ANALYSIS OF ROAD DAMAGE DUE TO OVER LOADING (CASE STUDY: DEMAK -TRENGGULI ARTERIAL ROAD, CENTRAL JAVA PROVINCE, INDONESIA)

Thesis

Submitted as Partial Fulfilling of the Requirement for the Degree of Master of Civil Engineering Diponegoro University

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Semarang

2011

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STATEMENT OF AUTHENTIFICATION

Herewith I stated that this thesis has never been published in other institution and there were no part of this thesis has been directly copied from published sources except citing from listed bibliographies attached.

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DEDICATION

To all those who love me and those that I love.

ACKNOWLEDGEMENTS

First and foremost, I would like to thank Allah S.W.T for giving me strength to endure the challenges i]n my quest to complete this research.

Secondly, I would like to thanks my supervisors, Ir. Bambang Pudjianto, MT. and Dr. Bagus Hario Setiadji, ST., MT. for their advice, patience and guidance throughout the process of completing this research. And thankful the head of master program Dr. Bambang Riyanto, DEA. for his kind help and support, and to all my lecturers who had taught me in DIPONEGORO UNIVERSITY, thank you for all the knowledge and guidance.

Also I would like to thank to my beloved family members. Their support was instrumental for the successful completion of this research project.

To them all, I am truly grateful.

ABSTRACT

The ability of a pavement structure in carrying out its function reduces in line with the increase of traffic load, especially if there are overloaded heavy vehicle passing through the road. In this thesis, the effect of overloaded vehicles on the road pavement service life was analyzed using the AASHTO 1993. Vehicle damage factor (VDF) and Structural Number (SN) were calculated on normal and overloading conditions. Remaining of pavement service life due to overloading condition was also presented. So it can be concluded how severe the effect of overloaded vehicles against pavement service life.

In this thesis it can be seen, that the presence of overloaded vehicles, particularly heavy vehicles (class 6B up to class 7C according to Bina Marga's vehicle classification) resulted in traffic load (W_{18}) value that was 200% greater than that of standard load condition. The increase of W_{18} value can affect the pavement service life. For the direction of Demak Trengguli, the pavement service life reduced by 70% due to overloading condition, while for the opposite direction, the service life was reduced by 40% caused by the same factor. In terms of layer thickness, overloading condition also increase the layer thickness than that of thickness at the load legal limit 10 ton. For the direction of Demak-Trengguli, the thickness reached 186% higher than of standard design, while for the direction of Trengguli-Demak, it's obtained that due to overloading condition, the layer thickness approximately 177% higher than that of standard design.

From the results, it can be concluded that overloaded vehicles on the road are very influential to the reduction in pavement service life. Therefore, it is expected that road users to comply with existing regulations in the conduct of transportation.

Key words: overload vehicle, damage factor, pavement service life, pavement thickness.

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NOMENCLATURE

a	Layer coefficient
ADT	Average daily traffic
AASHTO	American Association of State Highway and Transportation Officials
CBR	California Bearing Ratio
D	Layer thickness
D_D	Directional distribution factor
D_L	Lane distribution factor
EALFs	Equivalency Axle Load Factors
G_R	Annual growth factor
G_t	The log of the ratio of loss of serviceability
IRI	International Roughness Index
LEF	Axle Load Equivalency Factor
L_x	The axle group load
L_2	The axle code
m	Drainage factor
M_R	Resilient modulus
p_o	Initial Serviceability
P_s	Swelling probability
p_t	Terminal Servicability
SN	Sturcture number
So	Standard deviation
V_R	Maximum potential heave
W_{18}	Traffic load
W_{tx}	The number of x axle load applications at time t
W_{t18}	The number of 18 kip axle load applications at time t
Zr	Level of reliability

ΔPSI	Loss of serviceability
θ	Swell rate constant
β_x	The function of design and load variables,

CHAPTER 1
INTRODUCTION

1.1 Background

Pavements are engineering structures placed on natural soils and designed to withstand the traffic loading and the action of the climate with minimal deterioration and in the most economical way (Hudson et al., 2003). The majority of modern pavement structures may be classified as flexible or rigid pavement structures. A flexible pavement consists of a surface layer constructed of flexible materials (typically asphalt concrete) over granular base and sub base layers placed on the existing, natural soil. Rigid pavement is a pavement structure that deflects very little under loading because of the high stiffness of the Portland cement concrete used in the construction of surface layer. The rigid pavements can be further categorized depending on the types of joints constructed and use of steel reinforcement (Gillespie, 1993).

Each of these pavement types has specific failure mechanisms and each failure mechanism is caused by specific factors. Example of such failure mechanisms include: fatigue damage and roughness of rigid and flexible pavements, faulting of rigid pavements, and rutting of flexible pavements. These failure mechanisms are caused by the following factors: heavy vehicle loadings, climate, drainage, materials properties, and inadequate layer thicknesses (Hudson et al., 2003).

Among these factors, heavy vehicle loads are the major source for pavement damage. Magnitude and configuration of vehicular loads together with the environment have a significant effect on induced tensile stresses within flexible pavement (Yu et al., 1998).

Heavy vehicles load on the pavements subjects to high stresses causing damage. However, not all trucks have the same damaging effects; the damage on the road pavement depends on speed, wheel loads, number and location of axles, load distributions, type of suspension, number of wheels, tire types, inflation pressure and other factors (Gillespie et al., 1993).

The proper estimation of truck-induced damage is important for regulators since the fees and penalties applied to truck operators for using the roads are related to the distresses induced to the road pavement. Regulators need to allocate costs to vehicle operators in accordance with truck-induced damage to pavements. The proper evaluation of truck damage also helps the highway engineers in the optimization of pavement design and maintenance activities (Zaghloul and White, 1994).

In recent years, several studies have estimated the truck damage by computing the responses (stresses, strains and deflections) of pavements under heavy vehicles loadings using mechanistic approaches (Chen et al., 2002). In response to the need for mechanistic pavement design and analysis procedures, researchers are increasingly using three dimensional finite element analysis techniques to quantify the response of the pavement system to applied axle and temperature loading (Davids, 2000).

Another tool that would allow estimating the truck damages is the new Mechanistic Empirical Design Guide for pavements. Due to its advanced modeling capabilities, it is expected that federal and state transportation agencies will phase out the old empirical AASHTO Pavement Design Guide (1986, 1993) to let the new Mechanistic Empirical Design Guide handle nowadays pavement design challenges such as increased number and weight of heavy vehicles (FHWA, 2005).

The length of the bulge front/rear, Height of vehicles, will impact on increasing the carrying capacity of vehicles. Furthermore, this will directly increase load axis of the vehicle, so that the axle load will be heavier than the permitted (legal limit). This raises the problem of excessive load or overloading. The impact of overload conditions on the road pavement is premature failure, that is, a condition which the damage can reduce the life of roads before the design life of the road is reached. Research on excess load showed that it could significantly accelerate the damaging effect to the roads and endangering the safety of road users (Badan Litbang Departement PU, 2004)

1.2 The Location of Research

The case study of this thesis, i.e. Demak-Trengguli road segment, is a part of Java North Coastal Arterial Road, located in Kabupaten Demak, Central Java Province. The length of this road section is approximately 11 km that stretches from STA 24+300-STA 36+300 (see Fig. 1.1). Demak-Trengguli road segment is a flexible structure that consists of asphalt surface constructed on stabilised base and sub base course.

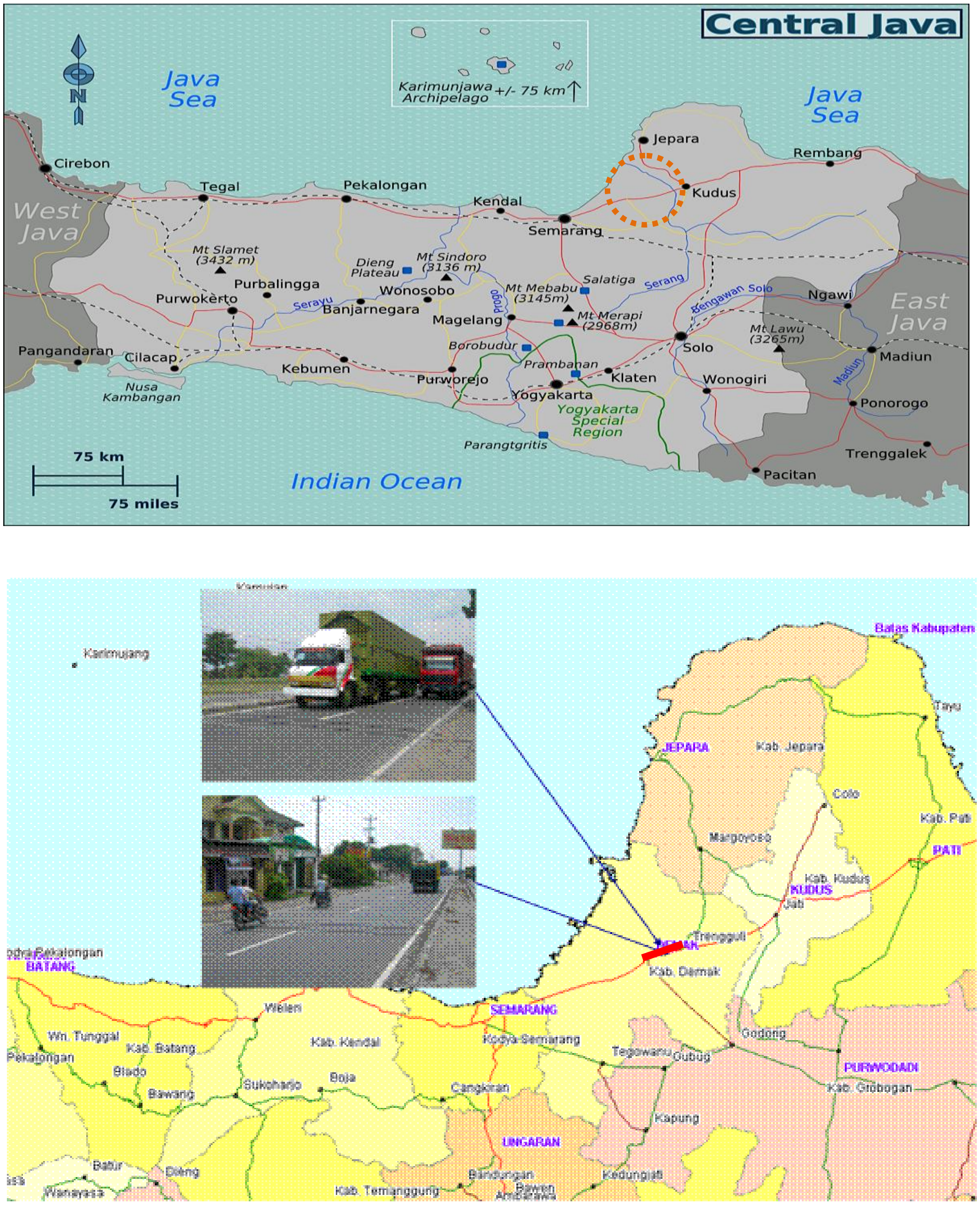


Figure1.1: Location of Demak-Trengguli Road Segment

1.3 Research Questions

Based on previous descriptions, it can be formulated several problems that arise, including the following:

a. How does the overloading heavy vehicles affect the pavement thickness and service life;

b. What is the difference in pavement service life between two conditions: road under standard traffic load and overloading condition?

1.4 Objectives

The objective of this research is to analyze the effect of overloaded heavy vehicle to the road pavement damage. The objective can be detailed into two targets:

a. To determine the reduction of pavement service life on the Demak-Trengguli road segment due to overloading.
b. To calculate the layer thicknesses required by pavement structure to withstand against overloading condition.

1.5 Research Scope and Limitations

In this study, the research scope and limitations are as follows.

a. The case study investigated in this research is Demak - Trengguli road segment, that is a two-way four-lane divided (4/2 D) flexible pavement road.
b. The calculation of pavement service life is based on ADT and CESAL of overloaded truck;
c. The standard method used in this study is AASHTO 1993

1.6 Organization of Thesis

In accordance with the Master Program, the proposal is organized into five chapters as follows:

Chapter 1 Introduction

This chapter consists of background, purpose, objectives, and location of case study.

Chapter 2 Literature Review

This chapter describes about characteristic of traffic, transport mode, the factors causing road damage and vehicle damage factor as well as pavement thickness design.

Chapter 3 Methodology

This chapter presents the descriptions of the approaches being taken to achieve objectives included in the secondary data processing, data analysis, and evaluation of results.

Chapter 4 Analysis Data

This chapter contains the data analysis due to overload, so that it will know how severe the overloaded truck will affect the road segment that is concerned.

Chapter 5 Conclusion and Recommendation

This chapter contains the conclusions that can be taken from the analysis results and recommendation for further works.

CHAPTER 2
LITERATURE REVIEW

2.1 Background

The current flexible pavement design methodology used by Bina Marga (Directorate General of Highway, DGH) is derived from the results of the American Association of State Highway Officials (AASHO) Road Test, conducted in the late 1950's. A basic nomograph and design equation were developed from the road test results including inputs of soil modulus, traffic, ride quality (serviceability), and the capacity of the pavement structure (structural number). Using the flexible pavement design equation or nomograph in conjunction with these inputs, the designer arrives at a design thickness for the pavement layers.

2.2 AASHO Road Test

2.2.1 Overview of the Test

The AASHO Road Test was conducted from 1958 to 1960 near Ottowa, Illinois. The primary purpose of the road test was to determine the effect of various axle loadings on pavement behavior. Both flexible and rigid pavements were tested in the study, along with several short span bridges. Six two-lane test loops were created for trafficking, including four large loops and two small loops. Hot mix asphalt (HMA) and base thicknesses were varied within each test loop to determine the effect of axle loadings on different pavement cross sections. Individual lanes were subjected to repeated loadings by a specific type and weight of vehicle. Single and tandem axle vehicles were used for trafficking. Bias-ply tires were used with pressures of approximately 70 psi. Only 2 million equivalent single axle loads (ESALs) were applied over the course of the test. One of the key products of the road test was the concept of load equivalency, which accounts for the effects of the axle loads on pavements in terms of an equivalent single axle load (ESAL). Under this concept, the damage imposed by any vehicle is based on its axle weights compared with a standard 18,000 lb axle load. The

ESAL values for other axles express their relative effect on pavement structure. If the number and types of vehicles using the pavement can be predicted, then engineers can design the pavement for anticipated a number of 18 kips equivalent single axle loads (18 kips ESAL). Virtually, all heavy-duty pavements built in the United States since the mid-1960s have been designed using the principles and formulas developed from the Road Test (Davis, 2009).

The test vehicles ranging in gross weight from 2,000 lb to 48,000 lb. The improved paving materials that are used today such as Superpave mixes, stone mastic asphalts, and open graded friction courses were not available during the road test. Within the pavement cross section, only one type of HMA, granular base material, and sub grade soil were used. The thickest HMA pavement was 6 inches. All results from the road test are a product of the climate of northern Illinois within a two-year period (HRB, 1962).

2.2.2 Results of AASHO Road Test

The results of the AASHO Road Test were used to develop the first pavement design guide, known as the AASHO Interim Guide for the Design of Rigid and Flexible Pavements. This design guide was issued in 1961, and had major updates in 1972, 1986, and 1993. The 1993 AASHTO Design Guide is essentially the same as the 1986 Design Guide for the design of new flexible pavements, and is still used today by many transportation agencies, including Bina Marga.

The primary objective for the AASHO Road Test was to determine the relationship between pavement loading and deterioration. Using replicate cross sections in different test loops (that were loaded with different axle weights), researchers at the road test were able to view the differences in pavement distresses such as rutting, cracking, and slope variance, that were caused by increasing axle loads. The relationship found was an approximate fourth power relationship: a unit increase in axle weight causes increased damage to the fourth power. To put this relationship into context, if the axle weight is doubled, it causes approximately sixteen times more damage to the pavement. Figure 2.1 illustrates the general relationship between loading and damage found at the road test (HRB, 1962).

