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Low Volume Road Pavement Design: A Review of Practice in the Upper Midwest

Jong Ryeol Kim
and
David Newcomb

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by

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16. Abstract

This report reviews the state-of-the-practice in low-volume road pavement design. The factors which dictate pavement design are discussed in terms of construction materials, environment, and traffic. Eight different design procedures are presented and discussed. These include the 1972 and 1986 AASHTO guides, The Asphalt Institute, Minnesota Department of Transportation, Wisconsin Department of Transportation, National Crushed Stone Association, Shell Netherlands, and National Association of Australian State Road Authorities. Examples of the use of these procedures on local roads are presented in the report. Finally, the Minnesota practice for low-volume roads is presented from the MN/DOT Aid Manual as well as the results of a survey of counties in the state.

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CHAPTER I
EXECUTIVE SUMMARY

The objective of this report is to present a synthesis of low volume road pavement design with an emphasis on how it is practiced in the Upper Midwest of the United States.

Several low volume road design methods are presented representing a range in approaches from empirical to mechanistic-empirical. The design procedures presented will also vary from those globally applicable to those which were developed for specific climatic regions. Finally, a review of local design practice in Minnesota will be given.

Factors in pavement design are materials (subgrade, subbase and base), environment (temperature, frost and rainfall) and traffic.

The subgrade is the most important portion of the pavement structure and is the surface of natural soil which is prepared to support a pavement system. If the subgrade material is not adequate as a foundation for the pavement, a special treatment should be required. The subbase course is the portion of the pavement structure between the subgrade soil and the base course. The subbase material should be of significantly better quality than the subgrade soil. However, if subgrade soil is higher quality, the subbase could be omitted. The base course is the portion of the pavement structure below the surface course. It usually consists of aggregates and its main function is structural support.

Temperature has a direct influence on the performance of the pavement and supporting materials. Sudden temperature change could cause cracking and spalling. The effect of frost action on pavements is the most severe of the environmental factors. Frost heave is the raising of a portion of the ground surface as a result of the formation of ice lenses in a frost susceptible soil or base course. The depth of freeze is relative to the freezing index which is the cumulative number of degree-days below 32 °F. Loss of pavement strength during the spring thaw period is one of the most serious problems associated with frost action. To reduce the damage of pavement structure during this period, load restrictions are usually applied. Rainfall influences the stability and strength of the supporting materials because it affects the moisture content of the subgrade and subbase.

Traffic can be represented by a number of 18,000-lb equivalent single axle loads (ESALs). For purposes of pavement design, it is necessary to estimate the cumulative number of ESALs for the design period. The calculation of future ESALs is usually based on truck factors by truck class.

Nine different pavement design procedures are reviewed and presented in this report. The design considerations for each of these are presented in Table I-1. These range from mechanistic approaches such as those proposed by The Asphalt Institute, Shell Oil Company, and Australia to empirical methods used by AASHTO, the Minnesota Department of Transportation, and the National Crushed Stone Association. The mechanistic based procedures are more flexible in their ability to accommodate changes in materials, traffic, and environment than the more empirical methods which are restricted to fairly narrow ranges of design parameters.

The primary impediment to using mechanistic design procedures at a local level in Minnesota is a lack of information on material properties and performance. The use of strain values in failure criteria require that resilient modulus values be used in the characterization of the materials. Equations exist which relate stabilometer R-value to resilient modulus, but these are rough approximations and have questionable use in design. The layer coefficients used in the AASHTO procedures have been related to resilient modulus for unbound and bound materials. However, these relationships suffer from the same lack of fit.

The relationship between traffic, material properties and structural failure must be understood in order to develop performance equations. Although generalized equations exist, these should be modified to reflect the materials, environment, and traffic in a given area.

Currently, low volume roads are designed in Minnesota using the MN/DOT procedure. This is sometimes verified by The Asphalt Institute and the 1972 AASHTO methods. Traffic is usually expressed as ESAL's estimated from HCADT. Traffic counts are normally provided by MN/DOT once every four years, and standard distribution models found in the MN/DOT design manuals can be used when classification counts are not provided with volume data. Stabilometer

R-values or assumed soil factors are used to characterize the subgrade. The gravel equivalency concept is used to incorporate material properties in the design process. In Minnesota, load restrictions begin in late March and last until mid May. Weight limitations for specific routes are based on field observations and deflection testing.

Design Procedure	Approach	Material Properties	Subgrade Properties	Environment	Traffic	Failure Criteria
1986 AASHTO	Structural Number	Layer Coefficient	Resilient Mudulus	Climatic Regions	ESAL	Terminal Service.
1972 AASHTO	Structural Number	Layer Coefficient	Soil Support Value	Regional Factors	ESAL	Terminal Service.
Asphalt Institute	Layered Elastic	Resilient Modulus	Resilient Modulus	Mean Ann. Air Temp.	ESAL	Strain Values
Mn/DOT Road Design	Gravel Equivalency	G.E.	R-Value	Inherent	ESAL	Terminal Service.
Wis/DOT	Structural Number	Layer Coefficient	Soil Support Value	Regional Factor	ESAL	Terminal Service.
Nat.Crushed Stone Assoc.	Gravel Equivalency	G.E.	CBR	Frost Susc. Class	Group Index	—
Shell Oil	Layered Elastic	Resilient Modulus	Resilient Modulus	Mean Ann. Air Temp.	80 kN Single Axle	Strain Values
Australia	Layered Elastic	Resilient Modulus	Resilient Modulus	Moist/ Temp.	80 kN Single Axle	Strain Values
Mn/DOT State Aid	Gravel Equivalency	G.E.	Soil Factor	Inherent	ESAL	Terminal Service

Table I-1. Pavement design procedures summary.

CHAPTER II

INTRODUCTION

BACKGROUND

Pavement design is the process of selecting the appropriate materials and layer thicknesses to withstand the expected level of traffic under the environmental conditions in a given location. This rather simple statement conceals many uncertainties that are inherent in the design of pavements. These uncertainties are translated into assumptions based upon a designer's experience. It also implies that all materials can be judged by common means and that pavement layer thicknesses can be determined by equating the structural benefit of one material to that of another.

In fact, pavements are very complex structural systems which must withstand dynamic loads of varying magnitude while resisting the effects of extremes in temperature and moisture. The primary objective of the pavement is to prevent the overstressing of the natural soils (subgrade). Thus, it follows that a thinner pavement can be used over a stronger subgrade; everything else being equal. To coincide with that, it could also be suggested that less stressing of the subgrade would occur where there is less overall traffic or fewer heavy loads. Again, the result would be a thinner pavement section.

Materials which are used in the construction of pavements are initially selected according to their availability, cost, and quality. The quality is evaluated in terms of the material's efficiency in distributing the load over the underlying layers, its ability to resist weathering, and its long-term performance characteristics. In general, better quality materials can be used in lesser quantities to obtain comparable performance. As such, the quality of the material must be weighed against the cost.

The climatic conditions to which a pavement will be subjected will affect both the materials selection and the layer thickness determination. Large daily and yearly temperature fluctuations create stresses in the surface materials which can cause them to crack as the material volume changes. These cracks are not generally harmful, but they do form paths for water to infiltrate the lower layers of the pavement and weaken them, especially unbound materials.

Besides weakening materials in underlying layers, water can cause volume changes to occur due to frost heave or the swelling of certain types of of clay. Many of the problems associated with moisture in pavements can be mitigated by removing as much of it as possible by passive drainage. By reducing the amount of weakening or volume change, it is possible to use thinner pavement cross-sections.

One of the greatest concerns for a pavement designer is whether or not the pavement will last as long as its design life. This will be influenced by a number of factors which are not known at the time of design. For instance, there is a great deal of uncertainty with respect to the amount of traffic forecasted for a road. Such things as changes in land use or traffic routing can significantly affect the types and volumes of vehicles. Futhermore, the designer has no idea of what level of quality will be present in the final constructed project and this will have a definite bearing on how well the pavement performs under traffic and the environment. To ensure that the facility will last for its intended design life, the engineer may decide to hedge his bets against the uncertainties. This can be done by overdesigning the pavement for the expected traffic.

It should be clear at this point that a rational economic evaluation must be an integral part of the design process. If all pavements were built such that there was very little chance of them failing prior to the end of their design lives, very few would be built and maintained. Likewise, if there was only a 50-50 chance of them surviving that long, then there would be a tremendous amount of money and effort expended in maintaining them. The engineer must make decisions with respect to the construction, maintenance, and rehabilitation of the pavement network. In the design phase of a particular pavement, several options should be reviewed which will provide the required level of service over a period of time. This period of time will include the initial construction and scheduled maintenance for the design life and one or more rehabilitation strategies. The total cost of the pavement over the period is called the life cycle cost, and the best alternative is the one with the lowest life cycle cost.

Generally speaking, the pavement design process becomes more arcane as the volume of traffic on a given roadway decreases. In other words, the designer is usually forced to work with less information because the budget for engineering is reduced on lower volume roads. Therefore, he may be less certain about subgrade conditions or material properties due to an inability to pay for extensive testing. Traffic data may also be sketchy or nonexistent in many instances. Thus, it is difficult to know if the combination of materials and thicknesses selected for the pavement structure is the most economical.

OBJECTIVE

The objective of this report is to present a synthesis of low volume road pavement design with an emphasis on how it is practiced in the Upper Midwest of the United States.

SCOPE

Several low volume road design methods will be presented representing a range in approaches from empirical to mechanistic-empirical. The design procedures presented will also vary from those that are globally applicable to those which were developed for specific climatic regions. Finally, a review of local design practice in Minnesota will be given.

CHAPTER III

DESIGN FACTORS

MATERIALS OF CONSTRUCTION

Subgrade

The subgrade is the surface of natural soil which is prepared to support a pavement system. Desirable properties for the subgrades are:

1. high compressive and shear strength,
2. permanency of strength under all weather and loading conditions,
3. ease of compaction,
4. permanency of compaction,
5. ease of drainage and
6. low susceptibility to volume changes and frost action.

Since soils vary considerably, the interrelationship of texture, density, moisture content and strength of subgrade materials is complex. In addition, the behavior of subgrades under repeated loads is difficult to evaluate and the properties of the subgrade extending underneath the pavement vary along the length and width of a highway. For this reason, the behavior of the subgrade should be well understood by the engineer.

If the subgrade material proves to be inadequate as a foundation for the pavement, a special treatment for the subgrade should be followed. This treatment may differ from one location to another, depending upon the reasons for inadequacy of the subgrade material, weather conditions and availability of better materials.

Compacting the subgrade soil generally increases its shear strength, and reduces its compressibility and absorption of water. These improvements are a result of the reduction of the voids of the compacted material and the increase in density.

Compaction tests should be made in the laboratory on each soil type to be used in construction to determine the practical maximum density that may be obtained. AASHTO Method of Test T 180 [1], "Moisture-Density Relations of Soils Using a 10-lb Rammer and a 18-inch Drop," is usually used.

For adequate subgrade support, several basic principles should be followed, including proper compaction of the subgrade and installation of adequate subdrains [2]. The subgrade strength values that are used for the design of pavements are generally based upon the results of laboratory tests of saturated samples. The tests should be performed only after a study of moisture-density-strength relationships of the various subgrade soils are determined. Arbitrary selection of standard densities and moisture contents gives results satisfactory for most routine work; however, a careful study should made of all the variables affecting the strength of the soil. If the laboratory tests indicate that the subgrade will lose its strength with increasing densities due to overcompaction, careful consideration should be given to closely controlling the moisture content. It may be necessary to limit the maximum weight of roller to be used.

Elevated grades should be used whenever possible, and transition zones should be provided between cut and fill. Careful consideration should be given to the suitability of the subgrade from the standpoint of frost action, shrinkage and swell, and other properties that may cause pavement distress to develop.

Principles have been presented wherein strength values can be selected to minimize total cost. These are based upon a knowledge of geology, climate, and other factors. These principles coupled with adequate construction control are important tools the engineer can use to improve pavement performance.

Subbase and Base Layers

The subbase course is the portion of the flexible pavement structure between the subgrade soil and the base course. It usually consists of a compacted layer of granular material, either treated or untreated, or of a layer of soil treated with a suitable admixture. In addition to its position in the pavement, it is usually distinguished from the base course material by less stringent requirements for strength, plasticity, and gradation. The subbase material should be significantly better than the subgrade soil. For reasons of economy, the subbase is often omitted if subgrade soils are of high quality.

When subgrade soils are of relatively poor quality and the design procedure indicates that a substantial thickness of pavement is required, several alternate designs should be prepared for structural sections with and without subbase. The selection of an alternate may then be made on the basis of availability and relative costs of materials suitable for base and subbase. Because lower quality materials may be used in the lower layers of a flexible pavement structure, the use of a subbase course is often the most economical solution for construction of pavements over poor subgrade soils.

The base course is the portion of the pavement structure immediately beneath the surface course. It is constructed on the subbase course, or, if no subbase is used, directly on the subgrade soil. Its major function in the pavement is structural support. It usually consists of aggregates such as crushed stone, crushed slag, crushed gravel and sand, or combinations of these materials. It may be used untreated or treated with suitable stabilizing admixtures, such as portland cement, asphalt, lime, cement-flyash and lime-flyash, i.e.,pozzolonic stabilized base. Specifications for base course materials are generally considerably more stringent than for subbase materials in requirements for strength, plasticity, and gradation.

When using pozzolonic stabilized bases under a relatively thin asphalt wearing surface, it can usually be expected that transverse reflection cracks will occur in the surface in a relatively short period of time, e.g., 1 to 3 years. Sawed and sealed joints (through the asphalt concrete into the base) may be used to minimize the adverse effects on appearance and to provide for better future sealing operations. Crack spacing may vary from 20 to 40 feet depending on local experience with past uncontrolled crack-spacing problems [3].

Environment

Both the pavement and the underlying supporting layers are exposed to strong climatic influences. For this reason, statistical data and general information on the following variables should be gathered before starting the design and construction of the pavement system.

Temperature

Air temperature has a direct influence on the performance of the pavement and supporting medium in flexible pavements. Sudden temperature variations accompanied by fluctuation in soil moisture cause deformation in the pavement system, which may result in cracking. Low temperatures cause soil shrinkage, particularly in cohesive soil. This may be accompanied by cracks which are filled with water during the next period of rain, resulting in a decrease in the bearing capacity of the soil. If the water in the cracks is frozen due to depressed air temperatures, then a break-up in the soil mass may result.

A temperature potential applied to soil will cause the movement of soil moisture from warmer regions to colder ones, thus changing the moisture distribution in the soil. If the colder region is then exposed to freezing temperatures, the migrating moisture acts as a supply, which causes the growth of ice lenses under the pavement and may contribute to frost damage.

It is the variation of air temperature that affects, to a large extent, the type and amount of bitumen that should be used in flexible pavements. Also, the variation in temperature between the top and bottom surfaces of the pavement affects its deflection and load-bearing capacity. Asphalt concrete stability usually decreases dramatically with increasing temperature. Rate of temperature change and temperature cycling (yearly and daily) are causes of thermal cracking of the asphalt concrete pavement.

Frost

The effect of frost action on pavements is the most severe of the environmental factors. In its broadest sense, frost action includes both frost heave and loss of subgrade support during the frost-melt period. Frost heave may cause a portion of the pavement to rise, due to ice lens formation (Figure III-1) in a frost susceptible subgrade or base course. Thawing of the frozen soil and ice lenses during the spring period may cause pavement damage under loads. This damage usually results in high maintenance costs. The loss of strength under the pavement due to thawing may be of such a magnitude as to require prohibition of heavy loads during the critical period. The economic loss resulting when trucks operate during this period while carrying much smaller loads than their capacity, may be very high.

The term 'frost' generally involves two concepts:

1. The existence of freezing temperatures below 32 °F and
2. The action of the freezing temperatures upon the soil, which leads to the state of frozen soils.

When the soil is subjected to freezing temperatures, several phenomena occur, the intensity of which depend upon the intensity of freezing temperatures. These

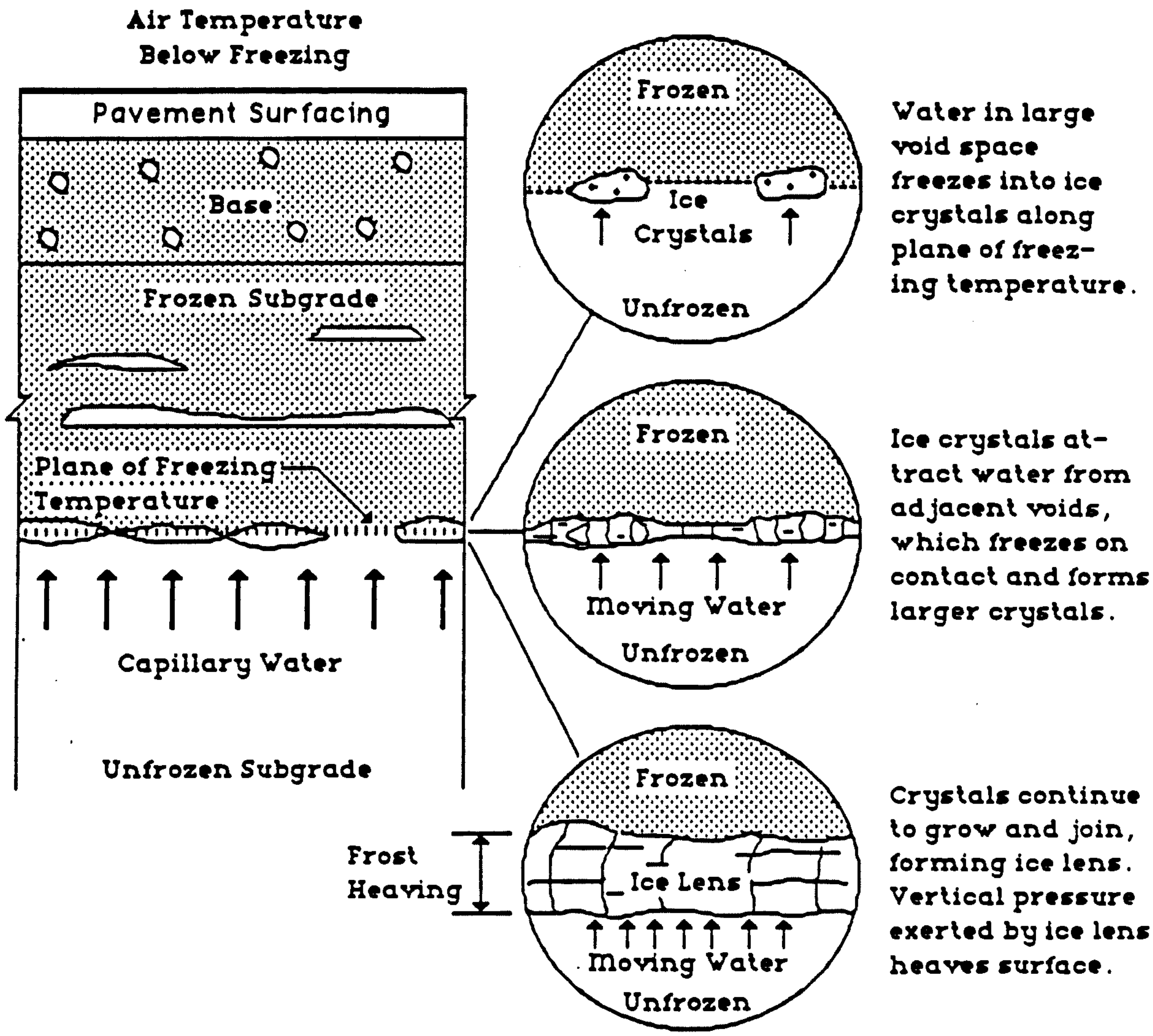


Figure III-1. Formation of ice lenses in a pavement structure [5].

phenomena are:

1. The penetration of frost into the soil occurs at a relatively low rate.
2. As a layer of water-bearing soil is frozen underneath the pavement, it becomes essentially dry, and as such has a high affinity for free water.
3. Free water in the underlying layers is attracted to the undersurface of the frozen soil layer and is consequently frozen as freezing temperature proceeds downward during cold weather.
4. Layers or lenses of clear ice, several inches in thickness, are built up in this way. When water freezes, it increases by 9 percent in volume.

The expansion of water as it freezes within the soil mass is not the main reason for heave and break-up of pavements due to freezing conditions. If a quick freeze process were to occur in the moist soil mass, total vertical expansion might amount to less than 1 inch even if freezing was carried to a considerable depth. It is mainly the nonuniform formation of the above-mentioned ice lenses that actually causes the pavement to heave.

It is important for the pavement designer to recognize that for frost heave to take place all of the following factors must be present:

1. a frost-susceptible soil,
2. slowly depressed air temperatures and
3. a supply of water.

If any of the above factors are not present, there will be no frost heave. Also, in the cases where the heave extends fairly uniformly over a considerable length of the pavement, the seriousness of the heaving problem can be greatly reduced.

Soil freezing depends to a large extent upon the duration of depressed air temperatures. Usually, time and temperature are measured by freezing degree days. One degree day represents 1 day with a mean air temperature 1 degree below freezing.

If a cumulative plot of degree days versus time is determined the difference between the maximum and minimum points on this curve is termed the freezing index (Figure III-2). In plotting this curve, any convenient day can be used as a start for the plot, since the data are cumulative.

Design freezing index is the average of freezing indices for the three coldest winters in the last 30 years if data are available or the coldest winter in the last 10 years [4].

The depth of frost penetration has been correlated with the surface freezing index which is based on the temperature of soil surface. If the data of soil or pavement surface temperature are available, the surface freezing index can be determined by multiplying a coefficient (n factor, Table III-1) to air freezing index.

The depth of frost penetration can be determined by the empirical method which was developed by the Corps of Engineers from the correlation between freezing index and measured frost depth based on well-drained non-frost

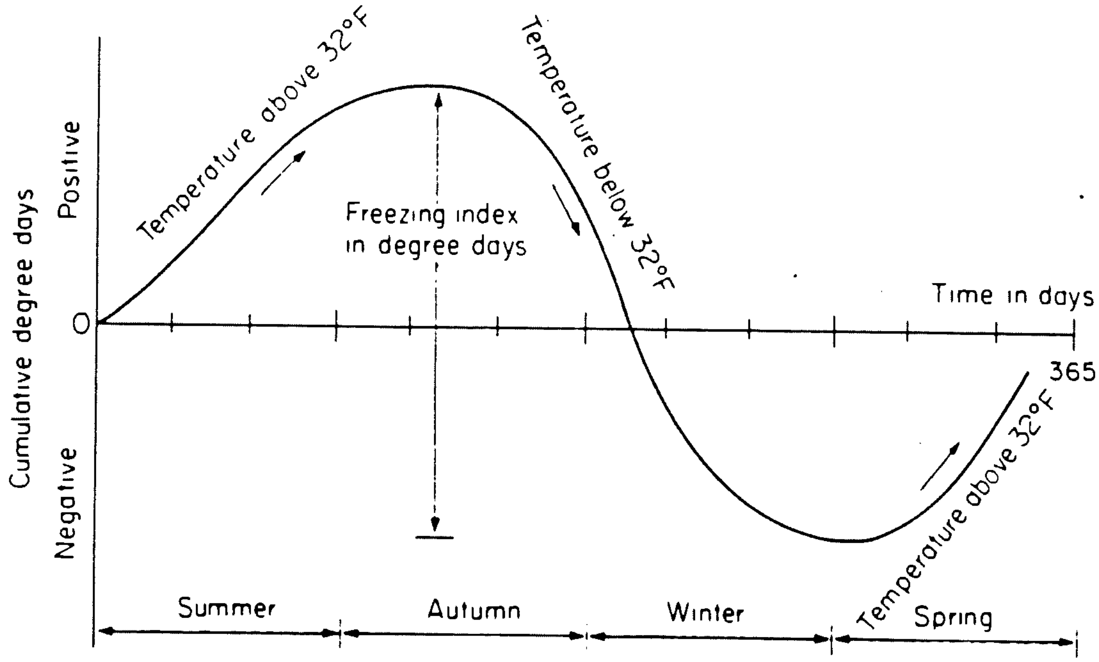


Figure III-2. Determination of the freezing index.

Table III-1. Values of n-factor [6].

Surface type	n-factor after Lunardini	n-factor recommended by the Corps of Engineers
Spruce trees, brush, moss over peat - soil surface	0.29*	-
As above with tree cleared - soil surface	0.25*	-
Turf	0.50	0.50
Sand and gravel	0.60 - 1.00	0.90
Pavement free of snow		
Asphalt concrete	0.29 - 1.00 or greater	0.90
P.C.C	0.25 - 0.95	0.90

*Under snow

susceptible base course material (Figure III-3). Figure III-4 presents a generalized map of the United States indicating freezing index contours.

One of the most serious problems associated with frost action is the rapidly decrease of pavement strength during the spring thaw. Since any decrease in material strength or increase in traffic load reduces the life of the pavement, the method of reducing loads during spring thaw period is a reasonable way to maintain the pavement structure. The criteria [5] which should be applied for load restrictions are the following:

1. Location (pavement sections)
   - A. Surface deflections during the spring thaw are 45 to 50 percent higher than summer values.
   - B. Surface thickness is equal or less than 2 inches and the freezing index is greater than 400 °F-days.
   - C. Fine-grained subgrades such as silts and clays.
   - D. Fatigue (alligator) cracking and rutting are occurred during the spring thaw.
   - E. Moisture conditions also should be considered.
2. Magnitude (Range of load restriction)
   - A. Average load restriction is about 44 percent. Minimum load restriction level is about 20 percent and maximum is 60 about percent.

