N_2 – B	Trans./Longl./ Diagonal Very Severely Deteriorated	20.3 / 7.3 / 27.6 m	(1343.9) (1807.3)	9.44	443	2870	Stone fragments Sand and gravel / Stone fragments Sand and gravel	53.79 / 43.48 / 46.00	470	0.8	4.0
San Jose – Aritao		T: 341 B: 99	N_3 – G	Corner Fine	0 / 1.0 / 1.0 m	(0) (66.2)	9.38	510	3080	Stone fragments Sand and gravel / Silty or clayey Sand and gravel	29.91 / 7.89 / 12.00	210	1.1	4.0
(168+3 — 168+47)		440	N_3 – F	Trans./Longl./ Diagonal Severely Deteriorated	8.1 / 3.4 / 11.5 m	(536.3) (761.3)	9.23	548	3190	Stone fragments Sand and gravel / Stone fragments Sand and gravel	39.33 / 40.06 / 40.00	440	0.8	4.0
1978 / 8 Yrs	Cut	10.34X10^6 (1.29X10^6)	N_3 – B	Trans./Longl./ Diagonal Very Severely Deteriorated	22.10 / 3.30 / 25.40	(1463.0) (1681.5)	8.60	429	2820	Stone fragments Sand and gravel / Stone fragments Sand and gravel	47.87 / 46.20 / 47.00	480	0.8	4.0
Sta. Tomas – Tiaong		T: 350 B: 204	S_1 – G	Transverse Fine	0 / 2.0 / 2.0 m	(0) (132.4)	11.25	478	2980	Stone fragments Sand and gravel / Clayey soils	5.38 / 1.52 / 2.00	80	0.8	4.0
(79+2 — 79+3)		554	S_1 – F	Trans./Corner Deteriorated	4.1 / 4.3 / 8.4 m	(271.4) (556.1)	8.63	451	2890	Silty or clayey Gravel and sand / Silty or clayey Gravel and sand	7.61 / 5.60 / 6.00	160	0.8	4.0
1972 / 14 Yrs	Embankment	8.53X10^6 (0.61X10^6)	S_1 – B	Trans./Corner	9.5 / 5.3 / 14.8 m	(628.9) (979.8)	8.71	447	2880	Silty or clayey Gravel and sand / Silty or clayey Gravel and sand	4.58 / 7.70 / 7.00	170	0.8	4.0
Lucena – Gumaca	Fault Zone	T: 281 B: 172	S_2 – G	Longitudinal Fine	0 / 4.5 / 4.5 m	(0) (297.9)	9.77	554	3210	Stone fragments Sand and gravel / Stone fragments Sand and gravel	22.51 / 22.68 / 23.00	280	0.8	4.0
(160+2 — 160+3)		453	S_2 – F	Trans./Longl./ Corner Deteriorated	3.0 / 13.0 / 26.0 m	(198.6) (1721.2)	9.60	521	3110	Stone fragments Sand and gravel / Silty soils	7.17 / 2.02 / 3.00	100	0.8	4.0
1978 / 8 Yrs	Cut	4.71X10^6 (0.59X10^6)	S_2 – B	Trans./Longl./ Diagonal Considerably Deteriorated	6.7 / 7.2 / 13.9 m	(443.5) (920.2)	9.27	576	3270	Stone fragments Sand and gravel / Silty or clayey Gravel and sand	4.73 / 1.79 / 2.00	80	0.8	4.0

TABLE 14.8-2 SUMMARY OF EVALUATION OF PAVEMENT DETERIORATION

	Survey Slab	Existing Condition			Results of Analysis						Probable Causes				Observation other than Traffic
		Cracking m/1000m²	Cracking Index m/1000m²	RRI	Design Standard DE/DR (Exit)(Req'd)	AASHTO W18/W18' (Act) (Exp)	AASHTO Performance	PCA Fatigue Ratio	PCA Erosion Ratio	Westergaard Fatigue Ratio	Primarily due to Traffic	Quality of Concrete	Subbase /Subgrade	Drainage Condition	
Sta. Rita ~ Gapan	N_1-G	0	132.4	4.1	0.833	1.072	Deteriorated	127	3.0	22		Fair	Weak Subgrade	Good	Relatively good condition because of fair concrete and good drainage.
(50+0 ~ 50+500)	N_1-F	569.3	602.4	1.7	0.801	1.089	Deteriorated	133	3.3	8	High fatigue ratio, low erosion ratio, W_{18} / W_{18}' higher than 1.0 and DE/DR lower than 1.0 show that the deterioration were primarily caused by traffic except S_2-G.	Good	Fair	Fair	Although good concrete and fair subbase are drainage, deteriorated due to thin slab.
1975 / 11 yrs.	N_1-B	913.6	1039.3	less than 1.0	0.705	1.153	Severely Deteriorated	139	6.3	101		Good	Both Poor	Poor	Although good concrete, deteriorated due to other poor conditions especially thin slab.
Sta. Rita ~ Gapan	N_2-G	238.2	304.5	2.6	0.773	1.102	Deteriorated	110	1.1	63		Poor	Weak Subbase	Fair	Condition is in near fatigue level. Probably deteriorated soon with evidence of cracking index. Fair drainage.
(75+6 ~ 76+8)	N_2-F	721.6	787.8	1.4	0.717	1.140	Deteriorated	184	1.4	117		Poor	Weak Subbase	Poor	Deteriorated due to all poor conditions, especially thin slab.
1976 / 10 yrs.	N_2-B	1343.9	1807.3	less than 1.0	0.695	1.147		189	3.4	230		Poor	Fair	Poor	Although fair base, severely deteriorated due to all poor conditions, especially thin slab.
San Jose ~ Arilao	N_3-G	0	66.2	4.1	0.923	1.031	Deteriorated	49	1.5	117		Fair	Fair	Poor	Relatively good condition because of other fair and good conditions. Slab is thick.
(168+3 ~ 168+4)	N_3-F	536.3	761.3	1.8	0.828	1.074	Deteriorated	6	1.1	5		Poor	Fair	Poor	Although fair concrete and subgrade, deteriorated due to poor drainage.
1978 / 8 yrs.	N_3-B	1463.0	1681.5	less than 1.0	0.682	1.157	Severely Deteriorated	213	1.5	Max. Stress over S'c		Poor	Fair	Poor	Severely deteriorated due to thin slab and other poor conditions except fair subgrade.
Sto. Tomas ~ Tiaong	S_1-G	0	132.4	4.1	0.896	1.047	Deteriorated	63	0.8	5		Poor	Both Weak	Poor	Relatively good conditions because of thick slab inspite of other poor conditions.
(79+2 ~ 79+3)	S_1-F	271.4	556.1	2.5	0.684	1.174	Deteriorated	150	1.4	Max. Stress over S'c		Poor	Weak Subbase	Poor	Condition is in near fatigue level. But, all indicator show deterioration. Probably deteriorated soon. High cracking index.
1972 / 14 yrs.	S_1-B	628.9	979.8	1.5	0.689	1.172	Deteriorated	220	2.4	Max. Stress over S'c		Poor	Weak Subbase	Poor	Severely deteriorated due to thin slab and other poor conditions.
Lucena ~ Gumaca	S_2-G	0	297.9	4.1	0.973	1.011	Fair	0.3	0.2	0.6		Fair	Fair	Poor	Fair conditions. Analysis also shown fair condition because of thick slab and other fair conditions except drainage.
(160+2 ~ 160+3)	S_2-F	198.6	1721.2	2.7	.880	1.056	Deteriorated	96	0.8	44		Fair	Both Weak	Poor	Conditions is in near fatigue level. Probably deteriorated soon with evidence of high cracking index.
1978 / 8 yrs.	S_2-B	443.5	920.2	1.9	0.886	1.055	Severely Deteriorated	37	1.1	32		Fair	Both Weak	Poor	Severely deteriorated due to weak base and poor drainage inspite of fair concrete.

CHAPTER 15
IMPROVEMENT FOR PAVEMENT REHABILITATION AND IDENTIFICATION OF REHABILITATION SECTIONS

15.1 SERVICEABILITY REQUIREMENT FOR PAVEMENT IMPROVEMENT

Since highways are defined as facilities for the comfort and convenience of the travelling public, the serviceability-riding quality-should be given by precedence over other consideration when the pavement rehabilitation is proposed. It should, however, be within the technical justification and the possible financial arrangements.

(1) AASHTO Guide 1986

AASHTO Guide 1986 suggest the following guidelines.

The primary measure of serviceability is the Present Serviceability Index (PSI) which ranges from 0 (impossible road) to 5 (perfect road). The terminal serviceability index (Pt) is the lowest acceptable level before resurfacing or reconstruction becomes necessary and suggested as follows:

Design of Major Highways: Pt = 2.5 or higher
Design of Highways with a low classification: Pt = 2.0
Design of Minor Highways: Pt = 1.5 (due to economic consideration)

One criterion for identifying a minimum level of serviceability may be established on the basis of public acceptance. Following are general guidelines for minimum levels of P_t obtained from studies in connection with the AASHO Road Test.

TABLE 15.1-1 PERCENTAGE OF PEOPLE STATING ACCEPTABLE/UNACCEPTABLE

Terminal Serviceability Level	Percent of People Stating Unacceptable
3.0	12
2.5	55
2.0	85

(2) Survey in the Study

During the surface condition survey, the assessors were asked to say whether the sections were acceptable or not. In this way, a level of acceptability on the rating scale was established.

Table 15.1-2 shows the survey results on acceptabilities for ranges of PSR (Present Serviceability Rating) and RRR (Rehabilitation Requirement Rating). The percents of people stating acceptable are graphically shown in Figure 15.1-1.

TABLE 15.1-2 PERCENT OF PEOPLE STATING ACCEPTABLE/UNACCEPTABLE

Present Serviceability Rating (PSR)

Range of Average PSR	Percent of People Stating as Follows: Unacceptable	Acceptable
.5 – 1.0	98.0	2.0
1.0 – 1.5	95.2	4.8
1.5 – 2.0	85.5	14.5
2.0 – 2.5	63.2	36.8
2.5 – 3.0	43.7	56.3
3.0 – 3.5	24.9	75.1
3.5 – 4.0	11.0	89.0
4.0 – 4.5	.7	99.3

Rehabilitation Requirement Rating (RRR)

Range of Average RRR	Percent of People Stating as Follows: Unacceptable	Acceptable
.5 – 1.0	99.7	.3
1.0 – 1.5	94.9	5.1
1.5 – 2.0	81.5	18.5
2.0 – 2.5	63.9	36.1
2.5 – 3.0	48.4	51.6
3.0 – 3.5	34.7	65.3
3.5 – 4.0	23.4	76.6
4.0 – 4.5	5.4	94.6

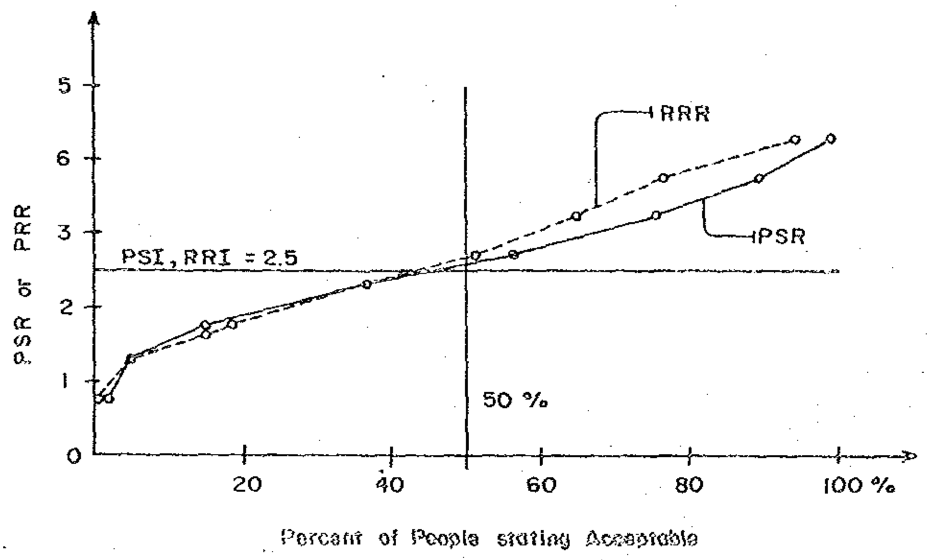


FIGURE 15.1-1 SERVICEABILITY AND ACCEPTABILITY

The survey results are summarized showing the percent of people stating acceptable/unacceptable at PSR and RRR of 3.0, 2.5 and 2.0 as shown in Table 15.1-3.

TABLE 15.1-3 PERCENT OF PEOPLE STATING ACCEPTABLE/UNACCEPTABLE

Terminal Serviceability Level	Percent of People Stating Acceptable		Percent of People Stating Unacceptable	
	PSR	RRR	PSR	RRR
3.0	66	58	34	42
2.5	47	44	53	56
2.0	26	27	74	73

At the level of 2.5 and 2.0, the percents are almost same as those as AASHO Road Test. But at 3.0, the percent of people stating unacceptable in this Study are 34 and 42, while AASHO Road Test shows only 12.

15.2 ENGINEERING REQUIREMENT FOR PAVEMENT IMPROVEMENT

(1) Failure Criteria of TRRL

Based on experimental pavements, Transport and Road Research Laboratory (TRRL) defines failure condition for reinforced concrete pavements is that the total length of cracking in each 100 m of the left-hand traffic exceeds 250 m. This is the time when wide cracks (width exceeding 1.2 mm. at the surface) are developing at a rate which makes reconstruction or overlaying a cheaper alternative to crack repair. See Figure 15.2-1.

For unreinforced concrete pavement, TRRL suggests that a substantial overlay would represent the economic solution to cracking when one in three of the slabs was cracked.

The 'failure' criterion used for experimental reinforced concrete roads in Britain is 250 m of total crack length per 100 cm of traffic lane. Experience shows that when such pavements are approaching the end of their lives about one-third of the total cracking falls into the Class 3 and Class 4 categories as defined in relation to the AASHO Road Test. It indicates that the British failure condition would correspond to a PSI level of about 2. See Figure 15.2-2.

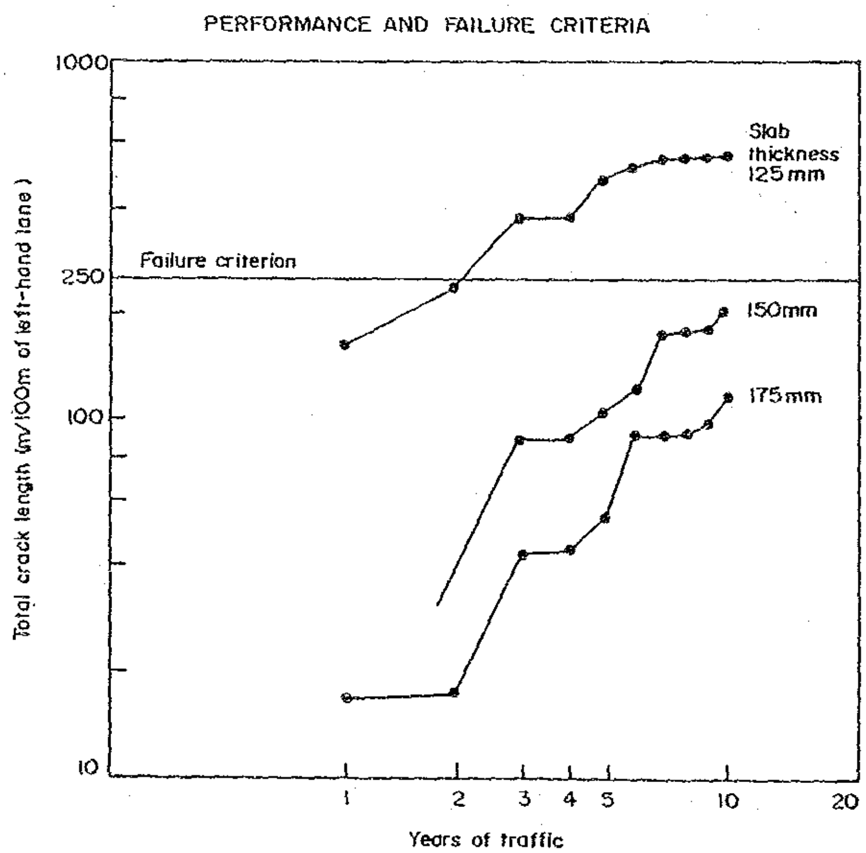


FIGURE 15.2-1 DEVELOPMENT OF CRACKS WITH AGE OF ROAD (REINFORCED CONCRETE)

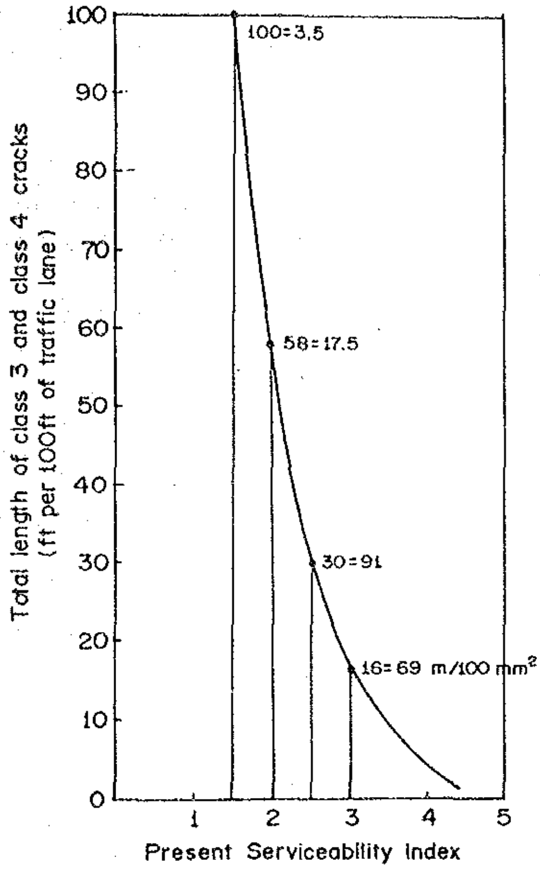


FIGURE 15.2-2 RELATIONSHIP BETWEEN DEGREE OF CRACKING AND PRESENT SERVICEABILITY INDEX

(2) Maintenance/Rehabilitation Criteria of JRA

The Road Maintenance/Rehabilitation Manual (Japan Road Association, JR 1978) proposes the criteria for maintenance/rehabilitation for concrete pavement as shown in Table 15.2-1.

TABLE 15.2-1 CRITERIA FOR MAINTENANCE/REHABILITATION (CONCRETE PAVEMENT)

	Rutting Depth (mm)	Faulting (mm)	Skid Resistance Coefficient	Longitudinal (mm)	Cracking[2/] Index Cm/m^2)	Joint Failure
Highway With Heavy Traffic	30–40	15	0.25	5.0[1/]	30	Whenever failures are observed
Highway With Light Traffic	40–50	–	–		50	

1/ Measured by 3 m Profilometer

2/ Cracks measured are those cracks reaching the bottom of the slab

15.3 RECOMMENDED IMPROVEMENT LEVEL FOR PAVEMENT REHABILITATION

(1) Comparison on Rehabilitation Criteria

Table 15.3-1 summarizes the comparison on the rehabilitation criteria required by AASHTO, TRRL, and JRA. The corresponding RRI's in the Study were calculated based on the same length of cracking of each guide line and roughness values associated with that length of cracking in the Study Section.

It is observed from Table 15.3-1 that

- Failure criteria is about RRI of 2.0.
- Rehabilitation criteria

  Major Highway: more than RRI of 2.5
  Highway and a low
   Classification: RRI of 2.0 or 2.2
  Minor Highway: RRI of 1.5

TABLE 15.3-1 COMPARISON ON REHABILITATION CRITERIA

A A S H T O	AASHTO Guide 1986			The Study		
	Suggested Terminal Serviceability	Typical Pavement Condition		Corresponding RRI	Typical Pavement Condition	
		Roughness	Cracking		Roughness	Cracking
Major Highway	3.0 or 2.5	220 260	50 110	3.2 2.7	280 310	50 110
Highway with a lower Classification	2.0	300	220	2.2	340	220
Minor Highway	1.5	360	390	1.5	370	400

T R R L	T R R L			The Study		
	Total Truck Length			Corresponging RRI	Typical Pavement Condition	
	(m/100 m)	($m/1000\ m^2$)	Cracking		Roughness	Cracking
Failure Criteria	250	810	270	1.9	350	270

Cracking; 1/3 of Total Crack Length

J R A	Japan Road Association			The Study		
	Mix Allowable Cracking Index			Corresponding RRI	Typical Pavement Condition	
	(cm/m^2)	($m/1000\ m^2$)	Cracking		Roughness	Cracking
Freeway	20	200	100	2.7	310	100
Highway with Heavy Traffic	30	300	150	2.5	320	150
Highway with Light Traffic	50	500	250	2.0	340	250

Cracking; 1/2 of Mix Allowable Cradking Index

(2) Recommended Improvement Level for Pavement Rehabilitation

From the comparison and observation, the improvement level for pavement rehabilitation as shown in Table 15.3-2 is recommended.

TABLE 15.3-2 RECOMMENDED IMPROVEMENT LEVEL FOR PAVEMENT REHABILITATION

Highway Class	RRI	Typical Pavement Condition Roughness	Cracking
Major Highway	2.5	320	140
Highway with a low Classification	2.0	340	250
Minor Highway	1.5	370	400

NOTE: Roughness: cm/km
Cracking: m/1000 m^2 cracks under Class 3 and 4

Pan-Philippine Highway, being the Study Road, is undoubtedly the most important Major Highway in the country, therefore, RRI of 2.5 is recommended as the improvement level.

It is, however, noted that pavements with 2.5 to 3.0 of RRI should be given attention when the pavement rehabilitation program is planned. Because, the cracks of these pavement might be progressed and RRI might be less than 2.5 when the actual rehabilitation work is started.