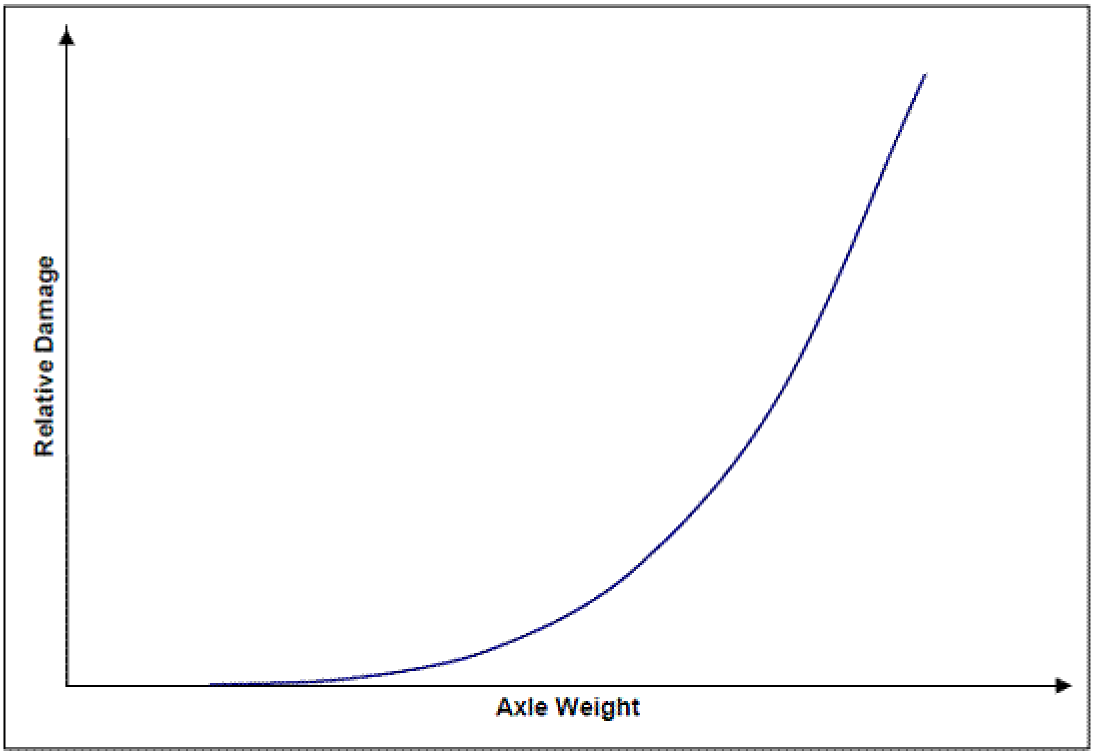


Figure 2.1: The Fourth Power Relationship between Axle Weight and Pavement Damage (HRB,1962)

2.3 Fundamental Equations

The 1993 AASHTO Design Guide is the current standard used for designing flexible pavement for many transportation agencies. In the AASHTO design methodology, the subgrade resilient modulus (M_R), applied ESAL (W_{18}), reliability (with its associated normal deviate, Z_R), variability (S_o), loss in serviceability (ΔPSI), and structural number (SN) are used in the nomograph (Figure 2.2) and the corresponding Equation 2.1 to design thickness of flexible pavements (AASHTO, 1993):

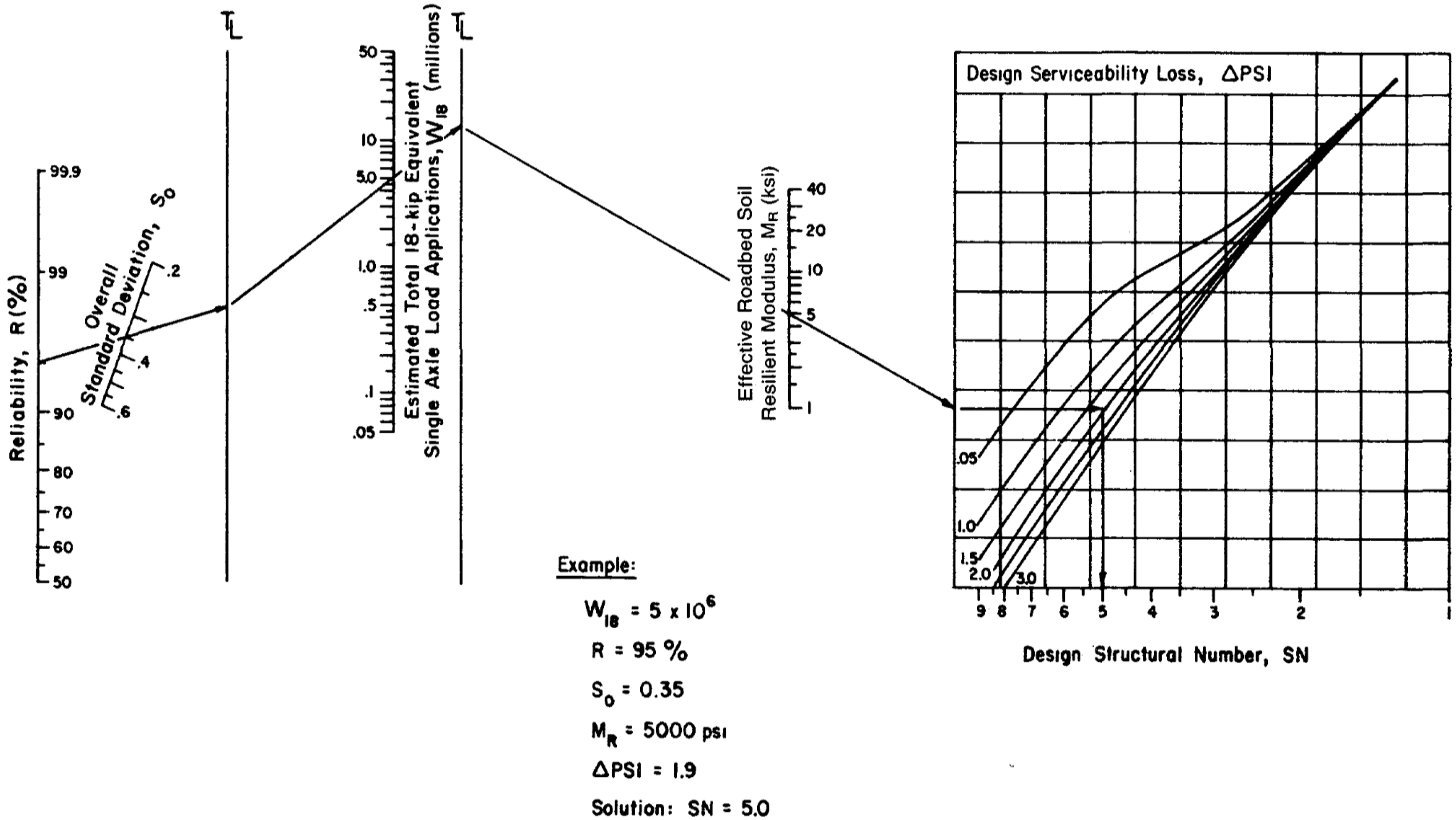


Figure 2.2: AASTHO Flexible Pavement Design Nomograph .(AASHTO, 1993)

$$logW18 = Zr(So) + 9.36\log(SN+1) - 0.20 + \frac{log\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40+\frac{1094}{(SN+1)^{5.19}}} + 2.32logMR - 8.07 \quad (2.1)$$

2.3.1 Traffic Load, W_{18} and Growth Rate, Gr

W_t is the number of single-axle load applications to cause the reduction of serviceability to the terminal level (p_t). and The standard deviation, S_o, is typically assumed to be 0.49 for flexible pavements based upon previous research (AASHTO, 1993).

Traffic load that used for determining flexible pavement design thickness in 1993 AASHTO is the cumulative traffic load during design life. The magnitude of the traffic load for two ways is obtained by summing the multiplication of three parameters, i.e. average daily traffic, axle load equivalency factor, and annual growth rate, for each type of axle load. Numerically, the formulation of cumulative traffic load is as follows:

$$\overline{W}_{18} = \sum_i \left(ADT_i \text{ x } E_i \text{ x } \mathrm{G}_{\mathrm{Ri}}\right) \text{x } 365 \quad (2.2a)$$

$$G_{Ri} = \frac{(1+g_i)^n - 1}{g_i} \quad (2.2b)$$

where:

$\overline{W}_{18}$ = cumulative standard single axle loads for two ways, ESALs

ADT_i = average daily traffic for axle load *i*

E_i = axle load equivalency factor (or vehicle damage factor) for axle load *i*

G_{Ri} = annual growth rate for vehicle *i*, %

g_i = traffic growth for vehicle type *i* (%)

n = service life, year

To obtain traffic on the design lane, the following formulation can be used:

$$W_{18} = D_D \text{ x } D_L \text{ x } \overline{W}_{18} \quad (2.3)$$

where:

W_{18} = cumulative standard single axle load on design lane, ESAL

D_D = direction distribution factor

D_L = lane distribution factors

D_D is generally taken 0.5. In some special cases, there are exceptions where heavy vehicles tend to run on a certain direction. Several studies indicate that the D_D varies from 0.3 to 0.7 depending on which direction that considers as major and minor (AASHTO 1993). The magnitude of D_L is determined based on the number of lanes in one carriageway (see Table 2.1)

Table 2.1: Lane Distribution factor (D_L)

Number of Lane per Direction	% Standard Axle Load in Design Lane
1	100
2	80-100
3	60-80
4	50-75

Source: AASHTO (1993)

2.3.2 Road Performance

Road performance can be defined as the ability of road structure to withstand against traffic load and environmental effects and denotes as PSI (present serviceability index). In pavement design, loss of serviceability or ΔPSI becomes the main concern, rather than PSI, because it indicates how far the pavement could survive before a rehabilitation work is required to extend its service life. The equation of ΔPSI is given by:

$$\Delta PSI = \Delta PSI_{traffic} + \Delta PSI_{SW, FH} \qquad (2.4)$$

In which

$\Delta PSI_{traffic}$ = serviceability loss because of traffic load = p_o - p_t

IP_o = 4,2 (AASHO road test),

IP_t = 2,5 - 3,0 for major highway and p_t equals to 2 for minor highway

$\Delta PSI_{SW, FH}$ = serviceability loss because of soil swelling (effect of moisture and frost)

$$\Delta PSI_{SW, FH} = 0{,}00335 \cdot V_R \cdot P_S \cdot (1 - e^{-\theta t}) \quad (2.5)$$

θ = swell rate constant (as a function of moisture supply and soil fabric)

V_R = maximum potential heave (as a function of plasticity index, compaction and subgrade thickness), inch.

P_s = swelling probability, %

2.3.3 The Relationship between PSI and IRI

The loss of serviceability (ΔPSI) is the difference between the initial serviceability of the pavement when opened to traffic and the terminal serviceability that the pavement will reach before rehabilitation, resurfacing or reconstruction is required. The present serviceability index (PSI), also known as the present serviceability rating (PSR), is a subjective measure by the road user of the ride quality, ranging from zero (impassible) to five (perfect ride). Studies conducted at the AASHO Road Test found that for a newly constructed flexible pavement, the initial serviceability (p_o) was approximately 4.2 (AASHTO, 1993).

The value of a terminal serviceability (p_t) was ranging between 2.0 and 3.5. The 1993 AASHTO Design Guide recommends the selection of p_t based upon the same traffic levels used for reliability selection: for low traffic, 2.5, for medium traffic, 3.0, and for high traffic, 3.5. To demonstrate the subjectivity of the measurement, studies from the AASHO Road Test found that an average of 12% of road users believe that a pavement receiving a rating of 3.0 is unacceptable for driving while 55% of road users believe that 2.5 is unacceptable (AASHTO, 1993).

Due to the subjective nature of serviceability measurements, most current road roughness measurements are now standardized to the international roughness index (*IRI*). This index provides a measure of the longitudinal wavelengths in the pavement profile in inches per mile or meters per kilometer. These measurements are taken by inertial profilers, and can be closely replicated from machine to machine (Sayers and Karamihas, 1998).

The use of this index can remove the subjectivity of assessing the ride quality, and therefore is a more accurate measurement. However, since the AASHTO flexible pavement design procedure still requires serviceability levels as inputs, a conversion must be made from *IRI* to *PSI* (Hall and Munoz, 1999).

In 1999, Hall and Munoz developed relationships for relating *IRI* and *PSI* for both asphalt and concrete pavements. They analyzed data from AASHO Road test that included parameters of slope variance (*SV*) and *PSI*, and then developed a correlation between *SV* and *IRI* for a broad range of road roughness levels. Their finding for flexible pavements can be expressed mathematically as

$$PSI = 5 - 0.2937X^4 + 1.1771X^3 - 1.4045X^2 - 1.5803X \tag{2.6}$$

Where:

$$X = \log(1 + SV) \tag{2.7}$$

$$SV = 2.2704\, IRI^2 \tag{2.8}$$

in which all variables are as previously defined.

Based upon the similarity of the Al-Omari and Darter (1994) and Holman (1990) equation, it was decided to focus on those relationships for this study. Since the equation developed by Al-Omari and Darter (1994) could produce much larger performance database, therefore, this equation was selected to convert the IRI data to present serviceability values.

2.3.4 Reliability (R) and Standard Deviation (S_o)

a. Reliability

Reliability concept is an effort to include a degree of certainty into the planning process to ensure a variety of alternatives will persist over the planned period. Planning reliability factors take into account possible variations of traffic estimate (w_{18}) and forecast performance (W_{18}), since both these factors provide a level of reliability where the section of pavement will persist for a planned period. Table 2.2 shows the recommended level of reliability for various road classifications. It should be noted that a higher level of reliability indicates the road that serves traffic at most, whereas the lowest level shows the local road (AASHTO, 1993).

Table 2.2 Recommendation of Reliability Level for Various Road Classifications

Functional Classification	**Recommended Level of Reliability, *R* (%)**	
	Urban	**Rural**
Toll road	85 – 99.9	80 – 99.9
Arterial road	80 – 99	75 – 95
Collector road	80 – 95	75 – 95
Local road	50 – 80	50 – 80

Source: AASHTO (1993)

Reliability of performance-design controlled with reliability factor (F_R) which is multiplied with the traffic estimates (w_{18}) over the design life to obtain performance predictions (W_{18}). For a given level of reliability, the reliability factor (F_R) is a function of the overall standard deviation (S_o), which takes into account the possibility of a variety of traffic estimates (w_{18}) and performance estimates (W_{18}) given. In flexible pavement design equation, the level of reliability accommodated with the parameters of the standard normal deviation (Z_R). Table 2.3 shows values of Z_R for certain service level.

Table 2.3: Standard Normal Deviation for Certain Reliability Service

Reliability (%)	**Normal Deviate (Z_R)**
50	-0.000
60	-0.253
70	-0.524
75	-0.674
80	-0.841
85	-1.037

Reliability (%)	**Normal Deviate (Z_R)**
90	-1.282
95	-1.645
98	-2.054
99	-2.327
99.9	-3.090
99.99	-3.750

Source: AASHTO (1993)

Application of the concept of reliability should consider the following steps:

1. Define the functional classification of roads and determine whether it is an urban road or inter-urban (rural) road.
2. Select the level of reliability from interval that given in Table 2.2.
3. Standard normal deviation of the corresponding reliability can be seen in Table 2.3.

b. Overall Standard Deviation (S_o)

Overall standard deviation is a combination of standard error of traffic prediction and road performance. This variable measures how far the probability of traffic prediction and road performance deviate from the design. For instance, it is predicted that the number of traffic is 2.000.000 ESAL for the next 20 year, however, in fact, there are 2.500.000 vehicles in that period. The larger the deviation is, the higher the value of S_o will be. For flexible pavement, the value of S_o is 0.35 – 0.40 (AASHTO, 1993).

2.3.5 Subgrade Bearing Capacity M_R

The M_R of the subgrade soil seen in the equation (2.1) has been adjusted to take into account for seasonal changes, and is termed as the effective M_R. This is done to take into account for differences in testing procedures from the road test and the current testing method using fallingweight deflectometer (FWD). At the road test, Screw driven laboratory devices were used to determine the soil stiffness. Due to slow response time of such devices, the apparent stiffness of the soil was very low (around 3,000 psi). With the much more rapid loading of FWD testing, the moduli are typically around three times higher, and therefore the moduli are divided by three to arrive at similar numbers to those used at the road test (Dives2009).

Subgrade bearing capacity can be represented by resilient modulus (M_R). It could be measured according to AASHTO T – 274 or based on relationship with other parameters, such as California Bearing Ratio (CBR).

$$M_R \text{ (psi)} = 1500 \times CBR \tag{2.9}$$

M_R measurement should be conducted routinely during a year to observe the relative damage on subgrade due to moisture effect.

2.3.6 Structural Number

All of the flexible pavement design methods up to 1993 concentrated on defining the structural number *SN* from the equation:

$$SN = a1D1 + a2D2m2 + a3D3m3 \qquad (2.\ 10)$$

where:

m_2 and m_3	=	drainage factor for layer 2 and 3, respectively;
D_1, D_2, and D_3	=	thickness (in) of layer 1, 2 and 3, respectively, and
a_1, a_2, and a_3	=	layer coefficient for layer 1, 2 and 3, respectively.