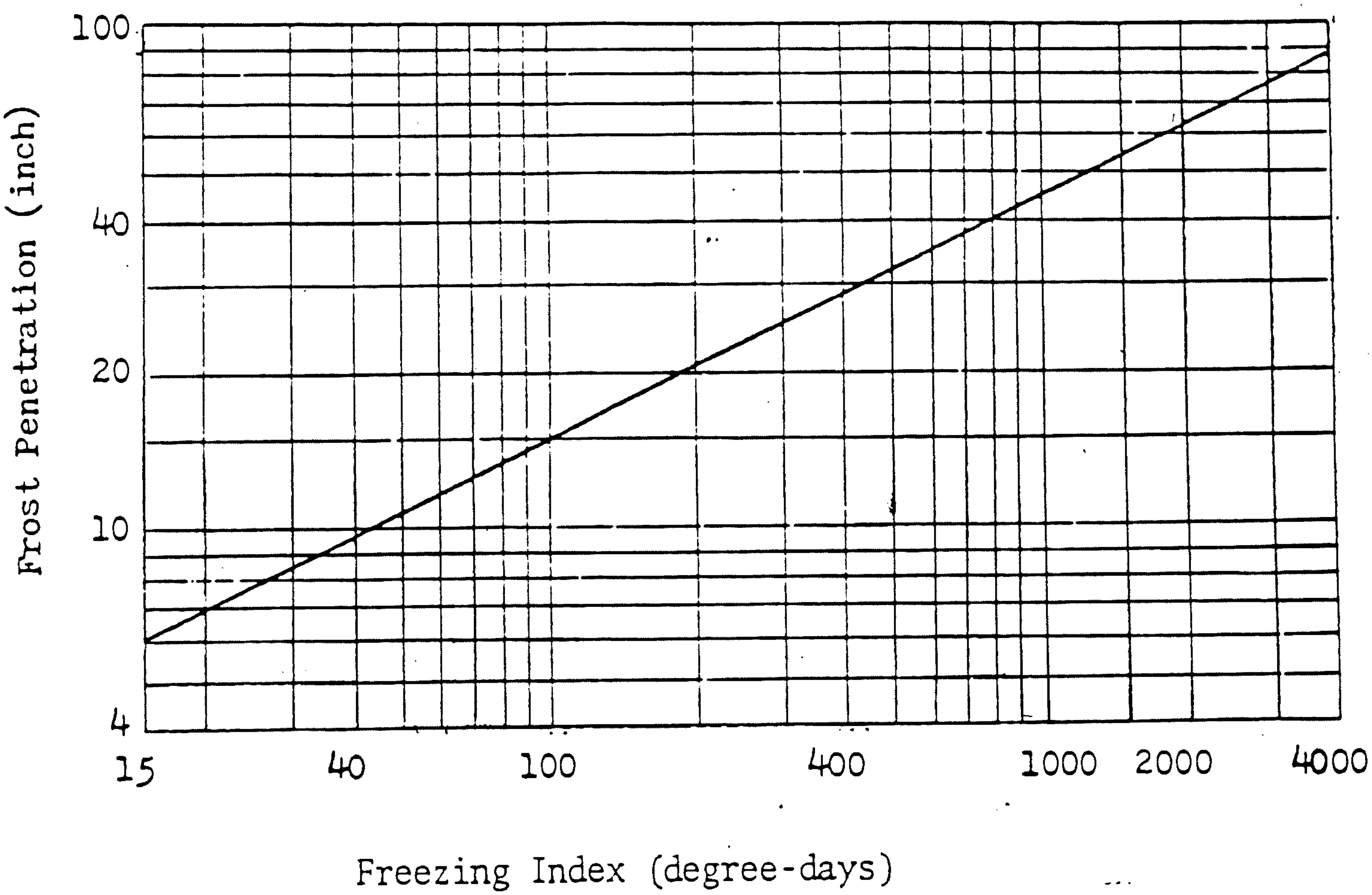


Figure III-3. Relationship between frost penetration and freezing index in well-drained, non-frost susceptible base course material [7].

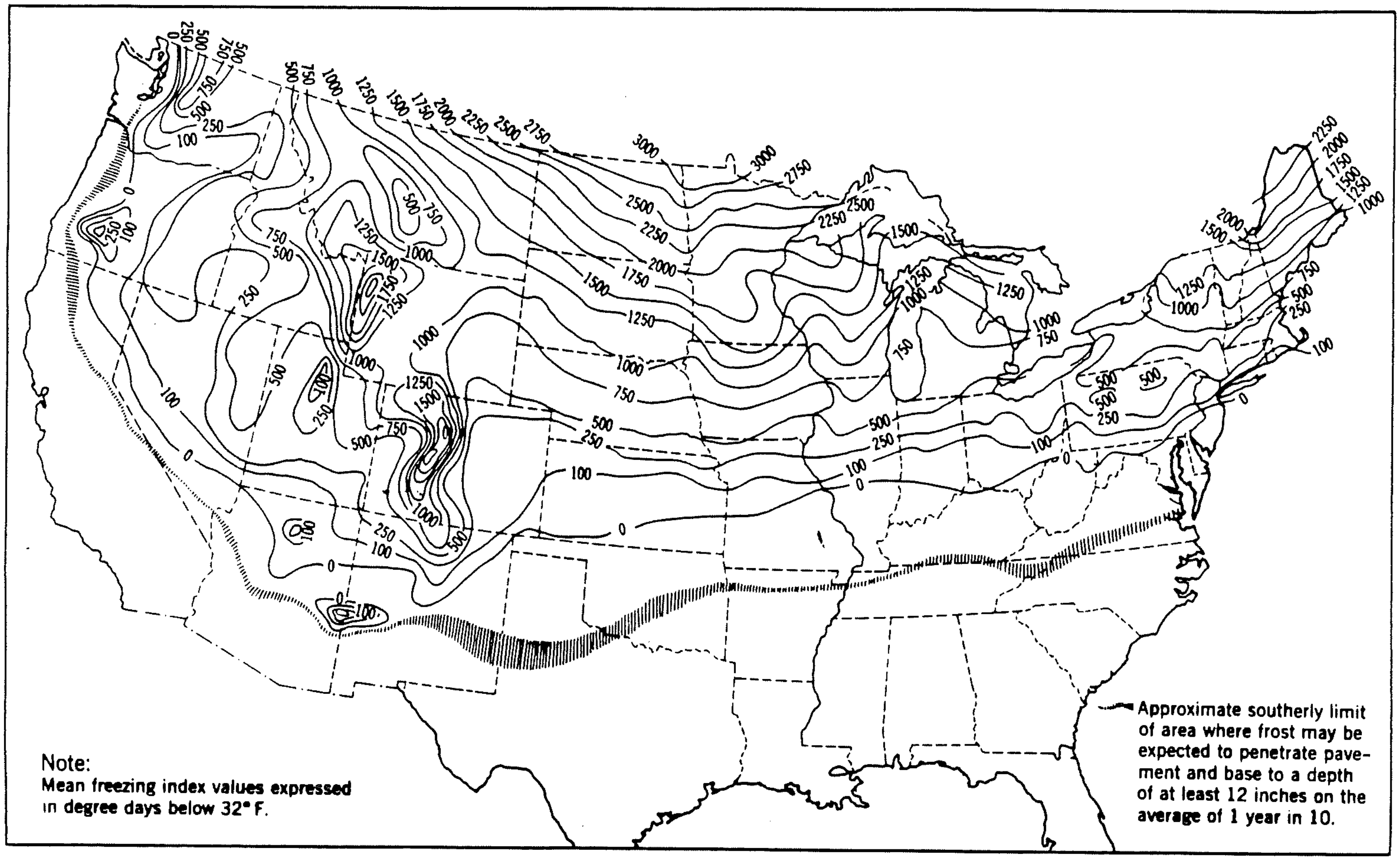


Figure III-4. Distribution of mean freezing-index values in the continental United States [8].

3. Time (placement of load restriction)

   A. For thin pavement structure (2 in. ≥ AC, and 6 in. ≥ base) load restrictions should be applied at a thawing index (based on air temperature datum of 29 °F) is 10 °F-days and must be applied at a thawing index is 45 °F-days.

   B. For thick pavement structure (2 in. < AC, and 6 in. < base) load restrictions should be applied at thawing index value is 25 °F-days and must be applied at thawing index is 50 °F-days.

4. Duration

   A. Approach with freezing index

   $D = 22.62 + 0.011\ (FI)$

   or

   $D \approx 25 + 0.01\ (FI)$

where

D = duration for complete thaw based on a start date when the air temperature is 29 °F or above in days

FI = freezing index in °F-days

   B. Approach with thawing index (TI)

   $TI = 4.154 + 0.259\ (FI)$

   or

$TI \approx 0.3\ (FI)$

These equations are based on fine-grained soils and 15 percent moisture content.

Rainfall

Rain has an influence on the stability and strength of the supporting medium because it affects the moisture content of the subgrade and subbase. Usually, the strength of a soil decreases with more moisture. Also, rainfall is well established as a factor affecting the elevation of the water table, the intensity of frost action, erosion, pumping and infiltration. The extent of the detrimental effects of frost action (in particular, loss of strength during the thaw period) are dependent on the combined effect of rainfall and air temperature. Where the frost problem is absent, the moisture content of the pavement will also vary with rainfall, and this will in turn affect the expansion and contraction of the pavement. Long periods of rainfall of low intensity can be more adverse than short periods of high intensity, because the amount of moisture absorbed by the soil is greater under the former condition.

Traffic

Among the most important factors to be evaluated in the structural design of highway pavements are the effects of vehicle type and traffic volume. In considering a particular vehicle type, one must consider such factors as the specific gear configuration, tire spacings, axle loads, tire pressures and vehicle speed.

The results of the AASHO Road Test have shown that the damaging effect of the passage of an axle of any mass can be represented by a number of 18,000-lb equivalent single axle loads (ESALs). The determination of design ESALs is one of the most important considerations for the design of pavement structures.

The procedure to convert a mixed traffic stream of different axle loads and axle configurations into a design traffic number is to convert each expected axle load into an equivalent number of ESALs to sum these over the design period.

To determine the traffic accurately and to determine the life expectancy of a pavement these following four keys [3] should be considered:

1. The correctness of the load equivalency values used to estimate the relative damage induced by axle loads of different mass and configurations,
2. The accuracy of traffic volume and weight information used to represent the actual loading projections,
3. The prediction of ESALs over the design period, and
4. The interaction of age and traffic as it affects changes in present serviceability index (PSI).

The available load equivalency factors, which are functions of pavement type, thickness, and terminal serviceability, are considered the best available at the present time, representing information derived from the AASHO Road Test.

State departments of transportation (DOTs) accumulate traffic information in the format of the Federal Highway Administration W-4 truck weight table, which is a tabulation of the number of axles observed within a series of load groups, with each load group covering a 2,000-lb interval. Traffic information relative to truck type is provided in W-2 tabulations. These tabulations can be used to estimate the number of equivalent single axle loads associated with mixed traffic at the particular reporting loadometer station. From this information it is possible to obtain average load equivalency factors for all trucks or for trucks by configuration [9].

Most states have taken the information from the W-4 tables and converted it into relatively simple multipliers (truck equivalency factors) which represent each truck type in the traffic stream. These multipliers can be used to convert mixed streams of traffic to ESALs.

Based on past traffic history, the future traffic can be predicted. Traffic may remain constant (some residential streets because of the use remains constant), or increase according to a straight line (minor arterial or collector-type highways) or at an accelerating (exponential) rate (new principal arterial or interstate highways) [3].

For purposes of pavement design, it is necessary to estimate the cumulative number of ESALs for the design period. The essential information required to calculate ESALs is truck traffic because the calculation of future ESALs is often based on truck factors by truck class. In regard to the use of truck factors, it will be important to use truck weight information representative of the truck traffic on the designed facility and to obtain information as nearly site specific as possible when estimating ESALs per truck for each truck classification. The percent trucks for the design period is often assumed to be constant.

The steps of ESALs calculations are:

1. Estimate growth in traffic,
2. Estimate heavy commercial vehicles as percent of traffic,
3. Determine direction and lane distributions,
4. Use AASHTO equivalency factors or
5. Use Asphalt Institute factors.

CHAPTER IV

DESIGN PRACTICES

AASHTO GUIDE FOR DESIGN OF PAVEMENT STRUCTURES 1986

The design procedure presented in the AASHTO Guide for Design of Pavement Structures 1986 [3] is summarized in this section. The design is based on identifying a flexible pavement structural number (SN) which is an abstract value expressing the structural strength of a pavement required for given combinations of soil resilient modulus (M_R), total traffic expressed in ESALs, terminal serviceability, and environment.

Environmental Considerations

According to this design method, there are six different climatic regions in the United States. Figure IV-1 is a map showing these regions and the environmental characteristics associated with each. Based on these regional characteristics, the season lengths needed for determining the effective roadbed soil resilient modulus for flexible pavement design can be defined (Table IV-1).

Subgrade Evaluation

The definitive material property used to characterize roadbed soil for pavement design in this manual is the resilient modulus (M_R). The procedure for determining the M_R is given in AASHTO Test Method T274 [1]. Because many agencies do not have equipment for performing the resilient modulus test, suitable factors are reported which can be used to estimate M_R from standard

CBR, R-value, and soil index test results or values.

1) Corps of Engineers' Method [10]

M_R (psi) = A * CBR

where

A = 1500, actual range is from 750 to 3000.

2) The Asphalt Institute's Method [11]

M_R (psi) = B + C * (R-value)

where

B = 772 to 1155 and

C = 369 to 555.

For fine-grained soils (R-value is less than or equal to 20)

M_R (psi) = 1000 + 555 * (R-value)

If the general quality of the roadbed material as a foundation for the pavement structure can be classified, the roadbed soil resilient modulus values in Table IV-2 may be used for low-volume road design. If the values in this table are combined with the suggested season lengths identified in the environmental condition part, effective roadbed soil resilient modulus values (Table IV-3) can be generated for each of the six U.S. climatic region.

Traffic Considerations

The vehicle loadings in this method are based on cumulative expected 18-kip equivalent single axle loads ($\hat{w}_{18}$) during the analysis period. The predicted traffic furnished by the planning group of an agency is generally the cumulative ESAL applications expected on the highway, whereas the designer requires the axle applications in the design lane. Thus, unless specifically furnished, the designer must factor the design traffic by direction and then by lane (if more than two). The following equation may be used to determine the traffic (w_{18}) in the design lane:

$$w_{18} = D_D * D_L * \hat{w}_{18}$$

where

D_D = a directional distribution factor, expressed as a ratio, that accounts for the distribution of ESAL units by direction (It may vary from 0.3 to 0.7).

D_L = a lane distribution factor (expressed as a ratio). For low-volume road, D_L is 100 %.

$\hat{w}_{18}$ = the cumulative two-directional 18-kip ESAL units predicted for a specific section of highway during the analysis period.

Analysis period of this method is from 15 to 25 years.

Failure Criteria

Selection of the lowest allowable Present Serviceability Index (PSI) or terminal serviceability index (p_t) is based on the lowest index that will be tolerated before rehabilitation, resurfacing, or reconstruction becomes necessary. An index of 2.5 or higher is suggested for design of major highways and 2.0 for highways with lesser traffic volumes. 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:

P_t level	% of People Stating Unacceptable
3.0	12
2.5	55
2.0	85

For relatively minor highways where economics dictate that the initial capital outlay be kept at a minimum, it is suggested that this be accomplished by reducing the design period or the total traffic volume, rather than by designing for a terminal serviceability of less than 2.0

Design Procedures

In this method, the level of reliability recommended for low-volume road design is 50 percent because of their low usage and the associated low level of risk. Depending on the importance of the road, however, the user may design at a higher reliability level of up to 80 percent.

The structural number (SN) can be obtained from Tables IV-4 and IV-5. Table IV-4 is based on the 50 percent reliability level and Table IV-5 is based on a 75 percent level. The range of SN values shown for each condition is based on a specific range of ESAL applications at each traffic level:

High :	700,000 to 1,000,000
Medium :	400,000 to 600,000
Low :	50,000 to 300,000

Once a design structural number is selected, it is up to the user to identify an appropriate combination of flexible pavement layer thicknesses which will provide the desired load-carrying capacity. This may be accomplished using the criteria for layer coefficients (a_i-values) which can be obtained using relationships presented in the AASHTO guide and the general equation for structural number:

$$SN = a_1D_1 + a_2D_2 + a_3D_3$$

where

a_1 = layer coefficient for surface course material

a_2 = layer coefficient for base course material

a_3 = layer coefficient for subbase course material

D_1 = thickness (in inches) of surface

D_2 = thickness (in inches) of base course

D_3 = thickness (in inches) of subbase course

Average values of layer coefficients for materials used in the AASHO Road Test were as follows:

Asphaltic concrete surface course	--- 0.44
Crushed stone base course	--- 0.14
Sandy gravel subbase course	--- 0.11

The following is provided as minimum practical thicknesses of asphalt concrete and aggregate base.

Traffic, ESALs	Asphalt Concrete	Aggregate Base
Less than 50,000	1.0	4.0
50,001 - 150,000	2.0	4.0
150,001 - 500,000	2.5	4.0
500,000 - 2,000,000	3.0	6.0

EXAMPLE

Given:

Location: Carver County, Minnesota

ESAL = 150,000

Assumption:

Reliability = 50 %

Relative quality of roadbed soil: good

Solution:

ESAL = 150,000 => Traffic level is low.

Minnesota is in the U.S. climatic region III (from Figure II-5).

Use 2.5 for SN (SN is between 2.0 and 2.7 from Table II-5).

$SN = a_1D_1 + a_2D_2 + a_3D_3$

$a_1 = 0.44$ (assumed)

$a_2 = 0.14$ (assumed)

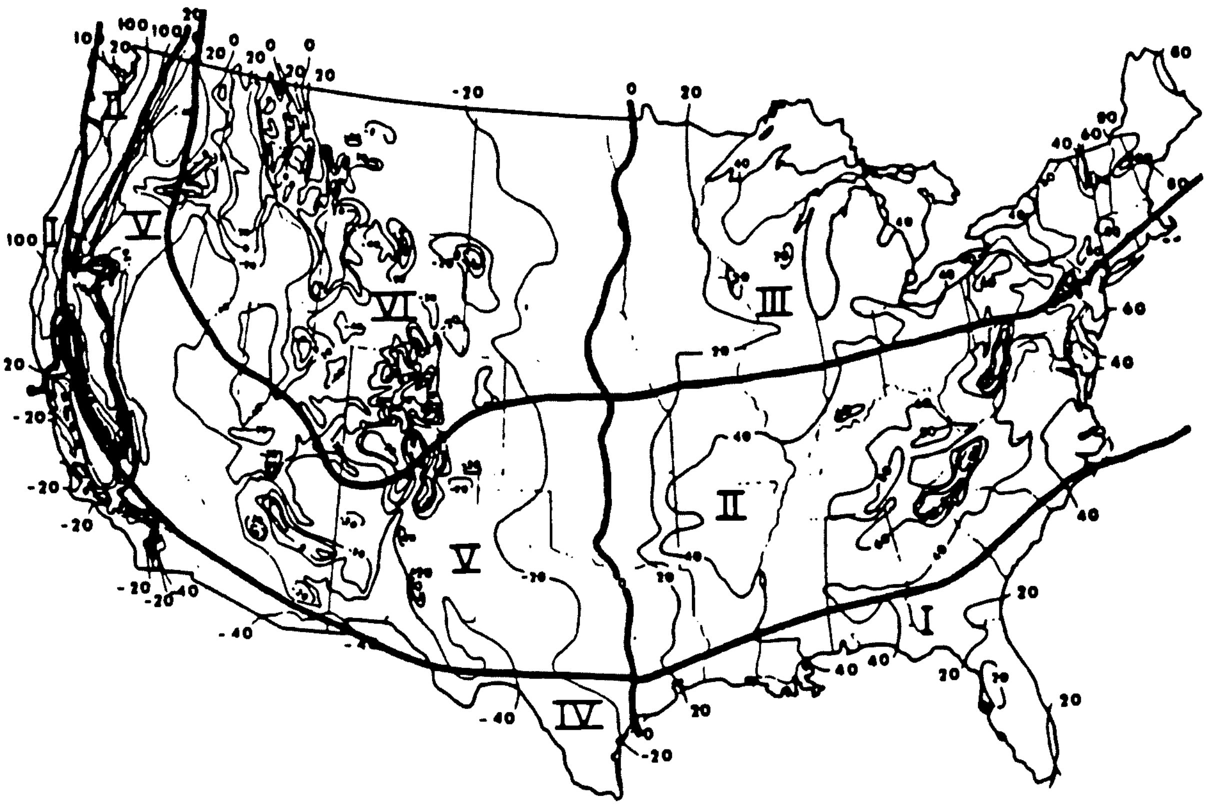
Minimum $D_1 = 2.0$

Minimum $D_2 = 4.0$

For two-layer, $D_3 = 0$

Therefore, the following design thicknesses would be acceptable:

layer	thickness (inches)			
D_1	2.5	3.0	3.5	4.0
D_2	10.0	8.5	7.0	5.5



REGION	CHARACTERISTICS
I	Wet, no freeze
II	Wet, freeze-thaw cycling
III	Wet, hard-freeze, spring thaw
IV	Dry, no freeze
V	Dry, freeze-thaw cycling
VI	Dry, hard freeze, spring thaw

Figure IV-1. The six climatic regions in the United States for 1986 AASHTO Procedure [3].

Table IV-1. Suggested season lengths (months) for the six U.S. climatic regions for 1986 AASHTO procedure [3].

U.S. Climatic Region	Season (Roadbed Soil Moisture Condition)			
	Winter (Roadbed Frozen)	Spring-Thaw (Roadbed Saturated)	Spring/Fall (Roadbed Wet)	Summer (Roadbed Dry)
I	0.0*	0.0	7.5	4.5
II	1.0	0.5	7.0	3.5
III	2.5	1.5	4.0	4.0
IV	0.0	0.0	4.0	8.0
V	1.0	0.5	3.0	7.5
VI	3.0	1.5	3.0	4.5

*Number of months for the season.

Table IV-2. Suggested seasonal roadbed soil resilient moduli, M_R (psi), as a function of the relative quality of the roadbed material [3].

Relative Quality of Roadbed Soil	Season (Roadbed Soil Moisture Condition)			
	Winter (Roadbed Frozen)	Spring-Thaw (Roadbed Saturated)	Spring/Fall (Roadbed Wet)	Summer (Roadbed Dry)
Very Good	20,000*	2,500	8,000	20,000
Good	20,000	2,000	6,000	10,000
Fair	20,000	2,000	4,500	6,500
Poor	20,000	1,500	3,300	4,900
Very Poor	20,000	1,500	2,500	4,000

*Values shown are Resilient Modulus in psi.

Table IV-3. Effective roadbed soil resilient modulus values, M_R (psi), that may be used in the design of flexible pavements for low-volume roads [3].

U.S. Climatic Region	Relative Quality of Roadbed Soil				
	Very Poor	Poor	Fair	Good	Very Good
I	2,800*	3,700	5,000	6,800	9,500
II	2,700	3,400	4,500	5,500	7,300
III	2,700	3,000	4,000	4,400	5,700
IV	3,200	4,100	5,600	7,900	11,700
V	3,100	3,700	5,000	6,000	8,200
VI	2,800	3,100	4,100	4,500	5,700

*Effective Resilient Modulus in psi

Table IV-4. Flexible pavement design catalog for low-volume roads (inherent reliability = 50%) [3].

Relative Quality of Roadbed Soil	Traffic Level	U.S. Climatic Region					
		I	II	III	IV	V	VI
Very Good	High	2.3 - 2.5*	2.5 - 2.7	2.8 - 3.0	2.1 - 2.3	2.4 - 2.6	2.8 - 3.0
	Medium	2.1 - 2.3	2.3 - 2.5	2.5 - 2.7	1.9 - 2.1	2.2 - 2.4	2.5 - 2.7
	Low	1.5 - 2.0	1.7 - 2.2	1.9 - 2.4	1.4 - 1.8	1.6 - 2.1	1.9 - 2.4
Good	High	2.6 - 2.8	2.8 - 3.0	3.0 - 3.2	2.5 - 2.7	2.7 - 2.9	3.0 - 3.2
	Medium	2.4 - 2.6	2.6 - 2.8	2.8 - 3.0	2.2 - 2.4	2.5 - 2.7	2.7 - 2.9
	Low	1.7 - 2.3	1.9 - 2.4	2.0 - 2.7	1.6 - 2.1	1.8 - 2.4	2.0 - 2.6
Fair	High	2.9 - 3.1	3.0 - 3.2	3.1 - 3.3	2.8 - 3.0	2.9 - 3.1	3.1 - 3.3
	Medium	2.6 - 2.8	2.8 - 3.0	2.9 - 3.1	2.5 - 2.7	2.6 - 2.8	2.8 - 3.0
	Low	2.0 - 2.6	2.0 - 2.6	2.1 - 2.8	1.9 - 2.4	1.9 - 2.5	2.1 - 2.7
Poor	High	3.2 - 3.4	3.3 - 3.5	3.4 - 3.6	3.1 - 3.3	3.2 - 3.4	3.4 - 3.6
	Medium	3.0 - 3.2	3.0 - 3.2	3.1 - 3.4	2.8 - 3.0	2.9 - 3.2	3.1 - 3.3
	Low	2.2 - 2.8	2.2 - 2.9	2.3 - 3.0	2.1 - 2.7	2.2 - 2.8	2.3 - 3.0
Very Poor	High	3.5 - 3.7	3.5 - 3.7	3.5 - 3.7	3.3 - 3.5	3.4 - 3.6	3.5 - 3.7
	Medium	3.2 - 3.4	3.3 - 3.5	3.3 - 3.5	3.1 - 3.3	3.1 - 3.3	3.2 - 3.4
	Low	2.4 - 3.1	2.4 - 3.1	2.4 - 3.1	2.3 - 3.0	2.3 - 3.0	2.4 - 3.1

*Recommended range of structural number (SN).

Table IV-5. Flexible pavement design catalog for low-volume roads (inherent reliability = 75%) [3].