15.4 SECTIONS IDENTIFIED FOR REHABILITATION

Pavement surface condition for every section of 100 meters including measurement roughness, cracking and patching and RRR/PSR value are presented in Appendix 13-4. RRI (Rehabilitation Requirement Index) and PSI (Present Serviceability Index in the Study) and PSI (AASHO) are also indicated.

Based on RRI proposed in the Study, the pavement rehabilitation section were identified in accordance with the rehabilitation criteria of RRI of 2.5 established in the Study. The locations of rehabilitation sections are shown in Appendix 22-1, and summarized in Table 15.4-1.

TABLE 15.4-1 SECTION LENGTH IDENTIFIED FOR REHABILITATION

Unit: Km per lane

North Study Section	Length (Km)	Manila Bound			Cagayan Bound			T o t a l		
		RRI ≦ 2.5	2.5 < RRI ≦ 3.0	RRI < 3.0	RRI ≦ 2.5	2.5 < RRI ≦ 3.0	RRI < 3.0	RRI ≦ 2.5	2.5 ≦ RRI < 3.0	RRI < 3.0
Segment N-1 (Sta. Rita-Gapan)	46	37.50 (82)	1.75 (4)	6.75 (14)	10.75 (23)	19.70 (43)	15.55 (34)	48.25 (52)	21.45 (23)	22.30 (25)
Segment N-2 (Gapan-Cabanatuan)	35	18.90 (54)	4.85 (14)	11.25 (32)	11.45 (33)	11.75 (34)	11.80 (34)	30.35 (43)	16.60 (24)	23.05 (33)
Segment N-3 (Cabanatuan-San Jose)	42	8.93 (21)	8.64 (21)	24.43 (58)	1.05 (3)	3.45 (8)	37.50 (89)	9.98 (12)	12.09 (14)	61.53 (74)
Segment N-4 (San Jose-Dalton)	38	31.73 (84)	3.40 (9)	2.87 (7)	3.10 (8)	30.28 (80)	4.62 (12)	34.83 (46)	33.68 (44)	7.49 (10)
Segment N-5 (Dalton-Aritao)	39	28.94 (74)	7.06 (18)	3.00 (8)	8.45 (22)	20.31 (52)	10.24 (26)	37.39 (48)	27.37 (35)	13.24 (17)
Sub-Total	200	126.00 (63)	25.70 (13)	48.30 (24)	34.80 (17)	85.49 (43)	79.71 (40)	160.50 (40)	111.19 (19)	128.01 (32)
South Study Section										
Segment S-1 (Calamba-Tiaong)	42	2.98 (7)	9.15 (22)	29.87 (71)	1.53 (4)	5.23 (12)	35.24 (84)	4.51 (5)	14.38 (17)	65.11 (78)
Segment S-2 (Tiaong-Pagbilao)	54	5.50 (10)	3.75 (7)	44.75 (83)	2.63 (5)	5.95 (11)	45.42 (84)	8.13 (8)	9.70 (9)	90.17 (83)
Segment S-3 (Pagbilao-Plaridel)	46	17.20 (37)	8.25 (18)	20.55 (45)	17.70 (38)	5.45 (12)	22.85 (50)	34.90 (38)	13.70 (15)	43.40 (47)
Segment S-4 (Plaridel-Calauag)	39	12.13 (31)	13.15 (34)	13.72 (35)	9.65 (25)	15.63 (40)	13.72 (35)	21.78 (28)	28.78 (37)	27.44 (35)
Sub-Total	181	37.81 (21)	34.30 (19)	108.89 (60)	31.51 (17)	32.36 (18)	117.23 (65)	69.32 (18)	66.56 (18)	226.12 (63)
T o t a l	381									

NOTE: Figure in ( ) show pavement

15.5 PAVEMENT CONDITION IDENTIFIED FOR REHABILITATION

(1) North Study Section

Average RRI of the North Study Section is 2.9 and 3.5 for Manila bound and Cagayan bound, respectively.

N_1 (Km. 39 – Km. 85, Sta. Rita-Gapan, L = 46 Km.)

The sections with severely deteriorated pavement are Km. 39 to 41, Km. 50 to 77 and Km. 79 to 85 of Manila bound. The pavements of the remaining sections show cracks in places scattered.

As to Cagayan bound, the pavement conditions are quite acceptable except the section from Km. 69 to 77. However, cracks were observed in other places.

For both lanes, the pavement lengths per lane with RRI of less than 2.5, 2.5 to 3.0 and more than 3.0 are 48.3 kms. 21.5 kms and 22.3 kms. respectively.

The geology of the area consists of flat alluvial terrain, therefore, one of the causes of pavement deterioration was presumed to be the consolidation settlement of soft ground foundation. However, cracks observed are rather regular one as transverse and longitudinal, and no blocking cracks nor slab rocking have appeared. It may indicate that cracks were caused by fatigue. Based on this observation and the different pavement condition of both bounds, the main causes are conjected to be primarily traffic load related. However, the drainage system which were not provided in the section might be responsible for the fast progress of pavement deterioration since the section runs in the flat field area.

N_2 (Km. 85 – Km. 120, Gapan-Cabanatuan, L = 35 km.)

The severely cracked pavements were observed at the sections between Km. 85 to 95 and Km. 98 to 105 of Manila Bound. Cracks were also observed in scattered places in the remaining sections.

As for the opposite lanes, the continuously deteriorated sections were not seen.

For both lanes, the pavement lengths per lane with RRI of less than 2.5, 2.5 to 3.0 and more than 3.0 are 30.4 kms, 16.6 kms. and 23.1 km, respectively.

Since the topography and pavement deterioration conditions in this sections are similar to those in N_1, the same observations with N_1 section, may be commented.

N_3 (Km. 120 – Km. 162, Cabanatuan-San Jose, L = 42 km.)

The pavement condition of this section is quite reasonable, except, a few kilometers of sections although the Manila bound lane shows more cracks in terms of degree and extent comparing with the Cagayan bound, as observed in other sections.

For both lanes, the pavement length per lane with RRI of less than 2.5, 2.5 to 3.0 and more than 3.0 are 10.0 kms, 12.1 kms. and 61.9 kms, respectively.

As reported in N_1 section, regular shaped fatigue cracks is the representative. The main causes might be traffic loads however, no provision of drainage system might be obliged for pavement deterioration.

N_4 (Km. 162 – Km. 200, San Jose-Dalton Pass, L = 38 Km.)

The sections with severely deteriorated pavement are those from Km. 168 to Km. 179 and Km. 183 to 199 of Manila bound. For the remaining sections which are scattered in various places, the progress of cracks were observed.

As for Cagayan bound, the present pavement condition may be judged as acceptable in general.

For both lanes, the pavement lengths per lane with RRI of less than 2.5, 2.5 to 3.0 and more than 3.0 are 34.8 kms, 33.7 kms. and 7.5 kms, respectively.

The road types of this section is the alternative of embankment and cut. However, no different observations were made on the probable causes of pavement deterioration mentioned in N_1 Section, because of the fatigue cracks same as N_1 Section.

N_5 (Km. 200 – Km. 239, Dalton Pass-Aritao, L = 39 km.)

The sections from Km. 200 to Km. 218 of Manila bound show considerably deteriorated pavement. After Km. 218, up to the end of Study Section, Km. 239, the cracked pavements were scattered in various places

As to Cagayan bound, no continuous sections with cracked pavements were observed, but scattered through the entire sections.

For both lanes, the pavement lengths less than RRI of 2.5, 2.5 to 3.0 and more than 3.0 are 37.4 kms, 27.4 km and 13.2 kms, respectively.

The topography of this section is steep mountainous and the road types are the combination of embankment and cut. Cracks observed are mainly transverse and longitudinal fatigue cracks, the primary causes of which were presumed to be traffic load related. However, the localized depression of cracked pavement was observed at near the top of Dalton Pass. It may indicate that the subgrade may be weak due to bad drainage conditions. No drainage system to discharge water from slopes of mountains were observed except for a few sections.

(3) South Study Section

Average RRI of the South Study Section is 3.5 and 3.6 for Manila bound and Bcicol bound, respectively.

S_1 (Km. 52 – Km. 92, Calamba-Tiaong, L = 42 km.)

The pavement condition of this section is finest in the Study Section. The cracked pavement were, however, scattered between Km. 52 to 61 and Km. 76 to 78.

For both lanes, the pavement length per lane with RRI of less than 2.5, 2.5 to 3.0 and more than 3.0 are 4.5 kms, 14.4 kms. and 65.1 kms.

The topography of this area is flat and the road type is mainly embankment. Cracks observed are traffic load related fatigue cracks judging from their transverse shape. Relatively reasonable maintenance works such as sealing of joints, cracks and partial reconstruction of slabs, were observed. But, shouldering of the road was ignored. No drainage systems were observed.

S_2 (Km. 42 – 146, Tiaong-Pagbilao, L = 54 km.)

The pavement of this section also shows reasonably acceptable conditions. The cracked pavements were only seen in few places.

For both lanes, the pavement lengths per lane with RRI of less than 2.5, 2.5 to 3.0 and more than 3.0 are 8.1 kms, 7.7 kms. and 90.2 kms., respectively.

The topography of this area is flat and the road type is mainly embankment. The same observation as S_1 Section may be made.

S_3 (Km. 146 – Km. 188, Pagbilao-Gumaca, L = 46 km.)

The pavement of this section is severely deteriorated especially for the sections from Km. 158 to 167 and Km. 169 to 179 for both lanes, Manila andBicol bounds. The remaining sections show relatively reasonable conditions.

For both lanes, the pavement lengths per lane with RRI of less than 2.5, 2.5 to 3.0 and more than 3.0 are 34.9 kms, 13.7 kms and 43.4 kms, respectively.

The topography of this section is rolling/mountainous terrain and road types consist of alternative or combination of embankment and cut. The pavement deterioration types of this section are very particular, represented by two.

This first type is characterized by very progressed cracks (third stage cracks) or multiple irregular breaks or separation of slabs. Observed in these slabs were depression which may have been caused by settlement or consolidation of the foundation soil. As the evidence of weak subbase/subgrade, pumping up of fine materials beneath slabs and slab rocking are observed.

The second type is depression localized in very narrow area (about 2 square meters) of the slabs. In these localized settled areas, pumping and rocking are observed. The probable causes may be weak foundation soil aggravated by water action resulting in loss of support beneath slabs.

For both types, no drainage systems were provided, even where high underground water level are anticipated.

S_4 (Km. 188 – Km. 227, Gumaca-Calauag, L = 39 km.)

The pavement of this section shows localized depression which are scattered throughout this section, for both Manila and Bicol bounds.

For both lanes, the lengths of pavement per-lane with RRI of less than 2.5, 2.5 to 3.0 and more than 3.0 are 21.8 kms, 28.8 kms and 27.4 kms, respectively.

The same observations mentioned in S_3 section may be made for this section.

CHAPTER 16
PROPOSED PAVEMENT REHABILITATION METHODS AND EVALUATION

16.1 APPROACH

The main concepts on pavement rehabilitation adopted in this Study are serviceability-performance concepts and rehabilitation concepts under the pavement management system, as described hereafter.

(1) Serviceability-Performance Concepts

The overall philosophy of the rehabilitation approach adopted in this study is based upon the AASHTO design-serviceability-performance concepts.

The serviceability of a pavement is expressed in terms of the Present Serviceability Index (PSI). The PSI is obtained from measurements of roughness and distress, e.g., cracking and patching at a particular time during the service life of the pavement.

Current concepts of pavement performance include some consideration of functional performance, structural performance and safety.

The functional performance of a pavement concerns on how well the pavement serves the user. In this context, riding comfort or ride quality is the dominant characteristic.

The structural performance of a pavement relates to its physical condition; i.e., occurrence of cracking, faulting, raveling, or other conditions which would adversely affect the load-carrying capability of the pavement structure or would require maintenance.

As discussed in Chapter 13, the Study Team inclined to give a preference to the structural performance primarily examined by cracking-rather than the functional performance-primarily evaluated by roughness, taking into consideration the present pavement technology in the country, though subject to review in compliance with the technical and economical development.

In line with this purpose, the Rehabilitation Requirement Index was established and justified to be utilized as the surface evaluation indicator or line of PSI, as discussed in Chapter 13.

(2) Rehabilitation Concepts

The basic purpose of a pavement management system is to achieve the best value possible for the available public funds and to provide safe, comfortable and economic transportation facilities.

Project management is described in terms of two generalized levels: (1) the network management level, sometimes called the program level and (2) the project management level, where technical management decision are made for specific projects.

The overall structure of a pavement management system, in terms of the logical sequence of activities involved is shown in Table 16.1-1.

Rehabilitation works fall into the project level. The term "rehabilitation" encompasses the activities described in the 4R programs – resurfacing, restoration, rehabilitation, and reconstruction. In short, major rehabilitation activities will be viewed as any work that is undertaken to significantly extend the service life of an existing pavement through the principles of resurfacing, restoration, and/or reconstruction.

Major rehabilitation activities differ remarkably from periodic maintenance activities (sometimes called normal, routine and/or preventive maintenance) in that the primary function of the latter activity is to preserve the existing pavement so that it can support the applied loading, while rehabilitation is undertaken to significantly increase the functional life.

In accordance with this rehabilitation concept, the Study was carried out. Major rehabilitation works are listed in Table 16.1-1 together with other works under the different activities of the PMS, for comparison.

TABLE 16.1-1 MAJOR WORKS OF PAVEMENT MANAGEMENT SYSTEM

Network Management Level (Program Level)	Project Level: Design	Construction	Maintenance	Evaluation	Rehabilitation
. Assess Network Deficiencies . Establish Priorities . Budgeting	. Information on Materials Traffic, Environment Costs, etc. . Alternative Design Strategies . Analysis . Economic Evaluation and Optimization	. Specifications and Contracts . Construction Schedules and Opreation . Quality Control . Construction Records	. Standards and Schedules . Maintenance Operations . Budget Control . Maintenance Records	. Periodic Monitoring . Evaluation Records	. Problem Definition . Potential Problem Solutions . Selection of Preferred Solution . Implementation
			Major Maintenance Works . Shoulders . Related Drainage . Thin resurfacing of short length . Patching . Filling Potholes . Sealing cracks and joints . Repair of minor failures	Major Periodic Monitoring . Structural Evaluation (Existing Distress, Load-Carrying Capacity) . Functional Evaluation (PSR/PSI, Roughness)	Major Rehabilitation Works . Resurfacing to provide structural capacity or serviceability . Replacing or restoring malfunctioning joints . Substantial pavement undersealing when essential for stabilization . Removing and Replacing deteriorated materials . Reworking or strengthening of bases or subbases . Cracking and sealing of PCC pavement with AC . Adding underdrains

16.2 PROPOSED REHABILITATION METHODS

(1) Rehabilitation Methods

Many of the rehabilitation methods available are presently being tried on an experimental basis and lack verification. Among these, major rehabilitation works are summarized in Table 16.2-1, subdividing into two categories.

TABLE 16.2-1 MAJOR REHABILITATION WORKS

Rehabilitation Methods Other Than Overlay
1. Full Depth Pavement Repair
2. Partial Depth Pavement Repair
3. Joint and Crack Sealing
4. Subsealing of Concrete Pavements
5. Grinding/Milling of Pavements
6. Subdrainage Design
7. Pressure Relief
8. Restoration of Joint Load Transfer
9. Surface Treatments
Rehabilitation Methods With Overlay
1. Flexible Overlay/Flexible Existing
2. Flexible Overlay/Rigid Existing
3. Rigid Overlay/Rigid Existing

In this Study, rehabilitation methods other than overlay were not main subjects for discussion. The emphasis of the Study was, therefore, put on full depth pavement repair and overlay.

(2) Proposed Rehabilitation Methods

The pavements type along the entire Study section is only portland cement concrete, but flexible existing was discussed here for a reference. Finally, the following five (5) rehabilitation methods were discussed.

PCC Reconstruction (Rigid Pavement)
AC Reconstruction (Flexible Pavement)
Rigid Overlay – Rigid Existing
Flexible Overlay – Rigid Existing
Flexible Overlay – Flexible Existing

Figure 16.2-1 summarizes the proposed five (5) rehabilitation methods, each of which includes several alternatives. Taking into account for the analysis periods of 25 years, the planned stage constructions was proposed. Figure 16.2-1 also shows First (Initial) Rehabiliation Works, Second and successive Third Works for each proposed rehabilitation methods.

(3) Typical Cross Section of Proposed Rehabilitation Methods

Typical cross sections of PCC Reconstruction and AC Reconstruction are presented in Figure 16.2-2 and 16.2-3, respectively as examples and sections of other overlay methods are shown in Appendix 16-1. The structural component of AC Reconstructions are also shown in Figure 16.2-4.

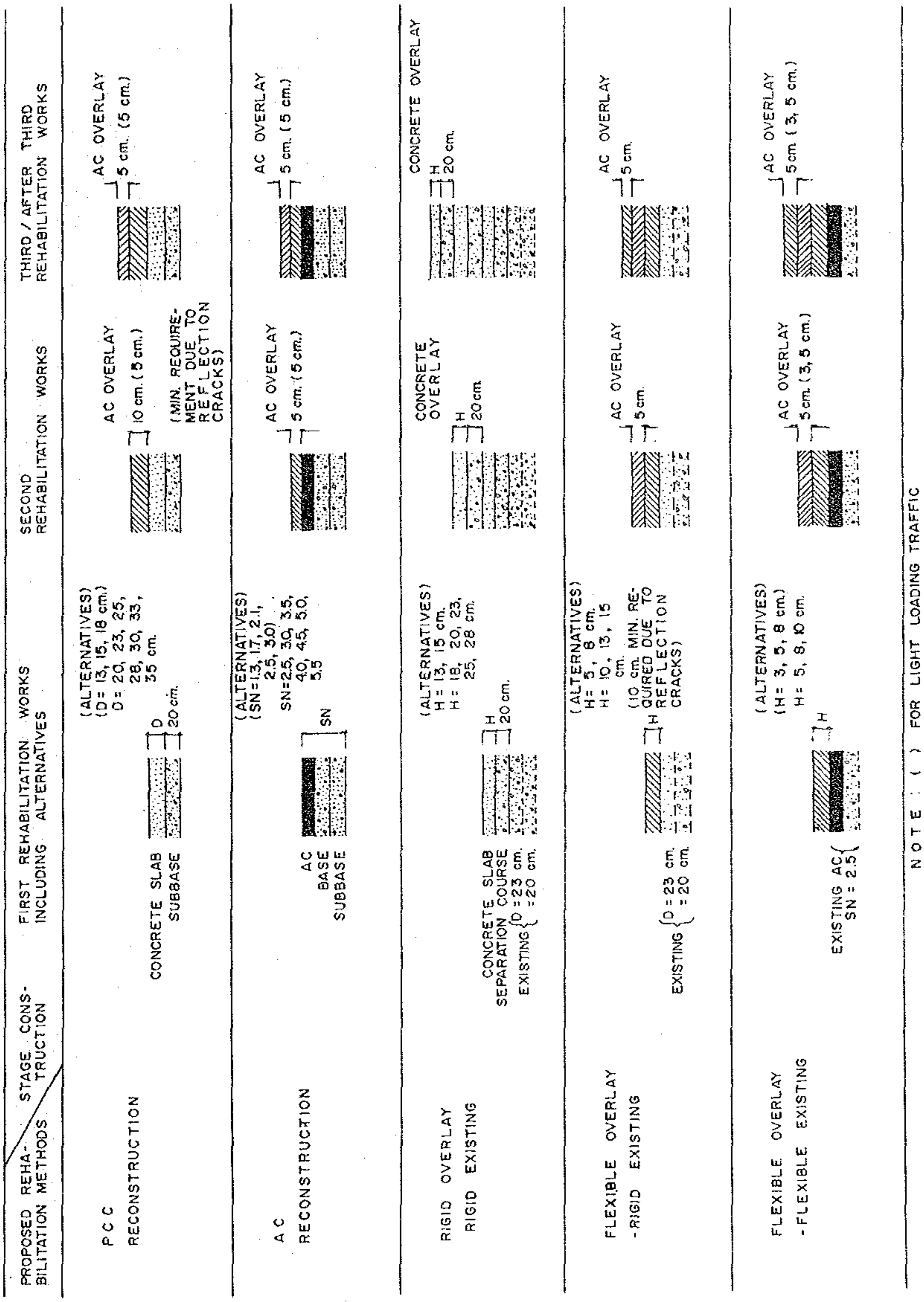
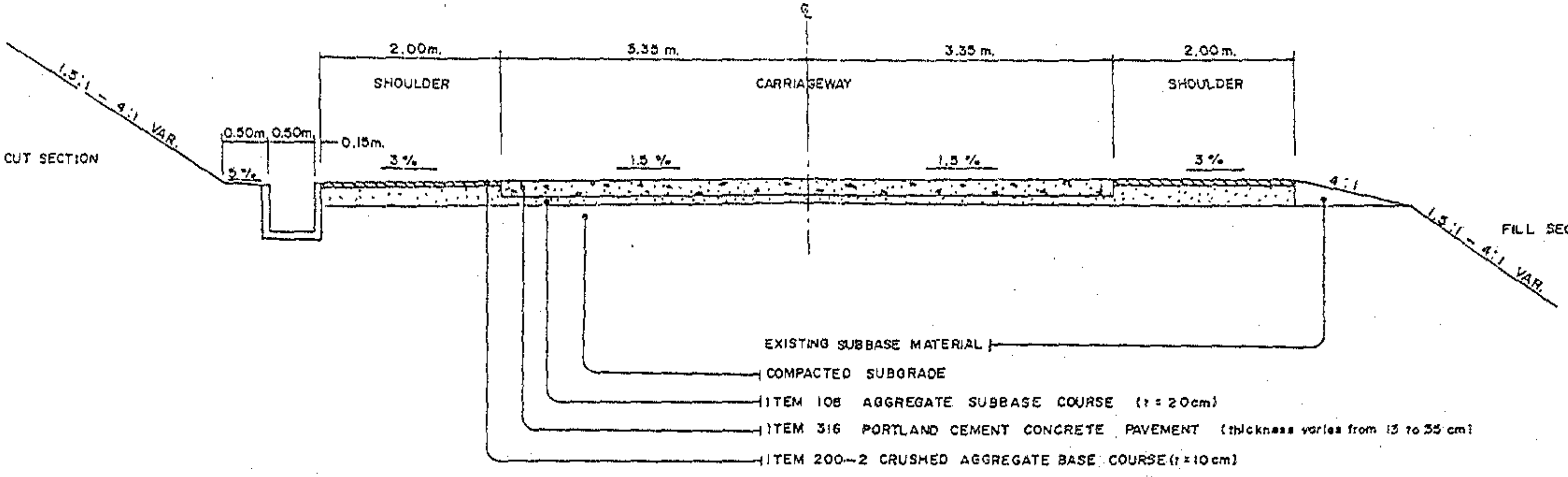