The layer thickness produced from *SN* equation does not have a single unique solution, i.e. there are many combination of layer thickness of the flexible pavement layers. It is necessary to consider their cost effectiveness along with the construction and maintenance constraints in order to avoid the possibility of producing an impractical design from a cost effective view. If the ratio of costs for layer 1 to layer 2 is less than the corresponding ratio of layer coefficients, then the optimum economical design is one where the minimum base thickness is used since it is generally impractical and uneconomical to place surface, base or subbase courses of less than some minimum thickness (AASHTO, 1993)

a. Drainage condition

The aim of involving this variable in the structural number is to evaluate the capability of the pavement in removing moisture. The category of quality of drainage corresponding on how long the moisture could be removed from the pavement, as follows.

Table 2.4: The Definition of Drainage Quality

Drainage Quality	Time for Water Disappeared
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very poor	water will not drain

Source: AASHTO (1993)

In 1993 AASHTO guidelines, it is introduced the principle of drainage coefficient to accommodate the quality of drainage system that owned by the road pavement. Table 2.5 shows drainage coefficient value (*m*) which is a function of drainage quality and percentage of time in a year pavement structure will be affected by water content that close to saturated.

Table 2.5: Drainage Coefficient (*m*)

Quality of drainage	Percentage of time pavement structure is exposed to moisture levels approaching saturation *)			
	< 1%	1 – 5%	5 – 25%	> 25%
Excellent	1.40 – 1.35	1.35 – 1.30	1.30 – 1.20	1.20
Good	1.35 – 1.25	1.25 – 1.15	1.15 – 1.00	1.00
Fair	1.25 – 1.15	1.15 – 1.05	1.00 – 0.80	0.80
Poor	1.15 – 1.05	1.05 – 0.80	0.80 – 0.60	0.60
Very poor	1.05 – 0.95	0.95 – 0.75	0.75 – 0.40	0.40

Remarks: *) depends on annual average rainfall and drainage condition at the road structure.
Source: AASHTO (1993)

b. Coefficient of Relative Strength

This guideline introduces a correlation between relative strength coefficient with mechanistic value, namely modulus resilient. Based on the type and function of pavement layer material, estimation of relative strength coefficient is grouped into 5 categories, namely asphalt concrete, granular base, granular subbase, cement-treated base (CTB), and asphalt-treated base (ATB) (AASTHO, 1993).

(1) Asphalt Concrete Surface Course

Figure 2.3 show the graphic that used to estimating the relative strength coefficient of Asphalt Concrete Surface Course (a_1) that has dense gradation based on Modulus Elasticity (E_{AC}) at 68°F temperature (AASTHO 4123). Although the modulus of asphalt concrete is higher, stiffer, and more resistant against deflection, but it is more susceptible to fatigue crack (AASHTO, 1993).

(2) Granular Base Layer

Relative strength coefficient (a_2) can be estimated by using Figure 2.4.

(3) Granular Subbase Layer

Relative strength coefficient (a_3) can be estimated by using Figure 2.5.

(4) Cement-Treated Base (CTB)

Figure 2.6 shows the graph that can be used to estimating relative strength coefficient, a_2 for cement-treated base (CTB).

(5) Asphalt-Treated Base (ATB)

Figure 2.7 shows the graph that can be used to estimating relative strength coefficient, a_2 for Asphalt-Treated Base (ATB).

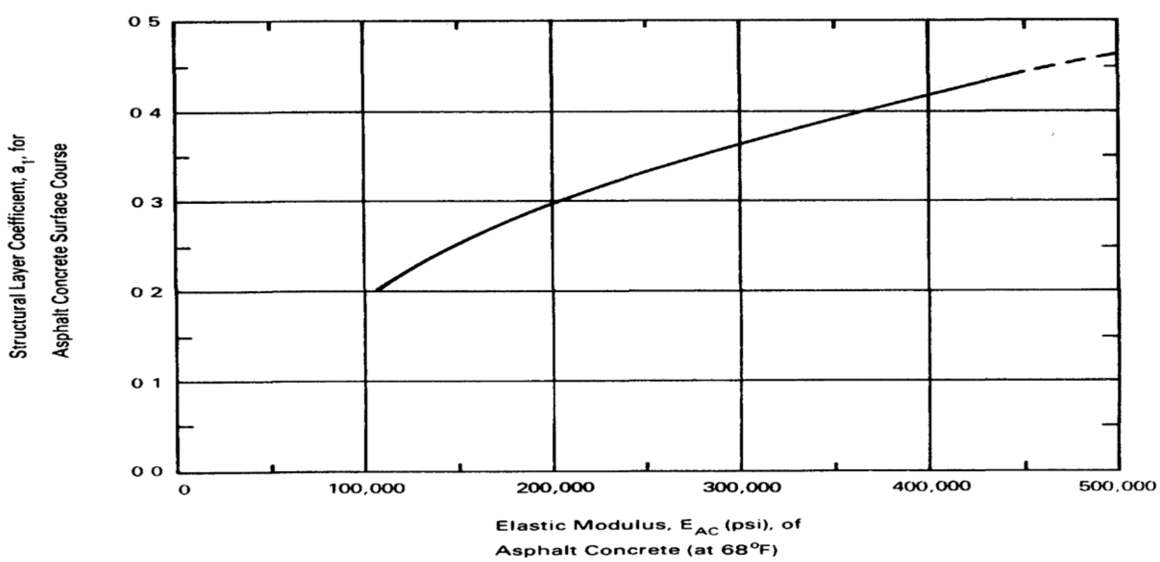
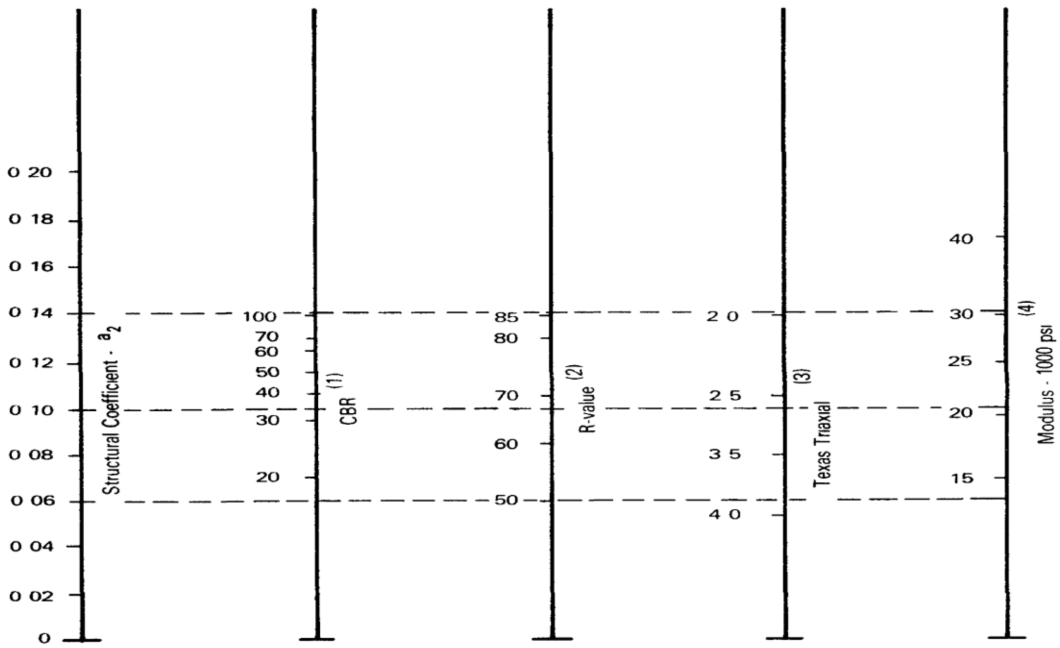
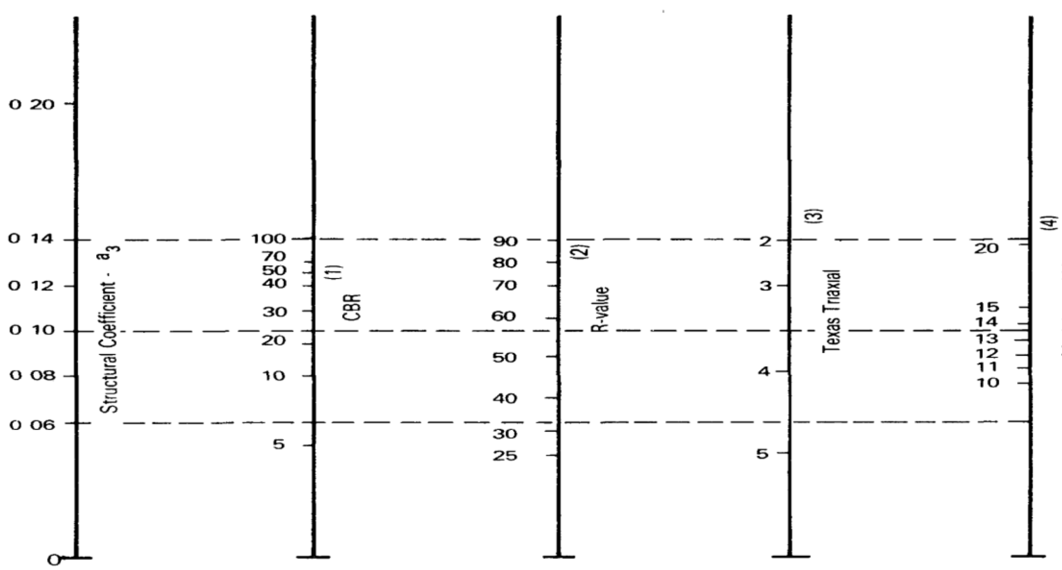


Figure 2.3: Relative Strength Coefficient of Asphalt Concrete Surface Course



(1) Scale derived by averaging correlations obtained from Illinois
(2) Scale derived by averaging correlations obtained from California, New Mexico and Wyoming
(3) Scale derived by averaging correlations obtained from Texas
(4) Scale derived on NCHRP project *(3)*

Figure 2.4: Variation of Relative Strength Coefficient of Granular Base Layers (a_2)



(1) Scale derived from correlations from Illinois
(2) Scale derived from correlations obtained from The Asphalt Institute, California, New Mexico and Wyoming
(3) Scale derived from correlations obtained from Texas
(4) Scale derived on NCHRP project *(3)*

Figure 2.5: Variation of Relative Strength Coefficient of Granular Subbase Layers (a_3)

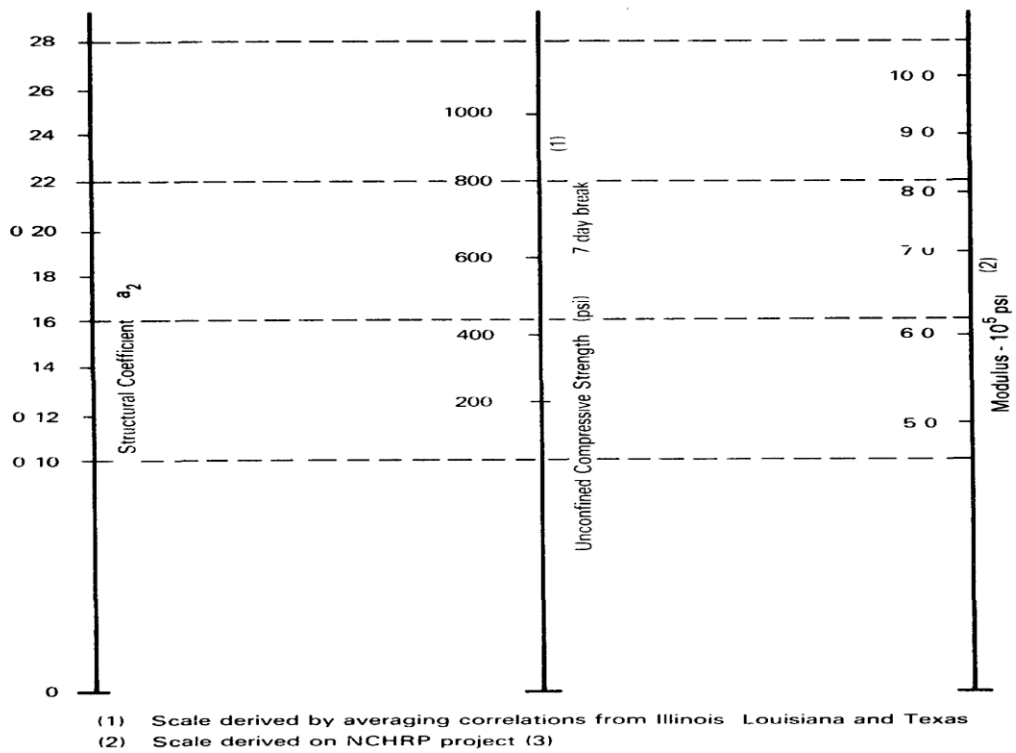


Figure2.6: Variation of Relative Strength Coefficient of Cement-Treated Base (a_2)

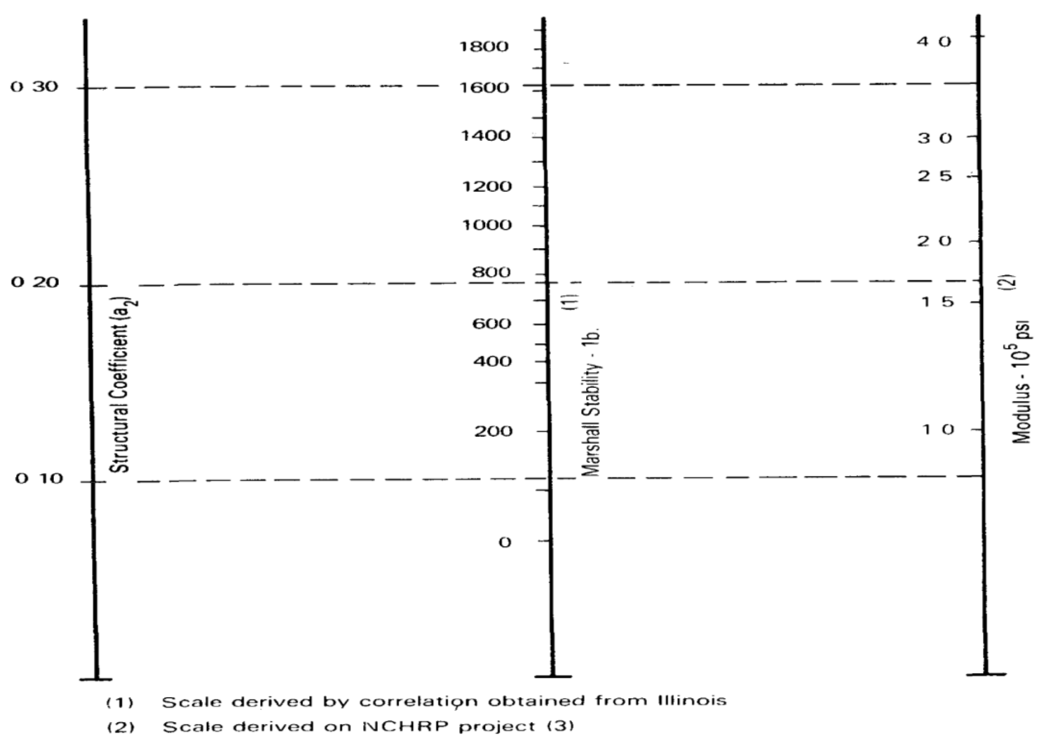


Figure 2.7: Variation of Relative Strength Coefficient of Asphalt-Treated Base (a_2)

2.3.7 Determination of Structural Number

Figure 2.2 shows the nomograph to determine Structure Number. The nomograph can be used if all following parameters are available:

1. Traffic estimation in the future (W_{18}) at the end of design life
2. Level of reliability (R)
3. Standard deviation (S_o)
4. Effective resilient modulus of subgrade material (M_R)
5. Loss of serviceability ($\Delta PSI = p_o - p_t$)

The calculation of pavement thickness in this guideline is based on the relative strength of each pavement layers, using formula as shown in Equation (2.1).

2.4 Single Axle Load Equivalency Factor (LEFs)

Using the fourth-power relationship found at the AASHO Road Test, equations were derived to relate axle loading to pavement damage. Replicate cross sections were constructed in different test loops to apply varying repeated axle loads on the same pavement structure. This allowed the researchers at the road test to view the damage caused by heavier axles, and create mathematical relationships based upon that damage. The resulting pavement damage was quantified using single axle load equivalency factors (LEFs), which are used to find the number of ESALs. An LEF is used to describe the damage done by an axle per pass relative to the damage done by a standard axle per pass. This standard axle is typically an 18-kip single axle, as defined in the road test. From the AASHO Road Test results, the LEF can be expressed in the following form according to Huang (2004). The EALF can be expressed in the following form according to Huang (2004):

$$EALF = \frac{Wt18}{Wtx} \tag{2.11}$$

To arrive at the design ESALs , it is necessary to assume a structural number (SN) and then select the equivalence factors listed in nine tables provided by AASHTO (1993). These tables vary by three types of axle and three values of terminal serviceability (p_t). The use of

SN of 5 for the determination of 18-kip single axle equivalence factors will normally give results that are sufficiently accurate for design purposes. Even though the final design may be somewhat different, this assumption will usually result in an over estimation of 18-kip equivalent single axle when more accurate results are desired and the computed design is appreciably different (1 inch of asphalt concrete) from the assumed value. A new value should be assumed and the design 18–kip ESAL traffic (W_{18}) recomputed. The procedure should be continued until the assumed and computed values are sufficiently close (AASHTO, 1993).