Relative Quality of Roadbed Soil	Traffic Level	U.S. Climatic Region					
		I	II	III	IV	V	VI
Very Good	High	2.6 - 2.7*	2.8 - 2.9	3.0 - 3.2	2.4 - 2.5	2.7 - 2.8	3.0 - 3.2
	Medium	2.3 - 2.5	2.5 - 2.7	2.7 - 3.0	2.1 - 2.3	2.4 - 2.6	2.7 - 3.0
	Low	1.6 - 2.1	1.8 - 2.3	2.0 - 2.6	1.5 - 2.0	1.7 - 2.2	2.0 - 2.6
Good	High	2.9 - 3.0	3.0 - 3.2	3.3 - 3.4	2.7 - 2.8	3.0 - 3.1	3.3 - 3.4
	Medium	2.6 - 2.8	2.7 - 3.0	3.0 - 3.2	2.4 - 2.6	2.6 - 2.9	2.9 - 3.2
	Low	1.9 - 2.4	2.0 - 2.6	2.2 - 2.8	1.8 - 2.3	2.0 - 2.5	2.2 - 2.8
Fair	High	3.2 - 3.3	3.3 - 3.4	3.4 - 3.5	3.0 - 3.2	3.2 - 3.3	3.4 - 3.5
	Medium	2.8 - 3.1	2.9 - 3.2	2.7 - 3.3	2.7 - 3.0	2.8 - 3.1	3.0 - 3.3
	Low	2.1 - 2.7	2.2 - 2.8	2.3 - 2.9	2.0 - 2.6	2.1 - 2.7	2.3 - 2.9
Poor	High	3.5 - 3.6	3.6 - 3.7	3.7 - 3.9	3.4 - 3.5	3.5 - 3.6	3.7 - 3.8
	Medium	3.1 - 3.4	3.2 - 3.5	3.4 - 3.6	3.0 - 3.3	3.1 - 3.4	3.3 - 3.6
	Low	2.4 - 3.0	2.4 - 3.0	2.5 - 3.2	2.3 - 2.8	2.3 - 2.9	2.5 - 3.2
Very Poor	High	3.8 - 3.9	3.8 - 4.0	3.8 - 4.0	3.6 - 3.8	3.7 - 3.8	3.8 - 4.0
	Medium	3.4 - 3.7	3.5 - 3.8	3.5 - 3.7	3.3 - 3.6	3.3 - 3.6	3.4 - 3.7
	Low	2.6 - 3.2	2.5 - 3.3	2.6 - 3.3	2.5 - 3.1	2.5 - 3.1	2.6 - 3.3

*Recommended range of structural number (SN).

AASHTO INTERIM GUIDE FOR DESIGN OF PAVEMENT STRUCTURES 1972

The design procedure presented in the AASHTO Interim Guide for Design of Pavement Structures 1972 [9] is summarized in this section. This procedure is based on data from the AASHO Road Test, supplemented and modified by data from other road tests and other design procedures. The use of this design procedure requires that values be established for the design variables of soil support (S), equivalent 18-kip single-axle loads, and regional factor (R).

Subgrade Evaluation

An empirical soil support scale (Figure IV-2) is used in this procedure with values from 1 to 10. Point 3.0 on the soil support scale represents the silty clay roadbed soils on the AASHO Road Test, and is a firm and valid point. Point 10.0 representing crushed rock base material, such as used on the Road Test, is a reasonably valid point. All other points on the scale were assumed from experience. The soil support value must be correlated with a particular soil test or identification method selected for use by the user agency.

Environmental Considerations

A regional factor is defined for pavements in areas with climatic and environmental conditions different from those at the AASHO Road Test. Based on AASHO Road Test information, values that may be used as a guide for such an analysis are:

Roadbed materials frozen to depth of 5 inches or more	0.2 to 1.0
Roadbed materials dry, summer and fall	0.3 to 1.5
Roadbed materials wet, spring thaw	4.0 to 5.0

Traffic Considerations

To use this design method the traffic must be converted into an equivalent number of 18-kip single-axle loads. The prediction of traffic for design purposes must rely on information from past traffic, modified by factors for growth or other expected changes.

Failure Criteria

Selection of the terminal serviceability index (P_t) is based on the lowest value that will be tolerated before resurfacing or reconstruction becomes necessary. An index of 2.5 is suggested as a guide for design of major highways, and 2.0 for highways with lesser traffic volumes.

Design Procedure

The steps required for designing the thickness of pavement layers are:

1. Determine the soil support value, the 18,000-lb single-axle loads and the regional factor.
2. Determine the structural number from the design nomographs (Figures IV-3 and IV-4).
3. Determine the thickness of the surface, base, and subbase layers using the equation for the standard number as discussed previous section.

The layer coefficients vary from region to region and the values used can be determined through Table IV-6.

The following are provided as minimum practical thicknesses for each pavement course:

Surface course	2 inches
Base course	4 inches
Subbase course	4 inches (if subbase is used)

The differences between this procedure and AASHTO 1986 procedure are as following:

1. Reliability

   AASHTO 1972 : reliability factors are not used.

   AASHTO 1986 : reliability factors are used.

2. Environment

   AASHTO 1972 : regional factors are used.

   AASHTO 1986 : six different climatic regions are used.

3. Soil support

   AASHTO 1972 : empirical soil support scale is used.

   AASHTO 1986 : effective roadbed soil resilient modules values are generated for each of the six U.S. climatic regions.

4. Layer coefficient

AASHTO 1972 : layer coefficient is decided in each region and pavement component.

AASHTO 1986 : layer coefficient scales are used.

EXAMPLE

Given:

ESAL = 150,000

Location : Carver County, Minnesota

Assumption:

Subgrade M_R = 4,500

P_t = 2.5

Regional factor = 2.5

Solution:

Soil support = 4 (from Figure IV-2, M_R = 4,500)

SN = 2.85 (from Figure IV-3)

$SN = a_1D_1 + a_2D_2 + a_3D_3$

From Table II-7, $a_1 = 0.315$

$a_2 = 0.14$

Minimum thickness D_1 = 2 inches

D_2 = 4 inches

and for two-layer, $D_3 = 0$

The following thicknesses would be acceptable:

layer	Thickness (inches)			
D_1	4.0	4.5	5.0	5.5
D_2	11.5	10.5	9.5	8.5

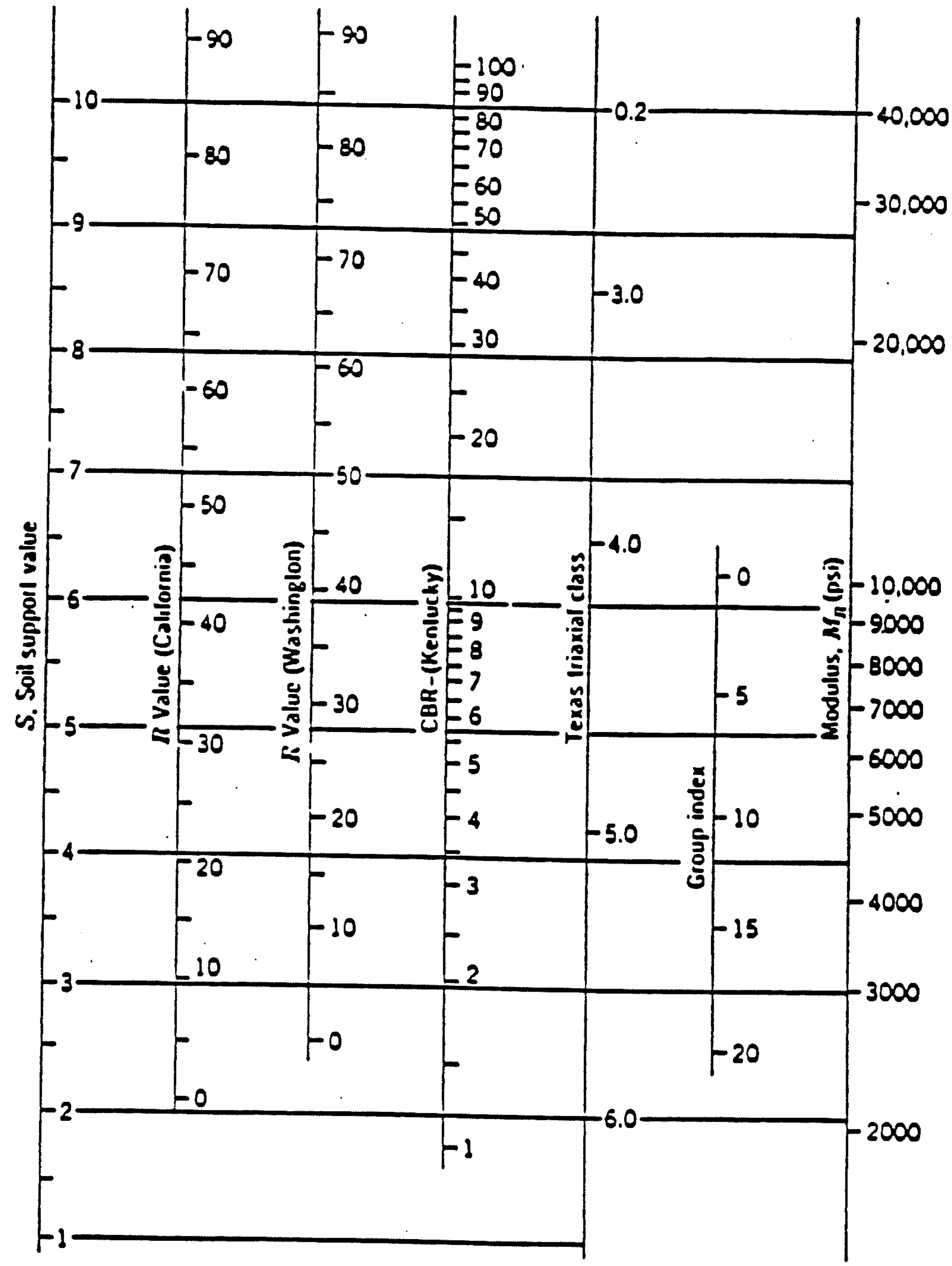


Figure IV-2. Correlation chart for estimating soil support value [12].

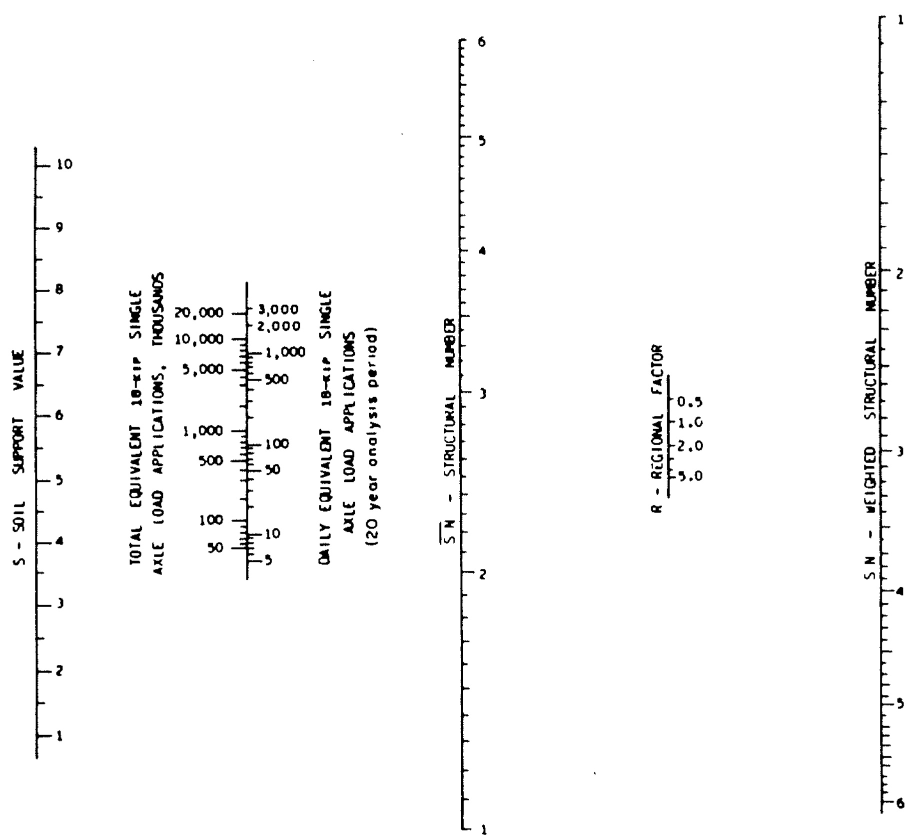


Figure IV-3. AASHTO Design Chart for Flexible Pavements, P_t = 2.5 [9].

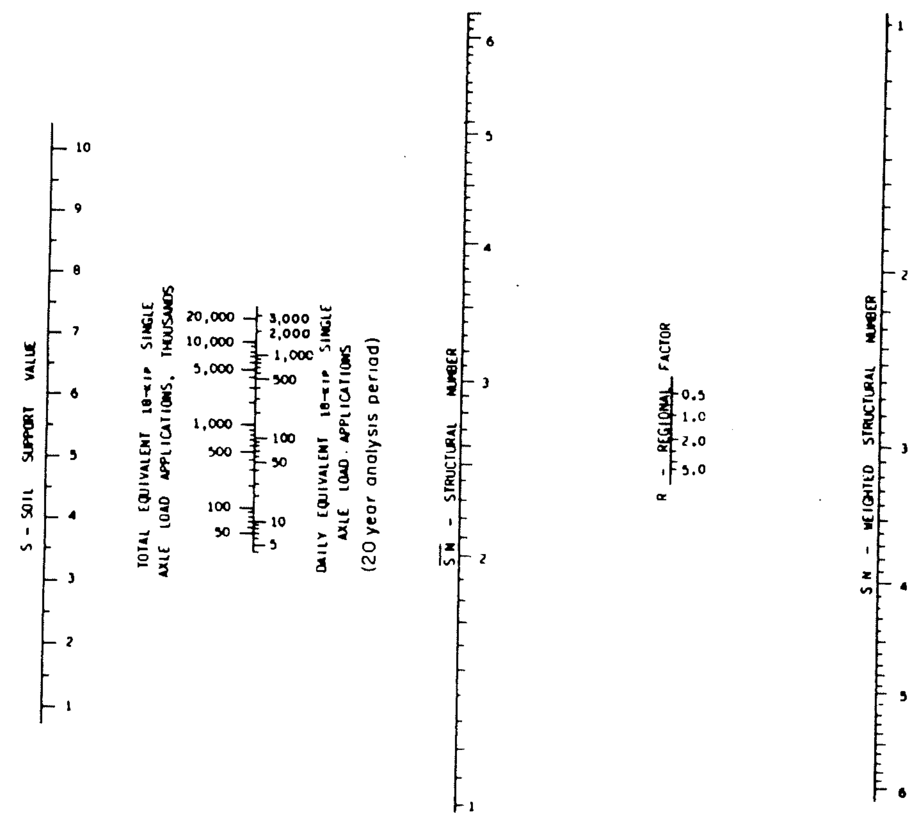


Figure IV-4. AASHTO Design Chart for Flexible Pavements, P_t = 2.0 [9].

Table IV-6. Summary of structural coefficients used for different pavement components for AASHTO Interim Guide 1972 Procedure [9].

COMPONENT	ALABAMA	ARIZONA	DELAWARE	MASSA-CHUSETTS	MINNESOTA	MONTANA	NEVADA	NEW HAMPSHIRE
SURFACE COURSES								
Plant Mix (high stab.)	0.44	.35-.44	.35-.40	0.44	0.315	.30-.40	.30-.35	0.38
Road Mix (low stab.)	0.20	.25-.38				0.20	.17-.25	0.20
Sand Asphalt	0.40	.25			Plant mix (low stab.) 0.28			0.20
Base Courses								
Untreated	limestone 0.14 slag 0.14 sand-stone 0.13 granite 0.12	sand & grav-el, well graded 0.14 cinders .12-.14 sandy gra-vel, most-ly sand .11-.13	Waterbound Macadam 0.20 crusher run .14 quarry waste 0.11 select borrow 0.08	crushed stone .14	crushed rock (Cl. 5 & 6 gravel) 0.14 sandy gravel 0.07	select surf 0.10 crushed gravel 0.12-0.14	crushed gravel .10-.12 crushed rock .13-.16	crushed gravel 0.10 bank run gravel .07 crushed stone 0.14
Cement Treated								
650 psi or more	0.23	500 psi (3.5MPa) .25-.30	soil-cement 0.20			400 psi (2.8MPa) or more 0.20		gravel 0.17
400 to 650 psi	0.20	300-500 psi (2.1-3.5MPa) .18-.25						
400 psi or less	0.15	less than 300 psi (2.1MPa) 0.15				.15		
Lime Treated						.15-.20		
Bituminous Treated	Coarse graded 0.30 sand 0.25	sand-gravel .25-.34 sand .20	asph. stab. .10	black base 0.34 penetrated crushed stone .29	0.175-0.21	plant mix 0.30 bit. stab. 0.20	plant mix .25-.34	bit. conc. 0.34 gravel 0.24
SUBBASE	sand & sandy clay 0.11 chert low P.I. 0.10 top soil 0.09 float gra-vel 0.09 sand & silty clay 0.05	sand-gravel, well graded 0.14 cr. stone or cinders 0.12 sand & silty clay .05-.10	select borrow 0.08	gravel 0.11 select material 0.08	sandy gravel (Cl. 3 & 4 gravel) 0.105 selected granular (12% minus 0.075mm (#200)) 0.07	sand 0.05 sp. borrow 0.07	gravel type 1 .09-.11 select material .05-.09	sand-gravel 0.05

Notes:

1. Indiana, Iowa, Montana, New Jersey, Tennessee, and Puerto Rico – conform to AASHTO Guides
2. North Carolina – conforms to AASHTO Guides, except 0.30 for Bituminous Treated Base
3. North Dakota – conforms to AASHTO Guides, except 0.30 for Bituminous Aggregate Base
4. Maine – conforms to AASHTO Guides with some modification. No further information
5. Maryland – substitution values for materials to replace design thickness of asphalt hot-mix are the AASHTO structural coefficients expressed in equivalent values, in inches

Table IV-6. Summary of structural coefficients used for different pavement components for AASHTO Interim Guide 1972 Procedure [9] (Continued).

COMPONENT	NEW MEXICO	OHIO	PENNSYL-VANIA	SOUTH CAROLINA	SOUTH DAKOTA	UTAH	WISCONSIN	WYOMING
SURFACE COURSES								
Plant Mix (high stab.)	.30-.45	0.40	0.44	0.40	.36-.42	0.40	0.44	.30-.40
Road Mix (low stab.)	0.20	A.C. Interim	0.20			0.20	0.20	
Sand Asphalt			0.35	AC binder 0.35			0.40	
Plant Mix Seal	.25					0.40		
BASE COURSES								
Untreated	.10-.15 quarry rock crushed rock .06-.12	aggregate 0.14 w.b. macadam 0.14	crushed stone .14 dense grade 0.18	crushed rock 0.14		0.12	crushed gravel 0.10 w.b. macadam .15-.20 sand-gravel 0.07	all untreated 0.05-0.12
Cement Treated 650 psi (4.5MPa) or more	0.23		soil cement 0.20				0.23	cement treated .15-.25
400 to 650 psi (2.8 to 4.5MPa)	0.17		cement aggr. plant mix 0.30			0.20	0.20	
400 psi (2.8MPa) or less	0.12						0.15	
Lime Treated	.05-.10		Soil-lime .20		0.15		.15-.30	.07-.12
Bituminous Treated	plant mix 0.30 road mix 0.15		soil bit. 0.20 plant mix 0.30	black base 0.30 sand 0.25	hot mix .30 coarse sand 0.24 fine sand 0.18 cold mix coarse .14 fine 0.10	coarse graded 0.30	coarse graded .34 sand 0.30 coarse H.M. 0.23	plant mix 0.20-0.30 emulsion 0.12-0.20
SUBBASE	aggregate .06-.12 borrow .05-.10	0.11	sand-gravel 0.11		sand-silty clay 0.07 3% lime .06 over 3% lime .05	sand & gravel 0.10 sand or sandy clay .06-.10	sand gravel. .11	special borrow & subbase 0.05-0.12

ASPHALT INSTITUTE "THICKNESS DESIGN - ASPHALT PAVEMENTS FOR HIGHWAYS AND STREETS MS-1"

The design procedure presented in the Asphalt Institute "Thickness Design - Asphalt pavements for Highways and Streets MS -1" [11] is summarized in this section. This method treats the pavement as a multilayer elastic solid. The principal design considerations are to preclude fatigue cracking in the asphalt-bound layer and to limit surface rutting by controlling the vertical compressive strain at the surface of the subgrade.

Subgrade Evaluation

The stiffness of the subgrade is defined by a resilient modulus, M_R, preferably determined from a laboratory repeated load triaxial compression test at a representative water content, dry density, and stress conditions. If such data are not available, the stiffness modulus can be estimated from:

$$M_R(psi) = 1500 * CBR$$

or

$$M_R(psi) = 1155 + 555 * R$$

where

CBR = California Bearing Ratio

R = stabilometer R value.

Environmental Considerations

Different environments are represented by mean annual air temperatures (MAAT). Design curves have been developed for conditions that encompass the temperature ranges in most of the United States: cold (MAAT < 45 °F), warm (45 °F < MAAT < 75 °F) and hot (MAAT > 75 °F). Frost effects have been included where appropriate as shown in Figure IV-5 by using an increased subgrade stiffness for the freezing period and reduced stiffness during the thaw period.

Traffic Considerations

Load repetitions are expressed in terms of 18,000-lb single axle loads and are determined using AASHTO equivalency factors. The Equivalent Axle Load (EAL) is the product of the number of vehicles in each weight class times its truck factor and growth factor as shown below:

$$\text{EAL} = \begin{bmatrix} \text{number of vehicles} \\ \text{in each weight class} \end{bmatrix} * \begin{bmatrix} \text{truck} \\ \text{factor} \end{bmatrix} * \begin{bmatrix} \text{growth} \\ \text{factor} \end{bmatrix}$$

The number of vehicles can be obtained through traffic counts or past data from highway departments. The truck factors (Table IV-7) are determined from axle-weight distribution data using Load Equivalency Factors (Table IV-8). The truck factor is equal to the equation below.

$$\text{Truck factor} = \frac{\text{number of axles} * \text{Load Equivalency Factor}}{\text{number of vehicles}}$$

The growth factor is calculated from the equation:

$$\text{Growth Factor} = ((1+r)^n - 1)/r$$

where

r = growth rate/100

n = design period in years

The design EAL can be calculated by summing the individual EAL's for each of the vehicle classes.

Failure Criteria

The horizontal tensile strain on the underside of the lowest asphalt-bound layer and the vertical compressive strain at the surface of the subgrade layer are critical in this method. If the horizontal tensile strain is excessive, cracking of the treated layer will result. If the vertical compressive strain is excessive, permanent deformation will result at the surface of the pavement structure from overloading the subgrade.

Design Procedures

1. Select or determine input data.

   A. traffic value, EAL,

   B. subgrade resilient modulus, M_R,

   C. surface and base types.

2. Determine design thickness from design chart (Figures IV-6) for the specific conditions described by the input data.

3. Prepare stage construction design, if appropriate.
4. Make an economic analysis of the various solutions arrived at for the design problem.
5. Select final design.

The minimum thicknesses of asphalt concrete over untreated aggregate base are listed following:

Traffic EAL	Minimum thickness of Asphalt concrete
1×10^4	3.0 inches
$10^4 - 10^5$	4.0 inches
1×10^5	5.0 inches

EXAMPLE

Given:

Location : Carver County, Minnesota

Traffic value, EAL = 150,000

Assumption:

Subgrade modulus, M_R = 5,000 psi

Base course : Untreated aggregate

Solution:

From Design charts in this manual, the following design thicknesses would be acceptable:

Layer	Thickness (inches)					
D_1	6.75	6.25	5.50	5.00	4.75	4.00
D_2	4.00	6.00	8.00	10.00	12.00	18.00

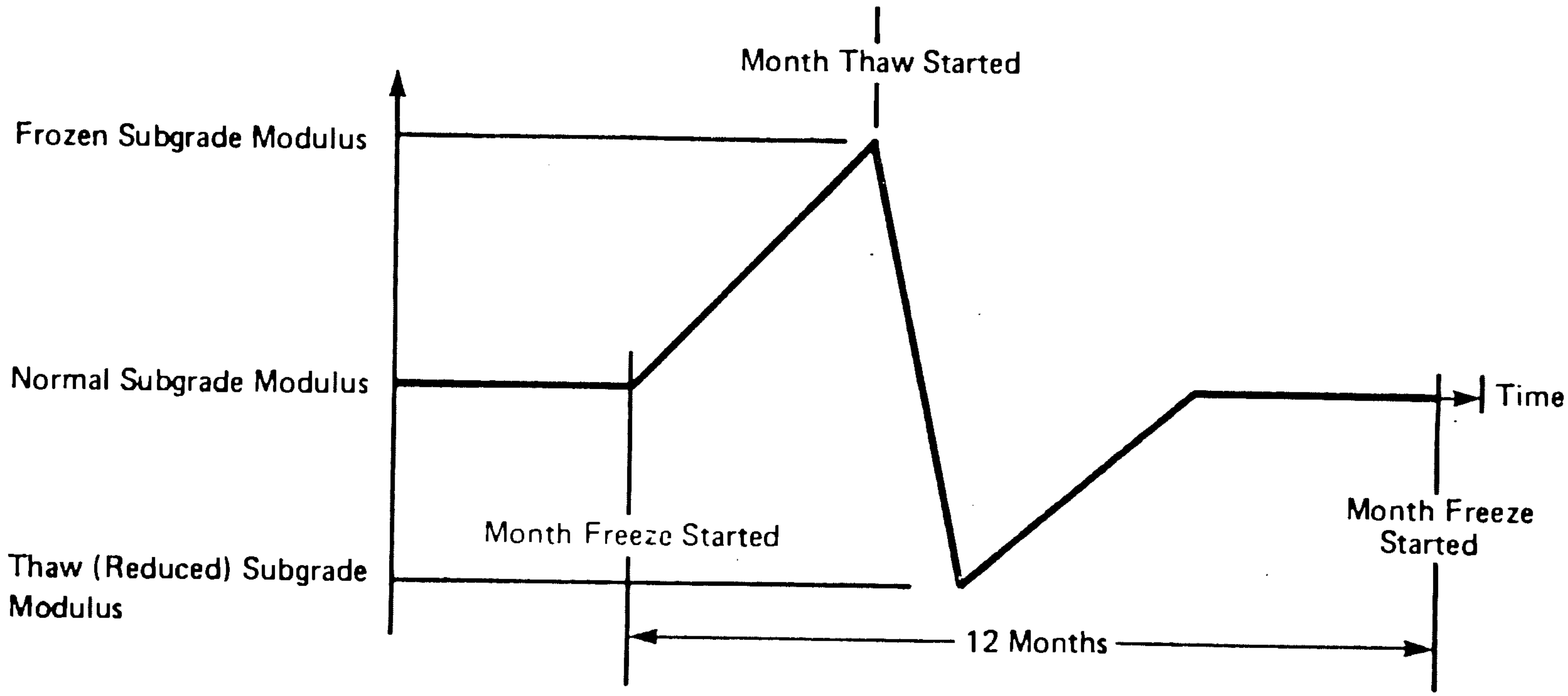


Figure IV-5. Subgrade modulus variations for the conditions where freeze-thaw occurs for Asphalt Institute (MS-1) Procedure [11].