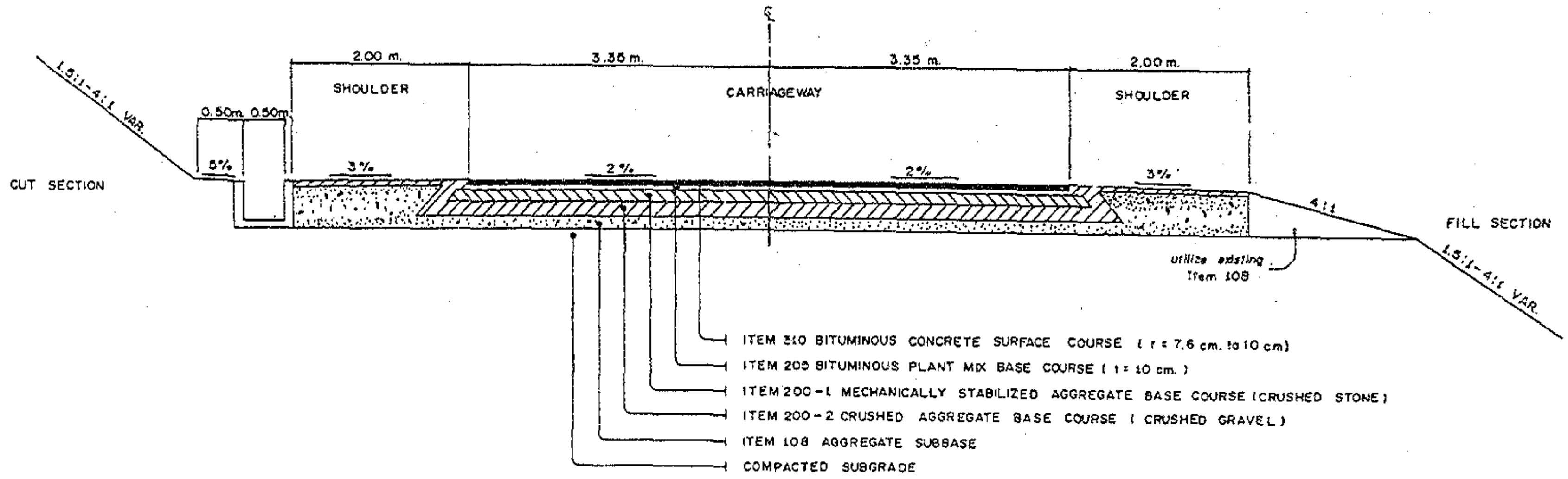
FIGURE 16.2-1 PROPOSED REHABILITATION METHODS



Unit : Peso per meter

Slab Thickness (cm.)	ITEM 104: Removal of Miscellaneous Structures	ITEM 105 Roadway Excavation	ITEM 108 Aggregate Subbase	ITEM 316: Portland Cement Concrete Pavement	ITEM 200-2: Crushed Aggregate Base	Total Cost (₱ / m)
20	176.01	132.29	529.72	1603.58	110.66	2552.26
23	176.01	147.80	554.74	1844.11	110.66	2833.32
25	176.01	158.13	571.43	2004.47	110.66	3020.70
28	176.01	173.64	596.45	2245.01	110.66	3301.77
30	176.01	183.97	613.14	2405.37	110.66	3489.15
33	176.01	199.48	638.16	2645.90	110.66	3770.21
35	176.01	209.82	654.85	2806.26	110.66	3957.60

FIGURE 16.2-2 TYPICAL CROSS – SECTION
PCC RECONSTRUCTION (2 – LANES)



SN	Item 104 Removal of Miscelleneous Structure	Item 105 Roadway and Drainage Excavation	Item 108 Aggregate Subbase	Item 200-1 Mechanically Stabilized Aggregate Base Course (Crushed Stone)	Item 202-2 Aggregate Base Course Crushed Gravel	Item 205 Bituminous Plant Mix Base Course	Item 310: Bituminous Concrete Surface Course Including Prime Coat and Tack Coat (Thickness = 10 cm)	Total Cost (₱/m)
2.5	176.01	171.56	571.01	—	510.70	—	1270.05	2699.33
3.0	176.01	209.82	699.69	—	514.57	—	1645.72	3245.61
3.5	176.01	287.34	882.17	—	716.52	—	1645.72	3707.76
4.0	176.01	339.02	953.07	448.04	547.77	—	1645.72	4109.63
4.5	176.01	416.54	1059.43	560.05	758.02	—	1645.72	4615.77
5.0	176.01	416.54	1059.43	560.05	572.67	1278.01	1645.72	5708.43
5.5	176.01	494.06	1165.79	672.06	782.92	1278.01	1645.72	6214.57

NOTE: SN 1.7 and 2.0 have no Tack Coat

FIGURE 16.2-3 TYPICAL CROSS – SECTION
AC RECONSTRUCTION (2 – LANES)

FIGURE 16.2-4 STRUCTURAL NUMBER (SN) OF AC PAVEMENT

SN 1.3

Layer	Thickness	T	BSC	DC	SN
SURFACE	5	2	0.39	-	0.78
CRUSHER RUN (CRUSHED GRAVEL)	15	6	0.105	0.8	0.504
	20				TOTAL = 1.284

SN 1.7

Layer	Thickness	T	BSC	DC	SN
SURFACE	5	2	0.39	-	0.78
CRUSHER RUN (CRUSHED GRAVEL)	15	6	0.105	0.8	0.504
SUBBASE	15	6	0.095	0.8	0.456
	35				TOTAL = 1.740

SN 2.1

Layer	Thickness	T	BSC	DC	SN
SURFACE	3.8	1.5	0.39	-	0.585
BINDER	3.8	1.5	0.39	-	0.585
CRUSHER RUN (CRUSHED GRAVEL)	15	6	0.105	0.8	0.504
SUBBASE	15	6	0.095	0.8	0.456
	37.6				TOTAL = 2.13

SN 2.5

Layer	Thickness	T	BSC	DC	SN
SURFACE	3.8	1.5	0.39	-	0.585
BINDER	3.8	1.5	0.39	-	0.585
CRUSHER RUN (CRUSHED GRAVEL)	20	8	0.105	0.8	0.672
SUBBASE	20	8	0.095	0.8	0.608
	47.6				TOTAL = 2.450

SN 3.0

Layer	Thickness	T	BSC	DC	SN
SURFACE	5	2	0.39	-	0.78
BINDER	5	2	0.39	-	0.78
CRUSHER RUN (CRUSHED GRAVEL)	20	8	0.105	0.8	0.672
SUBBASE	25	10	0.095	0.8	0.760
	55				TOTAL = 2.992

SN 3.5

Layer	Thickness	T	BSC	DC	SN
SURFACE	5	2	0.39	-	0.78
BINDER	5	2	0.39	-	0.78
CRUSHER RUN (CRUSHED GRAVEL)	30	12	0.105	0.8	1.008
SUBBASE	30	12	0.095	0.8	0.912
	70				TOTAL = 3.480

SN 4.0

Layer	Thickness	T	BSC	DC	SN
SURFACE	5	2	0.39	-	0.78
BINDER	5	2	0.39	-	0.78
MECHANICALLY STABILIZED (CRUSHED STONE)	20	8	0.125	0.8	0.80
CRUSHER RUN (CRUSHED GRAVEL)	20	8	0.105	0.8	0.672
SUBBASE	30	12	0.095	0.8	0.912
	80				TOTAL = 3.944

SN 4.5

Layer	Thickness	T	BSC	DC	SN
SURFACE	5	2	0.39	-	0.78
BINDER	5	2	0.39	-	0.78
MECHANICALLY STABILIZED (CRUSHED STONE)	25	10	0.125	0.8	1.00
CRUSHER RUN (CRUSHED GRAVEL)	30	12	0.105	0.8	1.008
SUBBASE	30	12	0.095	0.8	0.912
	95				TOTAL = 4.480

SN 5.0

Layer	Thickness	T	BSC	DC	SN
SURFACE	5	2	0.39	-	0.78
BINDER	5	2	0.39	-	0.78
BITUMEN STABILIZED	10	4	0.20	-	0.80
MECHANICALLY STABILIZED (CRUSHED STONE)	25	10	0.125	0.8	1.00
CRUSHER RUN (CRUSHED GRAVEL)	20	8	0.105	0.8	0.672
SUBBASE	30	12	0.095	0.8	0.912
	95				TOTAL = 4.944

SN 5.5

Layer	Thickness	T	BSC	DC	SN
SURFACE	5	2	0.39	-	0.78
BINDER	5	2	0.39	-	0.78
BITUMEN STABILIZED	10	4	0.20	-	0.80
MECHANICALLY STABILIZED (CRUSHED STONE)	30	12	0.125	0.8	1.2
CRUSHER RUN (CRUSHED GRAVEL)	30	12	0.105	0.8	1.008
SUBBASE	30	12	0.095	0.8	0.912
	110				TOTAL = 5.480

T = THICKNESS (in.)
B.S.C. = LAYER COEFFICIENT
D.C. = DRAINAGE COEFFICIENT
S.N. = STRUCTURAL NUMBER (T x B.S.C. x D.C.)

16.3 DESIGN CONSIDERATION

16.3.1 Traffic Loading Classes

(1) Equivalent Single Axle Load

In the pavement structural design, appropriate traffic input factors should be determined. Of primary concern are the number and weights of axle loads expected to be applied to the pavement during a given period of time. Since the axle loads from the light cars contribute very little to structural deterioration of the pavement, only heavy trucks and buses were considered for the purpose of designing pavement thickness.

The result of the AASHO Road Test has shown that damaging effect on pavement performance of the passage of an axle load of any mass can be represented by a number of 18-kip equivalent single axle loads or ESAL. This concept has been applied to the design equation of AASHTO Guide. The load equivalency factors derived from the AASHO Road Test are available in the same Guide.

(2) Traffic Loading Classes

Estimating the initial and future traffic volumes and loading for pavement structural design requires substantial study and analysis each as number of heavy trucks and buses, traffic growth, axle loads and axle configurations.

Table 16.3-1 shows the cummulative number of ESAL and traffic growth rate estimated from 1989 (initial year) and 2013 (project end year) along the Study Section. Figure 16.3-1 show them graphically.

For the purpose of the simplicity and the convenience for the future use of the output of the Study, traffic loadings on the Study Section were classified into thirteen (13) classes using the number of ESAL application at the initial year, as shown in Table 16.3-2.

16.3.2 Performance Period of Initial Pavement Structures

(1) Analysis on Most Economical Performance Period

When a pavement project is planned and assured economically feasible, it is required to achieve the maximum economy within that project. It involves a detailed economic evaluation of the possible alternatives within the project to optimize the project investment. It is essential in economic evaluation that all costs occuring during the life of the facility (life-cycle costs) be included.

An economic evaluation should consider many possible alternatives within the constraints of time and resources. It involves the selection of pavement types (rehabilitation method) and planned stage construction (performance period of initial pavement structures).

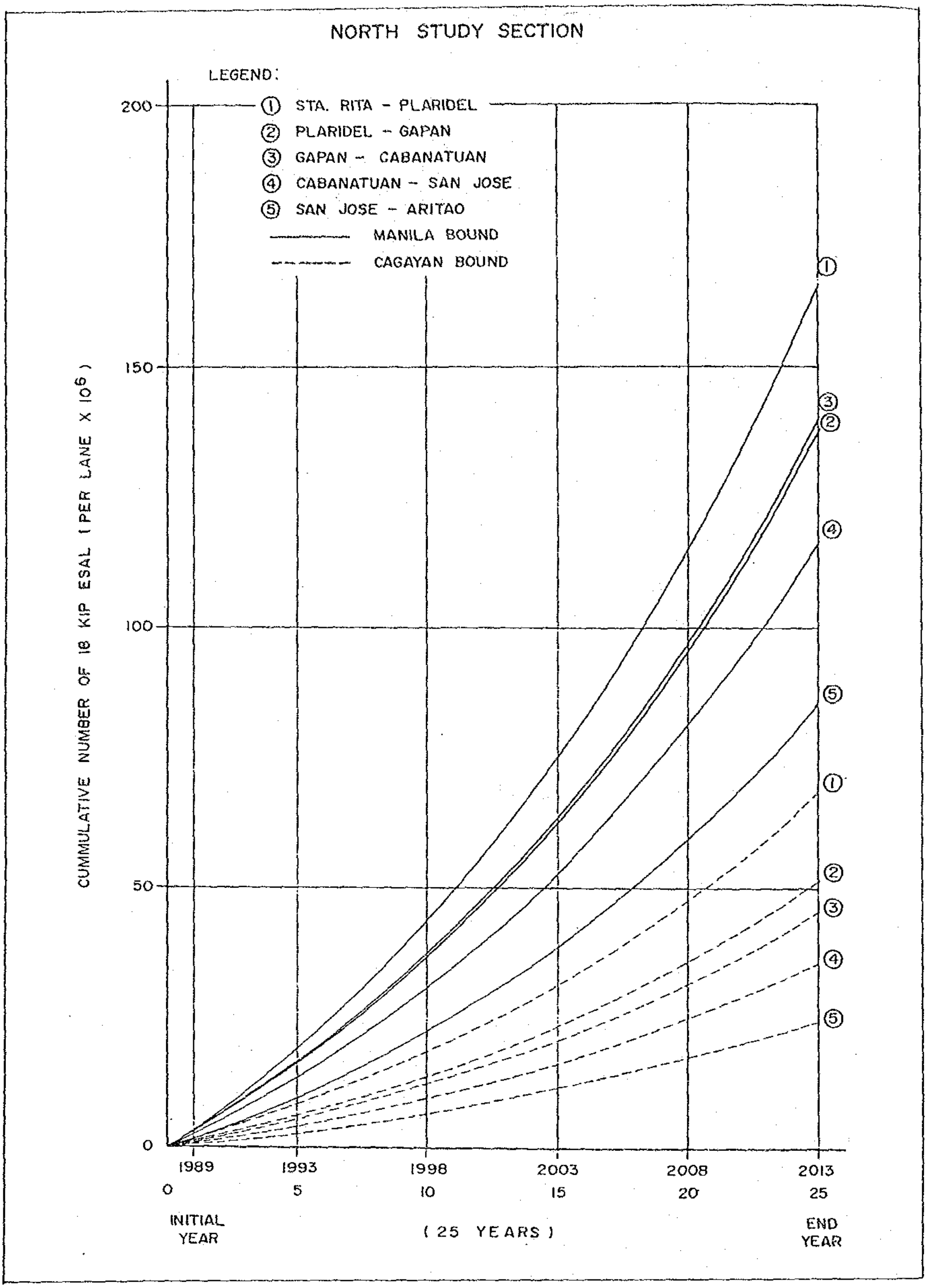


FIGURE 16.3-1 (1) CUMMULATIVE ESAL

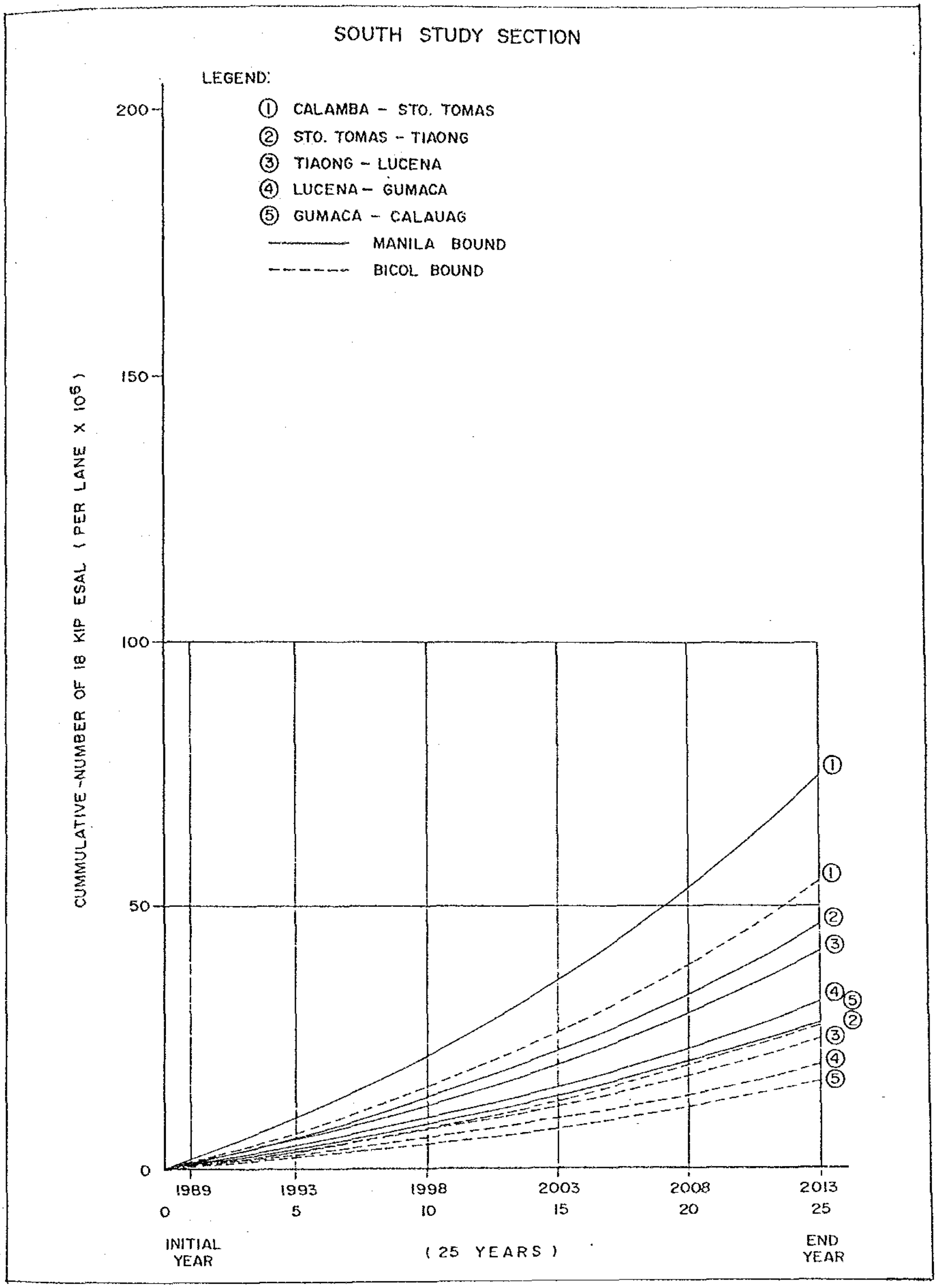


FIGURE 16.3-1 (2) CUMMULATIVE ESAL

TABLE 16.3-1 CUMMULATIVE NUMBER OF 18 KIP ESAL AND GROWTH RATE

Section	Lane	Number of ESALS in 1989 (x 10^6)	Number of ESALS in 2013 (x 10^6)	Cumulative Number of ESAL 1989-2013 (25 years)	Growth Rate 1989 – 2013
Sta. Rita – Plaridel	A	3.495	11.194	166.11	4.97
	B	1.451	4.602	68.55	4.93
Plaridel – Gapan	A	2.908	9.299	138.99	4.96
	B	1.098	3.461	51.67	4.90
Gapan – Cabanatuan	A	2.946	9.443	140.08	4.97
	B	0.964	3.050	45.47	4.92
Cabanatuan – San Jose	A	2.459	7.882	116.91	4.97
	B	0.763	2.413	35.98	4.91
San Jose – Aritao	A	1.787	5.762	85.27	5.00
	B	0.520	1.676	24.80	5.0
Calamba – Sto. Tomas	A	1.747	4.688	74.68	4.20
	B	1.258	3.436	54.34	4.28
Sto. Tomas – Tiaong	A	1.090	2.885	46.22	4.14
	B	0.637	1.724	27.38	4.24
Tiaong – Lucena	A	0.967	2.575	41.15	4.17
	B	0.577	1.575	24.90	4.27
Lucena – Gumaca	A	0.796	1.890	31.69	3.611
	B	0.480	1.167	19.36	3.771
Gumaca – Calauag	A	0.706	1.653	27.88	3.61
	B	0.407	0.972	16.26	3.69

NOTE: A: Manila Bound
B: Cagayan/Bicol Bound

TABLE 16.3-2 STANDARD TRAFFIC LOADING CLASSES

	Traffic Loading Class	Number of ESAL At Initial Year
Light Loading Traffic	L – 1	0.005 x 10^6
	L – 2	0.01
	L – 3	0.03
Heavy Traffic	A	0.03 – 0.1 x 10^6
	B	0.11 – 0.2
	C	0.21 – 0.4
	D	0.41 – 0.7
	E	0.71 – 1.0
Extra Heavy Loading Traffic	F	1.1 – 1.5 x 10^6
	G	1.6 – 2.0
	H	2.1 – 2.5
	I	2.6 – 3.0
	J	3.1 – 3.5

Analysis on most economical performance period of initial pavement structures were made, by an electric computer, selecting most economical pavement structure from alternatives of each proposed rehabilitation method under the following conditions:

- Rehabilitation Methods

  5 methods, Refer to Figure 16.2-1

- Alternative of Each Rehabilitation Method

  25 Alternatives for Heavy and Extra Heavy Loading Traffic and
  15 Alternatives for Light Loading Traffic Refer to Figure 16.2-1.