2.5 Previous of Studies

Several previous studies on road damage that have been done by previous researchers are as follows.

1. Rahim (2000). The analysis conducted was to calculate cost loss of road pavement distress resulted from overloading and therefore the amount of loss cost the overload car users shall bear can be determind. Overload heavy vehicle causes road pavement structure distress and service lifetime decreasing during design life time . The presence of overloading is indicated by the width area of rutting which is more than 60% of total road structural distress per km and by maximum axle load (MAL) of the heavy vehicle which is larger than the standard MAL. The cost loss of road pavement distress due to overloading is calculated based on damage factor (DF) and deficit design life (DDL). The loss of the overload car user shall bear 60% of total DFC (damage factor cost) and DDLC (deficit design life cost). Rahim (2000) was considering that not all pavement structural distresses are absolutely caused by overloading freight transport.
2. Koesdarwanto (2004) evaluated the service life of flexible pavement as a function of overloaded vehicles. Koesdarwanto concluded that the overloaded vehicles could decrease service life of road pavement from 5 years to 8 years.
3. Sulisty and Handayani (2002) evaluated the effect of heavy vehicle’s overloading to the pavement damage/service life on the road Muntilan-Magelang/Semarang. Sulisty and Handayani concluded that because of overloading on the road, there was a decrease of 1.4-year design life or 28% of the original design life 5 years.

CHAPTER 3
METHODOLOGY

3.1 Overview

The methodology is a flow chart or structural steps to solve a problem with a scientific approach. Every completed step should be evaluated with great accuracy in order to produce results as expected.

3.2 Research Methodology

In general, this research is conducted in several stages, as seen in Figure 3.1. The detail of each stage is presented in the following sections.

3.2.1 Preparation Stage

Preparatory work includes activities such as literature review of previous related studies in road sector, review the theories about the design of road pavement, and develop a methodology of the research.

3.2.2 Data Collection

At this stage, all data related to this research were collected. This study only employed secondary data, which is consisted of

- Traffic volume
- Vehicle weight that overloading was occurred (especially trucks).
- Soil strength in terms of California Bearing Ratio (CBR).
- Thickness of existing pavement layers in Demak-Trengguli road section.
- International roughness index (IRI).

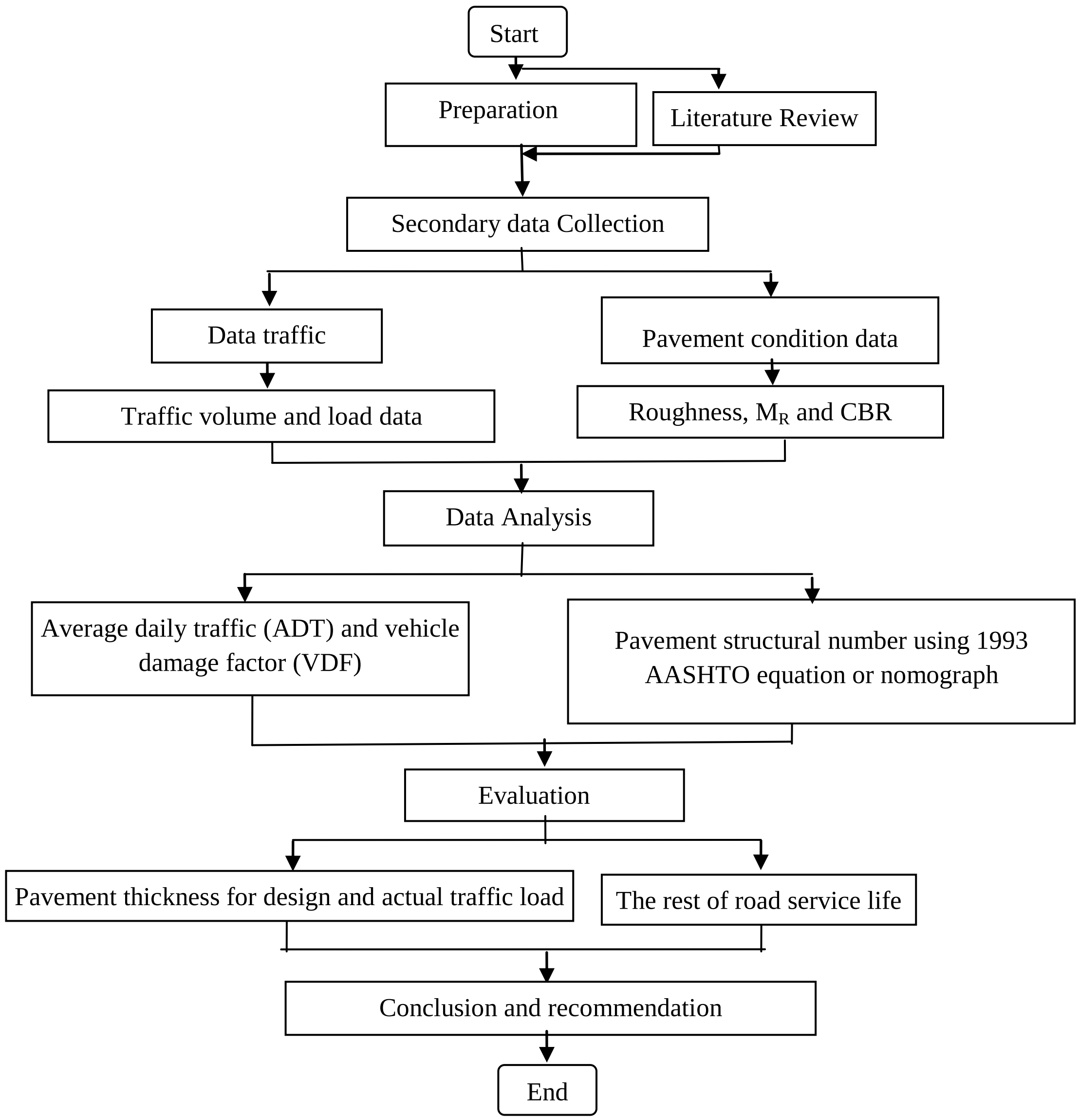


Figure 3.1: Methodology of This Study

3.2.3 Data Analysis

The secondary data obtained was analyzed on the basis of literature review and theories that had been learned. They are:

(1) Existing physical conditions

To know the existing physical condition, such as IRI, CBR, ADT obtained from Bina Marga.

(2) Design condition

Calculation of layer thickness using existing data was performed in order to know the differences between thickness of existing and design layers in Demak-Trengguli road section. The steps of calculation consist of:

a. Determine the volume of traffic (design ADT) from survey data
b. Calculate vehicle damage factor (VDF)
c. Calculate cumulative equivalent single axle load (CESAL) using actual VDF based upon design and existing condition.
d. Calculate pavement thickness based upon design and existing CBR and IRI.
e. Calculate pavement service life for design and existing condition.

3.2.4 Evaluation

The purpose of this stage is to evaluate the results obtained, by following the procedure:

a. Determine of the thickness difference between normal and overloaded conditions.
b. Determine the reduction of pavement service life for normal and overloaded conditions to know the rest of service life of Demak-Trengguli road section.

3.2.5 Conclusion

In this stage, conclusions from the results of evaluation can be drawn, to be followed up properly by the parties concerned.

CHAPTER 4
RESULTS AND ANALYSIS

4.1 Analysis of Traffic Data

In this study, analysis of traffic data in the means of calculating average daily traffic (ADT) was performed by summing all groups of vehicles for the entire survey period 24-hours a day and then divided on how many days to collect the 24-hour data. The data survey period was 5 days for each direction. However, the full-set data, i.e. 24-hour traffic data, available in this study was only 3 days. Therefore, all traffic analyses in this study were based upon these 3-day traffic data. According to Bina Marga standards (2009), vehicles could be categorized into 7 classes, however, only three classes of vehicles categorized as truck-type (heavy vehicle) that were considered in this study, in accordance with 1993 AASHTO Design Guide requirement. They are vehicle-class of 6B for 2-axis trailer, 7A for 3-axis trailer and 7C for more than 3-axis trailer. Vehicle-class 7C consists of three sub-classes; they are 7C1 for 4-axle trailer, 7C2 for 5-axle trailer and 7C3 for 6-axle trailer. The ADT of heavy vehicles for both directions for three-day survey is as shown in Table 4.1 below.

Table 4.1: ADT for Heavy Vehicles

Direction	**ADT of Each Vehicle Classes and Sub-classes (vehicle)**					
	6B	**7A**	**7C**			**Total**
			7C1	**7C2**	**7C3**	
Demak – Trengguli	921	781	74	130	119	2,025
Trengguli – Demak	814	968	83	146	134	2,144

Source: Bina Marga (2009)

From the table above, it can be seen that there is relatively no much deviation of ADT for heavy vehicles for both direction.

4.2 Determination of Vehicle Damage Factor (VDF)

VDF or axle load equivalency factor (LEF or E) of each heavy vehicle was determined using 1993 AASHTO Design Guide procedure, as follows.

1. The axle load unit was converted from ton to kips and the type of the axle load was determined whether it is single, tandem or triple axles.
2. VDF or E was determined by correlating the axle load (see the first column of Tables 4.2, 4.3 and 4.4 for single, tandem and triple axles, respectively) and its corresponding VDF value (see the rest columns of the tables). The selected VDF was the value under pavement structural number (SN) equals to 5, as recommended by 1993 AASHTO Design Guide. In this study, all axle load equivalency factor tables were associated with terminal serviceability (p_t) equals to 2.
3. The VDF of front and rear axles for every type of heavy vehicle were calculated based upon the configuration specification defined by Bina Marga (2009)
4. The VDF for each heavy vehicle was determined by summing its corresponding front and rear axles. Then, the total VDF for each type of heavy vehicle could be calculated. The results for VDF for each type of heavy vehicle are shown in Table 4.5.

The sample of VDF calculation can be seen in Appendix D.

Table 4.2: **Axle Load Equivalency Factors for Flexible Pavements, Single Axles and p_t of 2.0**

xle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	0002	0002	0002	0002	0002	0002
4	002	003	002	002	002	002
6	009	.012	011	010	009	009
8	030	035	036	033	031	029
10	075	085	090	085	079	076
12	165	177	189	183	174	168
14	325	338	354	350	338	331
16	589	598	613	612	603	596
18	1 00	1 00	1 00	1 00	1 00	1 00
20	1 61	1 59	1 56	1 55	1 57	1 59
22	2 49	2 44	2 35	2 31	2 35	2 41
24	3 71	3 62	3 43	3 33	3 40	3 51
26	5.36	5 21	4 88	4 68	4 77	4 96
28	7 54	7 31	6 78	6 42	6 52	6 83
30	10 4	10 0	9 2	8 6	8 7	9 2
32	14 0	13 5	12 4	11 5	11 5	12 1
34	18 5	17 9	16 3	15 0	14 9	15 6
36	24 2	23 3	21 2	19 3	19 0	19 9
38	31 1	29 9	27 1	24 6	24 0	25 1
40	39 6	38 0	34 3	30 9	30 0	31 2
42	49 7	47 7	43 0	38 6	37 2	38 5
44	61 8	59 3	53 4	47 6	45 7	47 1
46	76 1	73 0	65 6	58 3	55 7	57 0
48	92 9	89 1	80 0	70 9	67 3	68 6
50	113	108	97	86	81	82

Source: AASHTO (1993)

Table 4.3: **Axle Load Equivalency Factors For Flexible Pavements, Tandem Axles and p_t of 2.0**

Axle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	0000	0000	0000	0000	0000	0000
4	0003	0003	0003	0002	0002	0002
6	001	001	001	001	001	001
8	003	003	003	003	003	002
10	007	008	008	007	006	006
12	013	016	016	014	013	012
14	024	029	029	026	024	023
16	041	048	050	046	042	040
18	066	077	081	075	069	066
20	103	117	124	117	109	105
22	156	171	183	174	164	158
24	227	244	260	252	239	231
26	322	340	360	353	338	329
28	447	465	487	481	466	455
30	607	623	646	643	627	617
32	810	823	843	842	829	819
34	1 06	1 07	1 08	1 08	1 08	1 07
36	1 38	1 38	1 38	1 38	1 38	1 38
38	1 76	1 75	1 73	1 72	1 73	1 74
40	2 22	2 19	2 15	2 13	2 16	2 18
42	2 77	2 73	2 64	2 62	2 66	2 70
44	3 42	3 36	3 23	3 18	3 24	3 31
46	4 20	4 11	3 92	3 83	3 91	4 02
48	5 10	4 98	4 72	4 58	4 68	4 83
50	6 15	5 99	5 64	5 44	5 56	5 77
52	7 37	7 16	6 71	6 43	6 56	6 83
54	8 77	8 51	7 93	7 55	7 69	8 03
56	10 4	10 1	9 3	8 8	9 0	9 4
58	12 2	11 8	10 9	10 3	10 4	10 9
60	14 3	13 8	12 7	11 9	12 0	12 6
62	16 6	16 0	14 7	13 7	13 8	14 5
64	19 3	18 6	17 0	15 8	15 8	16 6
66	22 2	21 4	19 6	18 0	18 0	18 9
68	25 5	24 6	22 4	20 6	20 5	21 5
70	29 2	28 1	25 6	23 4	23 2	24 3
72	33 3	32 0	29 1	26 5	26 2	27 4
74	37 8	36 4	33 0	30 0	29 4	30 8
76	42 8	41 2	37 3	33 8	33 1	34 5
78	48 4	46 5	42 0	38 0	37 0	38 6
80	54 4	52 3	47 2	42 5	41 3	43 0
82	61 1	58 7	52 9	47 6	46 0	47 8
84	68 4	65 7	59 2	53 0	51 2	53 0
86	76 3	73 3	66 0	59 0	56 8	58 6
88	85 0	81 6	73 4	65 5	62 8	64 7
90	94 4	90 6	81 5	72 6	69 4	71 3

Source: AASHTO (1993)

Table 4.4: **Axle Load Equivalency Factors for Flexible Pavements, Triple Axles and p_t of 2.0**

Axle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	0000	0000	0000	0000	0000	000
4	0001	0001	0001	0001	0001	000
6	0004	0004	0003	0003	0003	000
8	0009	0010	0009	0008	0007	000
10	002	002	002	002	002	001
12	004	.004	004	003	003	003
14	006	007	007	006	006	005
16	010	012	012	010	009	009
18	016	019	019	017	015	015
20	024	029	029	026	024	023
22	.034	.042	042	038	035	034
24	049	058	060	055	051	048
26	068	080	083	077	071	068
28	093	107	113	105	098	094
30	125	.140	149	140	131	126
32	164	182	194	184	173	167
34	213	233	248	238	225	217
36	273	294	313	303	288	279
38	346	368	390	381	364	353
40	434	456	481	473	454	443
42	538	560	587	580	561	548
44	662	682	710	705	686	673
46	807	825	852	849	831	818
48	976	992	1 015	1 014	999	987
50	1 17	1 18	1 20	1 20	1 19	1 18
52	1 40	1 40	1 42	1 42	1 41	1 40
54	1 66	1 66	1 66	1 66	1 66	1 66
56	1 95	1 95	1 93	1 93	1 94	1 94
58	2 29	2 27	2 24	2 23	2 25	2 27
60	2 67	2.64	2 59	2 57	2 60	2 63
62	3 10	3 06	2 98	2 95	2 99	3 04
64	3 59	3 53	3 41	3 37	3 42	3 49
66	4 13	4 05	3 89	3 83	3 90	3 99
68	4 73	4 63	4 43	4 34	4 42	4 54
70	5 40	5 28	5 03	4 90	5 00	5 15
72	6 15	6 00	5 68	5 52	5 63	5 82
74	6 97	6 79	6 41	6 20	6 33	6 56
76	7 88	7 67	7 21	6 94	7 08	7 36
78	8 88	8.63	8 09	7 75	7 90	8 23
80	9 98	9 69	9 05	8 63	8 79	9 18
82	11 2	10 8	10 1	9 6	9 8	10 2
84	12 5	12.1	11 2	10 6	10 8	11 3
86	13 9	13.5	12 5	11 8	11 9	12 5
88	15 5	15.0	13 8	13 0	13 2	13 8
90	17 2	16 6	15 3	14 3	14 5	15 2

Source: AASHTO (1993)

Table 4.5: Total VDF for Each Type of Heavy Vehicle Used in This Study

Direction	**VDF for heavy vehicles**				
	Class 6B (2 axle)	**Class 7A (3axle)**	**Class 7C1 (4 axle)**	**Class 7C2 (5 axle)**	**Class 7C3 (6 axle)**
Demak – Trengguli	10.91	5.36	22.01	13.5	24.3
Trengguli – Demak	2.432	3.15	10.57	12.72	8.18

As seen in Table 4.5, the total VDF of Demak – Trengguli direction is higher than the opposite direction. The deviation of VDF is mainly contributed by VDF of classes 6B, 7C1 and 7C3.