Table IV-7. Distribution of Truck Factors (TF) for different classes of highways and vehicles - United States for Asphalt Institute (MS-1) Procedure [11].

Vehicle Type	Truck Factors									
	Rural Systems						Urban Systems		All Systems	
	Interstate Rural		Other Rural		All Rural		All Urban			
	Average	Range	Average	Range	Average	Range	Average	Range	Average	Range
Single-unit trucks										
2-axle, 4-tire	0.02	0.01-0.06	0.02	0.01-0.09	0.03***	0.02-0.08	0.03***	0.01-0.05	0.02	0.01-0.07
2-axle, 6-tire	0.19	0.13-0.30	0.21	0.14-0.34	0.20	0.14-0.31	0.26	0.18-0.42	0.21	0.15-0.32
3-axle or more	0.56	0.09-1.55	0.73	0.31-1.57	0.67	0.23-1.53	1.03	0.52-1.99	0.73	0.29-1.59
All single-units	0.07	0.02-0.16	0.07	0.02-0.17	0,07	0.03-0.16	0.09	0.04-0.21	0.07	0.02-0.17
Tractor semi-trailers										
3-axle	0.51	0.30-0.86	0.47	0.29-0.82	0.48	0.31-0.80	0.47	0.24-1.02	0.48	0.33-0.78
4-axle	0.62	0.40-1.07	0.83	0.44-1.55	0.70	0.37-1.34	0.89	0.60-1.64	0.73	0.43-1.32
5-axle or more**	0.94	0.67-1.15	0.98	0.58-1.70	0.95	0.58-1.64	1.02	0.69-1.69	0.95	0.63-1.53
All multiple units	0.93	0.67-1.38	0.97	0.67-1.50	0.94	0.66-1.43	1.00	0.72-1.58	0.95	0.71-1.39
All trucks	0.49	0.34-0.77	0.31	0.20-0.52	0.42	0.29-0.67	0.30	0.15-0.59	0.40	0.27-0.63

**Compiled from data supplied by the Highway Statistics Division, U.S. Federal Highway Administration.*
***Including full-trailer combinations in some states.*
****See Article 4.05 for values to be used when the number of heavy trucks is low.*

Table IV-8. Load Equivalency Factors for Asphalt Institute (MS-1) Procedure [11].

Gross Axle Load		Load Equivalency Factors		Gross Axle Load		Load Equivalency Factors	
kN	lb	Single Axles	Tandem Axles	kN	lb	Single Axles	Tandem Axles
4.45	1,000	0.00002		182.5	41,000	23.27	2.29
8.9	2,000	0.00018		187.0	42,000	25.64	2.51
13.35	3,000	0.00072		191.3	43,000	28.22	2.75
17.8	4,000	0.00209		195.7	44,000	31.00	3.00
22.25	5,000	0.00500		200.0	45,000	34.00	3.27
26.7	6,000	0.01043		204.5	46,000	37.24	3.55
31.15	7,000	0.0196		209.0	47,000	40.74	3.85
35.6	8,000	0.0343		213.5	48,000	44.50	4.17
40.0	9,000	0.0562		218.0	49,000	48.54	4.51
44.5	10,000	0.0877	0.00688	222.4	50,000	52.88	4.86
48.9	11,000	0.1311	0.01008	226.8	51,000		5.23
53.4	12,000	0.189	0.0144	231.3	52,000		5.63
57.8	13,000	0.264	0.0199	235.7	53,000		6.04
62.3	14,000	0.360	0.0270	240.2	54,000		6.47
66.7	15,000	0.478	0.0360	244.6	55,000		6.93
71.2	16,000	0.623	0.0472	249.0	56,000		7.41
75.6	17,000	0.796	0.0608	253.5	57,000		7.92
80.0	18,000	1.000	0.0773	258.0	58,000		8.45
84.5	19,000	1.24	0.0971	262.5	59,000		9.01
89.0	20,000	1.51	0.1206	267.0	60,000		9.59
93.4	21,000	1.83	0.148	271.3	61,000		10.20
97.8	22,000	2.18	0.180	275.8	62,000		10.84
102.3	23,000	2.58	0.217	280.2	63,000		11.52
106.8	24,000	3.03	0.260	284.5	64,000		12.22
111.2	25,000	3.53	0.308	289.0	65,000		12.96
115.6	26,000	4.09	0.364	293.5	66,000		13.73
120.0	27,000	4.71	0.426	298.0	67,000		14.54
124.5	28,000	5.39	0.495	302.5	68,000		15.38
129.0	29,000	6.14	0.572	307.0	69,000		16.26
133.5	30,000	6.97	0.658	311.5	70,000		17.19
138.0	31,000	7.88	0.753	316.0	71,000		18.15
142.3	32,000	8.88	0.857	320.0	72,000		19.16
146.8	33,000	9.98	0.971	325.0	73,000		20.22
151.2	34,000	11.18	1.095	329.0	74,000		21.32
155.7	35,000	12.50	1.23	333.5	75,000		22.47
160.0	36,000	13.93	1.38	338.0	76,000		23.66
164.5	37,000	15.50	1.53	342.5	77,000		24.91
169.0	38,000	17.20	1.70	347.0	78,000		26.22
173.5	39,000	19.06	1.89	351.5	79,000		27.58
178.0	40,000	21.08	2.08	356.0	80,000		28.99

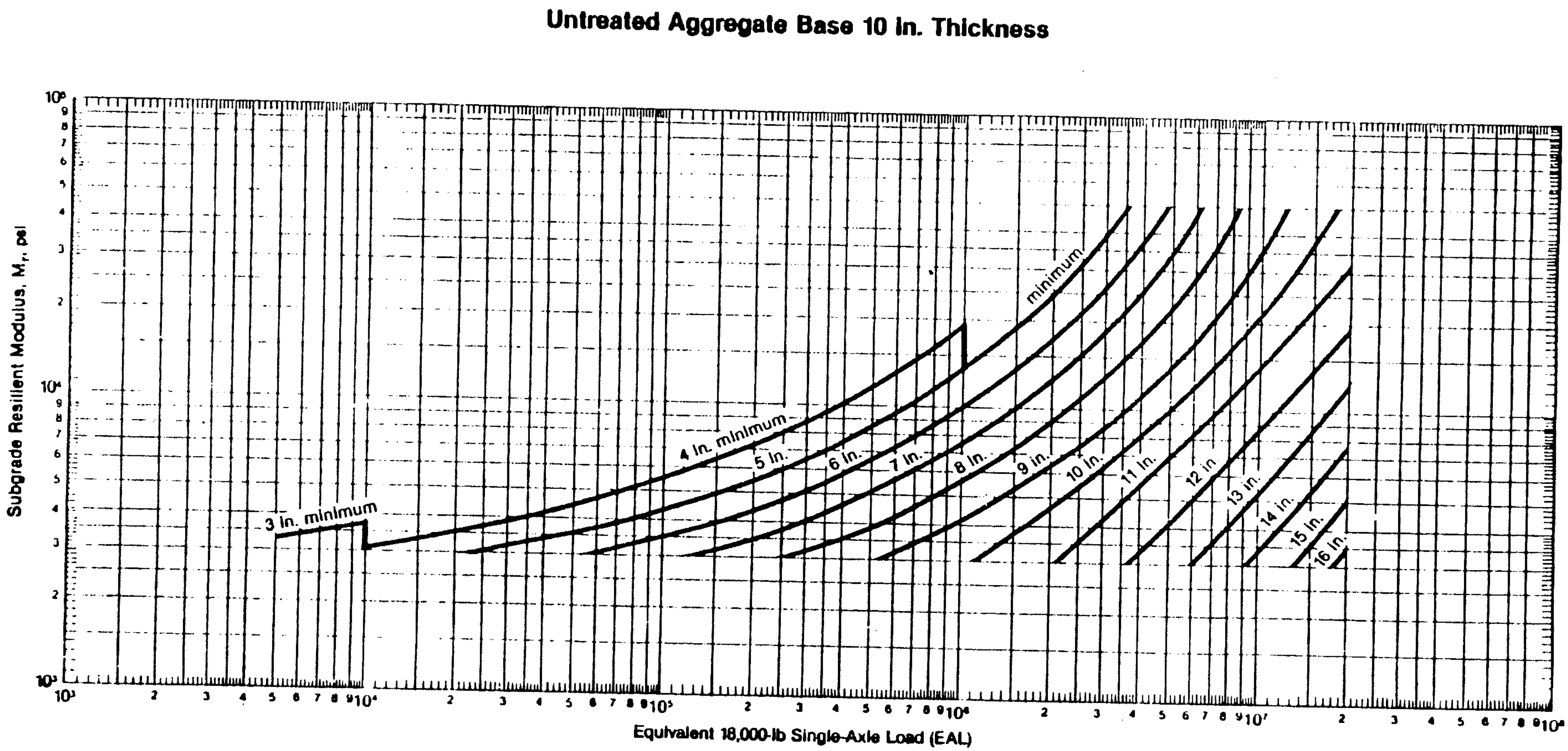


Figure IV-6. Typical design chart for Asphalt Institute (MS-1) Procedure [11].

MINNESOTA DEPARTMENT OF TRANSPORTATION "ROAD DESIGN MANUAL"

The design procedure presented in the Mn/DOT Road Design Manual [12] is summarized in this section. Structural requirements for flexible pavement in this method are a function of the cumulative damage effect of 18,000-pound equivalent single axle loads (Sigma N-18) of the heavy commercial average daily traffic (HCADT), the embankment soil strength (stabilometer R-value), and the whole granular equivalency (G.E.) concept.

Granular equivalency factors provide a means of equating the structural performance of all bituminous and aggregate courses which make up a pavement structure in terms of the structural performance of a aggregate base.

Subgrade Evaluation

The subgrade is evaluated using the stabilometer R-value, which is a measure of embankment soil resistance strength expressed on a scale of 1 to 100 as determined by a laboratory test using the Hveem Stabilometer method. The selection of the R-value is a critical step in the pavement design as structural requirements are considerably influenced by a small change in R-value. Table IV-9 establishes sampling frequency guidelines for Stabilometer R-values as a function of major soil texture. Table IV-10 illustrates typical R-values associated with AASHTO Soil Types. This table is used for small projects where it might be impractical to obtain and test R-value samples.

Environmental Considerations

Since the environmental factors were considered when this design procedure was developed by the Minnesota Department of Transportation for their use, it was not necessary to incorporate them separately.

Traffic Considerations

The MN/DOT method uses a Sigma N-18 value which is a convenient identification of the cumulative damage effect of heavy vehicles during the design life of a flexible pavement. Based upon equivalency factors developed during the AASHO Road Test, the damage effect (vertical deflections) of various types can be equated to that generated by one passage of a standard 18,000 pound single axle load. Identification numbers and descriptions of vehicle types are given in Table IV-11. The steps which are taken in determining the Sigma N-18 value are given below.

1. Convert 2-direction AADT to design-lane volume by multiplying by an approach factor of 0.5.
2. Determine volume of structurally significant Heavy Commercial Average Daily Traffic (HCADT) in the design lane by applying Vehicle Type Distribution Factors (Table IV-12) to the design -lane AADT.
3. If the traffic counts require adjustment because of seasonal variation in traffic, Table IV-13 should be used to adjust the values for the 16-hour class by vehicle type. This will give the seasonally adjusted volume.

4. Determine the present daily total damage effect by multiplying the daily total of each type HCADT vehicle by an average Sigma N-18 factor for that type (Table IV-14) and summarizing.

5. The present daily design-lane Sigma N-18 total is converted to Sigma N-18 for design period by using a Time Growth Factor from Table IV-15 or below equation:

$$\text{Growth Factor} = ((1 + i)^n - 1)/i$$

where

i = grow rate

n = design period (in year)

6. The design-lane Sigma N-18 is estimated by multiplying the total present daily N-18 by the selected time growth factor, and then by the number of days in the year (365).

Failure Criteria

There is no failure criteria in this method. However, a pavement is generally considered to have failed once the Present Serviceability Rating (PSR) has fallen below 2.5. At this level of serviceability there is generally some investment remaining in the pavement.

Design Procedures

The process for designing the structural section is as follows:

1. Determine the total structural requirement (in terms of G.E.) for a specific Sigma N-18 traffic value and embankment R-value with using Figure IV-7.
2. Subtracting the sum of minimum bituminous G.E. and minimum base G.E. from the total G.E. requirement over embankment defines the additional requirement for structural sufficiency.
3. Convert the minimum bituminous and minimum base into thickness of specific construction materials by multiplying the G.E value by the G.E.factor in Table IV-16.
4. Check that the thickness of materials selected possess a combined G.E. value at least equal to the required G.E. over embankment with Tables IV-17 and IV-18.

EXAMPLE

Given:

Location : Carver County, Minnesota

Traffic, Sigma N-18 = 150,000

Assumption:

1-way design-lane AADT = 240

Design R-value = 18

Solution:

1. Determine the minimum required wearing and base course from Table II-18 for AADT = 240.

   Wearing course = 1.5 in. specification 2331

   Binder course = 1.5 in. specification 2331

   Wearing and binder G.E. = 7 in.

2. Determine the minimum Bituminous G.E. and minimum base G.E. from Figure IV-7 for Sigma N-18 = 150,000 and R = 18.

   Minimum bituminous G.E. = 7 in.

   Minimum base G.E. = 10 in.

   G.E. for R-value = 16 in.

3. Calculate the minimum thicknesses of bituminous and aggregate bases in terms of G.E.

minimum bituminous base

$$= \frac{\text{min. bit. G.E. (Table II-18) - min. bit. G.E. (Fig. II-11)}}{\text{G.E. factor of specification}}$$

$$= \frac{\text{7 in. - 7 in.}}{2} = 0$$

minimum aggregate base (class 5)

$$= \frac{\text{minimum base G.E. - minimum bituminous G.E.}}{\text{G.E. factor of specification}}$$

$$= \frac{10 \text{ in.} - 7 \text{ in.}}{1} = 3 \text{ in.}$$

minimum aggregate base (class 3)

$$= \frac{\text{G.E. for R-value - minimum base G.E.}}{\text{G.E factor of specification}}$$

$$= \frac{16 \text{ in.} - 10 \text{ in.}}{0.75} = 8 \text{ in.}$$

4. Calculate actual G.E.

Thickness	Type	G.E. factor	**Actual G.E.**
1.5 in.	2331 wearing	2.00	3 in.
2.0 in	2331 binder	2.00	4 in.
0.0 in.	2331 base	2.00	0 in.
3.0 in	class 5 aggregate	1.00	3 in.
8.0 in.	class 3 aggregate	0.75	6 in.

Table IV-9. Stabilometer R-value sampling frequency guidelines for Mn/DOT Procedure [13].

For cases where R-value samples will be obtained, the following table may be used as a guide in sampling frequency.

Major Soil Texture*	Recommended Minimum Sampling Rate	Minimum Number of Samples
Sands	0 (assume a value of 70 or 75)***	0***
Clays, Clay Loams	1 every two miles	3**
Sandy Loams (nonplastic to slightly plastic)	3 per mile	5
Silt Loams		
Silty Clay Loams		
Plastic Sandy Loams		
Sandy Clay Loams		

* Major soil texture refers to a soil texture significant enough in areal extent to economically justify a change in pavement design.

** Given sufficient local experience, this may be reduced to 1 or 2 samples.

*** If % passing #200 exceeds 15%, then sample and select a Design R value in the same manner as for clay, clay loams. This means that a sufficient number of gradation checks of the sand areas will have to be made to determine if Stabilometer tests are required,

NOTE: Samples should be representive of the upper 4-feet of the proposed road grade as much as possible. In other words, in unbalanced jobs concentrate on the borrow sources; in balanced jobs concentrate on the cuts. If practical, resample the embankment after construction.

Table IV-10. Stabilometer R-values by soil type for Mn/DOT Procedure [13].

AASHTO Soil Type	Textural	Assumed R-Value	Comments*
A-1a	Sands Gravels	75	Excellent confidence in using assumed value.
A-1-b	Sands Sandy Loams (non-plastic)	70	If percent passing number 200 sieve is 15 to 25 percent, R-value may be as low as 25. In such cases, it is highly desirable to obtain laboratory R-values.
A-2-4 & A-2-6	Sandy Loams (non-plastic, slightly plastic, or plastic	30 (70 for LS and LFS)	Loamy Sands and Loamy Fine Sands commonly have R-value of 70. Laboratory R-values range from 10-80 for the entire A-2 classification. It is highly desirable to obtain laboratory R-values, for the Sandy Loams. See Table 7 5 03F for samping frequency.
A-3	Fine Sands	70	Excellent confidence in using assumed value.
A-4	Sandy Loams (plastic) Silt Loams Silty Clay Loams Loams Clay Loams Sandy Clay Loams	20	Laboratory R-values range from 10 to 75. It is highly desirable to obtain laboratory R-values. See Table 7 5.03F for sampling frequency.
A-6	Clay Loams Clays Silty Clay Loams	12	Laboratory R-values commonly occur between 8 and 20.
A-7-5	Clays Silty Clays	12	Data available is limited.
A-7-6	Clays	10	Laboratory R-values commonly occur between 6 and 18.

* Based on data collected by Mn/DOT through 1974.

Note: In using the above assumed R-values for bituminous pavement design it is essential that the subgrade be constructed of uniform soil at a moisture content and density in accordance with Mn/DOT Spec. 2105 and be capable of passing test rolling, Mn/DOT Spec. 2111. To minimize frost heaving and thaw weakening it is also essential that finished grade elevation be placed an adequate distance above the water table. This distance should be at least equal to the depth of frost penetration. In the case of silty soils the distance should be significantly greater.

Table IV-11. Vehicle types for Mn/DOT Procedure [13].

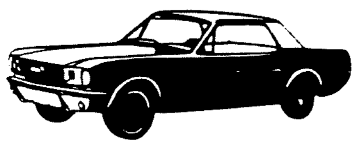
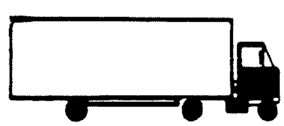
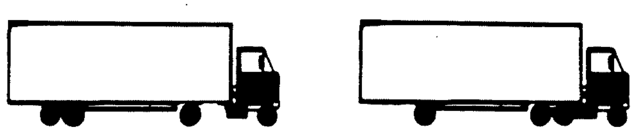
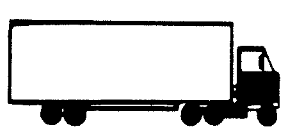
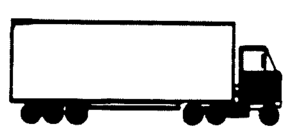
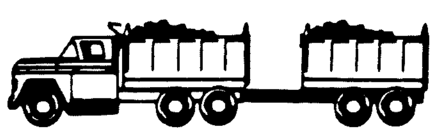
Vehicle Type Number	**Illustrated Example**	**Vehicle Description**
1		Passenger Cars
2		Panel and Pickups (Under 1 ton)
3		Single Unit — 2 axle, 4-tire
4		Single Unit — 2 axle, 6-tire
5		Single Unit — 3 axle and 4 axle
6		Tractor semitrailer Combination — 3 axle
7		Tractor Semitrailer Combination — 4 axle
8		Tractor Semitrailer Combination — 5 axle
9		Tractor Semitrailer Combination — 6 axle
10		Trucks with Trailers and buses

Table IV-12. Assumed Distribution Factors by vehicle type for Mn/DOT Procedure [13].

Vehicle Type	Description	Rural Trunk Highway % of AADT	Metro % of AADT	Local Rural and CSAH % of AADT **
1	Passenger Cars	78.1	83.5	75.7
2	Panels and Pickups (under 1 ton)	10.0	9.0	16.0
3	Single Unit - 2 axle, 4 tire	1.4	1.6	2.4
4	Single Unit - 2 axle, 6 tire	3.9	1.8	2.6
5	Single Unit - 3 axle & 4 axle	1.3	0.5	1.7
6	Tractor Semitrailer Combination - 3 axle	0.3	0.3	—
7	Tractor Semitrailer Combination - 4 axle	0.5	0.4	0.1
8	Tractor Semitrailer Combination - 5 axle	3.0	2.4	0.5
9	Tractor Semitrailer Combination - 6 axle	*	*	*
10	Trucks with Trailers and Buses	1.5	0.5	1.0

*Too few to establish a value at this time.

** Data for local rural roads is from 1975 and 1977 County Roads Pilot Project, and these should not be used in preference to current seasonally adjusted classification counts.

Table IV-13. Seasonal Adjustment Factors for vehicle types for Mn/DOT Procedure [13].

For Trunk Highways and Major Arterials in the Twin City Metro Area							
Time of Data Collection	Vehicle Type						
	1-3	4	5	6	7	8-9	10
January - April	1.44	0.93	1.34	1.03	1.06	1.18	1.33
May - August	1.03	0.91	0.91	0.95	0.87	1.01	1.20
September - December	1.17	0.83	0.88	0.96	0.93	1.04	1.01
For Trunk Highways in the Rural Area							
Time of Data Collection	Vehicle Type						
	1-3	4	5	6	7	8-9	10
January - April	1.54	1.04	1.13	0.94	0.96	1.13	1.35
May - August	0.89	0.84	0.90	0.99	1.09	1.14	1.10
September - December	1.17	0.86	0.96	0.90	0.96	1.02	1.18
Local Rural Roads (CSAH's & County Roads in the Rural Area)							
Time of Data Collection	Vehicle Type						
	1-3	4	5	6	7	8-9	10
December - February	1.32	1.26	1.43	1.00	0.98	1.36	0.95
March - May	1.23	0.84	0.95	0.69	1.14	1.21	0.72
June - August	1.00	0.92	0.74	1.25	0.88	0.95	1.07
September - November	1.10	0.72	0.68	0.92	0.88	0.54	0.70

The data for the Local Rural Roads is taken from the Pilot Project on County Roads conducted between 1975 and 1977.

Table IV-14. Average N-18 Factors by vehicle type for Mn/DOT Procedure [13].

Vehicle Type	Description	Rural T.H. N18 Factor	Metro N18 Factor	Local Rural CSAH & Municipal N18 Factors	Range		
					Max. Legal 10-Ton	Measured	
						Max.	Min.
1	Passenger Car	0.0004	0.0004	0.0004	–	0.0008	0.0003
2	Panels and Pickups (under 1 ton)	0.007	0.007	0.007	3.0	0.012	0.0006
3	Single Unit - 2 axle, 4 tire	0.01	0.01	0.01	3.0	0.070	0.003
4	Single Unit - 2 axle, 6 tire*	0.24	0.22	0.21	3.0	0.61	0.019
5	Single Unit - 3 axle and 4 axle****	0.41	0.57	0.45	2.61	1.40	0.015
6	Tractor Semitrailer Combination - 3 axle	0.58	0.21	0.15	2.20	2.45	0.028
7	Tractor Semitrailer Combination - 4 axle	0.53	0.41	0.30	2.62	3.91	0.060
8	Tractor Semitrailer Combination - 5 axle	0.88	0.63	0.59	2.20	4.10	0.028
9	Tractor Semitrailer Combination - 6 axle	***	***	***	–	–	–
10	Trucks with Trailers and Buses**	0.42	0.42	0.34	–	–	–

* Use 0.60 for 2 axle garbage trucks
** Use 1.25 for MTC Buses
*** Too few to establish a value at this time
**** Use 0.91 for Sugar Beet Trucks

Table IV-15. Time-Growth Factors for design periods of 10 ro 20 years for Mn/DOT Procedure [13].