- Traffic Loading Classes

  Heavy Loading Traffic, 5 cases (A to E)
  Extra Heavy Loading Traffic, 5 cases (F to J)
  Light Loading Traffic, 3 cases (L_1 to L_3)

- Strength of Subgrade : 3 cases of CBR (3, 6 and 10)

- Design Variables : Strength of Concrete

  $S'c = 580$ psi
  $Ec = 3.28 \times 10^6$ psi

- Drainage Factor ; $Cd = 0.9$

- Load Transfer Coefficient ; $J = 4$

- Analysis Period ; 25 years

The output of analysis by an electric computer is presented in Appendix 16-2.

Table 16.3-3 summarizes the most economical performance periods analyzed for the cases of CBR value of 3, 6 and 10 which are representative. While, Figure 16.3-2 graphically shows most economical alternative and the most economical performance period of initial pavement structure for PCC Reconstruction, AC Reconstruction and AC Overlay-PCC Existing for the case of Traffic Loading Class E and CBR 6 which is most representable along the Study Section.

(2) Recommended Performance Periods

Table 16.3-4 summarizes the performance periods of the initial pavement structures recommended based on the analysis for the five (5) proposed rehabilitation methods, subclassifying traffic loading class into five (5).

TABLE 16.3-3 MOST ECONOMICAL PERFORMANCE PERIOD OF INITIAL PAVEMENT STRUCTURE

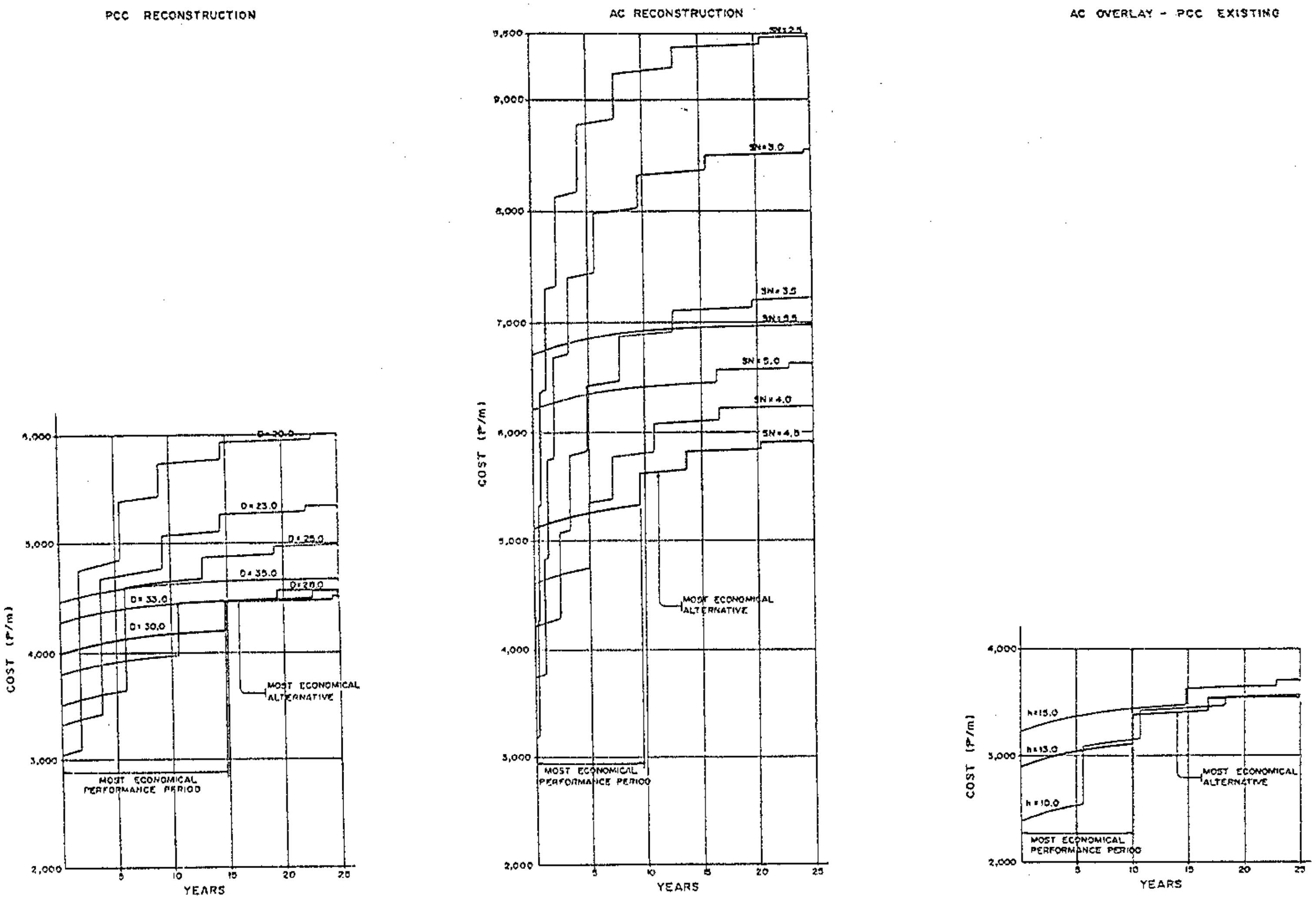
TRAFFIC LOADING CLASS \ METHOD / CBR	P C C Reconstruction				A C Reconstruction				PCC Overlay -PCC Existing				AC Overlay -PCC Existing				AC Overlay -AC Existing				Remarks
	3	6	10	Ave.	3	6	10	Ave.	3	6	10	Ave.	3	6	10	Ave.	3	6	10	Ave.	
L - 1	19.2**	24.2**	26.5**		18.2	15.0	14.2		49.1**	53.9**	56.0**		62.8**	83.3**	98.2**		5.0**	11.1**	18.2**		
L - 2	17.3	15.2**	16.9**	18.4	14.5	20.1	20.4	16.6	35.8**	40.2**	42.1**	38.0	48.4**	68.1**	82.7**	64.0	10.2	6.2**	10.9**	9.9	ESAL's ≅ 0.03×10^6
L - 3	19.0	14.1	13.3		16.0	15.4	16.0		19.3	21.9**	23.3**		28.3**	45.3**	58.9**		11.1	8.1	8.1		
A	18.6	14.0	14.0**		13.5	14.4	14.5		20.1	18.2	19.0		16.8**	30.4**	42.4**		3.2*	7.4*	11.0		
B	17.2	16.1	15.2	16.7	10.0	14.3	14.2	12.6	20.2	17.0	17.1	19.1	11.1	19.9**	29.9**	21.0	1.6*	4.0*	7.2*	-	ESAL's $=0.031 \sim 0.4 \times 10^6$
C	20.1	18.1	17.1		8.3	10.1	14.1		20.0	21.1	19.1		7.1	12.1**	19.6**		0.8*	2.1*	3.9*		
D	19.1	17.0	15.1	16.7	6.2	11.3	11.0	8.5	22.0	20.0	19.2	20.6	7.0	8.1	13.1	8.9	0.5*	1.2*	2.3*	-	ESAL's $=0.41 \sim 1.0 \times 10^6$
E	17.1	17.1	15.0		4.2	8.4	10.0		21.0	21.2	20.1		6.0	8.1	11.1		0.3*	0.9*	1.6		
F	18.3	15.2	15.1	15.8	9.5	6.8	11.1	8.2	20.0	20.1	20.0	18.9	4.0	9.1	7.1	6.5	0.2*	0.6*	1.1*	-	ESAL's $=1.1 \sim 2.0 \times 10^6$
G	17.1	14.1	15.0		7.4	5.3	9.3		17.1	17.2	19.1		3.6	8.0	7.0		0.2*	0.4*	0.8*		
H	14.2	15.0	13.1		6.1	4.3	7.8		14.5	15.2	16.3		3.1*	7.2*	9.3		0.1*	0.3*	0.7*		
I	12.7*	13.1	12.1	12.9	5.2	4.2	6.2	5.4	12.2	13.3	14.3	13.5	2.6*	6.1*	8.3	5.7	0.1*	0.3*	0.6*	-	ESAL's $=2.1 \sim 3.5 \times 10^6$
J	11.3*	12.3*	12.1		4.5	4.1	6.2		11.2*	12.3*	12.2		2.2*	5.3*	7.0		0.1*	0.2*	0.5*		

NOTE:
* Shows performance period of the maximum pavement structure (no period can be extended)
** Shows performance period of the minimum pavement structure (no. period can be shortened)
ESAL's means number of ESAL at the initial year

The ranges of pavement structures for the analysis are as follows:

Traffic Loading Class	P C C Reconstruction	A C Reconstruction	PCC Overlay -PCC Existing	AC Overlay -PCC Existing	AC Overlay -AC Existing
L-1 to L-3	D = 13 cm - 35 cm	SN = 1.3 - 5.5	h = 13 cm - 33 cm	h = 5 cm - 15 cm	h = 3 cm - 10 cm
A to J	D = 20 cm - 35 cm	SN = 2.5 - 5.5	h = 18 cm - 33 cm	h = 10 cm = 15 cm	h = 5 cm = 10 cm

D: Slab Thickness
SN: Structural Number
h: Overlay Thickness



NOTE : TRAFFIC LOADING CLASS : E
CBR : 6

FIGURE 16.3-2 ALTERNATIVE DESIGN STRATEGIES PLANNED STAGE, CONSTRUCTION, PERFORMANCE LIFE OF REHABILITATION

TABLE 16.3-4 RECOMMENDED PERFORMANCE PERIOD OF INITIAL PAVEMENT STRUCTURE

Traffic Loading Class	P C C Reconstruction	A C Reconstruction	PCC Overlay -PCC Existing	AC Overlay -PCC Existing	AC Overlay -AC Existing
L-1, L-2, L-3 ESAL's $\leqq 0.03\times10^6$	20 years 1) or Min. Thickness 13 cm	15 years	20 years 1) or Min. Thickness 13 cm	25 years 1) or Min. Thickness 5 cm	10 years 1)3) or Min. Thickness 3 cm
A, B, C ESAL's = $0.031\sim0.4\times10^6$	15 years	12 years	15 years	12 years 1) or Min. Thickness 10 cm	5 years 2)3) or Max. Thickness 10 cm
D, E ESAL's = $0.41\sim1.0\times10^6$	15 years	8 years	15 years	8 years 3)	5 years 2) 3) or Max. Thickness 10 cm
F, G ESAL's = $1.1\sim2.0\times10^6$	15 years	8 years	15 years	8 years 3)	Not Applicable
H, I, J ESAL's = $2.1\sim3.5\times10^6$	12 years 2) or Max. Thickness 35 cm	5 years 2) or Max. SN 5.5	12 years 2) or Max. Thickness 33 cm	5 years 2) 3) or Max. Thickness 15 cm	Not Applicable

NOTE: 1) Performance period is governed by the minimum structural requirement as the case may be.
2) Performance period is governed by the maximum pavement structure as the case may be.
3) Not applicable where performance period is too short (less than 5 years) even if the maximum pavement structure is applied (see "Basic Design")

16.4 DESIGN REQUIREMENTS

In this section, the preparation and/or selection of the input required for new or reconstruction pavement design are discussed.

Table 16.4-1 summarizes all possible design input requirement for rigid (PCC) pavement and flexible (AC), respectively.

Detailed discussion on design requirements are presented in Volume VI of the Study.

16.4.1 Design Variables

(1) Time Constraints

Time constraints involves the selection of performance periods and analysis period inputs which affect (or constrain) pavement design from the dimension of time.

Performance period refers to the period of time that an initial pavement structures will last before it needs rehabilitation.

Analysis period means the period of time for which the analysis is to be conducted. In this Study, an analysis period of 25 years was adopted.

(2) Traffic

The design analysis are based on cummulative expected 18-kip equivalent single axle loads (ESAL) during the analysis period (W18).

(3) Reliability

Reliability concept is introduced in AASHTO Guide 1986 to account for chance variations in both traffic prediction and performance prediction.

However, reliability was concluded not to considered in this Study after the discussion with representative of MPWH. It is, therefore, noted that reliability of pavement design of this Study is judged as 50 percent which is lowest level suggested by AASHTO Guide 1986.

(4) Environmental Impacts

In this Study, serviceability loss due to roadbed smelling (PSI_{SW}) was not accounted for because the effects of seasonal temperature and moisture changes on material properties are not known yet.

16.4.2 Performance Criteria

The primary measure of serviceability is the Present Serviceability Index (PSI) which ranges from 0 (impossible road) to 5 (perfect road). The original or initial

TABLE 16.4-1 DESIGN REQUIREMENTS

Category	Description
1. Design Variable	
1.1 Time Constraints	
. Performance Period	Life of Initial Pavement Structure
. Analysis Period	Planned Stage construction; 25 years
1.2 Traffic	W_{18} = 18 kip Equivalent Single Axle Load (ESAL) Application Traffic Loading Classes; 10 classes (A to J)
1.3 Reliability	Z_R = 1.645 for 95% Reliability, R) not considered S_0 = 0.3~0.4 for Standard Error)
1.4 Environmental Impact	
. Roadbed Swelling	PSI_{SW} = Loss of PSI; not considered
2. Performance Criteria	
2.1 Serviceability	$PSI = P_0 - P_t = P_0 - \Delta PSI_W - \Delta PSI_{SW}$ (ΔPSI_{SW}; not considered)
3. Material Properties for Structural Design	
3.1 Effective Roadbed Soil Resilent Modules (Flexible)	MR (pci); estimated based on CBR, 8 cases (2~20)
3.2 Effective Modulus of Subgrade Reaction (Rigid)	K-Value (pci); estimated based on CBR, 8 cases (2~20)
3.3 Pavement Layer Materials Characteri-zation	E_{SB} = Modulus of Subbase (8,000 psi) E_{BS} = Modulus of Base (22,000 psi) E_{AC} = Modulus of Asphalt Concrete (350,000 psi) E_C = Modulus of Elasticity of PCC(3.28×10^6 psi)
3.4 PCC Modulus of Rupture (Rigid) (Flexural Strength)	S'_c = Estimated Mean Value for PCC Modulus of Rupture (psi); 580 psi
3.5 Structural Layer Coefficient (Flexible)	Asphalt Concrete Layer Coefficient ; 0.39 Bitumen Stabilized : 0.2 Mechanically Stabilized (Crushed Stone); 0.125 Crusher Run (Crushed Gravel) ; 0.105 Subbase ; 0.095
4. Pavement Structural Characteristics	
4.1 Drainage	Flexible m = Layer Coefficent Modifying Factor; 0.8 Rigid CD = Drainage Coefficient; 0.9
4.2 Load Transfer (Rigid)	
. Jointed Pavement	
. Tied shoulder or Widened Outside Lane	J = Load Transfer Coefficient; 4
4.3 Loss of Support (Rigid)	LS = Loss of Support 1.0 ~ 3.0 for unbounded granular materials 2.0 ~ 3.0 for fine granular or natural subgrade materials 0 ~ 1.0 for cement Treated Granular Base
5. Reinforcement Variables (Rigid)	
5.1 Slab Length	Depending on local conditions, subbase type, course aggregate, etc.
5.2 Working Stress	
5.3 Friction Factors	

serviceability (Po) observed at the AASHO Road Test were:

Po = 4.5 for Rigid Pavement
= 4.2 for Asphalt Pavement

AASHTO Guide 1986 suggest the lowest allowable PSI or terminal serviceability Index (Pt).

Pt = 2.5 for design of major highway
= 2.0 for design of highway with lesser traffic

The pavement condition, especially the lengths of cracks of concrete pavements with PSI of 2.5 and 2.0 were compared those with the Rehabilitation Requirement Index (RRI) of the same value proposed in this Study. Then, it is concluded that the value of RRI can be used for the pavement design equations in AASHTO Guide 1986, since both values indicate almost same pavement condition.

In this Study, the terminal serviceability of 2.5 was adopted taking into account the importance and traffic volume of the Study road, as discussed in Chapter 15.

16.4.3 Material Properties for Structural Design

(1) Effective Roadbed Soil Resilient Modulus, (Flexible Pavement)

To determine effective roadbed soil resilient modulus (MR) exclusively for the design of flexible pavements based on serviceability criteria, laboratory resilient modulus test (AASHTO T274) should be performed on representative sample in stress and moisture conditions. Seasonal resilient modulus should also be determined based on conditions of primary moisture seasons, dry and wet seasons.

In this Study, effective roadbed soil resilient modulus (MR) were estimated only based on soil classification and CBR Test result because of absence of available data, as shown in Table 16.4-2.

(2) Effective Modulus of Subgrade Reaction (Rigid Pavement)

An effective modulus of subgrade reaction (k-value) will be developed for rigid pavement design, accounting for seasonal modulus value, effects of subbase characteristics, effects of rigid foundation, relative damage of slab thickness due to 18 kip ESAL, loss of support etc.

In this Study, effective modulus of subgrade reaction (k-value) were estimated, taking into account the effect of subbase, as shown in Table 16.4-2.

TABLE 16.4-2 STRENGTH OF ROADBED/SUBGRADE

CBR of Subgrade	k (pci) of Subgrade	MR (pci) of Subgrade	k (pci)
2	50	2,500	80
3	100	4,000	130
4	120	5,000	170
6	160	6,000	210
8	180	7,000	230
10	200	8,000	250
15	230	12,000	280
20	250	15,000	300

Note: K; estimated based on the suggestion by Portland Cement Association.

(3) Pavement Layer Materials Characteristics

In this Study, modulus of pavement layer materials were estimated following the suggestion by AASHTO Guide, 1986 as follows:

- Moduli for Subbase, E_{SB} ; 8,000 psi (at 68°F)
- Moduli for Base, E_{BS} ; 22,000 psi
- Moduli for Asphalt Concrete, E_{AC} ; 350,000 psi
- Moduli for Portland Cement Concrete E_c ; 3.28 x 10^6 psi

$$E_c = 57{,}000\ ('c)^{0.5}$$

Where:

Ec = PCC elastic modulus (psi)

'c = PCC compressive strength (psi)

(4) PCC Modulus of Rupture

The modulus of rupture (flexural strength) of portland cement concrete required by the design procedure is the mean value determined after 28 days using third-point loading (ASSHTO T97, ASTM C78) and estimated as follows:

S'_c (mean) = Sc + Z (SDs)

S'_c = Estimated mean value for PCC Modulus of Rupture (psi); 580 psi in the Study

Sc = construction specification in concrete modulus of rupture (psi)

SDs = estimated standard deviation of concrete modulus of rupture (psi)

Z = standard normal variate

In the Study, the modulus of rupture of 580 psi (40/kg/cm^2) was adopted in accordance with the specification requirement of 525 psi (36.8 kg/cm^2) at 14 days in the country.

(5) Layer Coefficients (for Flexible Pavement)

A value for layer coefficient is assigned to each layer material in the pavement structure in order to convert actual layer thickness into structural number (SN).

In this Study, the structural layer coefficients (Ai values were assumed as follows:

TABLE 16.4-3 STRUCTURAL COEFFICIENTS, Ai

Layer Material	Layer Coefficients
Asphalt Concrete Surface Course	0.39
Bitumen Stabilized	1.2
Mechanically Stabilized (Crushed Stone)	0.125 (CBR 40, R value 70)
Crusher Run (Crushed Gravel)	0.105 (CBR 25, R value 60)
Subbase	0.095 (CBR 8, value 40)

16.4.4 Pavement Structural Characteristics

(1) Drainage

The effects of certain levels of drainage on predicted pavement performance are important consideration in pavement design.

Table 16.4-4 presents the general definitions corresponding to different drainage levels from pavement structure.

TABLE 16.4-4 DRAINAGE LEVELS

Quality of Drainage	Water Removed Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	(water will not drain)

Depending on drainage level, AASHTO recommends m; values for modifying structural coefficients of base and subbase materials for flexible pavement and value of drainage coefficient Cd for rigid pavement.

In the Study, m; of 0.8 and Cd of 0.9 were adopted.

(2) Load Transfer (Rigid Pavement)

The load transfer coefficient, J, is a factor used in rigid pavement design to account for the ability of a concrete pavement structure to transfer (distribute) load across discontinuities, such as joints or cracks.

In this Study, load transfer coefficient of 4 was used considering from effect of plain joint.

(3) Loss of Support

This factor, LS, is included in the design of rigid pavements to account for the potential loss of support arising from subbase erosion and/or differential vertical soil movements. It is treated in the actual design procedure by diminishing the effective or composite k-value based on the size of the void that may develop beneath the slab.

16.4.5 Reinforcement Variables

Reinforcement variables were not discussed in this Study.

16.5 BASIC STRUCTURAL DESIGN

16.5.1 Assumed Design Variables and Costs

In this Study, AASHTO Guide for Design for Pavement Structure, 1986, was adopted, while others were used as the reference. Design procedures are discussed in Volume VI of this Study.

Design variables discussed in Chapter 16.4 are summarized in Table 16.5-1. Costs used for basic structural designing are also presented in Table 16.5-2.

16.5.2 Design Outputs

The pavement structural design was carried out by an electric computer, developing the programs for design equations. Design outputs are presented by Appendix 16-3.