4.3 Calculation of Traffic Load

The calculation of traffic load W_{18} in equivalent standard axle load (ESAL) should be based on the actual VDF and ADT. AASHTO Design Guide gives the following formula to determine the traffic load for design lane (W_{18}).

$$\overline{W}_{18} = \sum_i \left(ADT_i \text{ x } E_i \text{ x } \mathrm{G}_{\mathrm{Ri}} \right) \text{x } 365 \tag{2.2a}$$

$$W_{18} = D_D \text{ x } D_L \text{ x } \overline{W}_{18} \tag{2.3}$$

where:

ADT_i = average daily traffic for axle load *i*;

E_i = axle load equivalency factor or vehicle damage factor (VDF) for axle load *i*;

G_{Ri} = annual growth rate (depends on traffic growth rate, g, in percent; and service life, n, in year) axle load *i*;

D_D = directional distribution factor;

D_L = lane distribution factor;

$\overline{\mathrm{W}}_{18}$ = cumulative standard axle load for two directional 18-kip ESAL units predicted for a specific section of highway during the analysis period (from the planning group).

Road section Demak – Trengguli or Trengguli - Demak is a four-lane two- direction divided (4/2 D) road, therefore, in this case, D_D and D_L equal to 1 and 0.8, respectively. The traffic load on Demak – Trengguli or Trengguli – Demak road section was assumed to increase 6% per anuum and the road could serve traffic load for the next 10 years. Based on this assumption, the traffic load W_{18} on Demak – Trengguli and Trengguli – Demak in two conditions, i.e. standard (as designed) and overloaded conditions, are depicted in the following table.

Table 4.6: Traffic load (as designed and overloaded condition)

Direction	**W_{18} (in million ESAL)**	
	As Designed	**Overloaded**
Demak – Trengguli	19.542	100.850
Trengguli – Demak	20.715	40.380

Table 4.6 shows that there is no different on traffic load for both directions in standard condition, but in overloaded condition, traffic load of Demak -Trengguli direction is 2.5 higher than opposite direction. This is because the significant deviation of the VDF in Demak-Trengguli direction (see table 4.5), although the ADT of Demak-Trengguli was lower than the opposite direction.

4.4 Reduction of Pavement Service Life

Two impacts of overloaded heavy vehicles on road pavement that took into account in this study, namely, reduction of pavement service life and the need of structural capacity improvement in terms of layer thickness.

The reduction of service life could be indicated by the deviation of the pavement service life due to different magnitude of traffic load that have to withstand by the pavement structure. To calculate the reduction of service life, a relationship between traffic load and service life is able to be developed by using the 1993 AASHTO Design Guide equation as follows.

$$W_{18} = w_{18} \times \frac{(1+g)^n - 1}{g} \tag{4.1}$$

in which W_{18} is the predicted traffic load (in ESAL); w_{18} is the traffic load in basic year (in ESAL); the other parameter is as previously defined. The traffic loads in basic year for both conditions (standard and overloaded) are as shown in Table 4.6. These values and Equation (4.1) were used to plot predicted traffic load curves in Figures 4.1 and 4.2 for Demak – Trengguli and Trengguli – Demak directions, respectively.

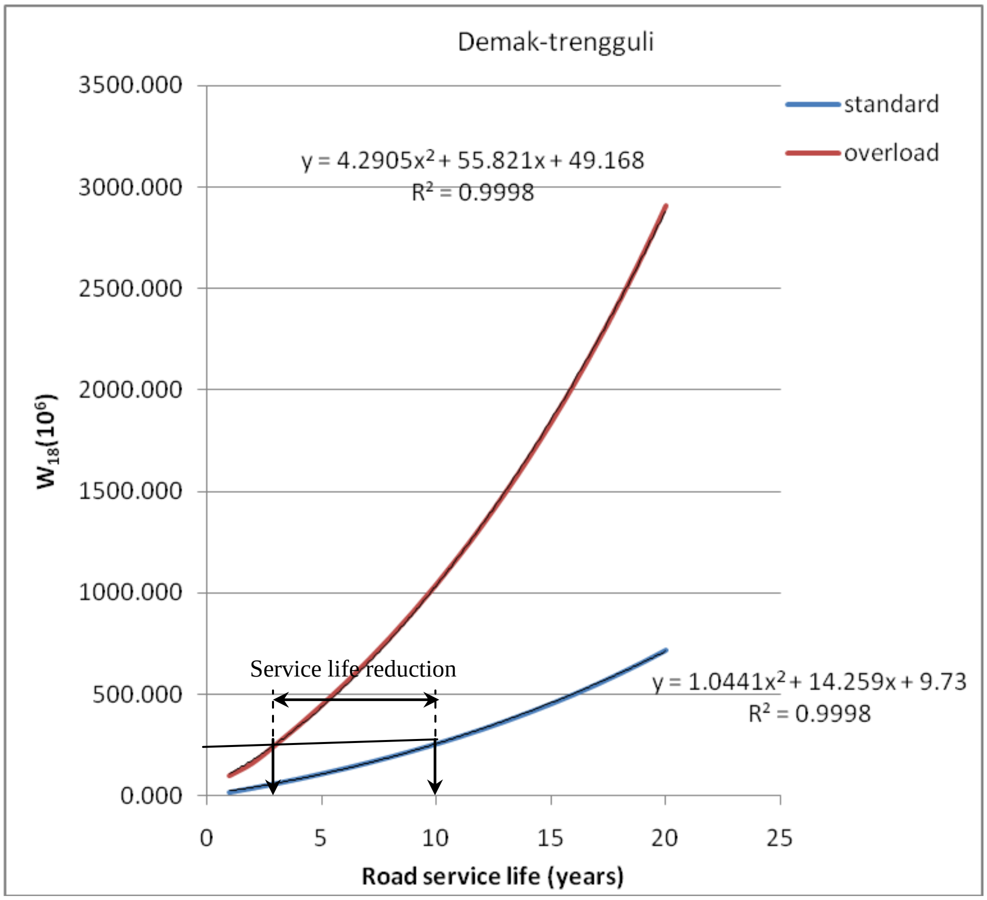


Figure 4.1: Service Relationship between Traffic Load and Service Life on Standard and Overloaded Conditions (Demak – Trengguli Direction)

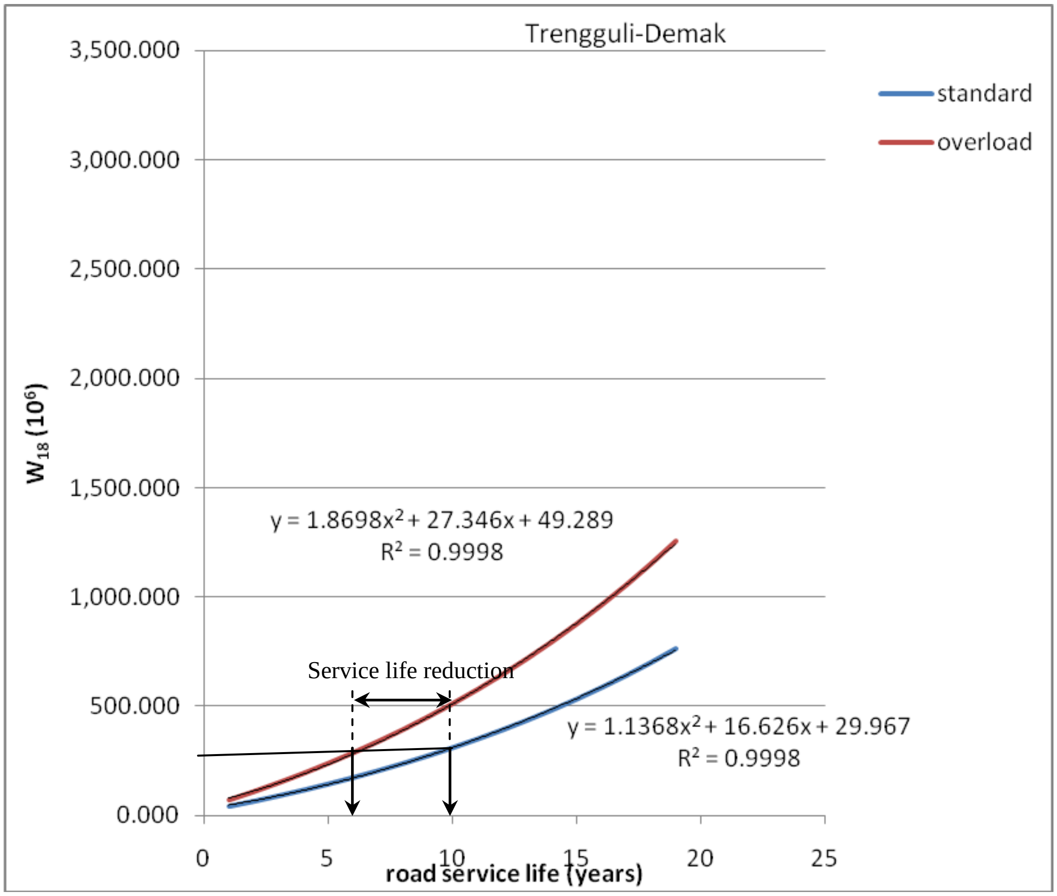


Figure 4.2: Relationship between Traffic Load and Service Life on Standard and Overloaded Conditions (Trengguli – Demak Direction)

As shown in the figures, the curves have the following equations.

$$Y = 1.044X^2 + 14.25X + 9.73 \tag{4.2}$$

$$Y = 4.219X^2 + 57.62X + 39.32 \tag{4.3}$$

for standard and overloaded conditions (Demak – Trengguli direction), respectively. And,

$$Y=1.136X^2 + 16.62X + 29.96 \tag{4.4}$$

$$Y=1.869X^2 + 27.34X + 49.28 \tag{4.5}$$

for standard and overloaded conditions (Trengguli – Demak direction), respectively.

Using the equations, the reduction of service life due to overloaded condition can be determined, as shown in Table 4.7 below.

Table 4.7: Relationship between Traffic Load and Service Life

No of.year	Traffic Load of Demak – Trengguli (million ESAL)		Traffic Load of Trengguli – Demak (million ESAL)	
	Standard	Overloaded	Standard	Overloaded
1	19.541	100.852	20.714	40.382
2	40.254	162.678	42.671	70.184
3	62.211	251.409	65.945	108.465
4	85.484	345.463	90.616	149.043
5	110.154	445.161	116.767	192.056
6	136.305	550.841	144.487	237.649
7	164.024	662.861	173.870	285.978
8	193.406	781.603	205.016	337.207
9	224.552	907.469	238.031	391.509
10	257.566	1040.887	273.027	449.070
11	292.561	1182.311	310.123	510.084
12	329.656	1332.219	349.444	574.759
13	368.976	1491.122	391.125	643.314
14	410.655	1659.560	435.306	715.983
15	454.836	1838.103	482.138	793.012
16	501.667	2027.360	531.781	874.663
17	551.308	2227.971	584.402	961.213
18	603.927	2440.619	640.180	1052.956
19	659.704	2666.027	699.304	1150.203
20	718.827	2904.958	761.977	1253.285

For example, the standard traffic load of Demak – Trengguli in 10 years is 257,566,000 ESAL, but this number in overloaded condition is reached in 3.077 years. This means that there is about 7 years reduction of service life because of overloaded heavy vehicles. In the same manner, the standard traffic load of Trangguli – Demak in 10 years will be reached in overloaded condition after 6.573 years, so that the reduction of service life due to overloading is about 4 years. It means that the overloaded condition could reduce the service life about 4 times and 2 times for Demak – Trengguli and Trengguli – Demak directions, respectively.

4.6 Calculation of Structural Capacity

To calculate the structural capacity of road pavement, as represented by structural number (*SN*), it is necessary to determine several parameters as follow.

4.5.1 Loss of Serviceability (ΔPSI)

The loss of serviceability can be determined by following the procedure below.

a. Average *IRI* was calculated from the existing data for 8 station of each direction. The *IRI* for all stations can be seen in Tables 4.8 and 4.9 below.

Table 4.8: Loss of Serviceability for Demak – Trengguli Direction

Station	**IRI (m/km)**	**SV**	**X**	**p_o**	**ΔPSI**
(26+900)	3.06	21.21	1.35	2.23	0.23
(28+350)	2.45	13.57	1.16	2.58	0.58
(30+000)	3.30	24.78	1.41	2.12	0.12
(32+100)	4.17	39.48	1.61	2.00	0.00
(33+300)	3.96	35.63	1.56	2.00	0.00
(34+700)	2.98	20.19	1.33	2.27	0.27
(35+700)	3.26	24.06	1.40	2.14	0.14
(36+300)	3.26	24.06	1.40	2.14	0.14
Average					0.185

Table 4.9: Loss of Serviceability for Trengguli – Demak Direction

Station	**IRI (m/km)**	**SV**	**X**	**p_o**	**ΔPSI**
(26+900)	3.29	24.51	1.41	2.12	0.12
(28+350)	3.34	25.33	1.42	2.10	0.10
(30+000)	3.07	21.35	1.35	2.23	0.23
(32+100)	3.97	35.81	1.57	2.00	0.00
(33+300)	2.72	16.80	1.25	2.41	0.41
(34+700)	3.03	20.89	1.34	2.25	0.25
(35+700)	3.30	24.72	1.41	2.12	0.12
(36+300)	3.30	24.72	1.41	2.12	0.12
Average					0.169

b. PSI (in this case, PSI was referred to initial serviceability, p_o) can be obtained by using the relationship between *PSI* and *IRI*, as follows.

$$PSI = 5 - O.2937X^4 + 1.1771X^3 - 1.4045X^2 - 1.5803X \quad (2.6)$$

where:

$X = \log(1 + SV)$ (2.7)

$SV = 2.2704\ IRI^2$ (2.8)

c. Loss of serviceability could be calculated using the following equation.

$$\Delta PSI = p_o - p_t \qquad (4.6)$$

where p_o is initial serviceability index (calculated by using equation 4.8 above) and p_t is terminal serviceability index. In this study, the terminal serviceability used equals to 2. The use of p_t = 2 is caused by the minimum terminal serviceability provided by AASHTO's axle load equivalency factors tables equals to 2. The calculation result ΔPSI for two directions are depicted in Table 4.8 and 4.9.

In Tables 4.8 and 4.9, there are road sections having high IRI values that cause the initial serviceability (p_o) is less than terminal serviceability (*pt*). To overcome this problem, all initial serviceability; that was less than two, was equated to two.

4.5.2 Resilient modulus (M_R)

The value of resilient modulus could be measured according to AASHTO procedure or based on relationship with other parameter, such as California Bearing Ratio (*CBR*). This relationship is represented by the following equation.

$$M_R \text{ (psi)} = 1500 \times CBR \qquad (2.9)$$

The CBR for every single station on the road and its corresponding M_R is shown in Table 4.10. It was assumed that the resilient modulus is similar for the subgrade of both directions.

Table 4.10: Modulus Resilient of Subgrade

STA	Subgrade CBR (%)	MR (psi)
(26+900)	4.5	6750
(28+350)	4.7	7050
(30+000)	6.3	9450
(32+100)	3.4	5100
(33+300)	6.8	10200
(34+700)	6.9	10350
(35+700)	7.2	10800
(36+300)	5.7	8550

4.5.3 Calculation of Structural Number (SN) and Layer Thickness (D)

The structural capacity of road structure, represented by SN, is determined by the following procedure.

a. SN_3, SN_2 and SN_1 were determined based on resilient modulus of subgrade, subbase and base layer, respectively, using AASHTO design thickness equation (see equation 2.1) The three values of SN were calculated using data from the following input parameter which is corresponding with standard and overloaded conditions: traffic load, W_{18} (see Table 4.7), and loss serviceability (ΔPSI) (see Tables 4.8 and 4.9). Other parameters, R or Z_R and S_o, were assumed to be similar for the two conditions, that are, R = 90% or Z_R = -1.282 and S_o = 0.35.

b. Coefficient of Relative Strength (*a*) and Drainage Coefficient (*m*) were determined. The coefficients of relative strength (*a*) for standard and overloaded conditions had similar values, except the values of a_3 (coefficient of relative strength for subgrade). The a_1 and a_2 were determined based on the assumption of resilient or elastic modulus as follows: E_1 = 400,000 psi, and E_2 = 30,000 psi. Using Figures 2.3, and 2.4, it will result in a_1 = 0.42, and a_2 = 0.14. For coefficient of relative strength of subgrade, there were two values based on different conditions: standard condition (based on E_3 = 15,000 psi), a_3 = 0.11, and overloaded condition, based on resilient modulus of subgrade as seen in Table 2.5. The drainage coefficient (*m*) was assumed to equal to 1 as the quality of drainage was flowing the water from the pavement structure within one day or it is categorized as “good” (see Table 2.4)

c. The layer thickness was calculated using the following equations

$$SN_3 = a_1 D_1 + a_2 D_2\, m_2 + a_3 D_3\, m_3 \tag{4.7a}$$

$$SN_2 = a_1 D_1 + a_2 D_2\, m_2 \tag{4.7b}$$

$$SN_1 = a_1 D_1 \tag{4.7c}$$

where:

a_1, a_2, a_3 = layer coefficients reprehensive of surface, base, and sub base courses respectively.