Annual Growth Factor in Present Daily N18	DESIGN PERIOD		20 Year Projection Factor
	10 Years	20 Years	
0%	10.00	20.00	1.0
0.5%	10.23	20.98	1.10
1%	10.46	22.02	1.22
1.5%	10.70	23.12	1.35
2%	10.95	24.30	1.49
2.5%	11.20	25.54	1.64
3%	11.46	26.87	1.81
3.5%*	11.73	28.28	1.99
4%	12.01	29.78	2.19
4.5%	12.29	31.37	2.41
5%	12.58	33.07	2.65
5.5%	12.88	34.87	2.92
6%	13.18	36.79	3.21

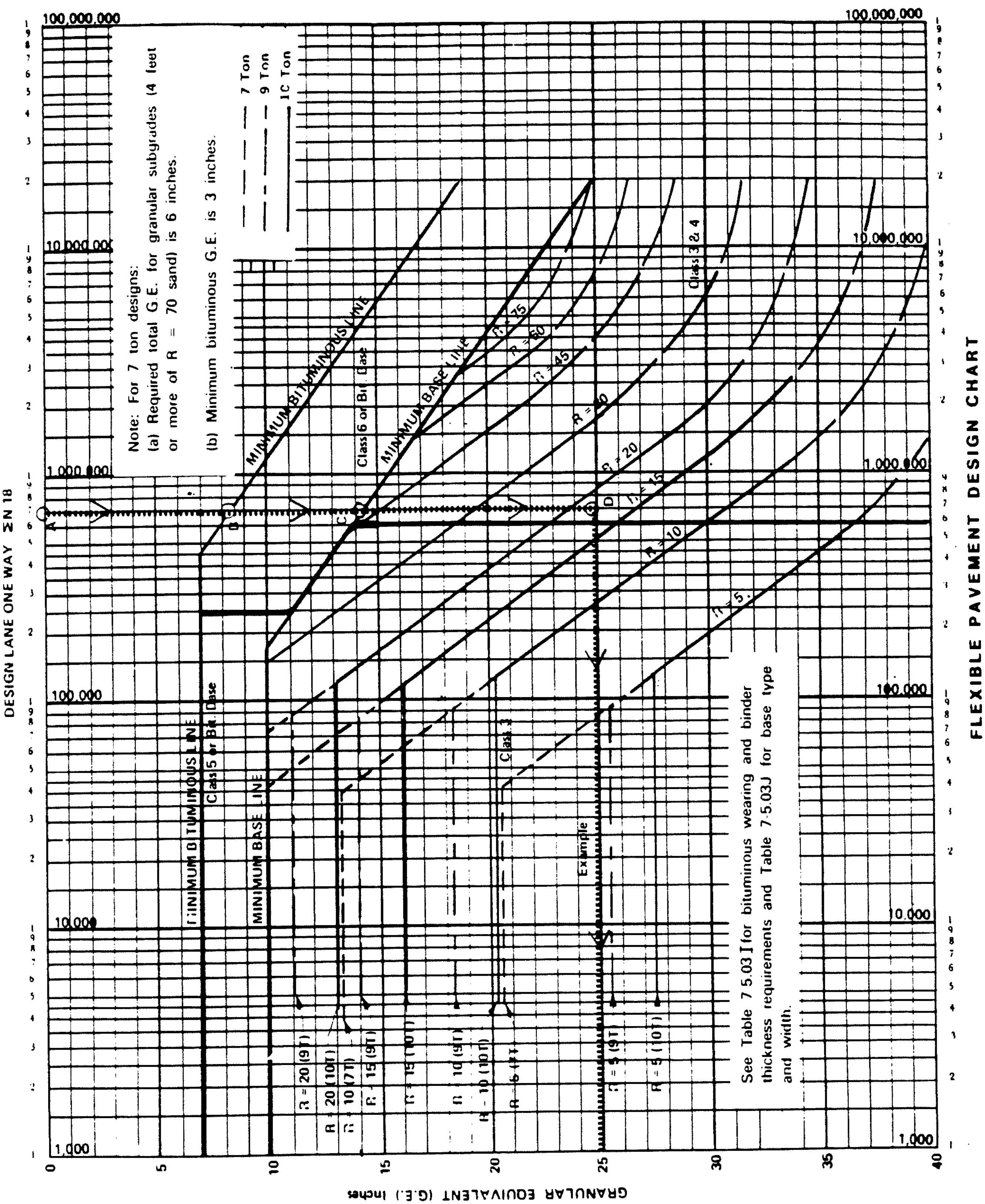


Figure IV-7. Bituminous Pavement Design Chart (Aggregate Base) for Mn/DOT Procedure [13].

Table IV-16. Granular Equivalent (G.E.) Factors for Mn/DOT Procedure [13].

Material	Specification	G.E. Factors
Plant-Mix Surface	2341, 2361	2.25
Plant-Mix Surface	2331	2.00
Plant-Mix Binder	2331	2.00
Plant-Mix Base	2331	2.00
Road-Mix Surface	2321	1.50
Road-Mix Base	2321	1.50
Bituminous Treat. Base	(Rich) 2204	1.50
Bituminous Treat. Base	(Lean) 2204	1.25
Aggregate Base	(Cl. 5, Cl. 6) 3138	1.00
Aggregate Base	(Cl. 3, Cl. 4) 3138	0.75
Selected Granular Material		0.50*

*May be used in design when so approved by the Subgrade and Base Design Engineer.

NOTE: Where the subgrade consists of granular material, the District Materials and or Soils Engineer may recommend the treatment of the upper portion of the selected granular material with 150 lbs sq yd or more of stabilizing aggregate (Specification 3149.2C).

Table IV-17. Bituminous wearing and binder course designs for Mn/DOT Procedure [13].

One-Way Design Lane AADT	Wearing [a. b.]		Binder		Wearing Plus Binder G.E.
	Specification	Thickness	Specification	Thickness	
Less than 500	2331	1-1/2"	2331	2"	7.0"
500 - 2,500	2341	1-1/2"	2331	1-1/2"	6.4"
More than 2,500	2361	3/4"	2331	1-1/2"	4.7"

a. Use a unit weight of 110 lbs./yd^2/in. for all 2331, 2341, and 2361 hot plant-mixed bituminous mixture for estimate and design purposes.

b. District Materials and/or Soils Engineer in conjunction with the Bituminous Engineer may substitute 2341 for 2361 2331 Wear for 2341 Wear.

c. May be specified as bituminous base in lieu of binder.

Table IV-18. Determination of type and width for aggregate base design for Mn/DOT Procedure [13].

One-Way Design Lane Σ N18	Base Type	Base Width[e]
Less than 250,000	Bituminous (2331, 2204, 2321)[a] Class 5 Class 3[b]	24 Feet Full Width Full Width
250,000 to 600,000	Bituminous (2331, 2204, 2321)[a] Class 6[c] Class 3[b]	24 Feet 30 Feet Full Width
More than 600,000	Bituminous (2331, 2204, 2321)[a] Class 6[c] Class 4[d] Class 3[d]	24 Feet 30 Feet Full Width Full Width

a. District Materials and/or Soils Engineer in conjunction with the Subgrade and Base Design Engineer may substitute Specification 2331, 2321, or 2204 for all or a portion of Class 5 and/or Class 6. If the thickness total of the Bit. Base exceeds 3", use width of 27'.

b. District Materials and/or Soils Engineer in conjunction with the Subgrade and Base Design Engineer may substitute the use of Class 4 in place of a portion of Class 3.

c. District Materials and/or Soils Engineer in conjunction with the Subgrade and Base Design Engineer may substitute Class 5 for Class 6.

d. When Class 3 and 4 are required the minimum thickness of Class 4 over Class 3 shall be 6 inches unless otherwise approved by the Subgrade and Base Design Engineer. If less than 6 inches of Class 3 and 4 are required use all Class 4 unless otherwise approved by the Subgrade and Base Design Engineer.

e. On urban sections, all bases are full width.

WISCONSIN DEPARTMENT OF TRANSPORTATION (Wis/DOT) "FACILITIES DEVELOPMENT MANUAL"

The design procedure presented in the Wis/DOT "Facilities Development Manual" [13] is summarized in this section. In general, this method follows the pavement design procedures provided in the "AASHTO Interim Guide for Design of Pavement Structures, 1972, Chapter III Revised, 1981" [9]. Modifications to the AASHTO design procedure have been made to in order to parameters conform to historical data and past experience in Wisconsin.

Subgrade Evaluation

Like the 1972 AASHTO method, the subgrade material strength is evaluated using the soil support scale.

Environmental Considerations

The Wisconsin DOT has altered AASHTO's two-fold 1972 design nomograph (which included a Regional Factor) into a single nomograph (Figures IV-8 and IV-9). Wis/DOT's design nomograph incorporates an AASHTO Regional Factor of 3 into the solution which yields the required structural number (SN).

Traffic Considerations

The traffic is calculated by reducing Wisconsin's truck weight data. This is done by expressing different types of truck loads in terms of equivalent ESAL through application of load factors as given below:

Truck Type	Designation	18,000-lb Equivalent Load Factor
2D	2 axles, 6 tires	0.3
3SU	3 axles	0.8
2S-1, 2s-2	3 or 4 axles	0.5
3S-2	5 axles and above	0.9
2-S1-2	5 axles and above (Double Bottom)	2.0

Multiplying these factors by the number of trucks of each type in the design lane and summing the results gives the total 18,000-lb equivalent load used in the pavement design procedure. Load factors are not given for automobiles and light trucks, as they are insignificant for pavement design purposes. On minor roads the 18,000-lb equivalent load is determined by multiplying the total number of trucks in the design lane by a load factor of 0.2. Five 18,000-lb equivalent loads per day is used as a standard minimum.

Failure Criteria

Major roadways, with a higher traffic volume, are assumed failed at a PSR (Present Serviceability Rating) of 2.5, while lower priority highways, with lower traffic volumes are assumed failed at a PSR of 2.0. A design nomograph has been established for each of these serviceability indices.

Design Procedure

This design procedure is based on the structural number (SN) concept of the AASHTO Interim Guide. Once both the 18,000-lb equivalent load and the soil support value have been determined, a structural number can be obtained directly from the nomograph. This structural number is then used to determine the thickness of the surface, base, and subbase layers.

Table IV-19 is the list of coefficients of relative strength for various construction materials and Table IV-20 is the list of minimum thicknesses of various layers.

EXAMPLE

Given:

Location : Wabasha County, Minnesota

ESAL = 150,000

Assumption:

Surface course = plant mix

Base course = crushed stone

Subgrade M_R = 4,500

P_t = 2.5

Solution:

From Figure IV-2, soil support = 4.

From Figure IV-8, SN = 4.8

$SN = a_1D_1 + a_2D_2$

From Table IV-19, $a_1 = 0.44$

$a_2 = 0.14$

From Table IV-20, $D_1 \geq 1.25$ in.

$D_2 \geq 6.00$ in.

Therefore, the following thicknesses would be acceptable:

layer	thickness (inches)				
D_1	5.0	6.0	6.5	7.0	7.5
D_2	19.0	15.5	14.0	12.5	11.0

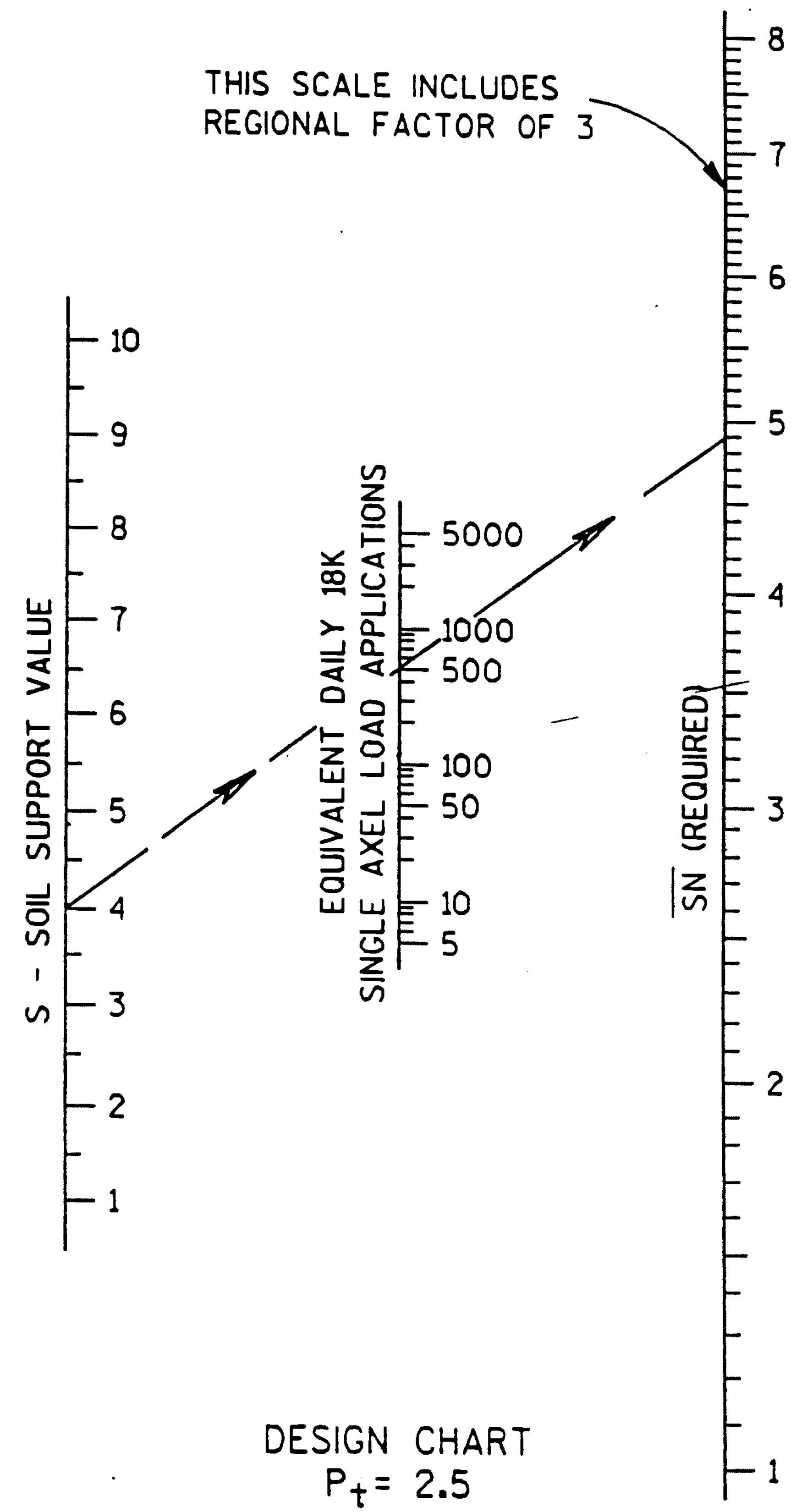


Figure IV-8. Design Chart, P_t = 2.5 for Wis/DOT Procedure [14].

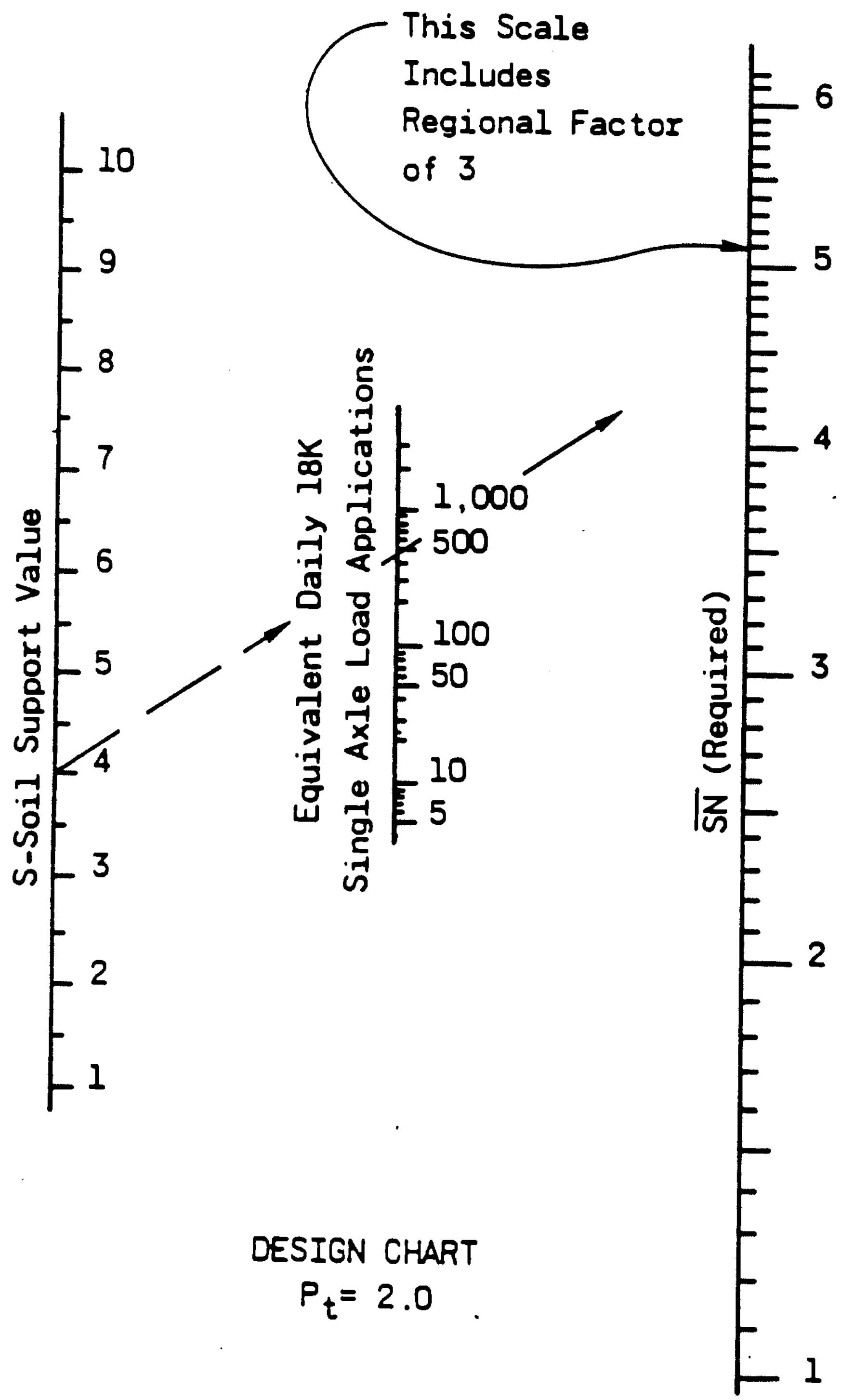


Figure IV-9. Design Chart, P_t = 2.0 for Wis/DOT Procedure [14].

Table IV-19. Relative Strength Coefficients for Flexible Pavements for Wis/DOT Procedure [14].

Pavement Components	Other Requirements	Coefficients a_1	a_2	a_3
Surface Course				
Road mix (low stability)	Marshall stability 500-1,000	0.20	--	--
Plant Mix (high stability)	Marshall stability 2,000	0.44**	--	--
Sand asphalt	Marshall stability 1,000-1,200	0.40	--	--
Base Course				
Sand gravel (uncrushed)	CBR 20-30	--	0.07*	--
Crushed gravel		--	0.10	
Crushed stone	CBR 105-110	--	0.14**	--
Water bound macadam		--	0.15-0.20	--
Lime treated	CBR	--	0.15-0.30	--
Sand asphalt (hot mix)	Marshall stability	--	0.30	--
Bituminous treated (coarse-graded hot mix)	Marshall stability	--	0.34*	--
Cement treated	650 psi 7-day	--	0.23*	--
	400-650 psi 7-day	--	0.20	--
	Less than 400 psi 7-day	--	0.15	--
Subbase				
Sandy gravel	CBR 20-30	--	--	0.11*
Sand or sandy-clay		--	--	See Figure 5

* Estimated on basis of AASHO Road Test data.

** Based on AASHO Test data. All other coefficients determined by assumption based on range of other values in the figure.

Table IV-20. Minimum thicknesses of various layers for Wis/DOT Procedure [14].

LAYER	TYPE		SERVICEABILITY INDEX $P_t = 2.0$	SERVICEABILITY INDEX $P_t = 2.5$
SURFACE COURSE	BITUMINOUS CONCRETE PAVEMENT	SURFACE	3/4"	1- 1/4"
	BITUMINOUS CONCRETE PAVEMENT	BINDER	1- 1/4"	1- 1/4"
	SINGLE AGGREGATE BITUMINOUS SURFACE		1- 1/2"	2"
	BITUMINOUS ROAD MIX SURFACE		1- 1/2"	2"
BASE COURSE	BITUMINOUS		2"	2- 1/2"
	ASPHALT STABILIZED		2- 1/2"	2- 1/2"
	CRUSHED AGGREGATE		5"	6"
SUBBASE COURSE	GRANULAR		6"	6"

NATIONAL CRUSHED STONE ASSOCIATION "DESIGN GUIDE FOR LOW VOLUME RURAL ROADS"

The design procedure presented in the National Crushed Stone Association "Design Guide for Low Volume Rural Roads" [14] is summarized in this section. In this method, one-inch of asphalt is equal to one-inch of gravel. A CBR value and the design index are required to use this method.

Subgrade Evaluation

This design procedure uses the California Bearing Ratio (CBR) to evaluate the subgrade soil. The CBR value is determined either by field testing, laboratory testing or by estimating it from the soil classification (AASHO, Unified Soil Classification (USC) or some other standard method). Based either on CBR tests or on the results of other tests and judicious use of the correlation chart, Figure IV-10, appropriate soil support categories should be assigned. Table IV-21 is the list of the suggested four categories.

Environmental Considerations

Frost classifications established by the Corps of Engineers based on experience and research on a variety of soils have been incorporated into this design method. Figure IV-11 is the U.S. Frost Penetration Map and general definition of four soil groups in ascending order of frost susceptibility are presented in Table IV-22.

Traffic Considerations

Traffic counts on secondary roads should be made separately for each of three groups of vehicle types:

Group 1 -	Passenger cars, panel and pick-up trucks.
Group 2 -	Two-axle trucks loaded, or larger vehicles obviously empty or carrying light cargoes.
Group 3 -	Trucks or combination vehicles having three, four, or more loaded axles

The traffic parameter is characterized by traffic categories called the Design Index (DI). This value is based on ranges of the average equivalent 18,000-lb single-axle loads per lane per day over a life expectancy of 20 years. Below is a listing of the Design Indices used for low-volume roads along with a general characters of each:

DESIGN INDEX	GENERAL CHARACTERS
DI-1	Light traffic (few vehicles heavier than passenger cars, no regular use by Group 2 or 3 vehicles)
DI-2	Medium-light traffic (similar to DI-1, maximum 1000 vehicles per day (VPD), including not over 10% Group 2, no regular use by Group 3 vehicles)
DI-3	Medium traffic (maximum 3000 VDP, including not over 10% Group 2 and 3, 1% Group 3 vehicles)

DI-4 Medium-heavy traffic (maximum 6000 VPD, including not over 15% Group 2 and 3, 1% Group 3 vehicles)

Failure Criteria

No failure criteria were listed in the manual. However,a pavement is generally considered to have failed once the Present Serviceability Rating (PSR) has fallen below 2.5 for major road or 2.0 for minor road.

Design Procedure

Once the CBR value and traffic Design Index have been determined the total design thickness is obtained using the design chart (Figure IV-12) or Table IV-23. This value is then compared to the thickness design value from the Frost Depth design table (Table IV-24). The maximum design value is then used as the design gravel thickness. The bituminous surfacing is determined from a table using the traffic Design Indices:

DESIGN INDEX	MINIMUM SURFACING REQUIRED
DI-1	1.0 inch (use surface treatments)
DI-2	2.0 inches
DI-3	2.5 inches
DI-4	3.0 inches

The surface thickness is subtracted from the design thickness, and the remaining thickness is subdivided into a base and a subbase thickness.

EXAMPLE

Given:

Location : Mille Lacs County, Minnesota

ADT = 500

Assumption:

CBR = 13

Subgrade soil frost group : F-2

Design index category for traffic : DI-2

Solution:

From Table IV-23, total design thickness = 8 in.

From Table IV-24, total design thickness = 12 in.

From Figure IV-12, total design thickness = 7 in.

Total thickness of 12 in is used.

Minimum thickness of D_1 = 2.0 in.

Therefore, the following thicknesses would be acceptable:

layer	thickness (inches)				
D_1	3.0	3.5	4.0	4.5	5.0
D_2	9.0	8.5	8.0	7.5	7.0

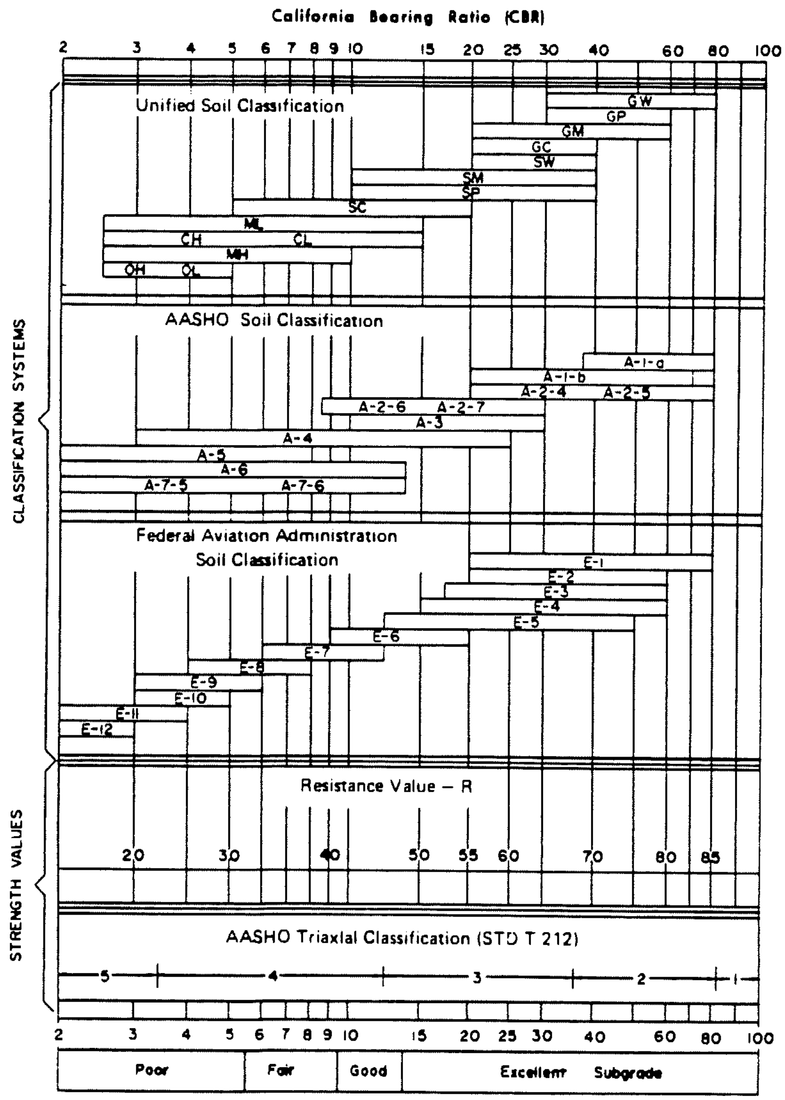


Figure IV-10. Correlation Chart for National Crushed Stone Association Procedure [15].