The rehabilitation methods design are:

- PCC Reconstruction (New Construction)
- AC Reconstruction (New Construction)
- Rigid Overlay – Rigid Existing
- Flexible Overlay – Flexible Existing
- Flexible Overlay – Flexible Existing

TABLE 16.5-1 ASSUMED DESIGN VARIABLES

Analysis	25 years
Performance Period	As indicated in Design Output
Traffic	As indicated in Design Output
Reliability	Not considered
Environmental Impact	Not considered because of no roadbed swelling
Serviceability	
Initial	4.5 for rigid, 4.2 for flexible
Terminal	2.5 for both
Effective Roadbed Soil Resilent Modulus (Flexible Pavement)	As indicated in Design Output
Effective Modulus of Subgrade Reaction (Rigid Pavement)	As indicated in Design Output
Pavement Layer Material Characteristics	See Table 16.4-1
PCC Modulus of Rupture	525 psi (36.8 kg/cm^2) at 14 days 580 psi (40.6 kg/cm^2) at 28 days
Structural Layer Coefficient	See Table 16.4-1 and 3
Drainage	m for flexible = 0.8 CD for Rigid = 0.9
Load Transfer Coefficient	4
Loss of Support	1 (Not considered)

The major outputs are as follows:

- Slab Thickness; D cm (PCC)
- Structural Number SN (AC)
- Thickness of Overlay; h (cm)
- Performance period of initial pavement structure; x years
- Initial Construction Cost; C_1 (Pesos per m of 2 lanes)
- Total Discounted Cost; C_2 (Pesos per m of 2 lanes)

Figure 16.5-1 summarizes the design outputs, for 13 cases of traffic loading, 8 cases of CBR for each proposed rehabilitation method.

(2) Construction Costs

The construction costs were roughly estimated as shown in Table 16.5-2

TABLE 16.5-2 ROUGHLY ESTIMATED COST

Unit: ₱/M-2 lanes

	Thickness	First (Initial) Rehabilitation Work	Improvement of Drainage	Second Rehabilitation Work	Third/After Third Rehabilitation Work	Maintenance Cost Per Year	
						After First Rehabilitation	After Second Rehabilitation
PCC Reconstruction	13 cm	1,896	496	1,093	1,093	30	37
	15 cm	2,084	496	1,093	1,093	30	37
	18 cm	2,365	496	1,093	1,093	30	37
	20 cm	2,552	496	1,898	1,093	30	37
	23 cm	2,833	496	1,898	1,093	30	37
	25 cm	3,021	496	1,898	1,093	30	37
	28 cm	3,302	496	1,898	1,093	30	37
	30 cm	3,489	496	1,898	1,093	30	37
	33 cm	3,770	496	1,898	1,093	30	37
	35 cm	3,958	496	1,898	1,093	30	37
AC Reconstruction	SN = 1.3	1,532	496	1,048	1,048	37	37
	1.7	1,934	496	1,048	1,048	37	37
	2.1	2,400	496	1,048	1,048	37	37
	2.5	2,699	496	1,048	1,048	37	37
	3.0	3,246	496	1,048	1,048	37	37
	3.5	3,708	496	1,048	1,048	37	37
	4.0	4,109	496	1,048	1,048	37	37
	4.5	4,616	496	1,048	1,048	37	37
	5.0	5,708	496	1,048	1,048	37	37
	5.5	6,214	496	1,048	1,048	37	37
PCC Overlay -PCC Existing	13 cm	1,857	496	1,857	1,857	30	30
	15 cm	2,034	496	2,034	2,034	30	30
	18 cm	2,300	496	2,300	2,300	30	30
	20 cm	2,477	496	2,477	2,477	30	30
	23 cm	2,742	496	2,742	2,742	30	30
	25 cm	2,919	496	2,919	2,919	30	30
	28 cm	3,185	496	3,185	3,185	30	30
	30 cm	3,362	496	3,362	3,362	30	30
	33 cm	3,628	496	3,628	3,628	30	30
AC Overlay -PCC Existing	5 cm	1,093	496	1,093	1,093	37	37
	8 cm	1,585	496	1,093	1,093	37	37
	10 cm	1,898	496	1,093	1,093	37	37
	13 cm	2,401	496	1,093	1,093	37	37
	15 cm	2,736	496	1,093	1,093	37	37
AC Overlay -AC Existing	3 cm	735	496	735	735	37	37
	5 cm	1,048	496	1,048	1,048	37	37
	8 cm	1,540	496	1,048	1,048	37	37
	10 cm	1,853	496	1,048	1,048	37	37

PCC RECONSTRUCTION

Required Thickness of Slab (cm)

	TRAFFIC LOADING CLASS \ CBR	2	3	4	6	8	10	15	20	DESIGN PERFORMANCE PERIOD
LIGHT TRAFFIC LOADING ROAD	L-1	15				15				20 YEARS
	L-2	18			15					
	L-3	20			18					
HEAVY TRAFFIC LOADING ROAD	A					23		20		15 YEARS
	B		25							
	C						28			
	D				30					
	E									
EXTRA-HEAVY TRAFFIC LOADING ROAD	F					33				
	G									
	H			35						12 YEARS
	I									
	J	(10.4)	(11.3)							

Note ; 1. Provision of Filter Layer is required for CBR less than 3.
2. For CBR less than 2, improvement method for weak subgrade should be applied.
3. ( ) Shows initial performance period less than design even adopting 35 cm. PCC slab.

AC RECONSTRUCTION

Required Structural Number

	TRAFFIC LOADING CLASS \ CBR	2	3	4	6	8	10	15	20	DESIGN PERFORMANCE PERIOD
LIGHT TRAFFIC LOADING ROAD	L-1			2.5			2.1		1.7	15 YEARS
	L-2	3.0								
	L-3	3.5			3.0		2.5			
HEAVY TRAFFIC LOADING ROAD	A	4.0				3.5	3.0	2.5		12 YEARS
	B	4.5		4.0					3.0	
	C					4.0				
	D									8 YEARS
	E					4.5				
EXTRA-HEAVY TRAFFIC LOADING ROAD	F			5.0					3.5	
	G									
	H		5.5					4.0		5 YEARS
	I									
	J									

Note ; 1. An improvement method for weak subgrade with CBR less than 3 is required to improve it to CBR more than 3.
2. For CBR less than 2, improvement method for weak subgrade should be applied.

FIGURE 16.5-1 (1) PAVEMENT STRUCTURE CAPACITY OF INITIAL PAVEMENT STRUCTURES

RIGID OVERLAY – RIGID EXISTING

Required Thickness of PCC Overlay (cm)

	CBR / TRAFFIC LOADING CLASS	2	3	4	6	8	10	15	20	DESIGN PERFORMANCE PERIOD
LIGHT TRAFFIC LOADING ROAD	L-1									20 YEARS
	L-2					13				
	L-3	15								
HEAVY TRAFFIC LOADING ROAD	A	20				18				15 YEARS
	B		23			20				
	C		25			23				
	D					28			25	
	E	30								
EXTRA-HEAVY TRAFFIC LOADING ROAD	F					30				
	G									12 YEARS
	H					33				
	I									
	J	(10.4)	(11.2)							

Note ; ( ) Shows initial performance period less than design even adopting 33 cm Rigid Overlay.

FLEXIBLE OVERLAY – RIGID EXISTING

Required Thickness of AC Overlay (cm)

	CBR / TRAFFIC LOADING CLASS	2	3	4	6	8	10	15	20	DESIGN PERFORMANCE PERIOD
LIGHT TRAFFIC LOADING ROAD	L-1									15 YEARS
	L-2					5				
	L-3	8								
HEAVY TRAFFIC LOADING ROAD	A	13								12 YEARS
	B	(10.8)	13							
	C	(6.0)	15	13			10			
	D									8 YEARS
	E		(7.0)	15	13					
EXTRA-HEAVY TRAFFIC LOADING ROAD	F		(4.9)	(7.7)						
	G			(6.0)		13				
	H	NOT			15					6 YEARS
	I	APPLICABLE								
	J									

Note ; ( ) Shows initial performance period less than design even adopting 15 cm Flexible Overlay.

FIGURE 16.5-1 (2) PAVEMENT STRUCTURE CAPACITY OF INITIAL PAVEMENT STRUCTURES

FLEXIBLE OVERLAY — FLEXIBLE EXISTING
Required Thickness of AC Overlay (cm)

	TRAFFIC LOADING CLASS \ CBR	2	3	4	6	8	10	15	20	DESIGN PERFORMANCE PERIOD
LIGHT TRAFFIC LOADING ROAD	L-1	8	5					3		10 YEARS
	L-2	10			5					
	L-3		10		8					
HEAVY TRAFFIC LOADING ROAD	A			10					5	8 YEARS
	B					10				
	C							8		
	D							10		5 YEARS
	E								10	
EXTRA-HEAVY TRAFFIC LOADING ROAD	F		NOT	APPLICABLE						
	G									
	H									5 YEARS
	I									
	J									

FIGURE 16.5-1 (3) PAVEMENT STRUCTURE CAPACITY OF INITIAL PAVEMENT STRUCTURES

16.6 EVALUATION OF PROPOSED PAVEMENT REHABILITATION METHODS

Discussed in this Section are the selection procedure of rehabilitation method and their economic evaluation. These discussion were performed only on the spots selected for the case study.

16.6.1 Selection of Case Study Spots

The factors to be considered in pavement rehabilitation design and evaluation are traffic loading, traffic volume by vehicle type, existing pavement conditions, roadbed soil property and drainage conditions among others. The case study spots were selected so as to cover existing range of such factors since the case study results case shall be applicable to every spot of the Studty Section and preferably to other roads with interpolation/extrapolation as necessary.

Table 16.6-1 shows the selected case study spots with description of traffic loading, traffic volumes, RRI, estimated CBR values of subgrade and drainage conditions.

16.6.2 Selection of Rehabilitation Methods for Case Study Spots

The selection of procedures of pavement rehabilitation methods are discussed in Chapter 22, covering technical, monetary and non-monetary considerations. Refer to Chapter 22 for selection procedure. However, rehabilitation methods were initially selected only based on technical consideration.

TABLE 16.6-1 SELECTED CASE STUDY SPOTS AND CONDITIONS

Segment		Case Study Spots	Location	Traffic Loading Class	AADT in 1989	RRI[1/]	Subgrade CBR (Assumed)	Drainage Condition (Assumed)
Segment N-1 (Sta. Rita-Gapan Sect.) L = 46 km	Sta. Rita -Plaridel	N-CS-1	40.5	J/F	12,510	1.0/2.2	3-4	Fair
		N-CS-2	41.0			1.0/3.0	3-4	Bad
	Plaridel - Gapan	N-CS-3	53.0	I/E	7,260	1.5/3.0	4	Fair
		N-CS-4	62.0			2.0/2.6	6	Fair
		N-CS-5	76.0			1.0/2.0	3	Fair
Segment N-2 (Gapan-Cabanatuan Section) L = 35 km		N-CS-6	110.0	I/E	7,150	2.2/3.0	8	Fair
Segment N-3 (Cabanatuan-San Jose Section) L = 42 km		N-CS-7	130.5	H/D	4,880	1.5/3.5	8	Fair
		N-CS-8	146.0			1.8/2.2	6	Bad
		N-CS-9	152.0			2.0/3.1	6	Fair
Segment N-4 (San Jose-Dalton Section) L = 38 km		N-CS-10	174.0	G/D	2,780	2.0/2.9	4	Fair
		N-CS-11	176.0			1.0/2.2	4	Fair
		N-CS-12	195.0			2.0/2.9	8	Fair
Segment N-5 (Dalton-Aritao Section) L = 39 km		N-CS-13	201.0	G/D	2,780	1.1/2.1	8	Fair
		N-CS-14	206.0			1.6/2.6	8	Fair
		N-CS-15	230.0			2.0/3.0	6	Fair
Segment S-1 (Calamba-Tiaong Section) L = 42 km	Calamba-Sto. Tomas	S-CS-1	55.0	G/F	14,200	2.0/2.0	3-4	Fair
	Sto. Tomas - Tiaong	S-CS-2	76.0	E/D	7,140	2.0/3.0	3	Bad
		S-CS-3	77.0			2.0/2.3	3	Bad
Segment S-2 (Tiaong-Pagbilao Section) L = 54 km		S-CS-4	106.0	E/D	5,040	2.3/3.0	8	Fair
		S-CS-5	138.0			2.1/3.0	8	Fair
		S-CS-6	142.0			0.5/0.5	8	Fair
Segment S-3 (Pagbilao-Plaridel Section) L = 46 km		S-CS-7	158.0	D/C	2,650	0.5/0.5	4	Bad
Segment S-4 (Plaridel -Calauag Section) L = 39 km		S-CS-8	213.0	D/C	2,140	1.8/2.8	8	Fair
		S-CS-9	214.0			1.8/1.9	8	Fair
		S-CS-10	217.0			2.2/2.2	6	Fair
		S-CS-11	220.0			2.1/2.8	4	Fair

[1/] Values for Manila Bound/Opposite Bound

The number of selected spots for case study is 15 for the North Study Section and 11 for the South. Table 16.6-2 summarizes the number of rehabilitation methods. Table 16.6-3 shows the selected rehabilitation method for each case study spots, the conditions of which are also mentioned.

TABLE 16.6-2 NUMBER OF SELECTED REHABILITATION METHODS FOR THE CASE STUDY

	1-Lane PCC Reconstruction	2-Lane PCC Reconstruction	2-Lanes AC Overlay	Total
North Study Section	6	5	4	15
South Study Section	4	5	2	11
Total	10	10	6	26

16.6.3 Economic Evaluation of Selected Rehabilitation Methods

The economic evaluation was carried were for the rehabilitation methods selected only based on technical considerations. The economic evaluation procedures are discussed in Chapter 22.

Aside from the economic indicators, e. g. benefit cost ratio (B/C) and internal rate of return (IRR), dl and dt value were calculated. The dl and dt are imaginary extra road length and time depending on road surface conditions when compared with the ideal surface conditions.

Figure 16.6-1 shows the examples of annual changes of pavement serviceability, AADT, construction cost, maintenance cost and traffic cost. The economic evaluation was made per unit length based on these costs and benefits stream.

The outputs of economic evaluation on selected rehabilitation methods are presented in Appendix 16-4, while Table 16.6-4 summarizes the results of evaluation.

The benefit/cost ratios and internal rates of return vary from 10.8 to 2.3 and from 42% to 43%, respectively, depending mainly upon traffic volume. High economic feasibilities were found in all cases. The savings in the dl-values vary from 0.29 to 0.38 km/actual-km/vehicle in case of PCC reconstruction and from 0.26 to 0.29 km/actual-km/vehicle in case of AC overlay. The savings in the dt-values vary from 28.3 to 33.8 sec/km/veh. in case of PCC reconstruction and from 24.4 to 27.0 sec/km/veh. in case of AC overlay. The savings in both dl- and dt- values are little lower in AC overlay than those in PCC reconstruction because of the shorter performance period of AC overlay.

The correlation between rehabilitation methods and economic indicator as well as dl and dt are graphically summarized in Chapter 19.

TABLE 16.6-3 SELECTED REHABILITATION METHODS

Case Study Spots	Location	Traffic Loading Class	AADT in 1989	RRI 1/	Subgrade CBR (Assumed)	Drainage Condition (Assumed)	Selected Rehabilitation Method
N-CS-1	40.5	J/F	12,510	1.0/2.2	3-4	Fair	2-lane PCC Reconst. (D=35/33 cm)
N-CS-2	41.0			1.0/3.0	3-4	Bad	1-lane PCC Reconst. (D=35 cm)
N-CS-3	53.0	I/E	7,260	1.5/3.0	4	Fair	1-lane PCC Reconst. (D=35 cm)
N-CS-4	62.0			2.0/2.6	6	Fair	2-lane AC Overlay (h=15 cm)
N-CS-5	76.0			1.0/2.0	3	Fair	2-lane PCC Reconst. (D=35/33 cm)
N-CS-6	110.0	I/E	7,150	2.2/3.0	8	Fair	2-lane AC Overlay (h=13 cm)
N-CS-7	130.5	H/D	4,880	1.5/3.5	8	Fair	1-lane PCC Reconst. (D=33 cm)
N-CS-8	146.0			1.8/2.2	6	Bad	2-lane PCC Reconst. (D=35/30 cm)
N-CS-9	152.0			2.0/3.1	6	Fair	1-lane PCC Reconst. (D=35 cm)
N-CS-10	174.0	G/D	2,780	2.0/2.9	4	Fair	1-lane PCC Reconst. (D=35 cm)
N-CS-11	176.0			1.0/2.2	4	Fair	2-lane PCC Reconst. (D=35/30 cm)
N-CS-12	195.0			2.0/2.9	8	Fair	2-lane AC Overlay (h=13 cm)
N-CS-13	201.0	G/D	2,780	1.1/2.1	8	Fair	2-lane PCC Reconst. (D=33/28 cm)
N-CS-14	206.0			1.6/2.6	8	Fair	1-lane PCC Reconst. (D=33 cm)
N-CS-15	230.0			2.0/3.0	6	Fair	2-lane AC Overlay (h=15 cm)
S-CS-1	55.0	G/F	14,200	2.0/2.0	3-4	Fair	2-lane PCC Reconst. (D=35/33 cm)
S-CS-2	76.0	E/D	7,140	2.0/3.0	3	Bad	1-lane PCC Reconst. (D=33 cm)
S-CS-3	77.0			2.0/2.3	3	Bad	2-lane PCC Reconst. (D=33/30 cm)
S-CS-4	106.0	E/D	5,040	2.3/3.0	8	Fair	1-lane PCC Reconst. (D=30 cm)
S-CS-5	138.0			2.1/3.0	8	Fair	2-lane AC Overlay (h=10 cm)
S-CS-6	142.0			0.5/0.5	8	Fair	2-lane PCC Reconst. (D=30/28 cm)
S-CS-7	158.0	D/C	2,650	0.5/0.5	4	Bad	2-lane PCC Reconst. (D=30/28 cm)
S-CS-8	213.0	D/C	2,140	1.7/2.8	8	Fair	1-lane PCC Reconst. (D=28 cm)
S-CS-9	214.0			1.8/1.9	8	Fair	2-lane PCC Reconst. (D=28/28 cm)
S-CS-10	217.0			2.2/2.2	6	Fair	2-lane AC Overlay (h=10 cm)
S-CS-11	220.0			2.1/2.8	4	Fair	1-lane PCC Reconst. (D=30 cm)

1/ Values for Manila Bound/Opposite Bound

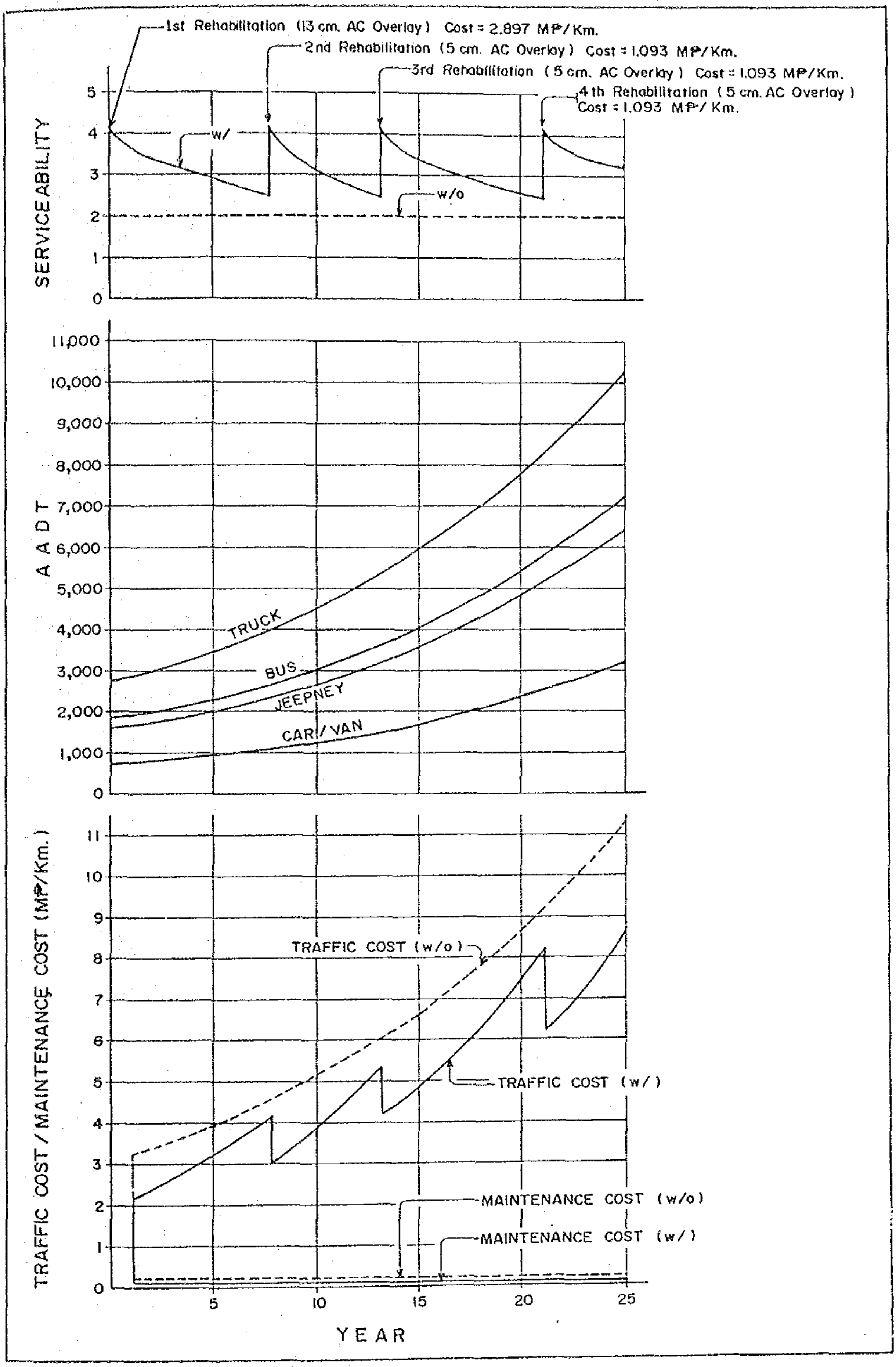


FIGURE 16.6-1 YEARLY CHANGE OF SERVICEABILITY AND COST (CASE STUDY SPOT N–CS–12)