D_1, D_2, D_3 = actual thicknesses (in inches) of surface, base and subbase courses, respectively

m_2, m_3 = drainage coefficients for base and sub base layers, respectively

Based on the procedure above, the structural number and thickness of each layer for two conditions (standard and overloaded) can be determined (see Tables 4.11 – 4.14). In rehabilitation work, it is common not to disturb the existing layers and add another layer on top of existing surface layer, called as overlay. The thickness of overlay can be obtained by subtracting the total thickness of surface layer (D_1 in Tables 4.12 and 4.14) with the thickness of existing surface layer (D_1 in Tables 4.11 and 4.13). The example of thickness calculation can be seen in Appendix D.

Table 4.11: SN and Layer Thickness of Road of Demak - Trengguli Direction (Standard Condition)

STA	SN3	SN2	SN1	D1 (in.)	D2 (in.)	D3 (in.)
(26+900)	5.02	3.88	3.06	8	4	10
(28+350)	4.96	3.88	3.06	8	4	10
(30+000)	4.52	3.88	3.06	8	4	6
(32+100)	5.48	3.88	3.06	8	4	15
(33+300)	4.42	3.88	3.06	8	4	5
(34+700)	4.40	3.88	3.06	8	4	5
(35+700)	4.33	3.88	3.06	8	4	4
(36+300)	4.67	3.88	3.06	8	4	7
AVERAGE	4.73	3.88	3.06	8.00	4.00	7.75

Table 4.12: SN and Layer Thickness of Road of Demak - Trengguli Direction (Overloaded Condition)

STA	SN3	SN2	SN1	D1 (in.)	D2 (in.)	D3 (in.)
(26+900)	8.30	6.40	4.63	16	4	10
(28+350)	6.82	5.19	3.52	13	4	10
(30+000)	21.05	18.67	15.54	48	4	6
(32+100)	24.07	18.67	15.54	53	4	15
(33+300)	20.63	18.67	15.54	47	4	5
(34+700)	7.76	6.87	5.15	16	4	5
(35+700)	9.43	8.55	6.80	21	4	4
(36+300)	10.09	8.55	6.80	21	4	7
AVERAGE	13.52	11.45	9.19	29.38	4.00	7.75

Table 4.13: SN and Layer Thickness of Road of Trengguli - Demak Direction (Standard Condition)

STA	SN3	SN2	SN1	D1 (in.)	D2 (in.)	D3 (in.)
(26+900)	5.06	3.91	3.08	8	4	11
(28+350)	5.00	3.91	3.08	8	4	10
(30+000)	4.55	3.91	3.08	8	4	6
(32+100)	5.52	3.91	3.08	8	4	15
(33+300)	4.44	3.91	3.08	8	4	5
(34+700)	4.42	3.91	3.08	8	4	5
(35+700)	4.36	3.91	3.08	8	4	4
(36+300)	4.71	3.91	3.08	8	4	8
AVERAGE	4.76	3.91	3.08	8.00	4.00	8.00

Table 4.14: SN and Layer Thickness of Road of Trengguli - Demak Direction (Overloaded Condition)

STA	SN3	SN2	SN1	D1 (in.)	D2 (in.)	D3 (in.)
(26+900)	10.98	8.75	7.03	22	4	11
(28+350)	13.84	11.27	9.26	29	4	10
(30+000)	10.00	8.75	7.03	21	4	6
(32+100)	28.81	21.81	18.20	64	4	15
(33+300)	8.18	7.27	5.69	17	4	5
(34+700)	9.50	8.52	6.83	20	4	5
(35+700)	11.69	10.67	8.73	26	4	4
(36+300)	12.46	10.67	8.73	27	4	8
AVERAGE	13.18	10.96	8.94	28.25	4.00	8.00

It can be seen from the tables above that there are significant differences between the structural number and thickness of each layer for two conditions (standard and overloaded). The summary of SN and thickness calculation is shown in Table 4.15 below.

Table 4.15: Summary of SN and Thickness Calculation

	Average SN_3	Average D_1 (in.)
Demak - Trengguli Direction		
Standard condition	4.73	8
Overloaded condition	13.52	29.38
Deviation	8.79	21.38
Trengguli – Demak Direction		
Standard condition	4.76	8
Overloaded condition	13.18	28.5
Deviation	8.42	20.5

From Table 4.15, it can be seen that there is a significant difference between structural number SN_3 and thickness of surface layer (D_1) for two conditions, i.e. standard load and overloaded conditions. It is interesting to know that the difference between the structural number and surface layer thickness for two directions was not too much, although the traffic load of Demak-Trengguli direction was 2.5 times higher than that of opposite direction. This could be contributed by the ability of the pavement structure of Demak-Trengguli direction to withstand load was higher than that of the opposite direction (see Tables 4.8 and 4.9).

CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

From the analysis of this study on "Identification of Damage due overloading on Demak-Treangguli Road”, it can be concluded that:

a. The average daily traffic (ADT) of heavy vehicle for Demak-Trengguli and Trengguli-Demak directions are 2,025 and 2,144 vehicles/day, respectively. It means that the ADT for Trengguli- Demak direction is more than the opposite direction

b. The average of vehicle damage factor VDF for Demak-Trengguli and Trengguli-Demak directions are 98.10 and 46.48 ESAL, respectively. It means that the heavy vehicles on Demak-Trengguli direction bring heavier goods than that of the opposite direction.

c. The equivalent standard axle load of heavy vehicle for Demak-Trengguli and Trengguli-Demak directions are about 100.85 and 40.38 million.ESAL, respectively.

d. Because of overloaded heavy vehicles, the service of life Demak-Trengguli direction is reducing from the original design 10 to 3 years, so that there is service life loss of 7 years. The similar condition is also encountered in the opposite direction Trengguli-Demak direction, where the service life reduces from 10 to 6.5 years, showing the service life loss of 4 years.

e. From the comparison of service life loss between both directions, it is very clear that the pavement layers of Demak-Trengguli direction needs more overlay thickness than that of the opposite direction.

5.2 Recommendations

From the conclusions mentioned above, there is a given suggestion to be considered or perhaps to be followed by some improvements, namely they are:

1. It needs a good coordination by the Public Works Department regarding the data archiving project, both designing and implementation, so that if one day data needed, can be reused in the best way.

2. It is necessary to evaluate the condition of the design with reality at the beginning of the design life.
3. It is recommended that all data should be measured in the same day
4. The use of primary data, instead of secondary data, in calculation is more recommended.

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Appendix A Traffic Data for Demak-Trengguli Direction

			VEHICLE CLASS VERSION Highways										
Day	**hour**	**Direction**	**type 1**	**type 2**	**type 3**	**type 4**	**type 5 A**	**type 5 B**	**type 6 A**	**type 6 B**	**type 7 A**	**type 7B**	**type 7 C**
Demak - Trengguli													
1	18	N	281	215	203	163	26	1	34	43	59	10	35
1	19	N	237	198	188	150	25	4	34	31	47	5	27
1	20	N	107	219	207	165	23	1	30	35	38	5	14
1	21	N	187	156	148	118	22	0	29	29	36	6	17
1	22	N	179	148	140	112	29	0	39	44	22	2	16
1	23	N	115	96	91	73	19	2	25	32	25	2	12
			1,106	1,032	976	781	144	8	191	214	227	30	121
2	00	N	59	63	60	48	21	0	28	29	23	1	9
2	01	N	23	48	45	36	22	1	30	25	21	0	12
2	02	N	26	37	35	28	10	2	14	28	38	3	6
2	03	N	34	30	28	22	22	1	29	35	24	0	7
2	04	N	79	50	47	38	28	3	38	39	31	4	6
2	05	N	193	107	101	81	32	3	43	40	29	2	10
2	06	N	329	138	131	105	37	3	48	34	31	1	17
2	07	N	263	167	158	127	29	1	39	40	32	4	15
2	08	N	299	235	223	178	32	2	42	28	35	2	6
2	09	N	399	275	260	208	30	0	39	35	31	4	10
2	10	N	364	271	256	205	27	2	35	57	35	7	16
2	11	N	464	253	239	192	34	1	44	57	41	9	15
2	12	N	458	196	186	149	29	3	39	41	32	7	14
2	13	N	233	320	303	242	26	1	35	37	22	7	11
2	14	N	378	267	253	202	27	1	36	42	31	5	22
2	15	N	359	303	286	229	25	2	33	48	31	10	21
2	16	N	456	305	288	230	27	4	35	65	39	10	17
2	17	N	355	324	306	245	20	3	26	45	27	3	9
2	18	N	284	210	199	159	23	2	30	49	36	5	12
2	19	N	248	204	193	155	18	0	25	44	35	1	16
2	20	N	254	211	199	159	16	1	21	52	36	6	15
2	21	N	192	210	198	159	22	2	29	48	41	5	20
2	22	N	180	162	154	123	23	1	31	43	41	6	12
2	23	N	77	83	78	63	25	0	32	39	24	2	13
		Result ADT	6,006	4,468	4,227	3,381	604	39	801	1,000	766	104	311
3	00	N	48	54	51	41	14	0	19	46	24	1	2
3	01	N	37	40	38	31	17	1	22	30	22	1	5
3	02	N	20	28	26	21	18	0	23	48	20	0	5
3	03	N	42	34	32	25	22	0	30	36	19	2	6
3	04	N	101	63	59	47	29	0	38	26	27	2	11
3	05	N	366	149	141	113	34	0	45	36	25	2	12
3	06	N	542	272	257	206	30	1	40	23	26	1	9
3	07	N	533	288	272	218	34	1	46	34	22	3	7
3	08	N	479	208	196	157	27	0	35	34	16	2	8
3	09	N	439	246	232	186	46	1	61	38	19	7	8
3	10	N	365	272	258	206	44	1	58	44	22	4	15
3	11	N	399	287	272	217	49	2	66	36	34	9	15
3	12	N	359	262	248	199	48	5	64	44	26	9	16
3	13	N	389	264	250	200	41	0	54	32	24	7	15
3	14	N	452	233	220	176	43	2	56	48	31	8	20
3	15	N	433	249	235	188	40	0	53	36	34	3	17
3	16	N	462	274	259	207	40	0	53	34	24	3	13
3	17	N	462	230	217	174	33	1	44	37	19	3	9
3	18	N	269	252	239	191	27	2	35	37	30	6	18
3	19	N	247	184	174	139	25	2	34	32	45	6	30
3	20	N	254	139	131	105	18	0	25	34	47	4	15
3	21	N	209	134	127	102	18	0	25	26	21	1	9
3	22	N	174	99	94	75	22	0	29	25	34	4	19
3	23	N	87	80	76	60	23	0	30	35	36	2	9
		Result ADT	7,168	4,340	4,106	3,285	742	19	984	851	647	90	293

			VEHICLE CLASS VERSION Highways										
Day	**hour**	**Direction**	**type 1**	**type 2**	**type 3**	**type 4**	**type 5 A**	**type 5 B**	**type 6 A**	**type 6 B**	**type 7 A**	**type 7B**	**type 7 C**
Demak - Trengguli													
4	00	N	53	65	62	49	23	0	31	29	26	1	5
4	01	N	40	29	28	22	28	0	36	24	22	1	5
4	02	N	31	30	28	22	27	0	36	25	22	0	4
4	03	N	32	44	41	33	41	0	54	55	32	1	6
4	04	N	42	72	68	55	39	1	51	37	36	0	4
4	05	N	218	124	117	94	48	2	64	37	35	2	9
4	06	N	419	213	201	161	56	2	74	22	35	7	23
4	07	N	425	240	227	181	37	1	50	26	24	0	8
4	08	N	476	238	225	180	38	1	51	26	15	4	10
4	09	N	451	264	250	200	49	3	66	38	29	3	17
4	10	N	385	279	264	211	70	1	93	28	40	4	19
4	11	N	337	295	279	223	55	0	74	44	35	2	19
4	12	N	451	200	189	151	48	2	63	52	45	3	15
4	13	N	508	218	206	165	41	2	54	33	51	3	16
4	14	N	486	200	189	151	44	1	59	41	33	5	21
4	15	N	427	208	197	158	41	2	55	52	65	6	14
4	16	N	498	273	258	206	45	3	59	45	67	10	25
4	17	N	447	346	328	262	30	1	40	53	26	3	10
4	18	N	263	235	223	178	27	1	36	57	45	9	16
4	19	N	189	233	220	176	31	1	40	43	63	4	32
4	20	N	258	160	151	121	34	1	44	46	69	3	26
4	21	N	193	193	183	146	26	0	34	28	41	2	19
4	22	N	193	140	132	106	30	1	39	44	31	4	22
4	23	N	91	83	79	63	20	0	27	27	44	1	19
		Result ADT	6,913	4,381	4,144	3,315	928	26	1,230	912	931	78	364
5	00	N	43	68	64	51	21	0	28	43	41	2	15
5	01	N	43	43	40	32	28	2	36	31	31	1	9
5	02	N	38	32	30	24	22	0	30	34	13	0	11
5	03	N	43	36	34	27	33	1	44	32	34	1	8
5	04	N	93	45	43	34	32	0	42	39	30	2	4
5	05	N	256	131	124	99	43	0	58	49	40	1	13
5	06	N	361	244	231	185	42	1	56	39	48	14	19
5	07	N	429	229	217	173	42	1	56	38	28	4	10
5	08	N	479	203	192	153	40	1	54	35	18	2	12
5	09	N	372	309	292	234	43	1	58	24	36	6	8
5	10	N	323	295	279	223	70	1	92	45	37	10	11
5	11	N	190	306	289	231	46	1	62	36	31	8	6
		Result ADT	6,696	4,396	4,159	3,327	758	28	1,005	921	781	91	323

Appendix A Traffic Data for Trengguli- Demak Direction

			VEHICLE CLASS VERSION Highways										
Day	**hour**	**Direction**	**type 1**	**type 2**	**type 3**	**type 4**	**type 5 A**	**type 5 B**	**type 6 A**	**type 6 B**	**type 7 A**	**type 7B**	**type 7 C**
Trengguli-Demak													
1	18	O	334	188	178	143	30	0	40	46	27	15	20
1	19	O	296	151	142	114	28	2	37	71	39	12	13
1	20	O	261	105	100	80	34	1	45	33	35	1	24
1	21	O	180	89	84	67	39	2	51	42	49	3	19
1	22	O	107	77	73	59	21	1	27	47	38	2	18
1	23	O	65	54	51	41	28	0	38	40	49	2	23
			1,243	665	629	503	180	6	238	279	237	35	117
2	00	O	49	36	34	27	22	2	29	33	35	2	18
2	01	O	36	33	32	25	20	0	27	31	44	1	12
2	02	O	32	29	27	22	20	0	27	24	28	3	17
2	03	O	46	35	33	27	15	1	20	28	48	2	10
2	04	O	102	51	48	39	15	0	20	37	45	7	6
2	05	O	303	106	100	80	22	1	29	51	44	1	7
2	06	O	434	169	160	128	25	1	34	59	48	9	16
2	07	O	455	220	208	167	31	1	40	45	48	9	22
2	08	O	500	250	237	190	39	2	52	43	60	8	19
2	09	O	267	352	333	266	24	3	32	39	36	14	15
2	10	O	472	250	237	190	33	2	43	37	48	17	23
2	11	O	430	212	201	161	32	1	42	38	64	3	15
2	12	O	425	176	167	133	30	5	39	34	34	6	23
2	13	O	502	204	193	155	28	2	37	23	33	8	20
2	14	O	557	220	208	166	31	3	41	34	31	9	15
2	15	O	585	242	229	183	36	1	47	52	33	8	15
2	16	O	769	292	276	221	30	2	39	36	38	7	16
2	17	O	403	136	129	103	16	2	21	29	18	7	7
2	18	O	284	210	199	159	23	3	30	44	43	7	13
2	19	O	248	204	193	155	18	1	25	39	42	4	17
2	20	O	254	211	199	159	16	2	21	47	43	7	16
2	21	O	192	210	198	159	22	3	29	43	48	5	21
2	22	O	180	162	154	123	23	2	31	38	48	6	13
2	23	O	77	83	78	63	25	1	32	34	31	3	14
		ADT	7,602	4,094	3,873	3,098	595	39	788	918	990	153	370
3	00	O	48	54	51	41	14	1	19	41	31	4	3
3	01	O	37	40	38	31	17	2	22	25	29	3	6
3	02	O	20	28	26	21	18	1	23	43	27	5	6
3	03	O	42	34	32	25	22	1	30	31	26	4	7
3	04	O	101	63	59	47	29	1	38	21	34	3	12
3	05	O	366	149	141	113	34	1	45	31	32	4	13
3	06	O	542	272	257	206	30	2	40	18	33	4	12
3	07	O	533	288	272	218	34	2	46	29	29	5	10
3	08	O	479	208	196	157	27	1	35	29	23	4	11
3	09	O	439	246	232	186	46	2	61	33	26	8	9
3	10	O	365	272	258	206	44	2	58	39	29	6	18
3	11	O	399	287	272	217	49	3	66	31	41	10	16
3	12	O	359	262	248	199	48	6	64	39	33	9	17
3	13	O	389	264	250	200	41	1	54	27	31	7	16
3	14	O	452	233	220	176	43	3	56	43	38	8	21
3	15	O	433	249	235	188	40	1	53	31	41	3	18
3	16	O	462	274	259	207	40	1	53	29	31	4	14
3	17	O	462	230	217	174	33	2	44	32	26	7	10
3	18	O	269	252	239	191	27	3	35	32	37	7	19
3	19	O	247	184	174	139	25	3	34	27	52	8	31
3	20	O	254	139	131	105	18	1	25	29	54	6	16
3	21	O	209	134	127	102	18	1	25	21	28	3	10
3	22	O	174	99	94	75	22	1	29	20	41	3	20
3	23	O	87	80	76	60	23	1	30	30	43	3	10
		Result ADT	7,168	4,340	4,106	3,285	742	36	984	731	815	128	325