Table IV-21. Soil Support Categories for National Crushed Stone Association Procedure [15].

GENERAL SOIL DESCRIPTION	STRENGTH-CBR
EXCELLENT	**15 plus**
Containing a uniformly high percentage of granular materials - Unified Soil Classes: GW, GM, GC, GP some SM, SP, and SC - AASHTO Soil Groups: A-1, A-2, some A-3's	
GOOD	**10 - 14**
Containing some granular materials intermixed with silt and/or light clay - Unified Soil Classes: SM, SP, SC some ML, CL, CH - AASHTO Soil Groups: A-2, A-3; some A-4's a few A-6's, or A-7's	
FAIR	**6 - 9**
Sand clays, sandy silts or light silty clays if low in mica content; may have some plasticity - Unified Soil Classes: ML, CL; some MH, CH - AASHTO Soil Groups: A-4 to A-7 (low group indices)	
POOR	**5 or less**
Plastic clays, fine silts, very fine or micaceous silty clay - Unified Soil Classes: MH, CH, OL, OH; (PT unsuitable) - AASHTO Soil Groups: A-4 to A-7 (higher group indices)	

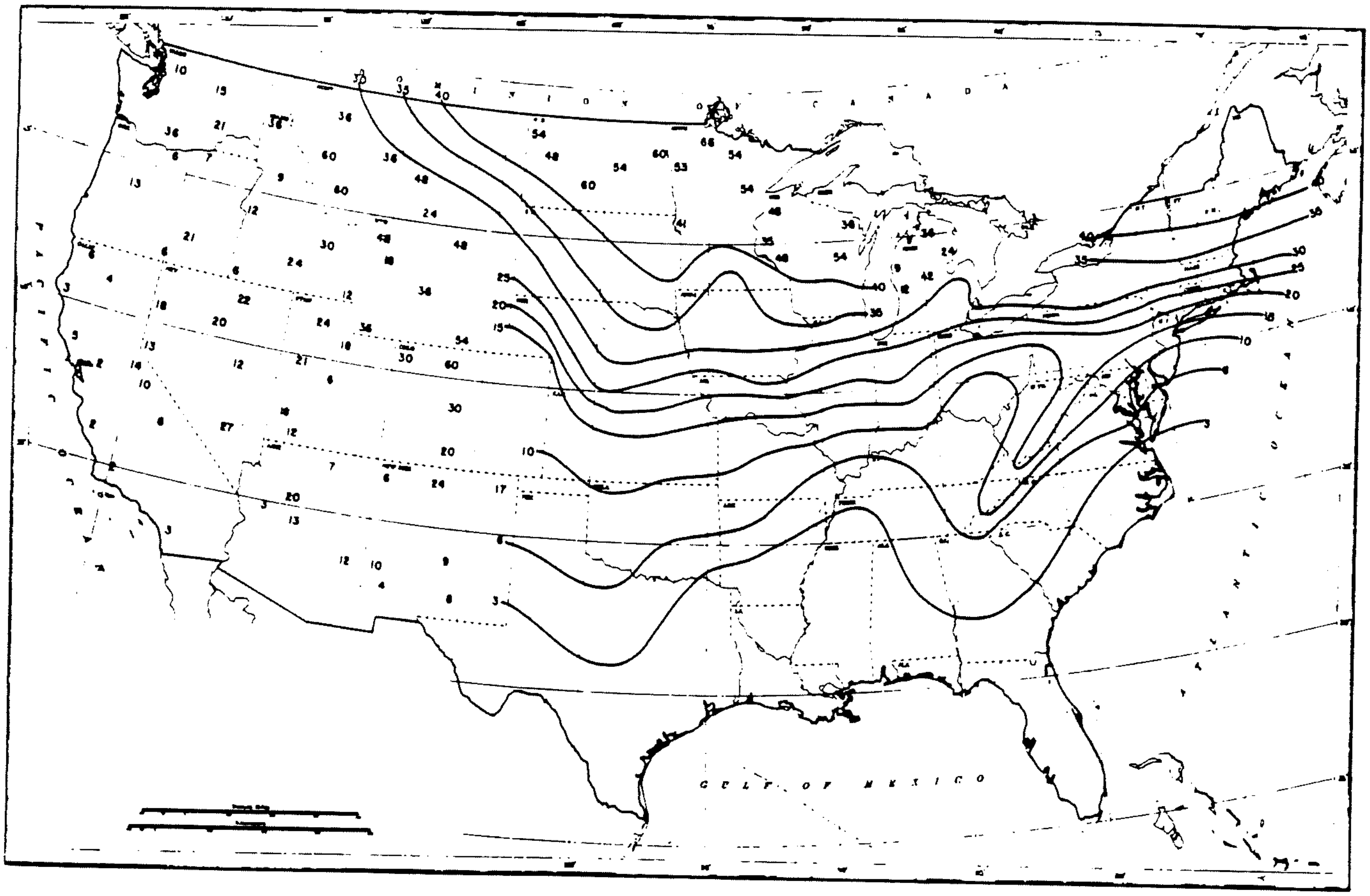


Figure IV-11. U.S. Frost Penetration Map for National Crushed Stone Association Procedure [15].

Table IV-22. General Definition of Four Soil Groups for National Crushed Stone Association [15].

Frost Group	% Finer than 0.02 mm	USC	Frost Susceptibility
F-1			
a) Gravelly Soils	3 - 10	GW, GP, GW-GM, or GP-GM	low
F-2			
a) Gravelly Soils	10 - 20	GM, GW-GM, or GP-GM	low to medium
b) Sands, Sand Clays	3 - 15	SW, SP, SM, SW-SM, or SP-SM	
F-3			
a) Gravelly Soils	over 20	GM or GC	
b) Sands, coarse to medium	over 15	SM or SC	high
c) Clays, PI>12	--	CL or CH	
F-4			
All silts, very fine silty sands, clays w/PI<12, etc.	over 15	ML, MH, SM, CL, CL-ML, CH and alternately banded deposits	very high

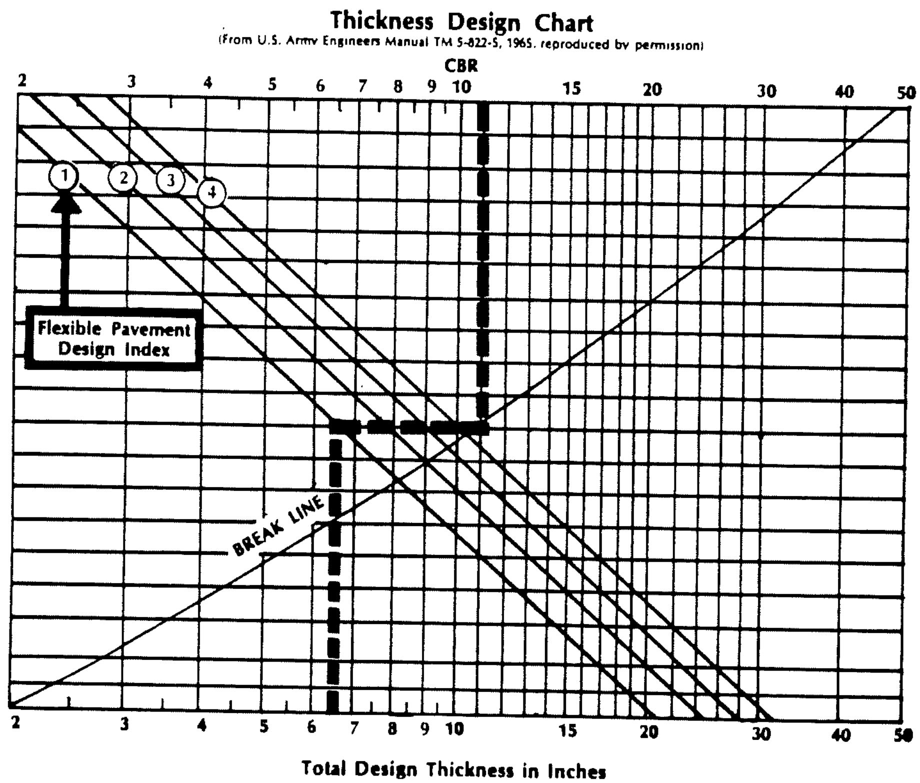


Figure IV-12. Thickness Design Chart for National Crushed Stone Association Procedure [15].

Table IV-23. Total design thickness (CBR basis) for National Crushed Stone Association Procedure [15].

Subgrade Soil		Design Thickness[1] (inches)			
Category	CBR	DI-1	DI-2	DI-3	DI-4
Excellent	15+	5	6	7	8
Good	14 - 10	7	8	9	10
Fair	6 - 9	9	11	12	14
Poor	< 6	See Note 2 below			

[1] Includes base and all bituminous layers.

[2] Subgrade Stabilization - Poor subgrade soils should be improved to the "fair" or better category before base construction is begun by admixing or beneficiating with two-inch maximum size crushed stone products in the top layer.

Table IV-24. Total design thickness (Frost Group basis) for National Crushed Stone Association Procedure [15].

Subgrade Soil — Frost Group	Design Thickness[1,2] (inches)			
	DI-1	DI-2	DI-3	DI-4
F-1 (Gravelly soils)	9	10	12	13
F-2 (Silty gravel, sand, sandy clays)	10	12	14	16
F-3 (Clay gravels, plastic sand clays)	See Note 2 below			
F-4 (Silts, silty sand, silty clays)	See Note 2 below			

[1] Includes base and all bituminous layers. Design thickness may be conservative except where both adverse moisture, conditions and deep freezes are common. Where soil moisture, conditions and drainage are both good, little modification from design based on CBR (Table IV-23) should be necessary. Check designs of successful road pavements in some vicinity for guidance.

[2] Subgrade Stabilization. F-3 and F-4 soils should be upgraded by admixing with crushed stone products in the top layer (Maximum size should be kept no greater than 2 inches). Use 100 to 400 lbs./sq. yd., mix, bring to suitable moisture content, and compact. This stabilization, which provides, a "working platform," should be carried out to a depth of expected frost penetration, considering economic feasibility. (See U.S. Frost Penetration Map, Figure IV-11).

SHELL PAVEMENT DESIGN MANUAL

The design procedure presented in the Shell Pavement Design Manual [15] is summarized in this section. In this method the pavement structure is regarded as a linear elastic multi-layer system in which the materials are characterized by Young's modulus of elasticity (E) and Poisson's ratio (ν). The materials are assumed to be homogeneous and isotropic. The layers are in the horizontal direction, and the bottom layer is infinite in the vertical direction as well.

In the basic design procedure, the pavement is regarded as a three-layer system (Figure IV-13). The lowest layer in the structure represents the subgrade. The middle layer represents the unbound base and subbase layer or, in composite-type constructions, the layers bound with cement, hydrated lime or granulated slag cement. The top layer represents all asphalt or bitumen-bound layers. All layers are considered to have complete friction between them.

Subgrade Evaluation

The dynamic modulus of the subgrade (E_3) is one of the principal design parameters in this method. It can be estimated from the laboratory tests (repeated load tri-axial tests) or following expression (Fig. IV-14):

$$E_3 = 10^7 * CBR \ (N/m^2),$$

where

CBR = California Bearing Ratio.

Environmental Considerations

The influence of climate is incorporated through the use of a weighted mean annual air temperature (w-MAAT). This value is derived from mean monthly air temperatures (MMAT) for a given location and is related to an effective asphalt temperature and thus to an effective asphalt stiffness.

The w-MAAT was obtained using the linear summation of cycle ratios concept. Weighing factors have been derived by which the MMAT is multiplied so that a single temperature for the year will produce the same damage as that resulting for 12 monthly temperatures summed throughout the year (Fig. IV-15).

It is also possible to include the effects of differing subgrade stiffnesses which may result from different environmental influences such as freezing and thawing.

Traffic Considerations

Traffic is represented in terms of repetitions of an 80 kN standard single axle on dual tires with a contact stress of 600 kN/m^2. Conversion of other axles to repetitions of the standard axle is done by the following relationship (Fig. IV-16):

$$N_e = 2.4 * 10^{-8} L^4,$$

where

N_e = conversion factor

L = the axle load in kN.

Tandem axles are treated as two separate axles. A time of loading of 0.02 sec is used and corresponds to a speed of about 50 - 60 km/hour.

Failure Criteria

In this method, fatigue and rutting are the two distress modes which are considered. Tensile strain repeatedly applied is the damage determinant for fatigue. The linear summation of cycle ratio is used as the cumulative damage hypothesis. This hypothesis can be stated as:

$$\sum_{i=1}^{n} \frac{n_i}{N_i} = 1$$

where

n_i = actual number of applications at strain level i

N_i = permissible number of applications at strain level i

To minimize rutting at the pavement surface from permanent deformation in the subgrade, the following expression relating vertical compressive strain at the subgrade surface, ε_z, to load (80 kN) applications is used.

$$\varepsilon_z = C * 10^{-2}(N)^{-0.25}$$

where

C = 2.8 when a confidence level is 50 percent

2.1 when a confidence level is 85 percent

1.8 when a confidence level is 95 percent.

Design Procedure

The Shell method of pavement design can summarized in the following steps:

1. Determine cumulative number of 80 kN axles (N) for design life.
2. Determine w-MMAT from MAATs.
3. Determine subgrade modulus and modulus of unbound base materials available.
4. Select appropriate mix code in accordance with stiffness and fatigue characteristics and bitumen type.
5. Read thickness design charts (Figures IV-17 to 21) for various values of w-MAAT, N and E_3 for appropriate mix codes; check required modulus for unbound base layers.
6. Make interpolations, if necessary, for given values of w-MAAT, N and E_3 for appropriate mix code.
7. Tabulate candidate constructions.
8. Estimate permanent deformation of candidate constructions.

EXAMPLE

Given:

Location : Ramsey County, Minnesota

ESAL = 1,000,000

Assumption:

w-MAAT = 12 C°

Subgrade modulus, $E_3 = 5 \times 10^7$ N/m^2

Surface : asphalt base mix (mix code is S1-F2-50) or

Lean sand asphalt (mix code is S2-F2-50)

Base : crushed gravel (CBR = 80) or

gravel/sand (CBR = 20)

Solution:

- Please see Table IV-25.
- Unbounded base thicknesses of 0, 300 and 500 mm are considered.
- The total asphalt thicknesses required for structures with h_2 = 0 and 300 mm are obtained from charts TN and with h_2 = 500 mm is obtained from charts HT.
- Any of the charts HN or HT for $E_3 = 5 \times 10^7$ N/m^2 can be used to select the sub-layers of the unbound base layer in accordance with the materials available.

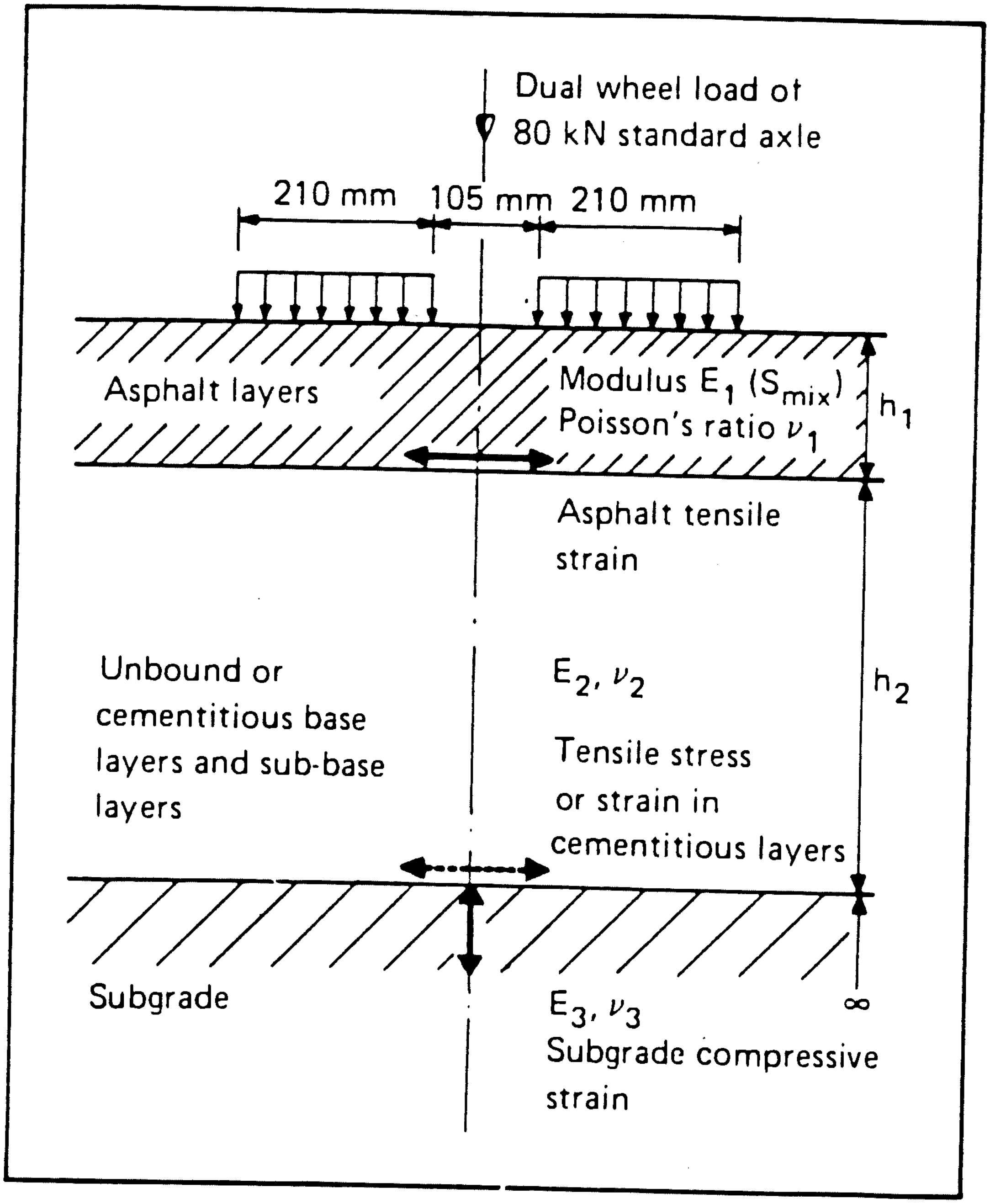


Figure IV-13. Simplified pavement structure for Shell Pavement Design Manual Procedure [16].

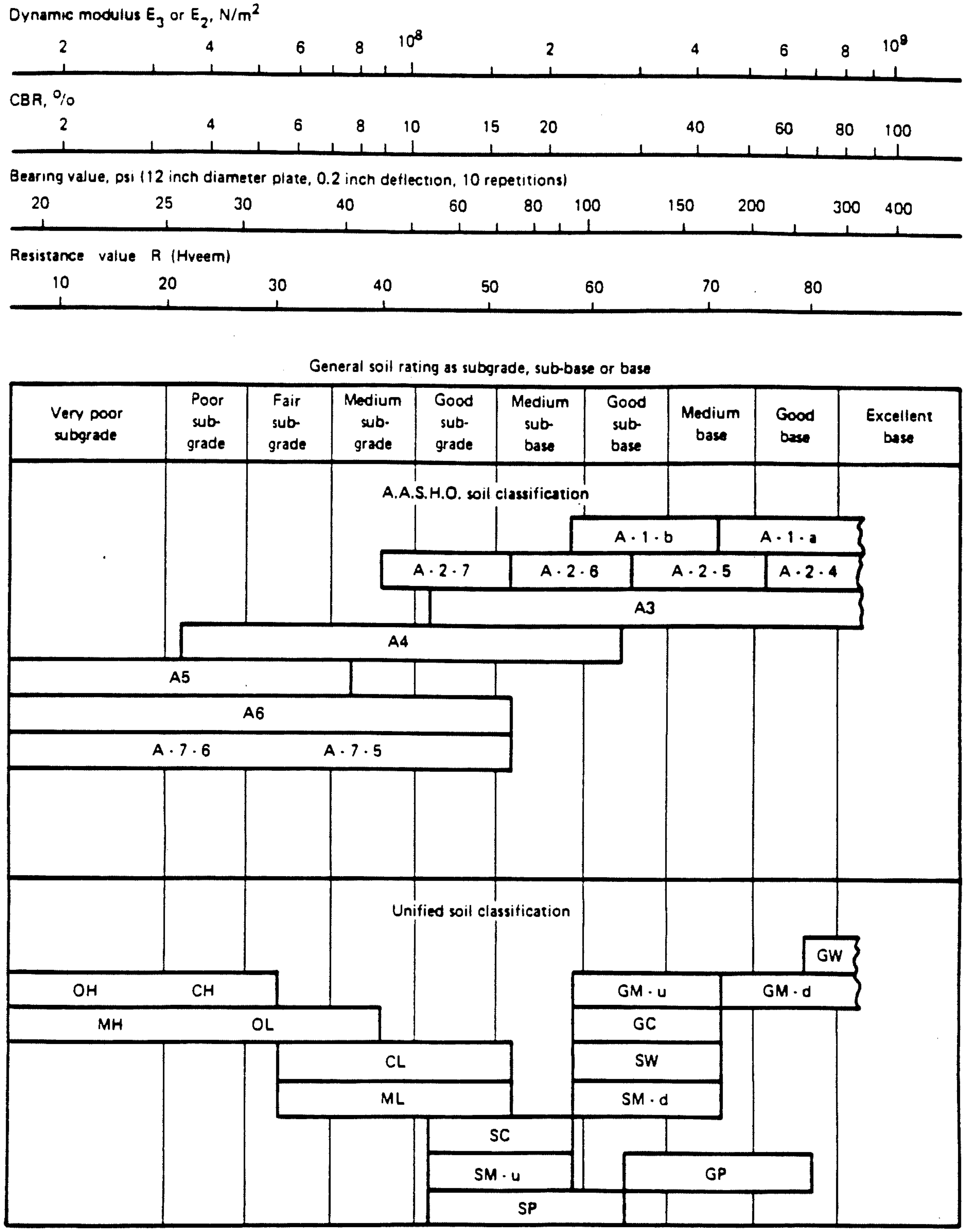


Figure IV-14. Estimation of dynamic modulus of subgrade (E_3) or of unbound base materials (E_2) for Shell pavement design manual procedure [16].