TABLE 16.6-4 EVALUATION OF SELECTED REHABILITATION METHODS

Case Study Spot	Traffic Loading Class	AADT in 1989 (Annual Growth Rate in %) Car/Van	Jeepney	Tricycle	Bus	Truck	Total	Selected Rehabilitation Method	Initial Const. Cost (MP/Km)	Economic Evaluation Total Dis'd. Cost (MP/Km)	Total Dis'd. Benefit (MP/Km)	B/C Ratio	IRR (%)	Savings in dl & dt dl (km. km./veh.)	dt (sec./km./veh.)
N-CS-1 N-CS-2	J/F	6,250 (6)	2,260 (5)	560 (5)	950 (4.5)	2,490 (5)	12,510	2-lane PCC Reconst. (D=35/33cm) 1-lane PCC Reconst. (D=35cm)	4,360 2,475	4,501 2,762	31,061 16,992	6.9 6.2	229.3 180.4	0.29 0.35	29.8 33.4
N-CS-3 N-CS-4 N-CS-5	I/E	2,860 (6)	1,390 (5)	290 (5)	950 (4.5)	1,770 (5)	7,260	1-lane PCC Reconst. (D=35cm) 2-lane AC Overlay (h=15cm) 2-lane PCC Reconst. (D=35/33cm)	2,475 3,232 4,360	2,574 3,667 4,377	11,458 15,929 21,054	4.5 4.3 4.8	96.5 108.0 115.5	0.38 0.26 0.31	32.9 26.2 29.0
N-CS-6	I/E	3,330 (6)	1,270 (5)	230 (5)	700 (4.5)	1,620 (5)	7,150	2-lane AC Overlay (h=13cm)	2,897	3,495	14,906	4.3	111.3	0.26	26.4
N-CS-7 N-CS-8 N-CS-9	H/D	1,310 (6)	1,270 (5)	460 (5)	570 (4.5)	1,270 (5)	4,880	1-lane PCC Reconst. (D=33cm) 2-lane PCC Reconst. (D=35/30cm) 1-lane PCC Reconst. (D=35cm)	2,381 4,220 2,475	2,566 3,979 2,428	7,449 14,716 7,663	2.9 3.7 3.2	58.5 69.8 57.4	0.37 0.35 0.38	35.1 33.8 35.6
N-CS-10 N-CS-11 N-CS-12	G/D	720 (6.5)	880 (5.5)	-	240 (5)	940 (5)	2,780	1-lane PCC Reconst. (D=35cm) 2-lane PCC Reconst. (D=35/30cm) 2-lane AC Overlay (h=13cm)	2,475 4,220 2,897	2,354 3,919 3,164	4,962 9,521 6,897	2.1 2.4 2.2	34.1 40.6 38.2	0.40 0.36 0.28	31.8 29.1 25.3
N-CS-13 N-CS-14 N-CS-15	G/D	720 (6.5)	880 (5.5)	-	240 (5)	940	2,780	2-lane PCC Reconst. (D=33/28cm) 1-lane PCC Reconst. (D=33cm) 2-lane AC Overlay (h=15cm)	4,032 2,381 3,232	3,821 2,396 3,377	9,267 4,790 6,785	2.4 2.0 2.0	41.5 34.3 34.0	0.35 0.37 0.28	29.2 30.0 25.1
S-CS-1	G/F	9,470 (6)	1,260 (5)	70 (5)	1,340 (5)	2,060 (4)	14,200	2-lane PCC Reconst. (D=35/33cm)	4,360	4,078	37,585	9.2	282.5	0.32	31.0
S-CS-2 S-CS-3	E/D	4,170 (6)	1,080 (4.5)	10 (4.5)	580 (5)	1,300 (4)	7,140	1-lane PCC Reconst. (D-33cm) 2-lane PCC Reconst. (D-33/30cm)	2,381 4,126	2,170 3,858	10,399 19,586	4.8 5.1	83.7 101.3	0.37 0.34	32.8 31.1
S-CS-4 S-CS-5 S-CS-6	E/D	2,500 (6)	810 (5)	-	610 (5)	1,120 (4)	5,040	1-lane PCC Reconst. (D=30cm) 2-lane AC Overlay (h=10cm) 2-lane PCC Reconst. (D=30/28cm)	2,241 2,394 3,892	2,200 2,674 3,696	7,751 11,490 14,955	3.5 4.3 4.0	66.3 114.3 79.1	0.36 0.28 0.34	30.7 26.5 30.1
S-CS-7	D/C	880 (5.5)	510 (4.5)	-	510 (4.5)	750 (3.5)	2,650	2-lane PCC Reconst. (D=30/28cm)	3,892	3,590	9,222	2.6	44.2	0.37	29.2
S-CS-8 S-CS-9 S-CS-10 S-CS-11	D/C	670 (5.5)	460 (4.5)	-	300 (4.5)	710 (3.5)	2,140	1-lane PCC Reconst. (D=28cm) 2-lane PCC Reconst. (D=28/28cm) 2-lane AC Overlay (h=10cm) 1-lane PCC Reconst. (D=30cm)	2,147 3,798 2,394 2,241	2,154 3,613 2,674 2,104	3,603 6,992 5,322 3,748	1.7 1.9 2.0 1.8	28.9 33.6 37.2 29.1	0.38 0.36 0.29 0.41	29.6 28.9 25.2 31.1

CHAPTER 17

ECONOMIC PAVEMENT TYPES

17.1 NEW CONSTRUCTION

The economic pavement types may differ by traffic loadings to be imposed, bearing capacity of roadbed soil on which the pavement are laid, structural conditions and environmental conditions.

The comparative analysis on PCC and AC pavement for new constructions was made to arrive at the most economical pavement types in accordance with traffic loading classes and CBR values.

17.1.1 Structural Capacity for Comparison

In analyzing the most economic pavement types, it is essential that both pavements, PCC and AC, should have the same structural capacity. The structural capacity of the initial pavement structure is required depending on traffic loading, CBR value and the performance period of initial pavement structure. From the output of the basic structure design discussed in Chapter 16, the required structural capacity (the thickness of concrete slab for rigid pavement and the structural design for flexible pavement) is determined under the given conditions of traffic loading class, CBR value and design performance period of the initial structure. See Figure 16.5-3. As an example, Figure 17.1.1 shows the thickness of PCC and structural numbers of AC pavement required for CBR value of 8 under all classes of traffic loading.

17.1.2 Comparison on Initial Construction Costs

(1) Initial Construction Cost

Initial construction costs of PCC and AC pavements are estimated as shown in Table 17.1-1 and Table 17.1-2.

TABLE 17.1-1 UNIT COST OF MAIN MATERIALS

M a t e r i a l s	Unit	Unit Cost (P)
• Portland Cement	bag	48.50
• Concrete Class A delivered	m³	795.75
• Portland Cement Concrete Pavement	m³	1,196.70
• Asphalt Concrete	ton	751.50
• Bituminous Concrete Surface Course	M.T.	1,061.95
• Aggregate Subbase	m³	208.55
• Crushed Aggregate Base Course	m³	276.65
• Mechanically Stabilized Base Course	m³	334.36

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TABLE 17.1-2 INITIAL CONSTRUCTION COST

PCC Pavement			AC Pavement		
Thickness	Including Shoulder (₱)	Excluding Shoulder (₱)	Structural Number	Including Shoulder (₱)	Excluding Shoulder (₱)
–	–	–	1.3	1,382	1,188
–	–	–	1.7	1,784	1,465
13 cm	1,747	1,444	2.1	2,250	1,909
15 cm	1,934	1,614	2.5	2,550	2,125
18 cm	2,215	1,871	3.0	3,096	2,610
20 cm	2,402	2,042	3.0	3,096	2,610
23 cm	2,683	2,298	3.5	3,558	2,947
25 cm	2,870	2,552	3.5	3,588	2,947
28 cm	3,152	2,724	4.0	3,960	3,265
30 cm	3,339	3,895	4.0	3,960	3,265
33 cm	3,620	3,151	4.5	4,446	3,647
35 cm	3,807	3,321	4.5	4,446	3,467
–	–	–	5.0	5,559	4,739
–	–	–	5.5	6,065	5,120

(2) Economic Type

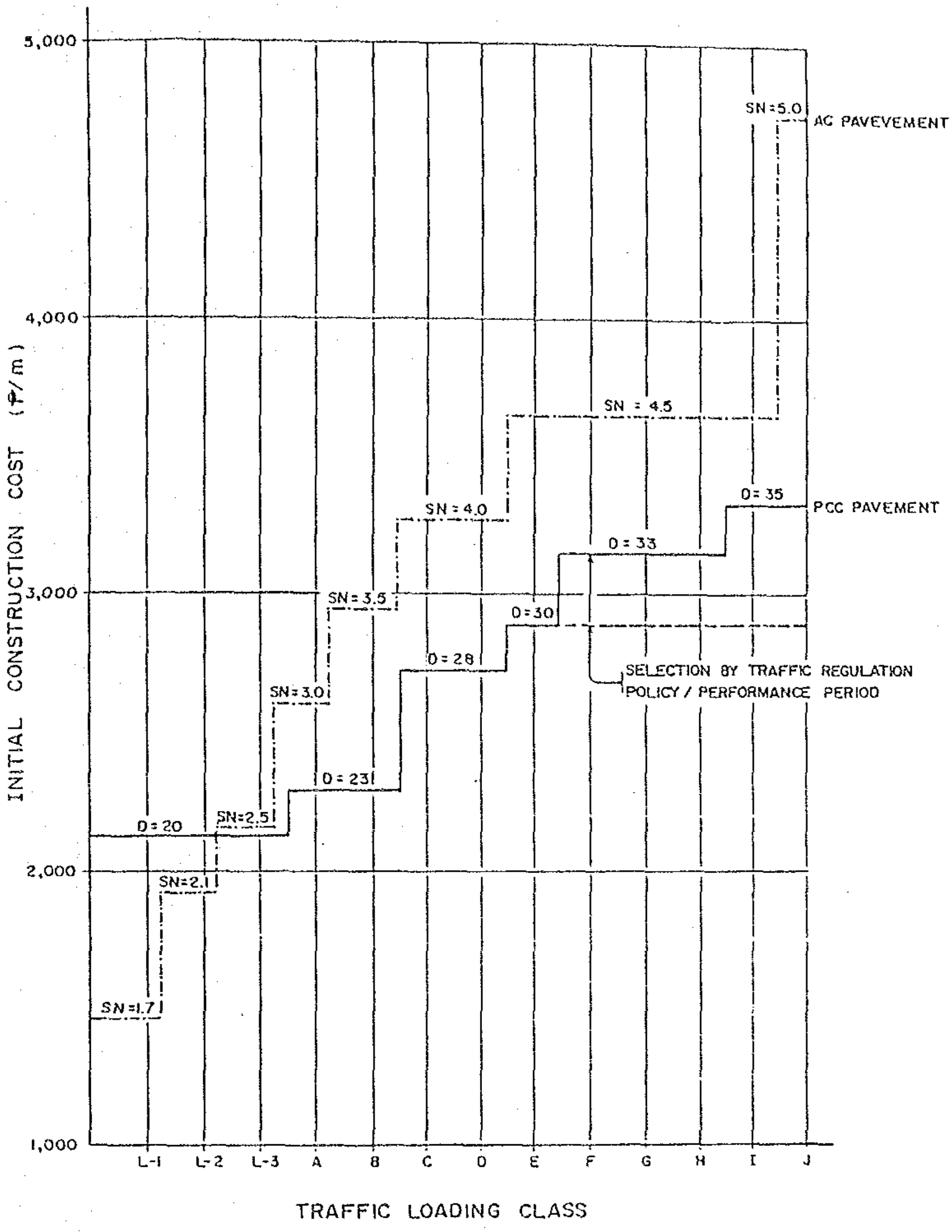
The economic pavement type should not be determined based on the initial construction cost alone. It should be also evaluated based on the total discounted cost which include the initial construction cost, second and third rehabilitation costs, maintenance cost among others. However, the initial constructions were only discussed for new construction. The discussion on total discounted costs for rehabilitation works mentioned in the following section can be referred even for new construction.

As discussed in Chapter 16, the benefit evaluation may be excluded in economic type evaluation since traffic cost may not be so different for any type of pavement providing that maintenance works will be performed to provide the reasonable road surface.

Figure 17.1-1 shows the thickness of PCC pavements and the structural number of AC required for CBR value of 8 under each traffic loading class and the corresponding initial construction cost of both pavement types.

This figure simply demonstrates that the concrete pavement is economical comparing with the Asphalt pavement under all traffic loading classes. It is true for all values of CBR except less than 3 and higher than 20 which are very special subgrades (3) Minimum Thickness of concrete slab.

It is noted that there is a limitation on thickness of concrete slab from the engineering point of view.



NOTE : D = Thickness of Slab (cm.)
SN = Structural Number
Cost in ₱ per meter with 6.7m in road width excluding shoulder.

FIGURE 17.1-1 INITIAL CONSTRUCTION/(NEW CONSTRUCTION) COST OF PCC AND AC PAVEMENT (CBR 8)

AASHTO Guide 1986 recommends the minimum PCC slab thickness to be 5 inches (13 cm) to 7 inches (18 cm) for low volume roads depending on the quality of roadbed soil. Mean PCC modulus of rupture is 600 psi (42 kg/cm^2).

TRRL requires 15 cm for unreinforced concrete with 28 MN/m^2 (285.42 kg/cm^2) crushing strength.

JRA regulates the minimum thickness of 15 cm for concrete with 45 kg/cm^2 and 20 cm for 40 kg/cm^2.

Considering the flexural strength of concrete of 580 psi (40 kg/cm^2) at 28 days (525 psi at 14 days, 40 kg/cm^2) required in BPH Memo Circular, the study adopted 20 cm as the minimum slab thickness in this country.

(4) Conventional Discussion on Economic Pavement Type in the Philippines

It is widely said in this country that the concrete pavement is permanent structure but costly type, and the asphalt concrete pavement is temporary but cheap.

This conventional opinion on pavement type may be explained by the following observation.

Standard Pavement Structures in the Philippines

As shown in Figure 12.2-1, standard types of pavement in the Philippines are established. Table 17.1-3 shows the structural components of standard pavement for major highways and their costs. AC pavement with the structural number of 3.5 which is analyzed to have almost same structural capacity with PCC slab of 23 cm thickness is also shown for the comparison.

When comparing the costs of standard sections of PCC and AC pavements AC is, of course, cheaper type than PCC. However, in comparing with both pavements which should have the same capacity, PCC might be more economical than AC types.

Performance Period of Standard Pavement Structures in the Philippines

The performance period of the standard pavement structures in the Philippines were analyzed for the average traffic loading for major highways, as shown in Table 17.1-4.

Traffic loading class C means the number of ESAL's application less than 0.4×10^6 during the performance period and D less than 0.7×10^6. For these classes of traffic loading, the standard AC pavement may have considerable short performance period of less than 2.2 years and PCC may have a little short of less than 8.6 years. It maybe, therefore, noted that the standard AC pavement proposed for major highways may not be applicable for heavy traffic loading class road.

TABLE 17.1-3 STANDARD PAVEMENT STRUCTURES IN THE PHILIPPINE AND COSTS

Unit: ₱ per M of 2 lanes excluding shoulder

Standard PCC Pavement	Thickness	Cost
PCC Slab	23	1844.1
Aggregate Base	15	278.1
Aggregate Subbase	0 – 20	0-279.4
TOTAL	38 – 58	2122.2-2401.6

Standard AC Pavement		
Bituminous Asphalt	5 – 7.6	840.8-1270.1
Aggregate Base	20	370.7
Aggregate Subbase	20 – 30	279.5-419.2
TOTAL	45 1/ – 57.6 2/ cm	1491.0 – 2060.0

AC Pavement with SN = 3.5 (comparable with PCC 23 cm)		
Bituminous Asphalt	10	1681.6
Aggregate Base	30	556.0
Aggregate Subbase	30	419.2
TOTAL	70 cm.	2656.8

NOTE: 1/ Total Thickness 45 cm. = Structural Number 2.1
2/ Total Thickness 57.6 cm. = Structural Number 2.8

TABLE 17.1-4 PERFORMANCE PERIODS OF STANDARD PAVEMENT STRUCTURES IN THE PHILIPPINES

CBR value of 8

		Traffic Loading Class C	Traffic Loading Class D
PCC	Thickness 23 cm	8.6 years	5.3 years
AC	SN = 2.1 1/	less than 1 year	less than 1 year
AC	SN = 2.8 1/	2.2 years	1.3 years
AC	SN = 3.5 2/	7.3 years	4.4 years

NOTE: 1/ Structural Number of Standard AC Pavement in the Philippines
2/ Structural Number of AC Pavement comparable with PCC Thickness 23 cm slab.

17.2 REHABILITATION FOR BOTH LANES

As pavement rehabilitation methods, the following five (5) were compared to analyze tha most economical types according to traffic loading classes and CBR values. Design procedure are discussed in Volume VI of the Study.

- PCC Reconstruction
- AC Reconstruction
- Flexible (AC) Overlay — Rigid Existing
- Rigid (PCC) Overlay — Rigid Existing
- Flexible (AC) Overlay — Flexible Existing

The analysis was made using the outputs of the basic structure design which includes initial construction costs and total discounted costs, among others. Refer to Appendix 16-3.

17.2.1 Economic Pavement Types by Traffic Loading Classes

As representative example of computer analysis, Figures 17.2-1 and 17.2-2 demonstrate the initial construction costs of the initial pavement structures and the total discounted costs including maintenance costs, respectively. These figures show all cases of traffic loading, but for CBR values of 4, 6 and 8.

These two figures summarize that when both lanes are planned to be reconstructed/ overlaid, AC Overlay (rigid existing) is the most economical type, followed by PCC Reconstruction and AC Reconstruction. PCC Overlay (rigid existing) is only shown in figure for the purpose of reference.

The representative comparisons are shown in Table 17.2-1 for the cases of Traffic Loading Classes C and G under CBR value of 6.

TABLE 17.2-1 COST COMPARISON BETWEEN PAVEMENT TYPES

Initial Construction Cost (₱/ both lanes, m) CBR = 6

Traffic Loading Class	C		H	
AC Overlay	2,394	(1.00)	3,232	(1.00)
PCC Reconstruction	3,798	(1.59)	4,454	(1.38)
AC Reconstruction	5,112	(2.14)	6,204	(1.92)

Total Discounted Cost (₱/ both lanes, m) CBR = 6

AC Overlay	2,907	(1.00)	4,183	(1.00)
PCC Reconstruction	4,094	(1.41)	4,925	(1.10)
AC Reconstruction	5,468	(1.88)	7,094	(1.70)