			VEHICLE CLASS VERSION Highways										
Day	hour	Direction	type 1	type 2	type 3	type 4	type 5 A	type 5 B	type 6 A	type 6 B	type 7 A	type 7B	type 7 C
Trengguli-Demak													
4	00	O	53	65	62	49	23	1	31	24	33	3	6
4	01	O	40	29	28	22	28	1	36	19	29	4	6
4	02	O	31	30	28	22	27	1	36	20	29	3	5
4	03	O	32	44	41	33	41	1	54	50	39	5	9
4	04	O	42	72	68	55	39	2	51	32	43	1	5
4	05	O	218	124	117	94	48	3	64	32	42	3	10
4	06	O	419	213	201	161	56	3	74	17	42	7	24
4	07	O	425	240	227	181	37	2	50	21	31	4	9
4	08	O	476	238	225	180	38	2	51	21	22	7	11
4	09	O	451	264	250	200	49	4	66	33	36	4	20
4	10	O	385	279	264	211	70	2	93	23	47	6	20
4	11	O	337	295	279	223	55	1	74	39	42	3	20
4	12	O	451	200	189	151	48	3	63	47	52	4	16
4	13	O	508	218	206	165	41	3	54	28	58	4	17
4	14	O	486	200	189	151	44	2	59	36	40	6	22
4	15	O	427	208	197	158	41	3	55	47	72	7	15
4	16	O	498	273	258	206	45	4	59	40	74	8	26
4	17	O	447	346	328	262	30	2	40	48	33	4	11
4	18	O	263	235	223	178	27	2	36	52	52	10	17
4	19	O	189	233	220	176	31	2	40	38	70	5	33
4	20	O	258	160	151	121	34	2	44	41	76	4	27
4	21	O	193	193	183	146	26	1	34	23	48	5	20
4	22	O	193	140	132	106	30	2	39	39	38	5	23
4	23	O	91	83	79	63	20	1	27	22	51	2	20
		Result ADT	6,913	4,381	4,144	3,315	928	43	1,230	792	1,099	114	392
5	00	O	43	68	64	51	21	1	28	38	48	3	16
5	01	O	43	43	40	32	28	3	36	26	38	2	10
5	02	O	38	32	30	24	22	1	30	29	20	3	12
5	03	O	43	36	34	27	33	2	44	27	41	3	9
5	04	O	93	45	43	34	32	1	42	34	37	6	7
5	05	O	256	131	124	99	43	1	58	44	47	2	14
5	06	O	361	244	231	185	42	2	56	34	55	14	20
5	07	O	429	229	217	173	42	2	56	33	35	5	13
5	08	O	479	203	192	153	40	2	54	30	25	4	13
5	09	O	372	309	292	234	43	2	58	19	43	9	11
5	10	O	323	295	279	223	70	2	92	40	44	11	12
5	11	O	190	306	289	231	46	2	62	31	38	9	7
		Result ADT	**2,670**	**1,940**	**1,835**	**1,468**	**464**	**17**	**614**	**385**	**471**	**71**	**144**

Appendix B for International Roughness' Index Normal Demak-Trengguli

Normal (Demak-Trengguli)

SECTION ID	SUBDISTANCE	TOTALDISTANCE		IRI	EVENT	GOOD	MUDIUM	POOR	VERY POOR
0	0	0		0	START				
1	0.1	0.1		7.2			0.1		
1	0.2	0.2		3.1		0.1			
1	0.3	0.3		2.3		0.1			
1	0.4	0.4		2		0.1			
1	0.5	0.5		2.5		0.1			
1	0.504	0.504		2		0.004			
2	0.1	0.604		2.3		0.1			
2	0.2	0.704		2.5		0.1			
2	0.3	0.804		2.4		0.1			
2	0.4	0.904		3.2		0.1			
2	0.5	1.004		2.8		0.1			
2	0.505	1.01		2.6		0.006			
3	0.1	1.11		2.5		0.1			
3	0.184	1.194		4.8			0.084		
4	0.1	1.294		4.5			0.1		
4	0.2	1.394		2.2		0.1			
			Average	3.06					
4	0.3	1.494		4.2		0.1			
4	0.309	1.503		1.4		0.009			
5	0.1	1.603		1.9		0.1			
5	0.2	1.703		1.8		0.1			
5	0.3	1.803		2.9		0.1			
5	0.4	1.903		2.4		0.1			
5	0.5	2.003		2.3		0.1			
5	0.501	2.004		1.9		0.001			
6	0.025	2.029		2.7		0.025			
7	0.1	2.129		2.3		0.1			
7	0.2	2.229		1.6		0.1			
7	0.3	2.329		2.1		0.1			
7	0.4	2.429		1.8		0.1			
7	0.473	2.502		1.7		0.073			
8	0.1	2.602		2.4		0.1			
8	0.2	2.702		2.1		0.1			
8	0.3	2.802		2.7		0.1			
8	0.4	2.902		3		0.1			
8	0.5	3.002		4.6			0.1		
9	0.1	3.102		3.1		0.1			
			Average	2.445					
9	0.154	3.156		3.4		0.054			
10	0.1	3.256		6.3			0.1		
10	0.2	3.356		3.9		0.1			
10	0.3	3.456		3.7		0.1			
10	0.346	3.502		2.6		0.046			
11	0.1	3.602		3.8		0.1			
11	0.2	3.702		3.3		0.1			
11	0.3	3.802		3.1		0.1			
11	0.4	3.902		2.7		0.1			
11	0.5	4.002		3.7		0.1			
11	0.509	4.011		4.9			0.009		
12	0.005	4.016		10.4					
13	0.1	4.116		5.8			0.1		
13	0.2	4.216		2.8		0.1			
13	0.3	4.316		2.3		0.1			
13	0.4	4.416		2.7		0.1			
13	0.486	4.502		3.7		0.086			
14	0.1	4.602		6.7			0.1		
14	0.2	4.702		5			0.1		
			Average	3.30					

Normal (Demak-Trengguli)

SECTION ID	SUBDISTANCE	TOTALDISTANCE		IRI	EVENT	GOOD	MEDIUM	POOR	VERY POOR
14	0.3	4.802		3.5		0.1			
14	0.4	4.902		4.4			0.1		
14	0.498	5		2.9		0.098			
15	0.1	5.1		2.5		0.1			
15	0.2	5.2		3.3		0.1			
15	0.3	5.3		4		0.1			
15	0.4	5.4		3.9		0.1			
15	0.5	5.5		5.5			0.1		
16	0.1	5.6		6.5			0.1		
16	0.2	5.7		2		0.1			
16	0.3	5.8		3.4		0.1			
16	0.4	5.9		4.3			0.1		
16	0.499	5.999		3.7		0.099			
17	0.1	6.099		3.6		0.1			
17	0.2	6.199		3.6		0.1			
17	0.247	6.246		9.6	FINISH				
			Average	4.16875					
	TOTAL	6.246	RATA2	3.38		5.001	1.193	0.052	0

Normal (Demak-Trengguli)

SECTION ID	SUBDISTANCE	TOTALDISTANCE		IRI	EVENT	GOOD	MEDIUM	POOR	VERY POOR
1	0.1	0.1		8.2					0.1
1	0.2	0.2		4.1			0.1		
1	0.3	0.3		3.3		0.1			
1	0.4	0.4		3		0.1			
1	0.5	0.5		4.1			0.1		
1	0.504	0.504		1.8		0.004			
2	0.1	0.604		3.5		0.1			
2	0.2	0.704		4.2			0.1		
2	0.3	0.804		2.7		0.1			
2	0.349	0.843		6.2			0.05		
3	0.1	0.954		4.7			0.1		
3	0.2	1.054		3.2		0.1			
3	0.3	1.154		2.5		0.1			
			Average	3.96					

Normal (Demak-Trengguli)

SECTION ID	SUBDISTANCE	TOTALDISTANCE		IRI	EVENT	GOOD	MEDIUM	POOR	VERY POOR
3	0.4	1.254		2.9		0.1			
3	0.5	1.354		2.9		0.1			
3	0.6	1.454		3.9		0.1			
3	0.7	1.554		5.1			0.1		
3	0.8	1.654		2.6		0.1			
3	0.9	1.754		1.9		0.1			
3	1	1.854		1.9		0.1			
3	1.1	1.954		2.5		0.1			
3	1.2	2.054		2.7		0.1			
3	1.3	2.154		3.2		0.1			
3	1.4	2.254		3.2		0.1			
			Average	2.98					
3	1.5	2.354		3.4		0.1			
3	2.6	2.454		3.7		0.1			
3	1.651	2.505		2.3		0.051			
4	0.1	2.605		3.7		0.1			
4	0.2	2.705		2.3		0.1			
4	0.3	2.805		2.6		0.1			
4	0.4	2.905		2.8		0.1			
4	0.497	3.002		3.5		0.097			
5	0.082	3.084		5			0.082		
			Average	3.26					
	TOTAL	3.084	RATA2	3.34		2.352	0.632	0.1	0

Appendix B for International Roughness' Index Opposite Trengguli -Demak

opposite (trengguli-Demak)

SECTION ID	SUBDISTANCE	TOTALDISTANCE		IRI	EVENT	GOOD	MEDIUM	POOR	VERY POOR
0	0	0		0	start				
1	0.1	0.1		7					
1	0.2	0.2		2.4		0.1			
1	0.3	0.3		2.9		0.1			
1	0.4	0.4		2.8		0.1			
1	0.5	0.5		2.2		0.1			
1	0.511	0.511		2.3		0.011			
2	0.1	0.611		3.9		0.1			
2	0.2	0.711		2.9		0.1			
2	0.3	0.811		3.4		0.1			
2	0.4	0.911		3.3		0.1			
2	0.498	1.009		2.7		0.098			
3	0.1	1.109		4.1		0.1			
3	0.2	1.209		2.8			0.1		
3	0.3	1.309		3.3		0.1			
			0	3.285714					
3	0.4	1.409		4.1		0.1			
3	0.494	1.503		2.8			0.094		
4	0.1	1.603		2.4		0.1			
4	0.2	1.703		2.7		0.1			
4	0.3	1.803		3.5		0.1			
4	0.4	1.903		2.5		0.1			
4	0.5	2.003		1.2		0.1			
4	0.513	2.016		2.8		0.013			
5	0.1	2.116		3.9		0.1			
5	0.2	2.216		3.1		0.1			
5	0.202	2.218		4.8		0.002			
6	0.1	2.318		3.6			0.1		
6	0.2	2.418		2.6		0.1			
6	0.286	2.504		3.1		0.086			
7	0.1	2.604		2.5		0.1			
7	0.2	2.704		2.8		0.1			
7	0.3	2.804		5		0.1			
7	0.4	2.904		4.2		0.1			
7	0.5	3.004		5			0.1		
7	0.545	3.05		4.2			0.046		
			0	3.34					
8	0.1	3.15		4.6			0.1		
8	0.2	3.25		5.6			0.1		
8	0.3	3.35		3.2		0.1			
8	0.4	3.45		5.1			0.01		
8	0.454	3.504		3.6		0.054			
9	0.1	3.604		2.5		0.1			
9	0.2	3.704		1.8		0.1			
9	0.3	3.804		2.5		0.1			
9	0.4	3.904		1.9		0.1			
9	0.5	4.004		1.9		0.1			
9	0.503	4.007		0.7		0.003			
10	0.1	4.107		1.5		0.1			
10	0.2	4.207		1.6		0.1			
10	0.247	4.254		2.4		0.047			
11	0.1	4.354		2.8		0.1			
11	0.2	4.454		1.6		0.1			
11	0.3	4.554		3.4		0.1			
11	0.4	4.654		1.8		0.1			
11	0.5	4.754		1.3		0.1			
11	0.6	4.854		1.5		0.1			
11	0.7	4.954		2.2		0.1			
11	0.8	5.054		3.1		0.1			
11	0.808	5.061		11				0.007	
12	0.1	5.161		6			0.1		
			Average	3.066667					

SECTION ID	SUBDISTANCE	TOTALDISTANCE		IRI	EVENT	GOOD	MEDIUM	POOR	VERY POOR
12	0.2	5.261		2.1		0.1			
12	0.3	5.361		2.7		0.1			
12	0.4	5.461		5.9			0.1		
12	0.404	5.466		7.8			0.005		
13	0.1	5.566		4.2			0.1		
13	0.2	5.666		4.4			0.1		
13	0.3	5.766		3.1		0.1			
13	0.4	5.866		2		0.1			
13	0.5	5.966		3.8		0.1			
13	0.533	5.998		3.9		0.032			
14	0.1	6.098		3.1		0.1			
14	0.2	6.198		3.4		0.1			
14	0.286	6.284		2.1		0.086			
15	0.1	6.384		7.1			0.1		
			Average	3.971429					
15	0.2	6.484		3.2		0.1			
15	0.3	6.584		2.3		0.1			
15	0.4	6.684		2.6		0.1			
15	0.5	6.784		2.6		0.1			
15	0.504	6.788		2.1		0.004			
16	0.1	6.888		2.5		0.1			
16	0.2	6.988		2.5		0.1			
16	0.3	7.088		4.3			0.1		
16	0.4	7.188		3		0.1			
16	0.499	7.287		3.3		0.1			
17	0.1	7.387		2.1		0.1			
17	0.2	7.487		2.7		0.1			
17	0.3	7.587		1.9		0.1			
17	0.4	7.687		2.2		0.1			
17	0.5	7.787		3.5		0.1			
			Average	2.72					
17	0.522	7.809		2.6		0.022			
18	0.1	7.909		2.9		0.1			
18	0.2	8.009		2.7		0.1			
18	0.3	8.109		2.3		0.1			
18	0.4	8.209		3.2		0.1			
18	0.476	8.285		2.9		0.076			
19	0.1	8.385		2.8		0.1			
19	0.2	8.485		3.2		0.1			
19	0.231	8.516		3.3		0.031			
20	0.1	8.616		4.6			0.1		
20	0.2	8.716		3.1		0.1			
20	0.27	8.786		2.8		0.07			
			Average	3.033333					
21	0.1	8.886		2.9		0.1			
21	0.2	8.986		3.3		0.1			
21	0.3	9.086		2.9		0.1			
21	0.4	9.186		2.7		0.1			
21	0.497	9.283		4.5			0.097		
22	0.071	9.354		3.5		0.072			
			Average	3.3					
	TOTAL	497.301	RATA2	3.34		2.352	5.102	0.007	0

Appendix C Data of California Bearing Ration (CBR)

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Dynamic Cone Penetration Test (Sub Grade)

Proyek	:	Peningkatan Jalan
Ruas	:	Demak - Kudus
		KM. SMG. 26+900 Kiri

Lokasi	:	TP. 1 Demak - Kudus
Teknisi	:	Tatang
Tanggal	:	21/11/2009

Jumlah Tumbukan Kumulatif	Penetrasi Kumulatif (mm)
0	0
1	40
3	120
5	190
10	260
15	370
20	550
25	770
29	1000