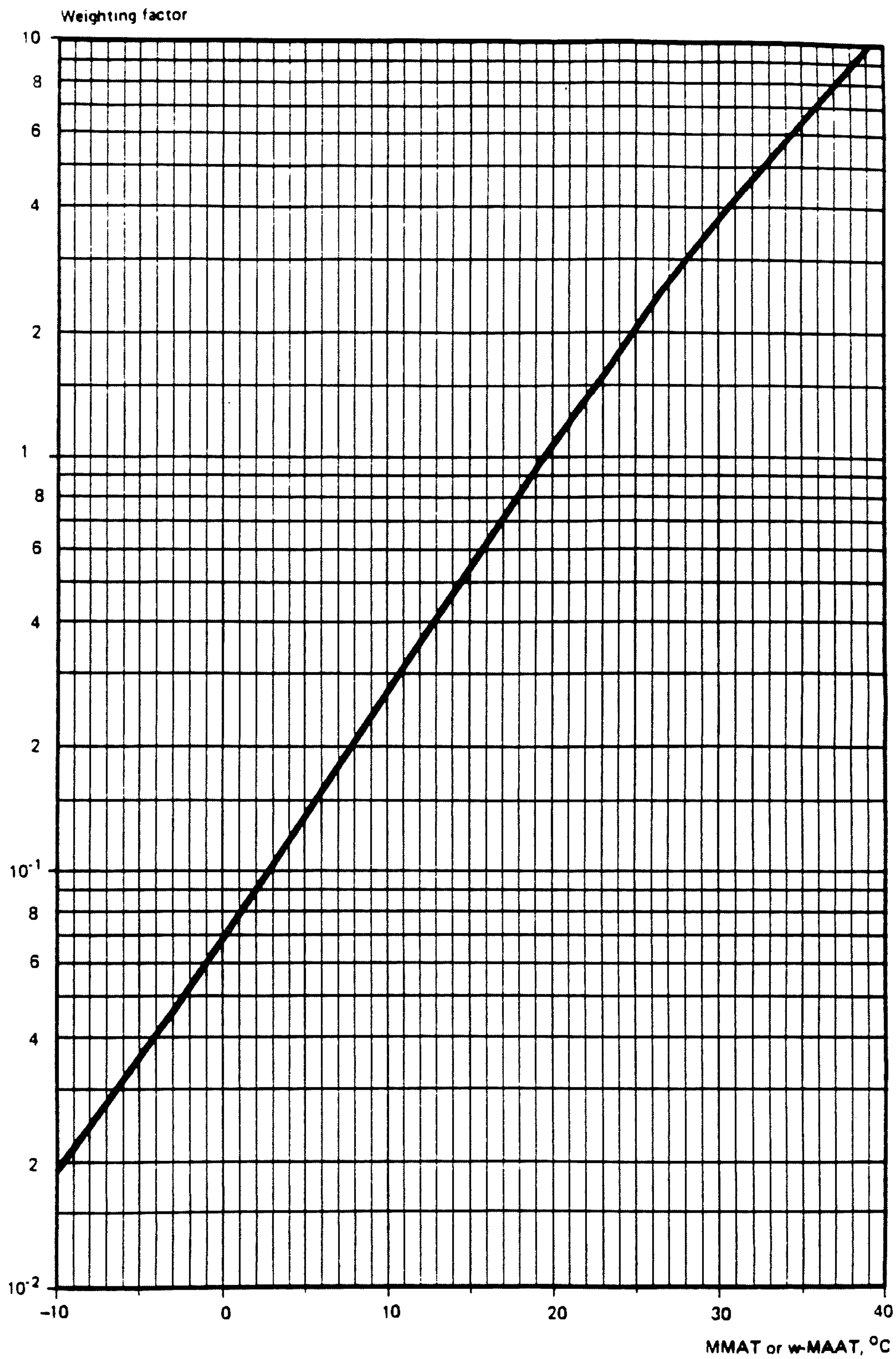
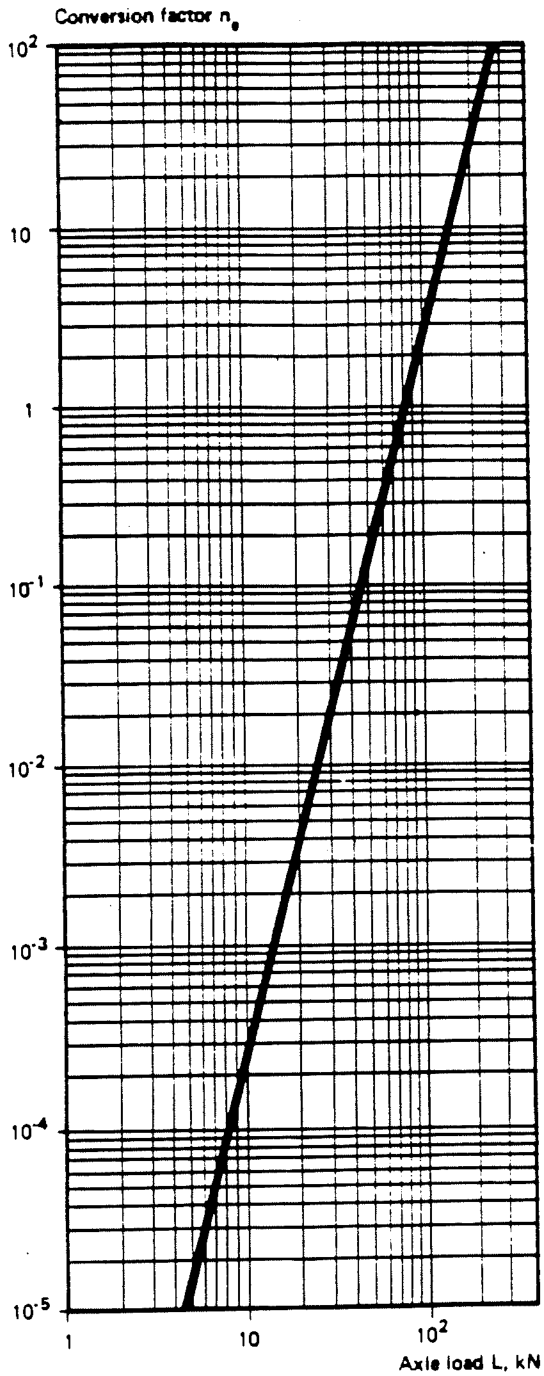


Figure IV-15. Temperature weighing curve for Shell pavement design manual procedure [16].



Axle load L, kN	Conversion factor n_e
5	0.000015
9	0.000157
10	0.000240
20	0.00384
27	0.0127
30	0.019
40	0.061
44	0.090
50	0.15
60	0.31
62	0.35
70	0.58
80	1.00
90	1.57
100	2.40
110	3.51
120	4.98
130	6.85
140	9.22
150	12.15
160	15.73
170	20.05
180	25.19
190	31.28
200	38.40

$n_e = 2.4 \times 10^{-8} \times L^4$

Figure IV-16. Axle load conversion for Shell pavement design manual procedure [16].

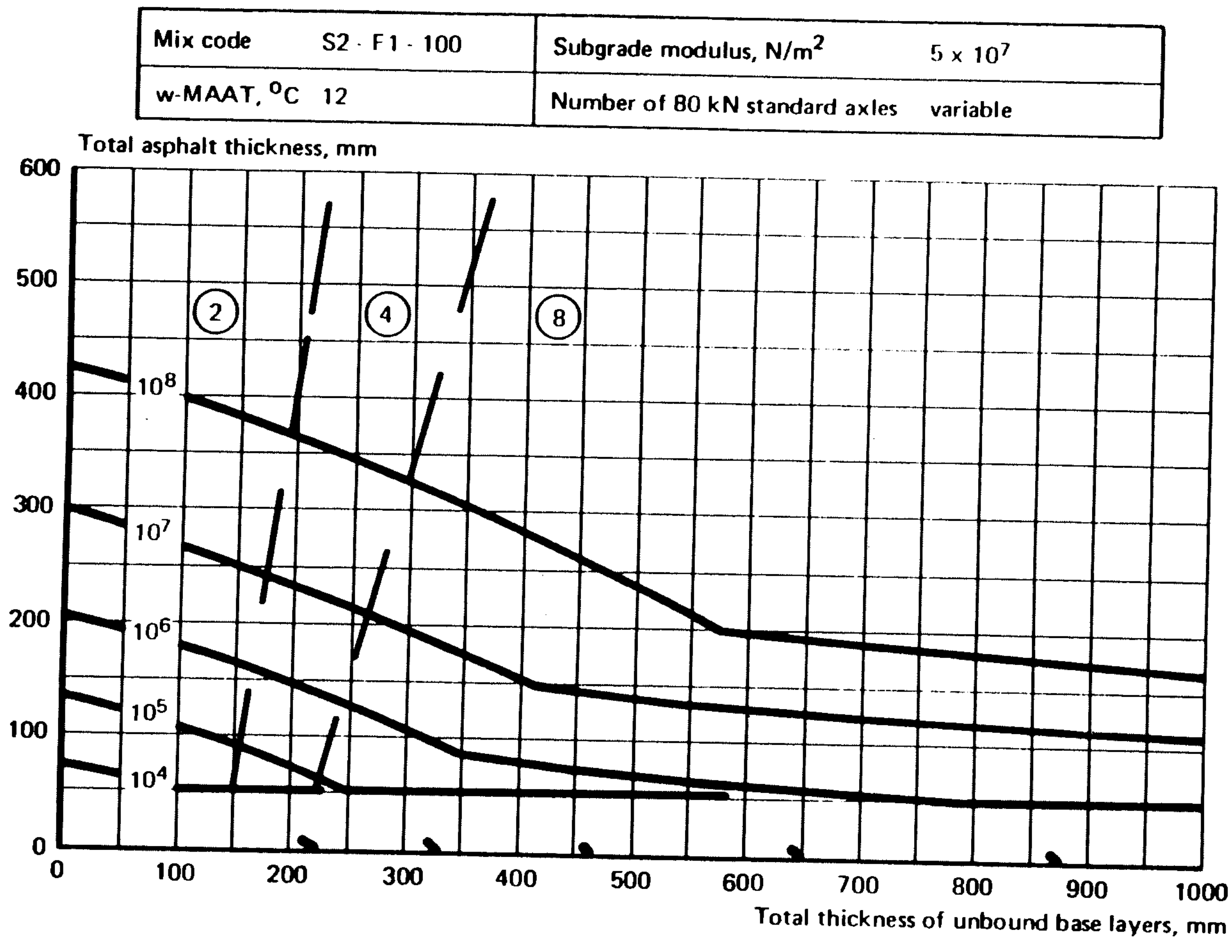


Figure IV-17. Typical design chart for various 80 kN axle load applications for Shell Pavement Design Manual [16].

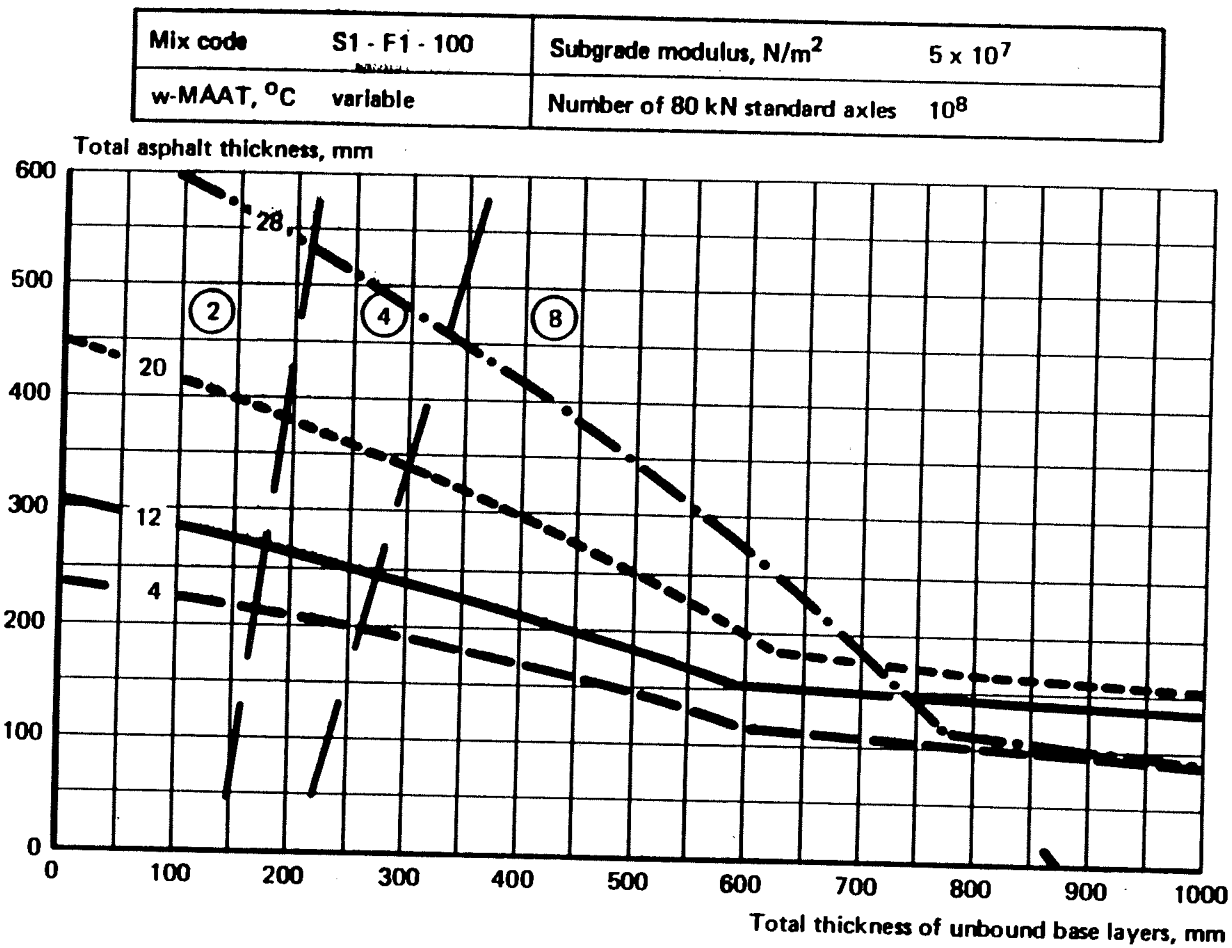


Figure IV-18. Typical design chart for various 80 kN axle load applications for Shell Pavement Design Manual [16].

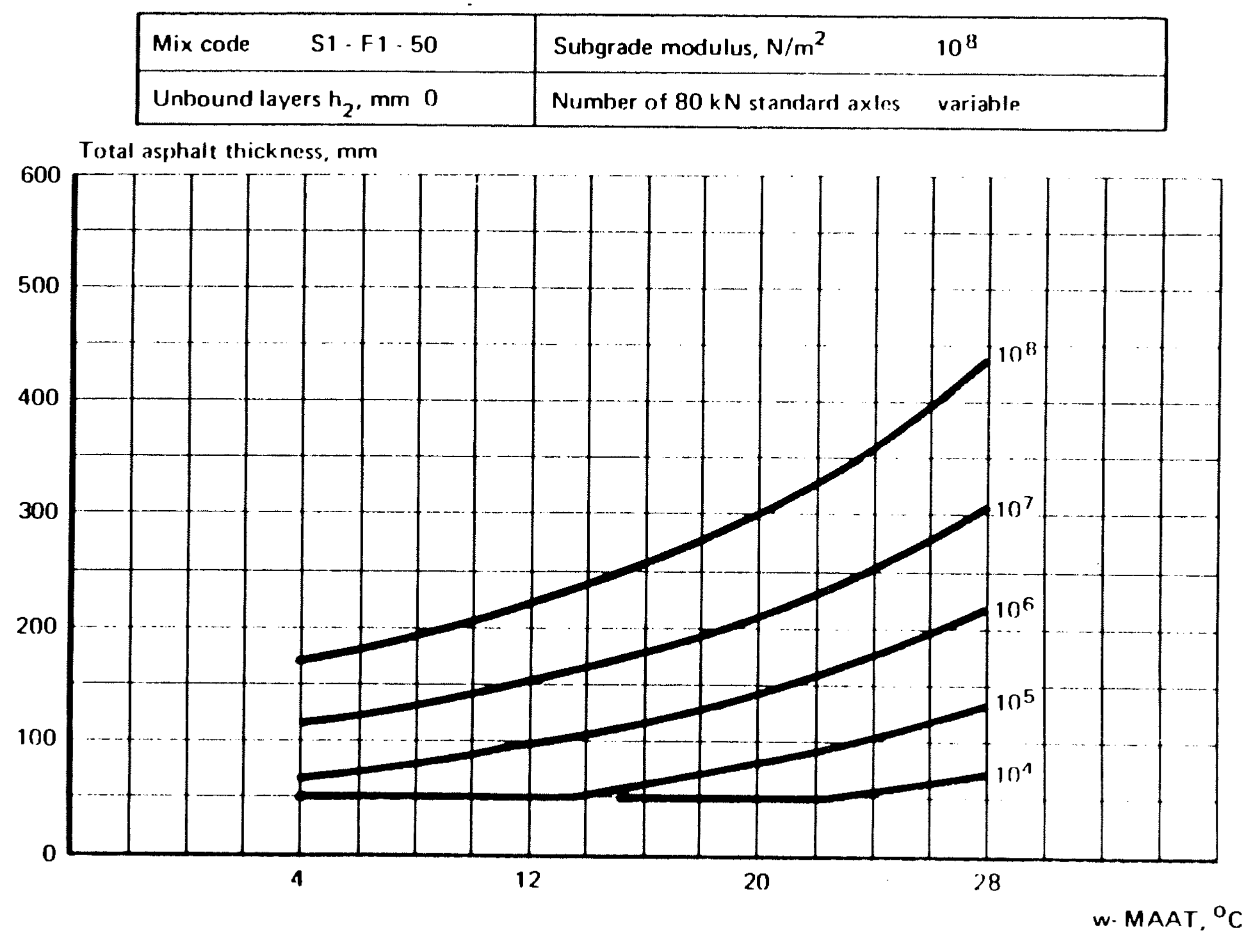


Figure IV-19. Typical design chart for various 80 kN axle load applications for Shell Pavement Design Manual [16].

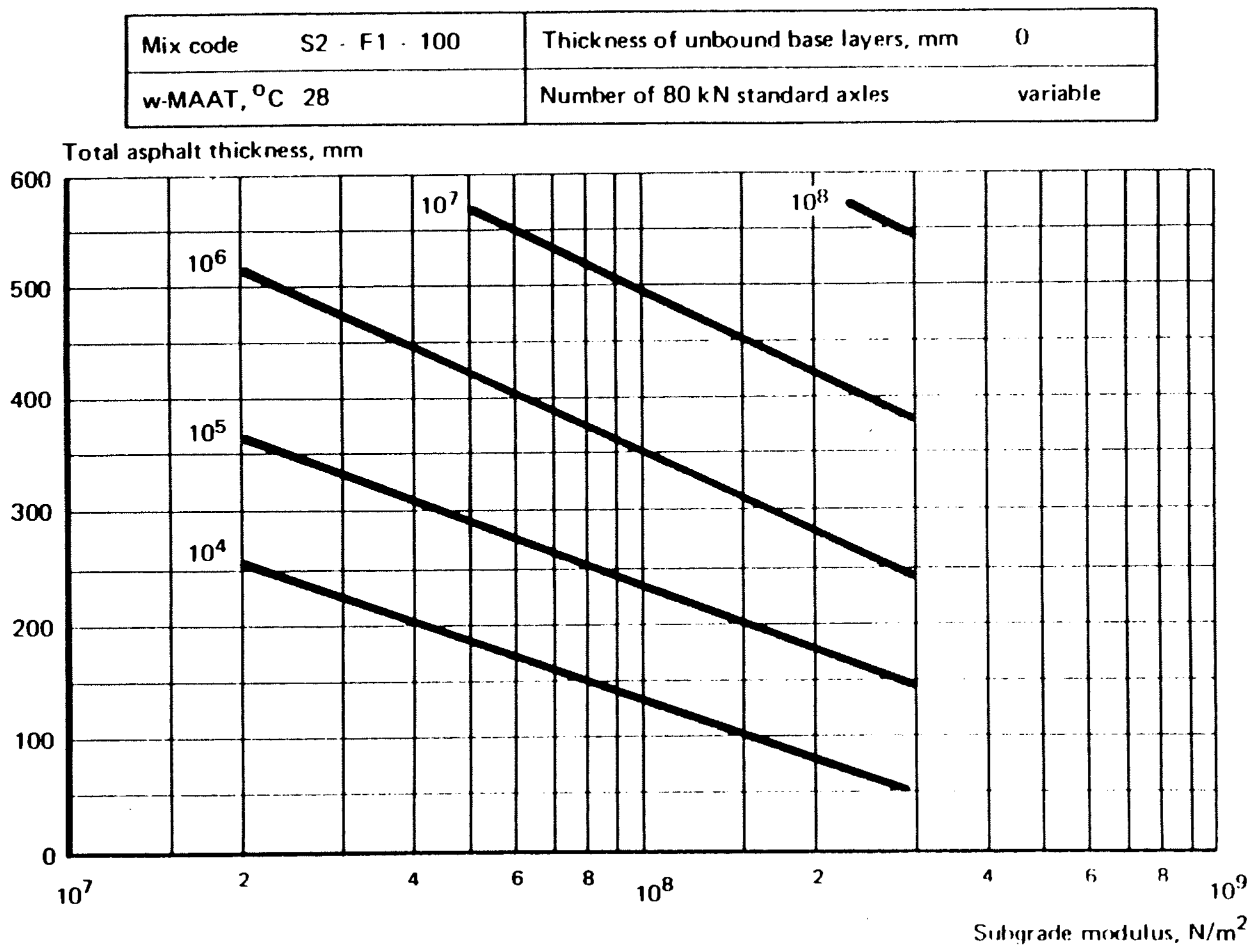


Figure IV-20. Typical design chart for various subgrade moduli for Shell Pavement Design Manual [16].

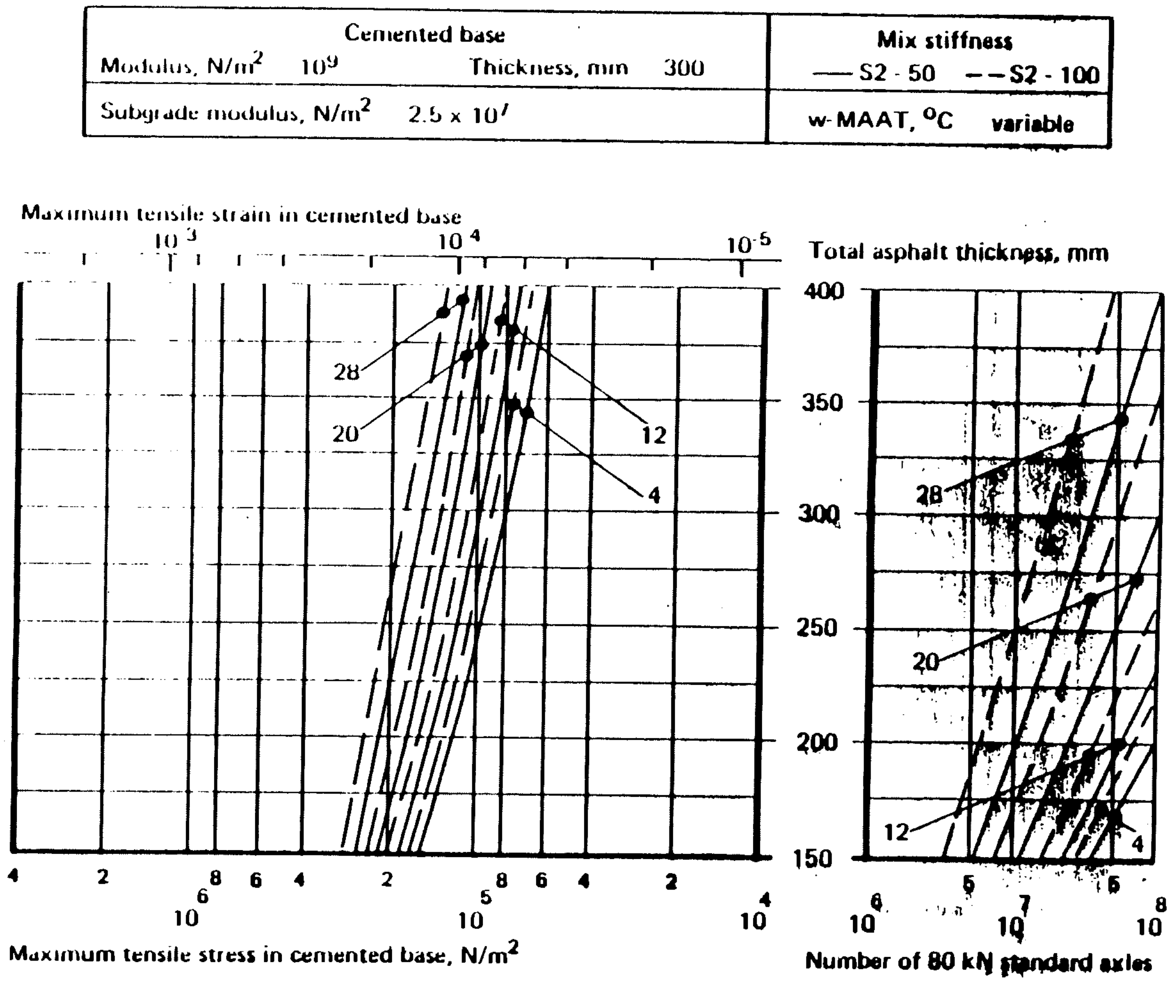


Figure IV-21. Typical design chart for various subgrade moduli for Shell Pavement Design Manual [16].

Table IV-25. Design thicknesses (example) for secondary road with Shell Pavement Design Manual [after 16].

Design parameters

(1) Worksheet A: Number of 80 kN standard axles N:	1 x 10^6
(2) Worksheet B: w-MAAT, °C	12 °C
(3) Subgrade modulus E_3, N/m^2 (see also Chart E)	5 x 10^7

Unbound base materials available

(4) Material description	(5) Attainable dynamic modulus, 10^8 N/m^2 (see also Chart E)
crushed gravel (CBR 80)	8
gravel / sand (CBR 20)	2

Thicknesses from Charts HN, HT, TN, EN or CT
(for full depth constructions first read Chart P: $N^{2/3}/E_3$ =)

Candidate construction	1	2	3	4	5	6

(6) Mix description	(7) Mix code	(8) Total asphalt thickness h_1, mm					
asphalt base mix	S1-F2-50	125	90	75			
lean sand asphalt	S2-F2-50				180	130	125

(9) Total thickness of unbound layers h_2, mm	0	300	500	0	300	500

Subdivision of unbound layer (see Charts HN or HT and step nos. (4) and (5))

(10) Material	(11) Modulus $E_{2...}$ 10^8 N/m^2		(12) Sub-layer thickness $h_{2...}$, mm					
crushed gravel	8	$h_{2\cdot1}$		140	350		130	340
gravel / sand	2	$h_{2\cdot2}$		160	150		170	160
		$h_{2\cdot3}$						
		$h_{2\cdot4}$						

NATIONAL ASSOCIATION OF AUSTRALIAN STATE ROAD AUTHORITIES 1987 PAVEMENT DESIGN

The design procedure presented in the National Association of Australian State Road Authorities 1987 "Pavement Design" [16] is summarized in this section. This design method is based on the structural analysis of a multi-layered pavement subject to traffic loading. Pavement materials are considered to be homogeneous, elastic and isotropic and response to loading is calculated using linear elastic theory in this method. A typical pavement model is shown in Figure IV-22.

Subgrade Evaluation

The measures of subgrade support used in this method are California Bearing Ratio (CBR) and elastic parameters for the flexible pavement. The shear modulus and vertical modulus can be calculated with following equations:

Shear Modulus = Vertical Modulus / (1 + Poisson's Ratio)

Vertical Modulus = 10 * CBR

Poisson's Ratio = 0.45 (cohesive material)

= 0.35 (non-cohesive material)

The following factors must be considered in determining the design strength/stiffness of the subgrade:

1. Sequence of earthwork construction,
2. Compaction moisture content used and field density achieved,
3. Moisture changes during service life,

4. Subgrade variability, and
5. Total pavement thickness may be governed by the presence of weak layers below the design subgrade level.

Environmental Considerations

The environmental factors which significantly affect pavement performance are moisture and temperature in this guide. Freeze and thaw conditions are not included because they rarely occur in Australia. The most important factor in determining the modulus of asphalt is temperature. Some data which have been obtained by testing typical Australian asphalt mixes are shown in Figure IV-23.