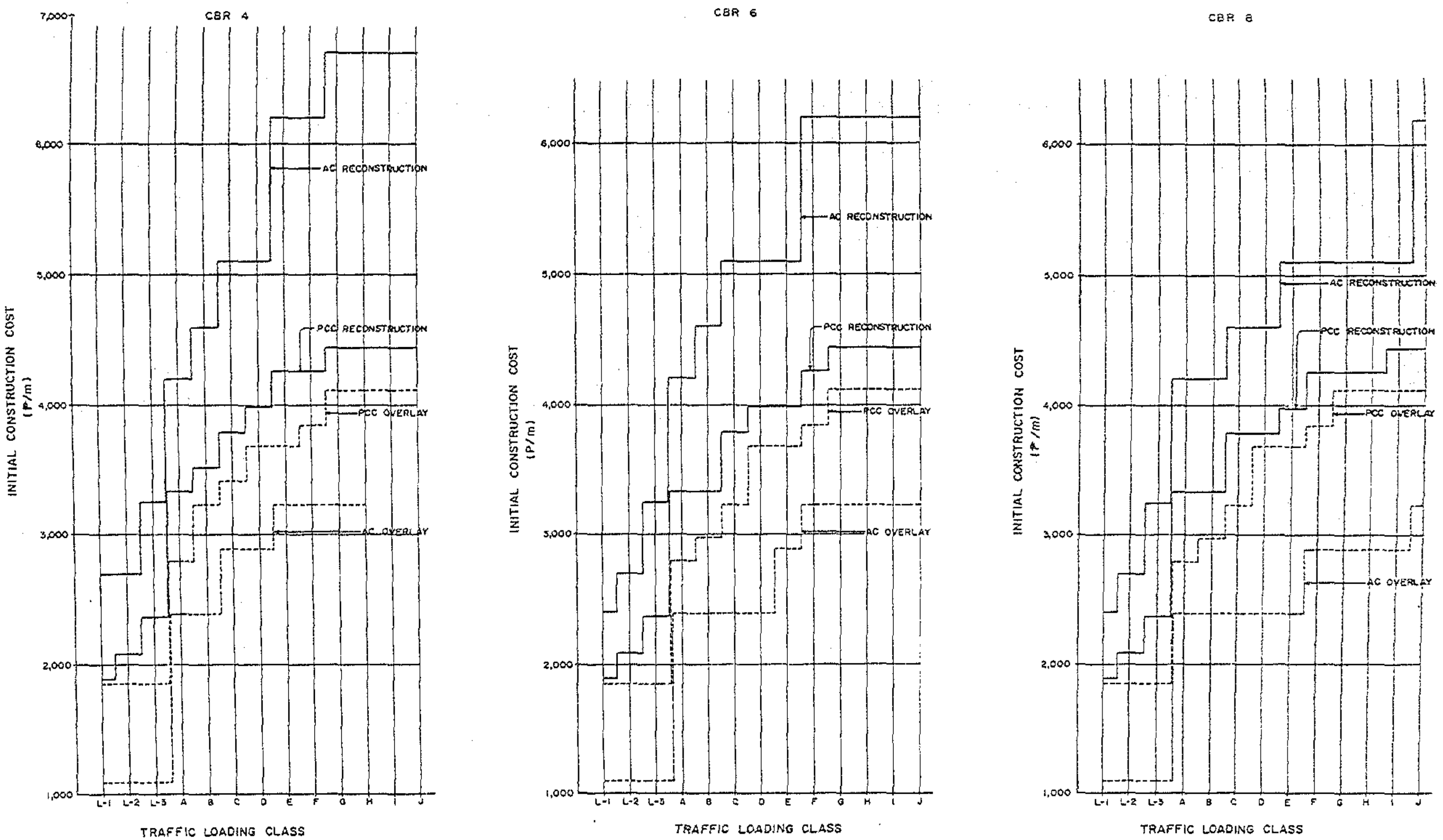
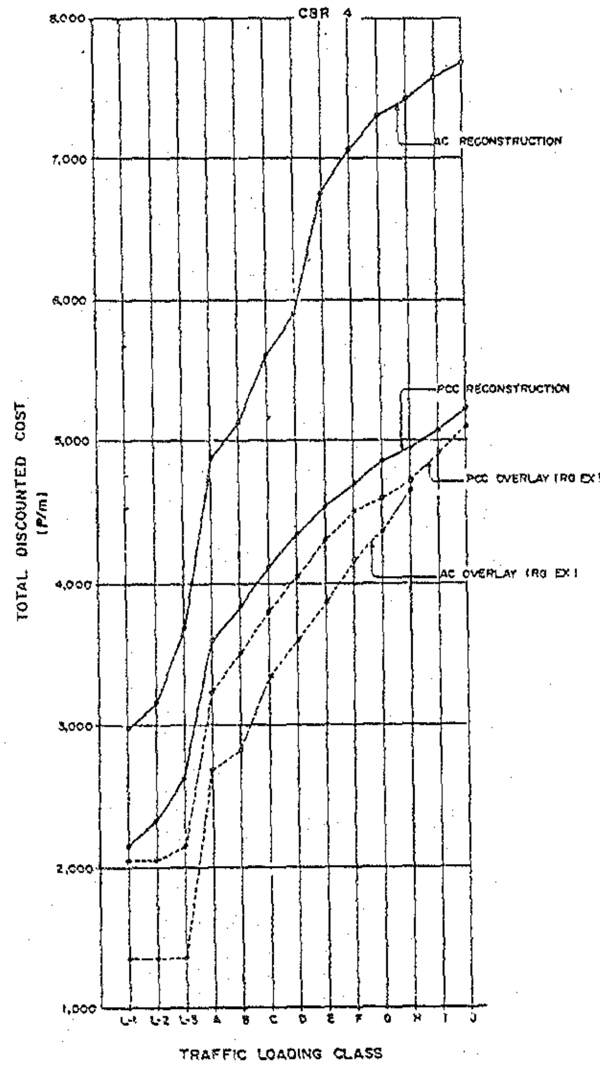
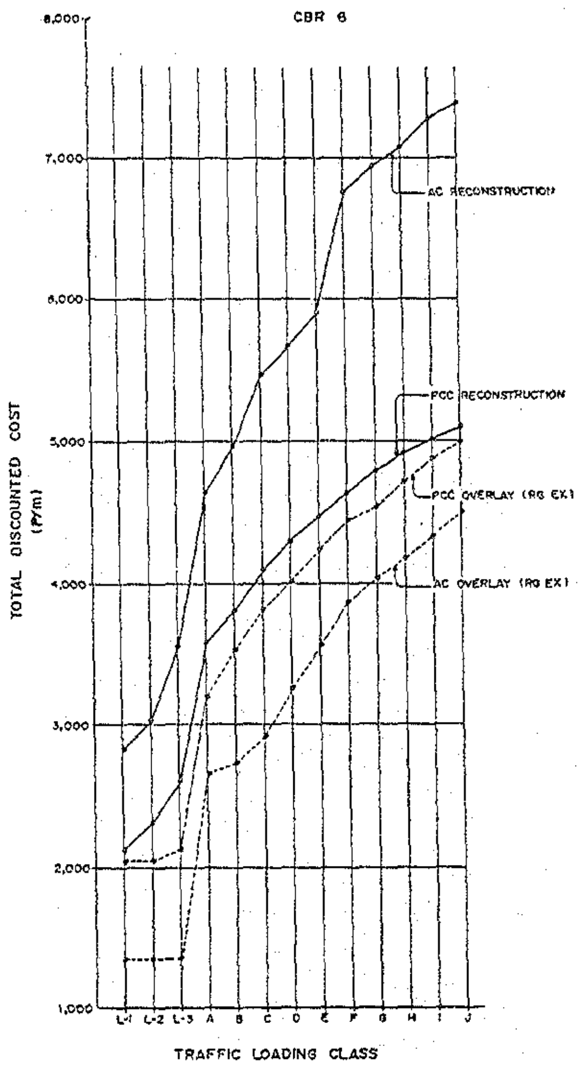


FIGURE 17.2-1 INITIAL CONSTRUCTION COST VS. TRAFFIC LOADING CLASS

Note: 1) Construction Cost for Reconstruction/Overlay of both lanes

2) Cost in ₱ per meter with 6.7 m in road width





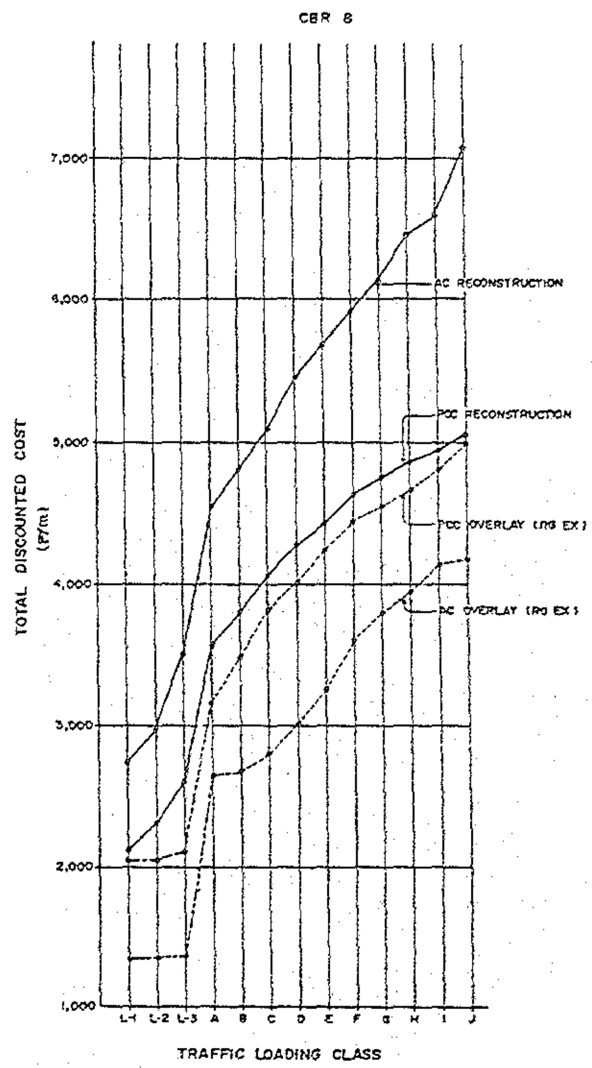


FIGURE 17.2-2 TOTAL DISCOUNTED COST VS. TRAFFIC LOADING CLASS
Note: 1) Discounted Cost for Reconstruction/Overlay of both lanes
2) Cost in ₱ per meter with 6.7 m. in Road width

17.2.2 Economic Pavement Types By CBR Value

The initial construction costs and total discounted cost for each CBR value under the traffic loading classes of B, E and I are shown in Figure 17.2-3 and 17.2-4, respectively.

Regardless of CBR values, AC overlay is still the most economical type, followed by PCC Reconstruction and AC Reconstruction. Only exception is the initial construction cost for the case that CBR values are more than 15 and traffic loading classes are heavier than E.

It is concluded that when the both lanes are planned to be rehabilitated, AC overlay is recommended as the most economical method, followed by PCC Reconstruction and AC Reconstruction in order. Only for the case, CBR values are more than 20, AC reconstruction may be slightly economical comparing with PCC reconstruction.

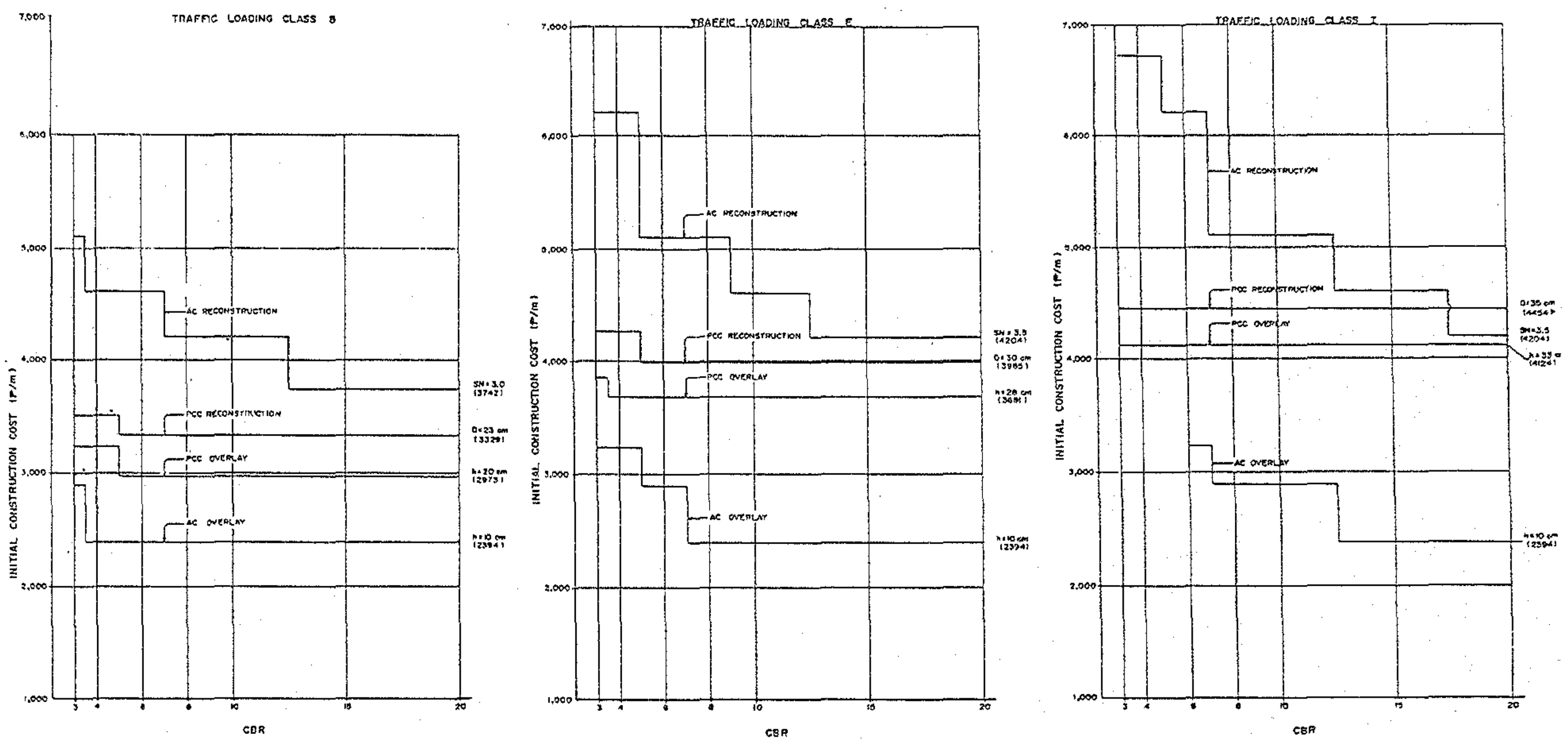


FIGURE 17.2-3 INITIAL CONSTRUCTION COST VS. CBR

Note: 1) Construction Cost for Reconstruction/Overlay of both lanes

2) Cost in ₱ per meter with 6.7 m. in width

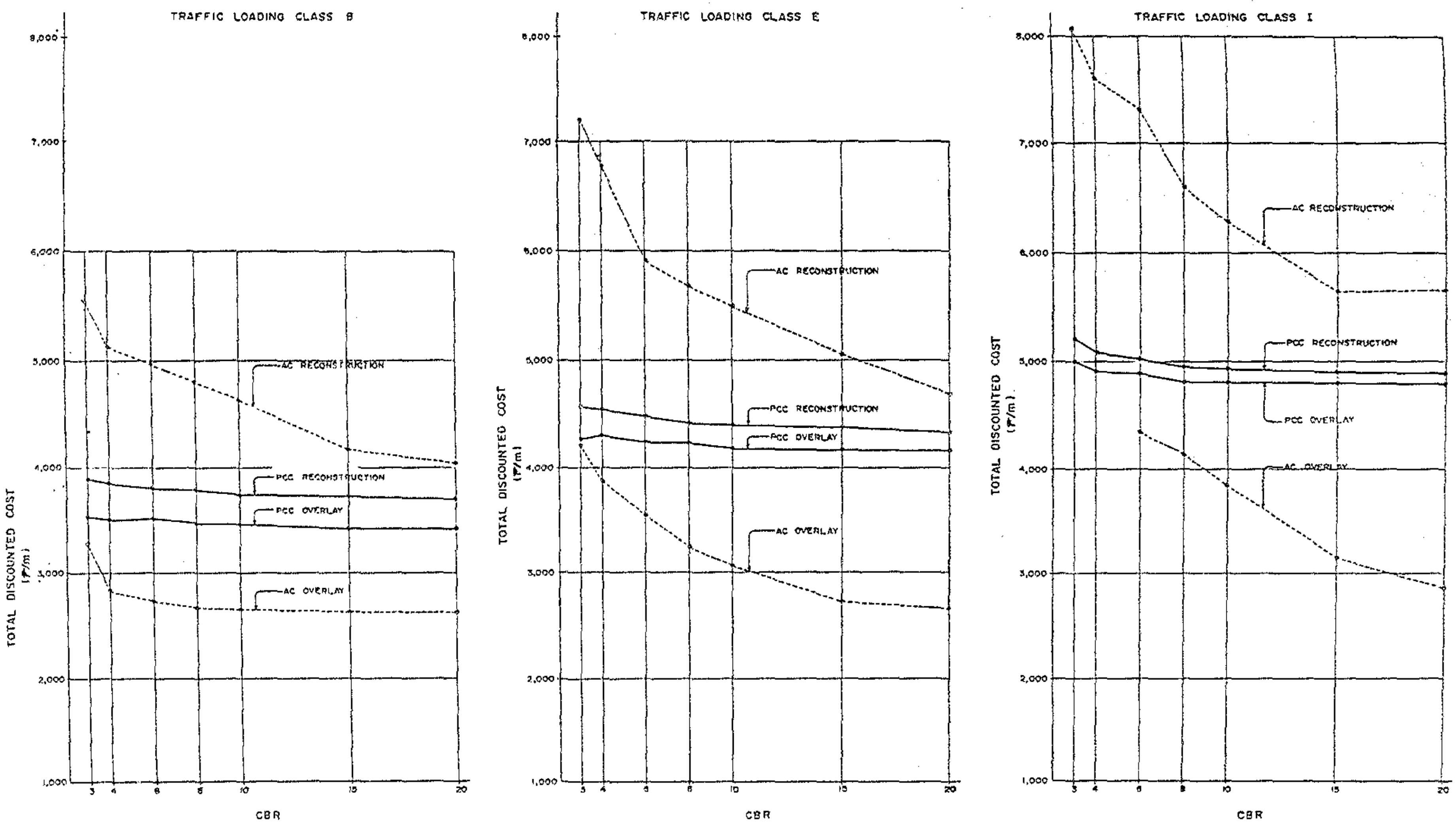


FIGURE 17.2-4 TOTAL DISCOUNTED COST VS. CBR

Note: 1) Discounted Cost for Reconstruction/Overlay of both lanes

2) Cost in ₱ per meter with 6.7 m. in Road width

17.3 PCC RECONSTRUCTION (ONE-LANE) AND AC OVERLAY (TWO-LANE)

17.3.1 Assumed Conditions for Comparative Study

It was remarkably observed during the field investigation that the concrete pavement of Manila bound of North Study Sections were seriously deteriorated, but the opposite lane were not too deteriorated. This fact can be proved by the weight of vehicles and axle loads.

The many sections of Manila bound require the urgent rehabilitation at present, while the rehabilitation for the opposite lane may be deferred depending on its remaining life of the existing pavement structures predicted.

As the rehabilitation methods for this case, the following two alternatives were proposed.

Method 1; PCC Reconstruction of One-Lane deteriorated

The opposite lane which does not require the rehabilitation at present will be rehabilitated by PCC reconstruction after XP years when the pavement condition will reach to RRI less than 2.5.

Method 2; AC Overlay of Both Lanes

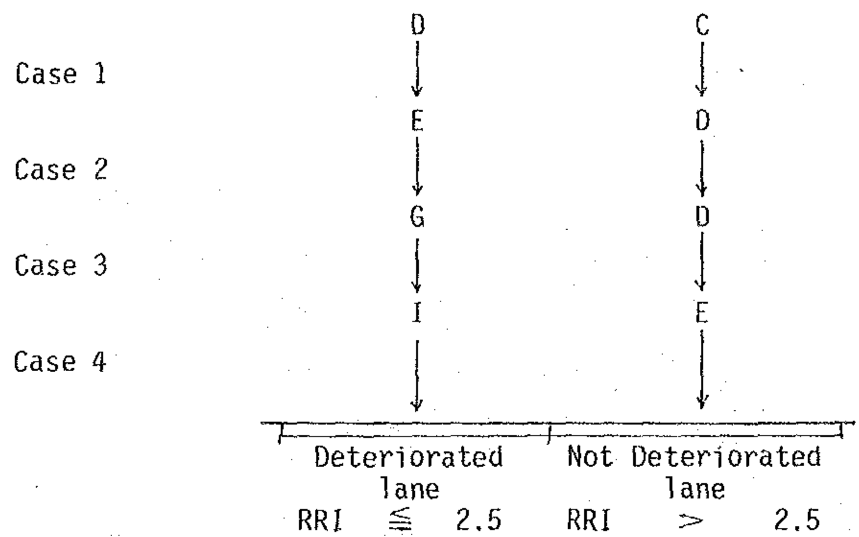
The minimum thickness of AC overlay is 10 cm so that both lanes may be overlaid at present because of the tapering of overlaid asphalt concrete.

In order to evaluate both methods, a comparative study was done using an electric computer. The output are presented in Appendix 17-1.

The pavement conditions assumed for the comparative study are the following three cases as shown in Figure 17.3-1.

FIGURE 17.3-1 ASSUMED CONDITIONS FOR COMPARATIVE STUDY

(TRAFFIC LOADING CLASSES)



17.3.2 Comparison on Initial Costs

Since the initial cost of Method 1 is almost half of two lanes PCC reconstruction, PCC reconstruction of one lane is economical, about 80% of two lanes AC overlay for traffic loading classes less than E, but almost same for more than F. See Figure 17.3-2.

17.3.3 Comparison on Total Discounted Costs

The total discounted costs for two methods were calculated including the initial cost, the second and third reconstruction/overlays and the maintenance cost for the analysis period of 25 years. The remaining life of the not-deteriorated lane is expressed with XP which is the year that the lane will be deteriorated and rehabilitation will be undertaken.

The output of the analysis are reported in Appendix 17-1 and Figure 17.3-3 presents an example of total discounted costs for CBR values of 6.

According to the output of comparative analysis, the economic pavement type of two method depends on CBR values, traffic loading classes and remaining life of pavement of not-deteriorated lane. The observation are summarized in Table 17.3-1.