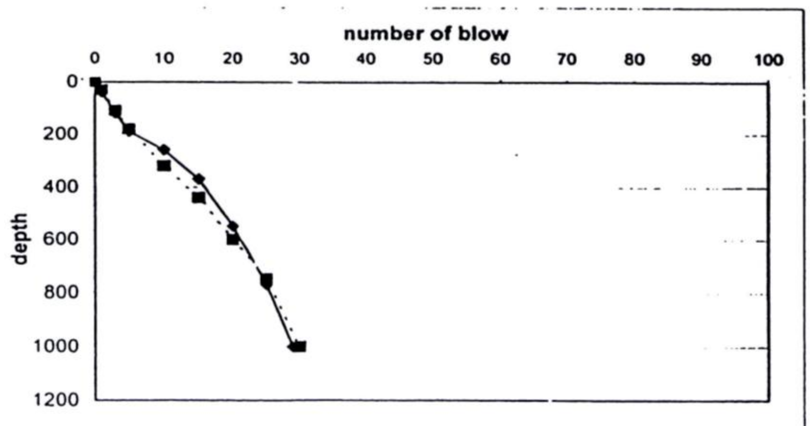
Jumlah Tumbukan Kumulatif	Penetrasi Kumulatif (mm)
0	0
1	30
3	110
5	180
10	320
15	440
20	600
25	750
30	1000

Test Result

DCP (mm/blow)	53,8	57,5
		4,17
CBR (%)	4,5	

50,0	
4,83	

Kedalaman DCP : -0,50 meter



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Dynamic Cone Penetration Test (Sub Grade)

Proyek :	Peningkatan Jalan	Lokasi :	TP. 2 Demak - Kudus
Ruas :	Demak - Kudus	Teknisi :	Tatang
	KM. SMG. 28+350 Kanan	Tanggal :	21/11/2009

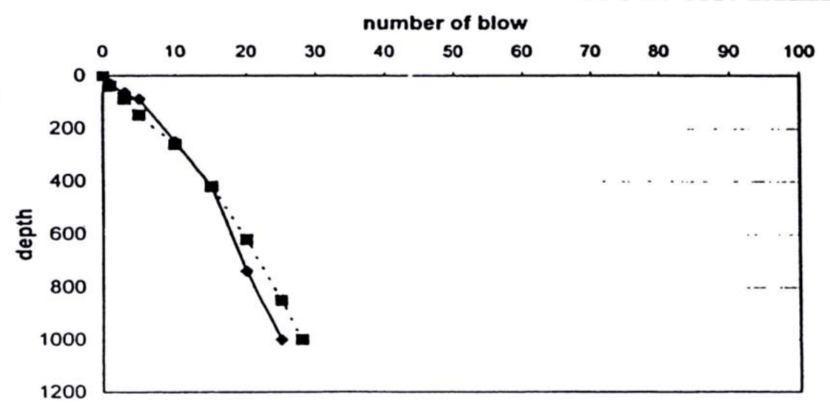
Jumlah Tumbukan Kumulatif	Penetrasi Kumulatif (mm)
0	0
1	30
3	65
5	90
10	250
15	420
20	740
25	1000

Jumlah Tumbukan Kumulatif	Penetrasi Kumulatif (mm)
0	0
1	40
3	90
5	150
10	260
15	420
20	620
25	850
28	1000

Test Result

DCP (mm/blow)	51,0	52,0	50,0	
		4,64	4,83	
CBR (%)	4,7			

Kedalaman DCP : -0,80 meter



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Dynamic Cone Penetration Test (Sub Grade)

Proyek : Peningkatan Jalan	Lokasi : TP. 4 Demak - Kudus
Ruas : Demak - Kudus	Teknisi : Tatang
KM. SMG. 32+100 Tengah	Tanggal : 23/11/2009

Jumlah Tumbukan Kumulatif	Penetrasi Kumulatif (mm)
0	0
1	70
3	190
5	330
10	590
15	860
18	1000

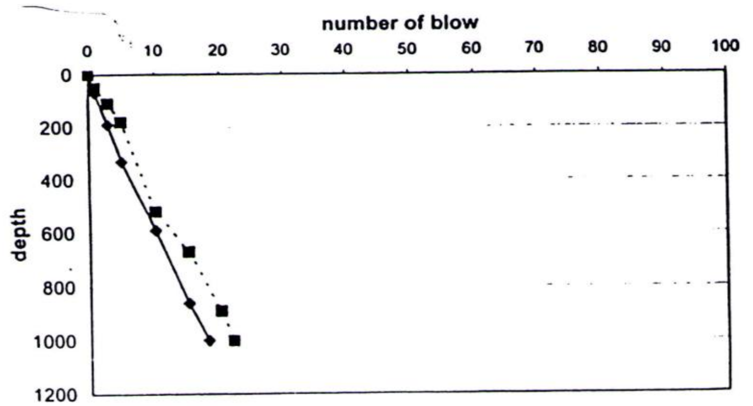
Jumlah Tumbukan Kumulatif	Penetrasi Kumulatif (mm)
0	0
1	50
3	110
5	180
10	520
15	670
20	890
22	1000

Test Result

DCP (mm/blow)	69,0	70,0
		3,39
CBR (%)	3,4	

68,0	
3,49	

Kedalaman DCP : -1,10 meter



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Dynamic Cone Penetration Test (Sub Grade)

Proyek	:	Peningkatan Jalan
Ruas	:	Demak - Kudus
		KM. SMG. 33+300 Kiri

Lokasi	:	TP. 5
		Demak - Kudus
Teknisi	:	Tatang
Tanggal	:	23/11/2009

Jumlah Tumbukan Kumulatif	Penetrasi Kumulatif (mm)
0	0
1	35
3	90
5	130
10	220
15	290
20	330
25	390
30	400
35	425
40	440
45	470
50	530
55	590
60	690
65	790
70	900
75	1000

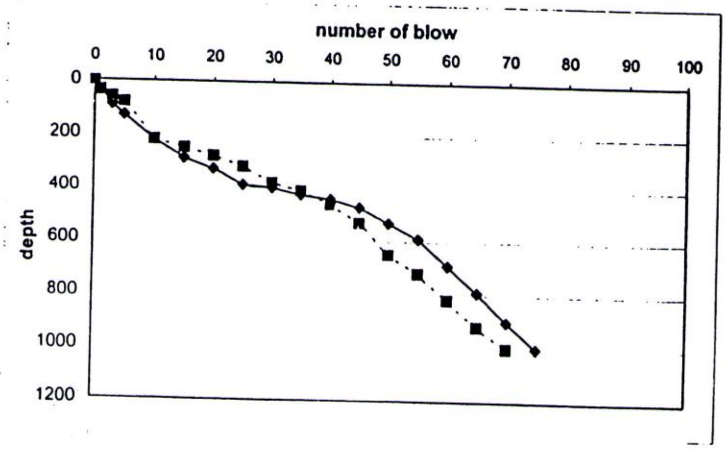
Jumlah Tumbukan Kumulatif	Penetrasi Kumulatif (mm)
0	0
1	37
3	60
5	80
10	220
15	250
20	280
25	320
30	380
35	410
40	460
45	530
50	650
55	720
60	820
65	920
70	1000

Test Result

DCP (mm/blow)	36,0	35,0
		7,05
CBR (%)	6,8	

37,0	
6,64	

Kedalaman DCP : -0,65 meter



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Dynamic Cone Penetration Test (Sub Grade)

Proyek	:	Peningkatan Jalan
Ruas	:	Demak - Kudus
		KM. SMG. 34+450 Kiri

Lokasi	:	TP. 6 Demak - Kudus
Teknisi	:	Tatang
Tanggal	:	23/11/2009

Jumlah Tumbukan Kumulatif	Penetrasi Kumulatif (mm)
0	0
1	35
3	85
5	120
10	190
15	310
20	470
25	590
30	690
35	850
40	1000

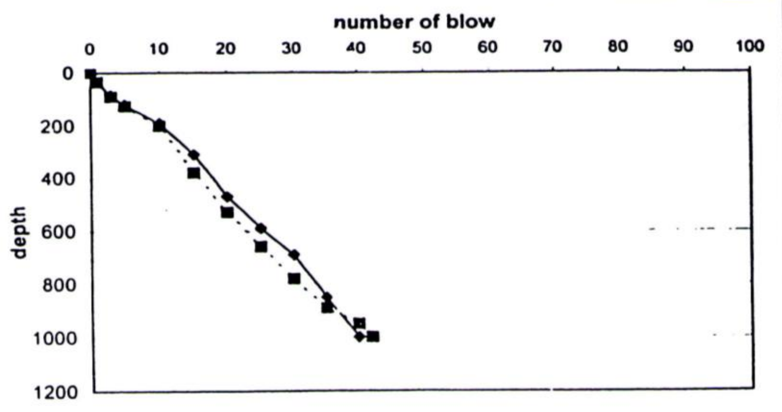
Jumlah Tumbukan Kumulatif	Penetrasi Kumulatif (mm)
0	0
1	35
3	90
5	125
10	200
15	380
20	530
25	660
30	780
35	890
40	950
42	1000

Test Result

DCP (mm/blow)	35,5	35,0
		7,05
CBR (%)	6,9	

36,0	
6,84	

Kedalaman DCP : -0,65 meter



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Dynamic Cone Penetration Test (Sub Grade)

Proyek	:	Peningkatan Jalan
Ruas	:	Demak - Kudus
		KM. SMG. 35+700 Tengah

Lokasi	:	TP. 7 Demak - Kudus
Teknisi	:	Tatang
Tanggal	:	23/11/2009

Jumlah Tumbukan Kumulatif	Penetrasi Kumulatif (mm)
0	0
1	30
3	40
5	50
10	85
15	180
20	280
25	320
30	370
35	520
40	670
45	840
50	1000

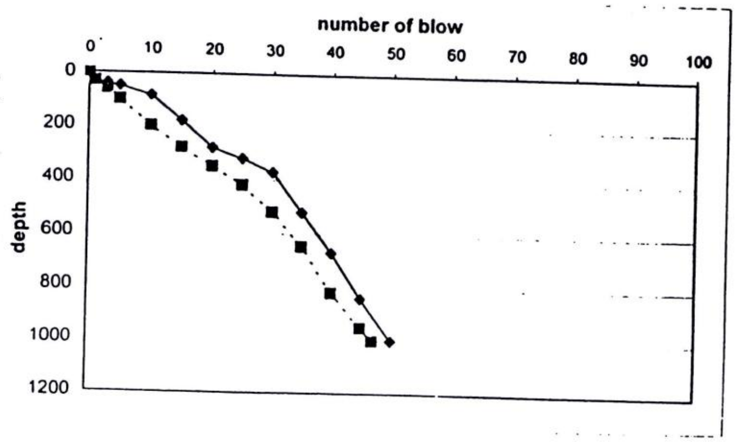
Jumlah Tumbukan Kumulatif	Penetrasi Kumulatif (mm)
0	0
1	30
3	60
5	100
10	200
15	280
20	350
25	420
30	520
35	648
40	820
45	950
47	1000

Test Result

DCP (mm/blow)	34,2	34,0
		7,26
CBR (%)	7,2	

34,4	
7,18	

Kedalaman DCP : -1,20 meter



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Dynamic Cone Penetration Test (Sub Grade)

Proyek :	Peningkatan Jalan
Ruas	Demak - Kudus
	KM. SMG. 36+300 Kanan

Lokasi :	TP. 8 Demak - Kudus
Teknisi :	Tatang
Tanggal :	23/11/2009

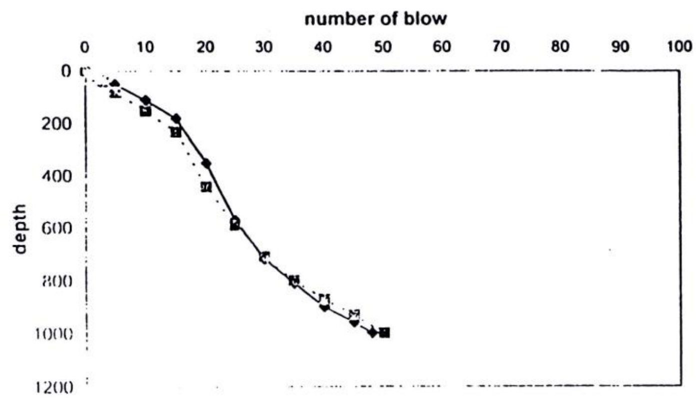
Jumlah Tumbukan Kumulatif	Penetrasi Kumulatif (mm)
0	0
1	20
3	35
5	50
10	110
15	180
20	350
25	570
30	720
35	810
40	900
45	960
48	1000

Jumlah Tumbukan Kumulatif	Penetrasi Kumulatif (mm)
0	0
1	20
3	40
5	80
10	150
15	230
20	440
25	590
30	710
35	800
40	870
45	930
50	1000

Test Result

DCP (mm/blow)	43,0	44,0	42,0	
		5,53	5,81	
CBR (%)	5,7			

Kedalaman DCP : -1,00 meter



APPENDIX D
EXAMPLES OF CALCULATION

D.1 Calculation of Vehicle Damage Factor (VDF)

In this section, the calculation of VDF for 6B-class truck is presented. In this study, the calculation of VDF was performed based on axle load equivalency factor (LEF) tables with p_t = 2, i.e. Tables 4.2-4.4. The 6B-class truck has one single axle on the front and rear. The detail calculation is as follows

- Axle load on the front : 4,965 ton
- Axle load on the rear : 5,193 ton

To enable using the axle load in 1994 AASHTO axle load equivalency factor (LEF) tables (see Tables 4.2-4.4), it is necessary to change the unit of the axle load parameter, from ton to kips, by multiplying the value of axle load (in ton) with a constant 0.002206. This result in 10.95 and 11.46 kips for front and rear axle loads, respectively. Since there is only one type of axle load in this calculation, that is, single axle load, therefore only Table 4.2 was used.

In this table, it not possible to find axle load equals to 10.95 and 11.46 kips in the first column; therefore an interpolation is required by interpolating the axle load values between 10.95 and 11.46 kips, and its corresponding LEF values (under structural number SN = 5) 0.079 and 0.174 ESAL, respectively. The following interpolation equation was used in this study.

$$Y \text{ (ESAL)} = (X-X_2)/(X_1-X_2)*(Y_1-Y_2)+Y_2 \qquad \text{(D.1)}$$

where:

X_1 = the first axle load value (kips)

X_2 = the second axle load value (kips)

Y_1 = the LEF that corresponding with the first axle load value (ESAL)

Y_2 = the LEF that corresponding with the second axle load value (ESAL)

X = the axle load value under consideration (kips)

Using Equation (D.1), the LEF that corresponding with axle load equals to 10.95 kips is as follows.

$Y = (10.95\text{-}12)/(10\text{-}12)*(0.079\text{-}0.174)+0.0174 = 0.12$ ESAL

Using the same equation, the LEF that corresponding with axle load equals to 11.46 kips is 0.15 ESAL. The VDF for 6B-class truck is 0.12 + 0.15 = 0.27 ESAL.

D.2 Calculation of Layer Thicknesses

The calculation of layer thickness is carried out by using the equations below.

$$SN_3 = a_1D_1 + a_2D_2\, m_2 + a_3D_3\, m_3 \qquad (4.7a)$$

$$SN_2 = a_1D_1 + a_2D_2\, m_2 \qquad (4.7b)$$

$$SN_1 = a_1D_1 \qquad (4.7c)$$

For example, if it is known all *SN* and *a*, i.e. $SN_3 = 8.3$, $SN_2 = 4.63$, $SN_1 = 6.4$, $a_1 = 0.42$, $a_2 = 0.14$, $a_3 = 0.11$, $m_2 = 1$ and $m_3 = 1$, then the layer thicknesses can be calculated as follows.

$D_1 = SN_1 / a_1 = 11$ (rounding up, D1 = 12 inchi)

$SN_1^* = a_1 \times D_1 = 0.42 \times 8 = 5.04$

$D_2 = (SN_2 – SN_1^*)/(a_2 \times m_2) = 9.71$ (rounding up, $D_2 = 10$ inch)

$SN_2^* = SN_1^* + a_2 \times D_2 \times m_2 = 5.04 + 0.14 \times 12 \times 1 = 6.04$

$D_3 = (SN_3 – SN_2^*)/(a_3 \times m_3) = 17$(rounding up, $D_3 = 17$ inchi)

In order to calculate overlay thickness, the following equation was used.

$$D_{OL} = (SN_3 - (a_2 \times D_2 \times m_2 + a_3 \times D_3 \times m_3)/a_1 \qquad (D.2)$$

Using the previous data, the overlay thickness D_{OL} can be calculated as follows.

$D_{OL} = (8.3 – (0.14 \times 10 \times 1 + 0.11 \times 17 \times 1)/0.42 = 15.08$ (rounding up, D_{OL} =16 inchi)