TABLE 17.3-1 COMPARATIVE ON PCC RECONSTRUCTION (ONE-LANE) AND AC OVERLAY (TWO-LANES)

CBR Value	Traffic Loading Class	Remaining Life XP	Economic Pavement Type
Less Than 4	All Cases	All Cases	PCC Reconstruction (One-Lane)
6	More Than F	More Than 4 years	PCC Reconstruction (One-Lane)
		Less Than 3 years	AC Overlay (Two-Lanes)
	Less Than E	All Cases	AC Overlay (Two-Lanes
More Than 8	All Cases	All Cases	AC Overlay (Two-Lanes)

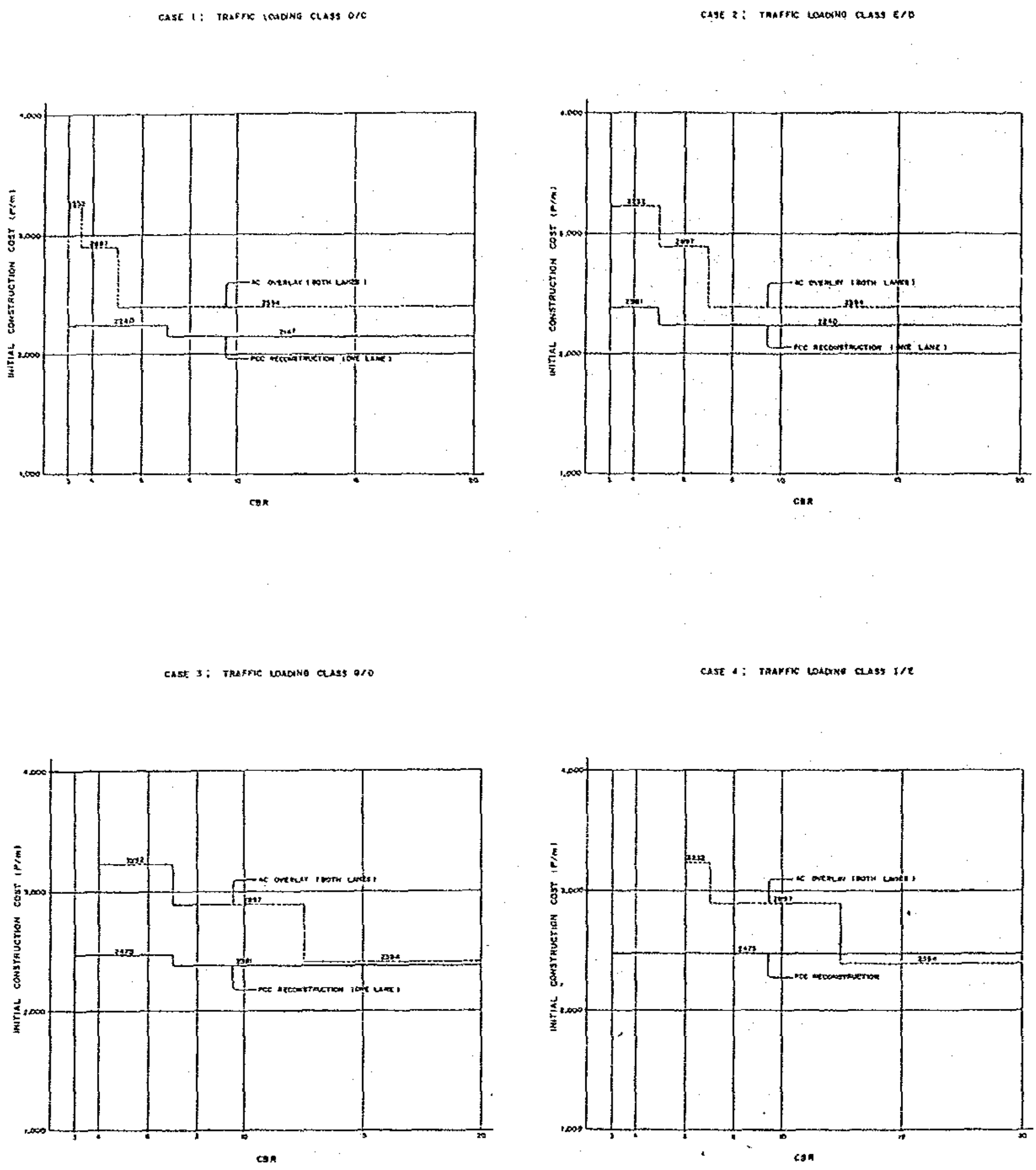


FIGURE 17.3-2 INITIAL CONSTRUCTION COST OF AC OVERLAY AND PCC RECONSTRUCTION
(Both Lanes) (One Lane)
NOTE: cost = ₱/m

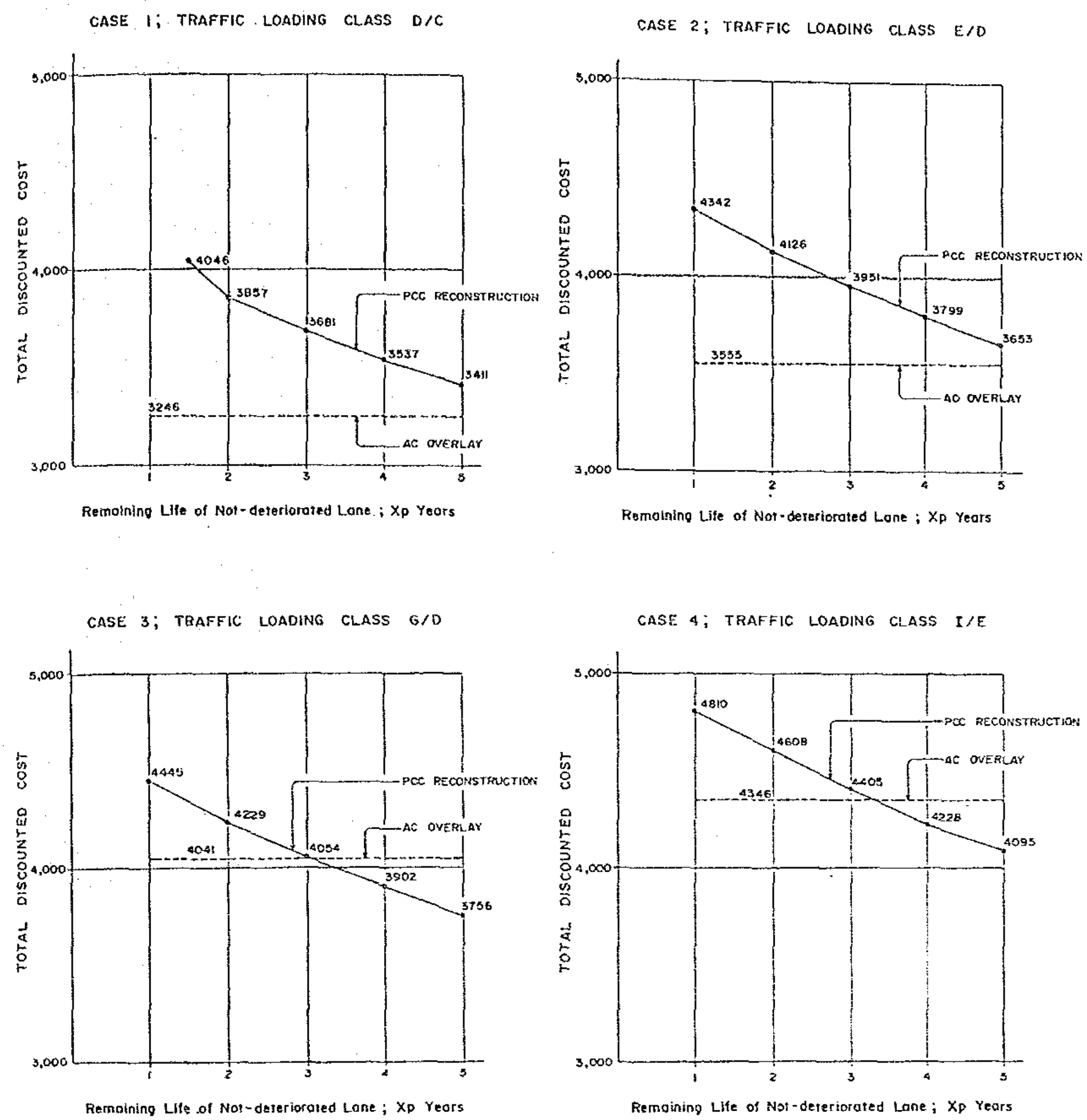


FIGURE 17.3-3 PCC RECONSTRUCTION AND AC OVERLAY
CBR 6

Note: 1) Discounted Cost for Reconstruction/Overlay of Both Lanes
2) Cost in ₱ per meter with 6.7 m in Road width

17.4 PAVEMENT COST AND TRAFFIC REGULATION

Republic Act 4136 promulgated in 1964 states that the weights of good vehicles in the Philippines to 8 tons (18 kip) for most heavily loaded wheel and 14.5 tons for most heavily loaded axle group.

The traffic regulations is, however, not necessarily implemented strictly. Many trucks, both single and tandem axles, loading heavier weights than legal limitations run along the Study Road resulting in damaging the pavements.

In order to evaluate the damage to the pavements due to overloaded vehicles, the comparative study was made to verify the pavement cost savings and the extension of the initial performance period of the initial pavement structure by the implementation of the traffic regulation.

17.4.1 Number of ESAL under Traffic Regulation

Number of vehicles loading heavier than this legal limits were converted into number of vehicles which can carry the maximum legal limit loading, under the simple assumption that the total weight of loads transported by the vehicles are the same.

The results of the conversion analysis on number of vehicles are summarized in Table 17.4-1. Considered were only trucks and buses.

17.4.2 Comparative Analysis With/Without Traffic Regulation

The comparative analysis was carried out by an electric computer assuming the following two cases.

Case 1; The pavement are assumed to be designed and constructed in accordance with the present traffic regulation.

In this case, the initial construction costs and the total discounted costs including maintenance costs will be reduced.

Case 2; The pavement are assumed to be designed and constructed based on the present axle loading patterns (without traffic regulation). But, just after the construction, the traffic regulation will be enforced.

In this case, the initial construction costs are the same as the case without the regulation. But, the initial performance period of the initial pavement structure will be prolonged, resulting in the saving of the total discounted cost.

The analysis output for the whole Study Section are presented in Table 17.4-2, while Figure 17.4-1 presents the analysis results for Segment N-2 and S-2, as examples. The comparative analysis are summarized in Table 17.4-3.

TABLE 17.4-1 NUMBER OF TRUCKS AND BUSES WITH TRAFFIC REGULATIONS

Section	Direction	Vehicle Type	Present Condition: AADT in 1,989 (Both Dir.)	Present Condition: Relative Damaging Effect	Present Condition: No. of ESALs Per Lane in 1,989 ($x10^6$)		With Regulation: AADT in 1,989 (Both Dir.)	With Regulation: Relative Damaging Effect	With Regulation: No. of ESALs Per Lane in 1,989	
Sta. Rita - Plaridel	Manila Bound	Bus	947	1.3	0.225	3.496	1,042	0.9	0.171	1.079
		Truck	2,489	7.2	3.271		3,982	1.25	0.908	
	Cagayan Bound	Bus		1.3	0.225	1.451		0.9	0.171	0.534
		Truck		2.7	1.226			0.5	0.363	
Plaridel - Gapan	Manila Bound	Bus	947	1.3	0.225	2.908	1,042	0.9	0.171	0.818
		Truck	1,771	8.3	2.683		2,834	1.25	0.647	
	Cagayan Bound	Bus		1.3	0.225	1.098		0.9	0.171	0.430
		Truck		2.7	0.873			0.5	0.259	
Gapan - Cabanatuan	Manila Bound	Bus	696	1.3	0.165	2.946	766	0.9	0.126	0.718
		Truck	1,621	9.4	2.781		2,594	1.25	0.592	
	Cagayan Bound	Bus		1.3	0.165	0.964		0.9	0.126	0.363
		Truck		2.7	0.799			0.5	0.237	
Cabanatuan - San Jose	Manila Bound	Bus	571	1.3	0.135	2.458	628	0.9	0.103	0.568
		Truck	1,273	10.0	2.323		2,037	1.25	0.465	
	Cagayan Bound	Bus		1.3	0.135			0.9	0.103	0.289
		Truck		2.7	0.627			0.5	0.186	
San Jose - Aritao	Manila Bound	Bus	243	1.3	0.058	1.787	267	0.9	0.044	0.386
		Truck	938	10.1	1.729		1,501	1.25	0.342	
	Cagayan Bound	Bus		1.3	0.058	0.520		0.9	0.044	0.181
		Truck		2.7	0.462			0.5	0.137	
Calamba- Sto. Tomas	Manila Bound	Bus	1,343	1.3	0.319	1.747	1,477	0.9	0.243	0.732
		Truck	2,059	3.8	1.428		2,677	1.0	0.489	
	Bicol Bound	Bus		1.3	0.319	1.258		0.9	0.243	0.585
		Truck		2.5	0.939			0.7	0.342	
Sto. Tomas - Tiaong	Manila Bound	Bus	579	1.3	0.137	1.090	637	0.9	0.105	0.415
		Truck	1,305	4.0	0.953		1,697	1.0	0.310	
	Bicol Bound	Bus		1.3	0.137	0.637		0.9	0.105	0.322
		Truck		2.1	0.500			0.7	0.217	
Tiaong - Lucena	Manila Bound	Bus	614	1.3	0.146	0.967	675	0.9	0.111	0.378
		Truck	1,125	4.0	0.821		1,463	1.0	0.267	
	Bicol Bound	Bus		1.3	0.146	0.577		0.9	0.111	0.298
		Truck		2.1	0.431			0.7	0.187	
Lucena - Gumaca	Manila Bound	Bus	514	1.3	0.122	0.796	565	0.9	0.093	0.272
		Truck	754	4.9	0.674		980	1.0	0.179	
	Bicol Bound	Bus		1.3	0.122	0.480		0.9	0.093	0.218
		Truck		2.6	0.358			0.7	0.125	
Gumaca - Calauag	Manila Bound	Bus	297	1.3	0.070	0.705	327	0.9	0.054	0.222
		Truck	710	4.9	0.635		923	1.0	0.168	
	Bicol Bound	Bus		1.3	0.070	0.407		0.9	0.054	0.172
		Truck		2.6	0.337			0.7	0.118	

TABLE 17.4-2 COMPARATIVE STUDY OF PAVEMENT COST WITH OR WITHOUT TRAFFIC REGULATION

CBR = 6

	Segment		Without Regulation								With Regulation											
			Traffic Loading				Cost				Traffic Loading				Cost (Case 1)*				Cost (Case 2)**			
			AADT in 1989 (Bus Truck)	Relative Damaging Effect (Bus Truck)	ESAL in 1989 Total) (x10^6)	Traffic Loading Class	Slab Thick-ness (Cm)	Initial Perfor-mance Period (Year)	Initial Const. Cost (P/M)	Total Dis'd. Cost (P/M)	AADT in 1989 (Bus Truck)	Relative Damaging Effect (Bus Truck)	ESAL in 1989 (Total) (x10^6)	Traffic Loading Class	Slab Thick-ness (Cm)	Initial Perfor-mance Period (Year)	Initial Const. Cost (P/M)	Total Dis'd. Cost (P/M)	Slab Thick-ness (Cm)	Initial Perfor-mance Period (Year)	Initial Const. Cost (P/M)	Total Dis'd. Cost (P/M)
North Study Section	Segment N - 1	Sta. Rita -Plaridel	950 2,490	1.3/1.3 7.2/2.7	3.50/ 1.45	J/F	35/33	12.3	4,360	5,015	1,040 3,980	0.9/0.9 1.25/ 0.5	1.08/ 0.53	E/D	30/30	14.9	3,985	4,483	35/33	28.1	4,360	4,619
	(l=46km)	Plaridel-Gapan	950 1,770	1.3/1.3 8.3/2.7	2.91/ 1.10	I/E	35/30	13.9	4,220	4,796	1,040 2,830	0.9/0.9 1.25/ 0.5	0.82/ 0.43	E/C	30/28	14.9	3,892	4,390	35/30	27.2	4,220	4,432
	Segment N-2 (l = 35 km)		700 1,620	1.3/1.3 9.4/2.7	2.95/ 0.96	I/E	35/30	13.9	4,220	4,796	770 2,590	0.9/0.9 1.25/ 0.5	0.72/ 0.36	D/C	30/28	19.2	3,892	4,219	35/30	27.2	4,220	4,432
	Segment N-3 (l = 42 km)		570 1,270	1.3/1.3 10.0/2.7	2.46/ 0.76	H/D	35/30	15.8	4,220	4,691	630 2,040	0.9/0.9 1.25/ 0.5	0.57/ 0.29	D/C	30/28	19.2	3,892	4,219	35/30	27.2	4,220	4,432
	Segment N-4 (l = 38 km)		240 940	1.3/1.3 10.1/2.7	1.79/ 0.52	G/D	35/30	18.5	4,220	4,581	270 1,500	0.9/0.9 1.25/ 0.5	0.39/ 0.18	C/B	28/23	14.4	3,564	4,044	35/30	39.3	4,220	4,392
	Segment N-5 (l = 39 km)		240 940	1.3/1.3 10.1/2.7	1.79/ 0.52	G/D	35/30	18.5	4,220	4,581	270 1,500	0.9/0.9 1.0/0.7	0.39/ 0.18	C/B	28/23	14.4	3,564	4,044	35/30	39.3	4,220	4,392
South Study Section	Segment S - 1	Calamba-Sto.Tomas	1,340 2,060	1.3/1.3 3.8/2.5	1.75/ 1.26	G/F	35/33	17.3	4,360	4,750	1,480 2,680	0.9/0.9 1.0/0.7	0.73/ 0.59	D/D	30/30	19.2	3,985	4,312	35/33	28.1	4,360	4,566
	(l=42 km)	Sto.Tomas -Tiaong	580 1,300	1.3/1.3 4.0/2.1	1.09/ 0.64	E/D	30/30	14.9	3,985	4,483	640 1,700	0.9/0.9 1.0/0.7	0.42/ 0.32	C/C	28/28	20.8	3,798	4,094	30/30	27.2	3,985	4,197
	Segment S-2 (l = 54 km)		610 1,120	1.3/1.3 4.0/2.1	0.97/ 0.58	E/D	30/30	14.9	3,985	4,483	660 1,460	0.9/0.9 1.0/0.7	0.38/ 0.30	C/C	28/28	20.8	3,798	4,094	30/30	27.2	3,985	4,197
	Segment S-3 (l = 46 km)		510 750	1.3/1.3 4.9/2.6	0.79/ 0.48	D/C	30/28	19.2	3,892	4,219	570 980	0.9/0.9 1.0/0.7	0.27/ 0.22	C/B	28/23	14.4	3,564	4,044	30/28	27.2	3,892	4,104
	Segment S-4 (l = 39 km)		300 710	1.3/1.3 4.9/2.5	0.71/ 0.41	D/C	30/28	19.2	3,892	4,219	330 920	0.9/0.9 1.0/0.7	0.22/ 0.17	B/B	23/23	14.4	3,329	3,809	30/28	31.4	3,892	4,088

NOTE: *Case 1: Economic design according to expected traffic loading
**Case 2: Same design as "without" case

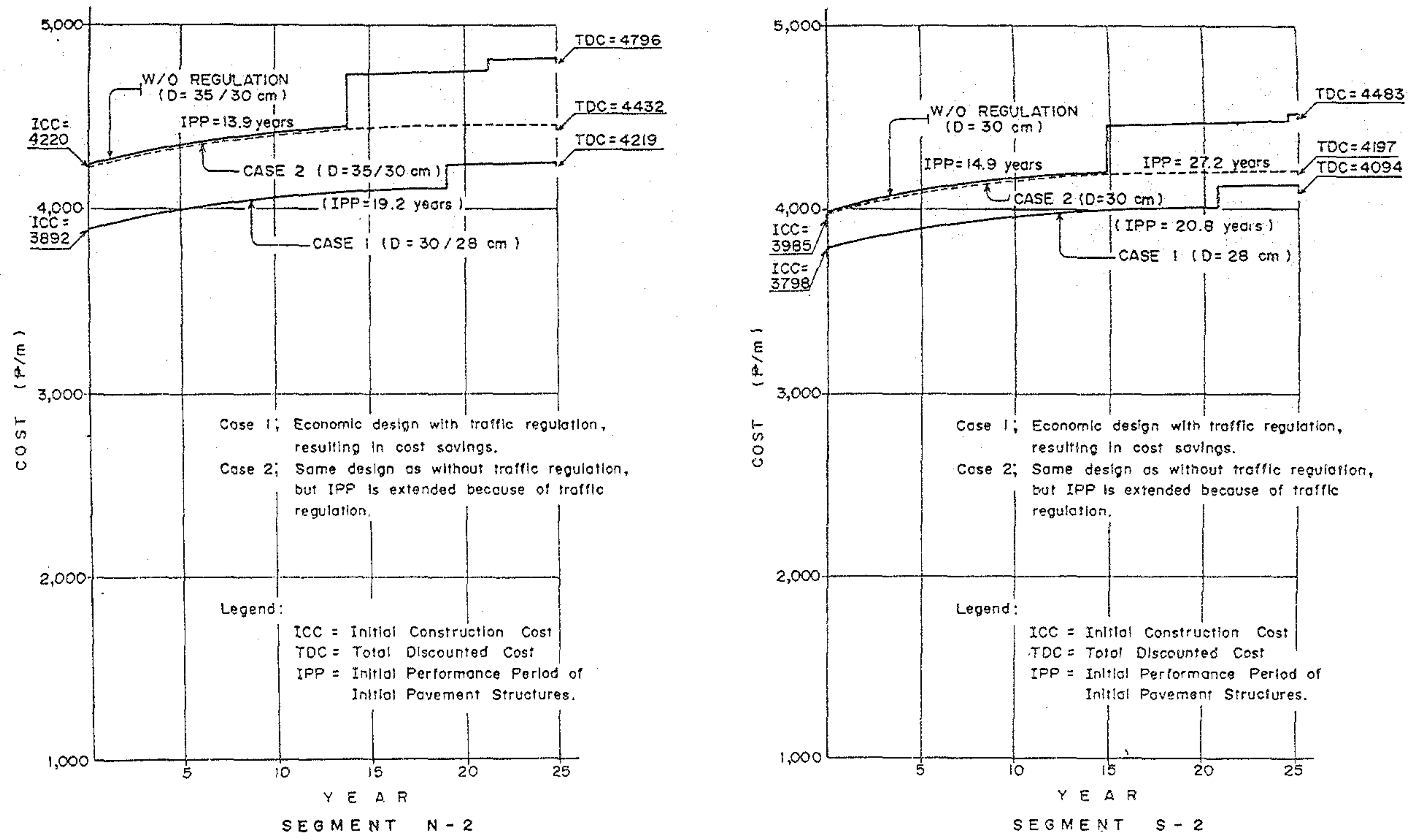


FIGURE 17.4-1 PAVEMENT COST AND TRAFFIC REGULATION

TABLE 17.4-3 PAVEMENT COST AND TRAFFIC REGULATIONS

	Without Present Traffic Loading	Case 1 Design in accordance with Traffic Regulation, complete Implementation of Regulation	Case 2 Design in accordance with Present Traffic Loading, but after construction, regulation
A A D T (Bus and Truck)	N_1-1; 3,440 N_2 ; 2,320 S_1-1; 3,400 S_1-2; 1,880	5,020 3,360 4,160 2,340	Same as without case
Traffic Loading Class	N_1-1; J, 3.5×10^6 N_2 ; I, 2.9×10^6 S_1-1; G, 1.7×10^6 S_1-2; E, 1.1×10^6	E, 1.1×10^6 D, 0.7×10^6 D, 0.7×10^6 C, 0.4×10^6	Same as without case
Slab Thickness Required	N_1-1; 35 cm N_2 ; 35 cm S_1-1; 35 cm S_1-2; 30 cm	30 cm 30 cm 30 cm 28 cm	35 cm 35 cm 35 cm 30 cm
Performance Period	N_1-1; 12.3 years N_2 ; 13.9 years S_1-1; 17.3 years S_1-2; 14.9 years	14.9 years 19.2 years 19.2 years 20.8 years	28.1 years 27.2 years 28.1 years 27.2 years
Initial Construction Cost	N_1-1; ₱4,360/m N_2 ; 4,220/m S_1-1; 4,360/m S_1-2; 3,985/m	₱3,985/m 3,892/m 3,985/m 3,798/m	Same as without case
Total Initial Construction Cost	North ₱844.4 M (200 Km) South ₱714.6 M (181 Km)	₱753.4 ₱657.2	Same without case

N_1-1; Sta. Rita-Plaridel
N_2 ; Gapan-Cabanatuan
S_1-1; Calamba-Sto. Tomas
S_1-2; Sto. Tomas-Tiaong