Traffic Considerations

In this method, traffic is represented in terms of repetitions of the standard axle which is defined as a single axle with dual wheels that carries a load of 80 kN. The repetitions of the standard axle to other axles can be calculated by the following equation:

$$A = (B / C)^4$$

where

A = No. of standard axles

B = Load on axle group

C = Appropriate load from Table IV-26

Tandem axles which have dual wheels on one axle and single wheels on the other may be considered to be equivalent to tandem axles (both with dual wheels), which are loaded to 1.2 times the load on the six-wheeled tandem.

The design number of ESAs (equivalent standard axles) are calculated as follows:

1. Estimate AADT for the design lane and percent commercial vehicles (C%) from traffic counts.
2. Estimate the number of ESAs per commercial vehicle (F).
3. Calculate initial daily ESAs (N) as follows:

$$N_E = AADT * F * C /100$$

4. Calculate N_{SA}, N_{SS} and N_{SC} as follows:

$$N_{SA} = 1.1\ N_E$$

$$N_{SS} = 1.1\ N_E$$

$$N_{SC} = 20\ N_E$$

where

N_{SA} = Number of standard axles that produce the same cumulative damage in asphalt as the design traffic

N_{SS} = Number of standard axles that produce the same cumulative damage in the subgrade as the design traffic

N_{SC} = Number of standard axles that produce the same cumulative damage in cemented materials as the design traffic

5. Calculate the design loading as follows. Design number of standard axles for:

$$asphalt = N_{SA} * 365 * GF$$

$$subgrade = N_{SS} * 365 * GF$$

$$cemented\ materials = N_{SC} * 365 * GF$$

where

GF = the cumulative growth factor from Table II-28

Failure Criteria

The only failure mode considered for asphalt is fatigue cracking. It is assumed that asphalt mixes are designed with sufficient stability so that permanent deformation does not need to be considered at the design stage. The general relationship between the maximum tensile strain in asphalt produced by a specific load and the allowable number of repetitions of that load is:

$$N = [\ 6918\ (0.856\ V_B + 1.08)\ /\ Smix^{0.36}\mu\varepsilon]^5$$

where

N = allowable number of repetitions of the load

$\mu\varepsilon$ = tensile strain produced by the load (in microstrains)

V_B = percentage by volume of bitumen in the asphalt

S_{mix} = mix stiffness (modulus) MPa

Design Procedures

A. Mechanistic procedure

1. Select a trial pavement.
2. Determine the following elastic parameters for the subgrade E_V, $E_H = 0.5\ E_V$, $\nu_V = \nu_H$, $F = E_V / (E(1 + \nu_V))$.
3. Determine the elastic parameters (as above) of the top sublayer of the granular layer (if relevant).
4. Determine the elastic parameters and thickness of the other granular sublayers (if relevant).
5. Determine the elastic parameters for cemented materials and asphalt (if relevant).
6. Adopt a subgrade strain criterion.
7. Determine fatigue criteria for cemented materials and asphalt (if relevant).
8. Determine design number of standard axles for each relevant distress mode.
9. Approximate the standard wheel loading as two circular vertical loads (total) load 40 kN uniform vertical stress distribution in the range 550 - 700 kPa center to center spacing of the loads 330 mm. Radius of each load is $R = 2523\ p^{-0.5}$

where

R = radius (mm)
p = vertical stress (kPa) = tire pressure

10. Determine critical locations in the pavement for the calculation of strains as follow: Bottom of each asphalt or cemented layer, top of subgrade and directly beneath one wheel load and midway between the two wheel loads.

11. Determine the maximum vertical compressive strain at the top of the subgrade and the maximum horizontal tensile strain at the bottom of each cemented and/or asphalt layer.

12. Determine the allowable number of standards axles for each of the relevant distress modes.

13. For each distress mode, compare allowable number of standard axles with the design number of standard axles.

14. If, for all distress modes, the allowable number of standard axles exceeds the design number of standard axles, the pavement is acceptable. If not, select a new trial pavement and repeat steps 1 to 14.

B. Using design charts

A primary application of this method is to provide a basis for developing design charts for specific circumstances (Figure IV-24 to IV-27).

1. Determine the input parameters.
2. Determine the type of pavement structure.
3. Determine the design chart from Table IV-28.
4. Read the thicknesses.

EXAMPLE

Given:

Location : Somewhere Minnesota

Design traffic for 20 year design life = 1×10^6 ESAs.

Assumption:

NASRA Road functional class = 3

Design subgrade CBR = 5

Asphalt modulus, M_R = 2,800 MPa

Base : Unbound granular

Solution:

From the Table IV-28, design chart number is EC4.

Therefore, the following thickness would be acceptable:

layer	thickness (mm)			
D_1	75	100	125	150
D_2	290	250	180	50

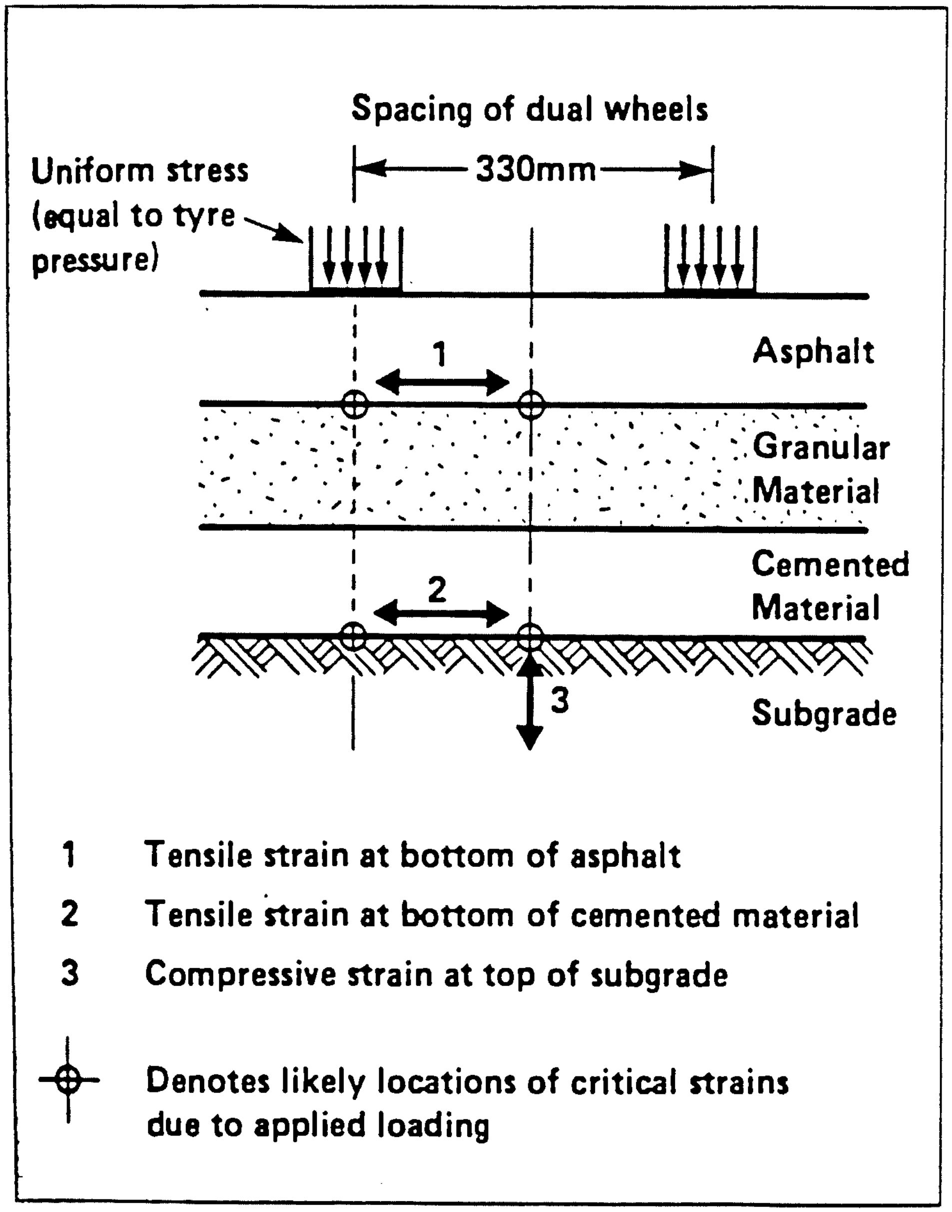


Figure IV-22. Pavement model for Australian Procedure [17].

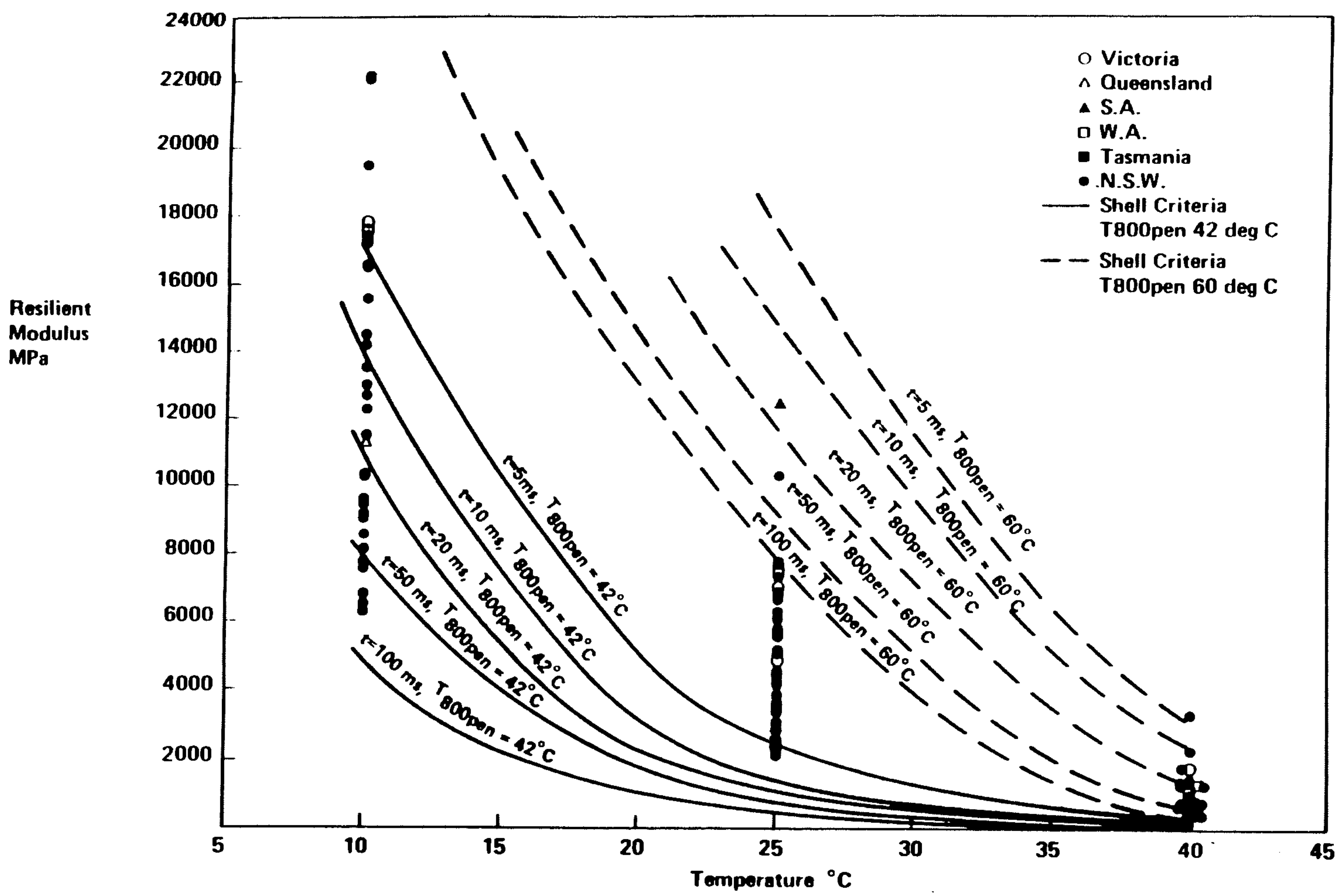


Figure IV-23. Asphalt mix stiffness vs. temperature and load time for Australian Procedure [17].

Table IV-26. Axle load which cause equal damage for Australian Procedure [17].

Axle Configuration	**Single Single**	**Single Dual**	**Tandem Dual**	**Triaxle Dual**
Load (kN)	53	80	135	181

Table IV-27. Cumulative growth factors (GF) for Australian Procedure [17].

Design Period (Years)	**Growth Rate (%)**					
	0	2	4	6	8	10
5	5	5.2	5.4	5.6	5.9	6.1
10	10	10.9	12.0	13.2	14.5	15.9
15	15	17.3	20.0	23.3	27.2	31.8
20	20	24.3	29.8	36.8	45.8	57.3
25	25	32.0	41.6	54.9	73.1	98.3
30	30	40.6	56.1	79.1	113.3	164.5
35	35	50.0	73.7	111.4	172.3	271.0
40	40	60.4	95.0	154.8	259.1	442.6

Table IV-28. Catalog of example design charts for Australian Procedure [17].

Layer	Thickness
Asphalt 2,800 MPa modulus	Varying
Granular	Varying
Subgrade	

Subgrade CBR	Chart Number
2	EC1
3	EC2
4	EC3
5	EC4
7	EC5
10	EC6
15	EC7

Layer	Thickness
Asphalt 2,800 MPa modulus	Varying
Cemented Material 2,000/5,000 MPa modulus	Varying
Subgrade	

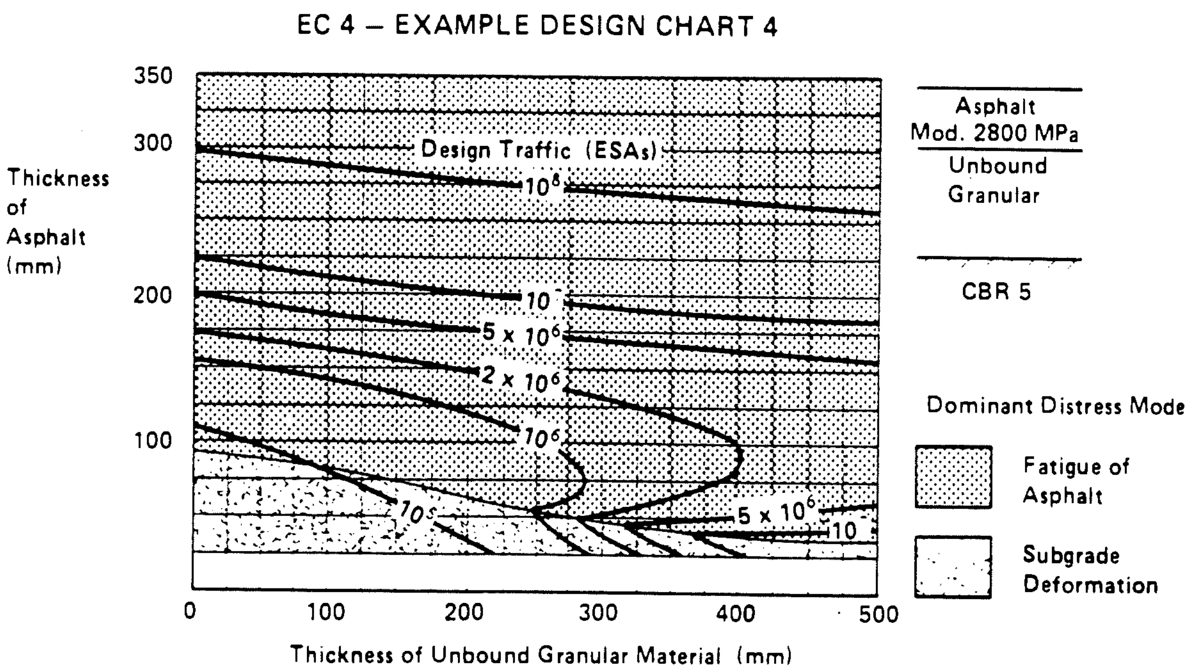
	MODULUS OF CEMENTED MATERIAL	
Subgrade CBR	2,000 MPa Chart No	5,000 MPa Chart No
2	EC8	EC15
3	EC9	EC16
4	EC10	EC17
5	EC11	EC18
7	EC12	EC19
10	EC13	EC20
15	EC14	EC21

Layer	Thickness
Asphalt 2,800 MPa modulus	Varying
Granular	100 mm
Cemented Material 2,000 MPa modulus	Varying
Subgrade	

Subgrade CBR	Chart Number
2	EC22
3	EC23
4	EC24
5	EC25
7	EC26
10	EC27
15	EC28

Layer	Thickness
Asphalt range of moduli	Varying
Subgrade	

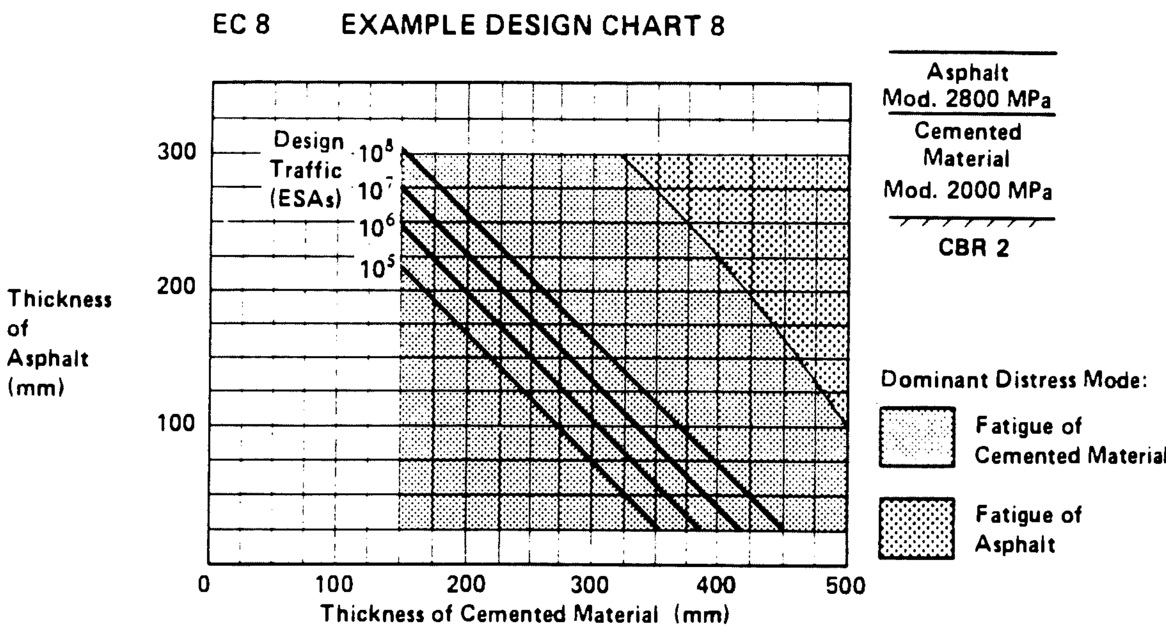
Asphalt Modulus (MPa)	Chart Number
750	EC29
1000	EC30
1600	EC31
2800	EC32
3500	EC33
4500	EC34



NOTE 1. Allowance to be made for construction tolerances

2. For explanation why more than one asphalt thickness is satisfactory refer to appendix F

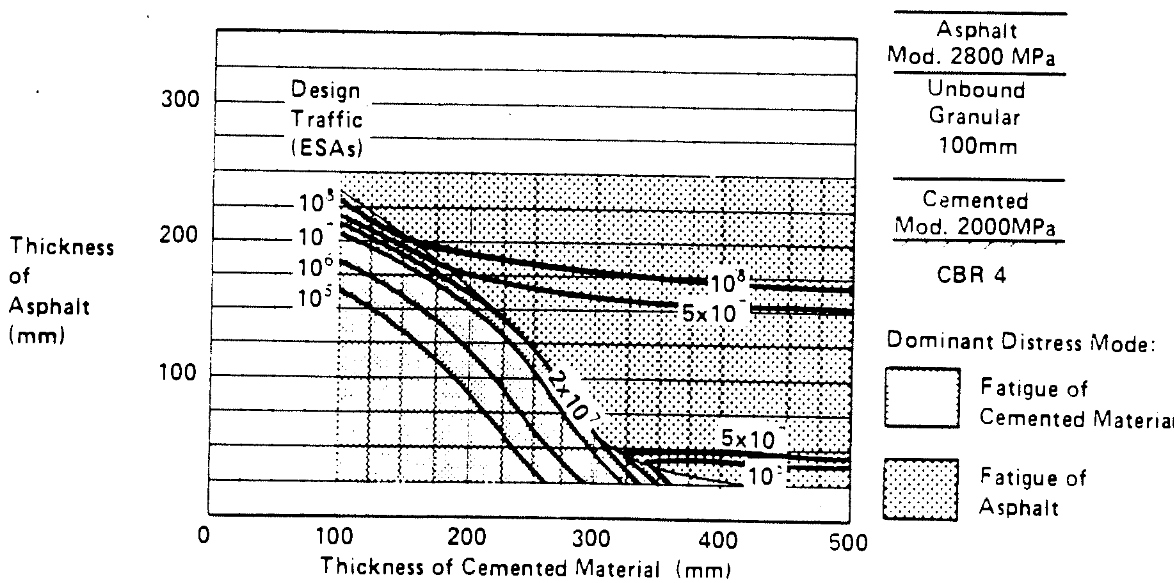
Figure IV-24. Typical example design chart for Australian Procedure [17].



NOTES
1. Allowance to be made for construction tolerances
2. For pavements where the cover over the cemented material exceeds 100mm the second phase of life of the pavement after the cracking of the cemented material may be considered. For guidelines see Sec. 8.5
3. For designs with asphalt thickness < 100mm, the upper 150mm of subgrade should consist of material of CBR > 15 to provide resistance to infiltration through shrinkage cracks

Figure IV-25. Typical example design chart for Australian Procedure [17].

EC 24 EXAMPLE DESIGN CHART 24



NOTES

1. Allowance to be made for construction tolerances
2. For explanation as to why more than one asphalt thickness is satisfactory refer to appendix F
3. For pavements where the cover over the cemented material exceeds 100mm the second phase of life of the pavement after the cracking of the cemented material may be considered. For guidelines see Section 8.5

Figure IV-26. Typical example design chart for Australian Procedure [17].

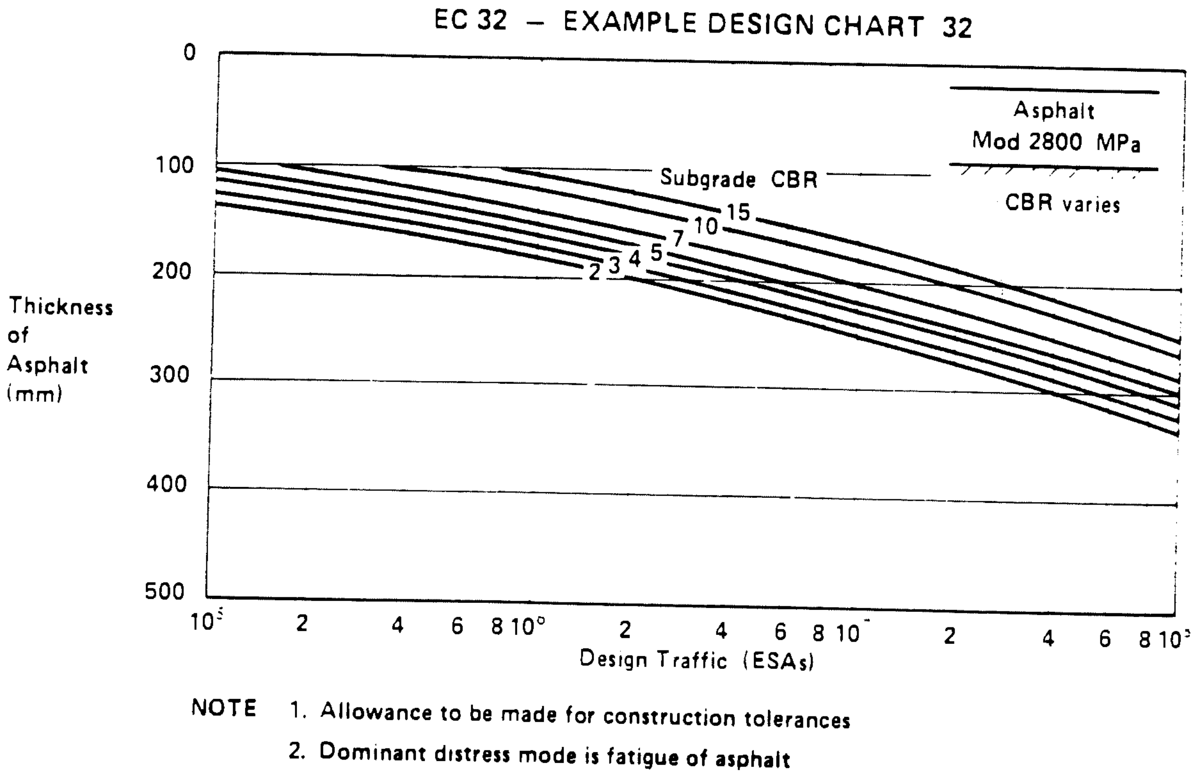


Figure IV-27. Typical example design chart for Australian Procedure [17].

CHAPTER V

REVIEW OF LOCAL PRACTICE

STATE AID MANUAL

The design procedure presented in the MN/DOT State Aid Design Manual [17] is summarized in this section. This design method is based on the soil factor (S.F.) concept. Soil factor is a percentage that reflects both the strength and the frost susceptibility of the soil relative to the AASHTO soil class A-6 [18].

Subgrade Evaluation

The soil factors based on the AASHTO soil classes are used in this design method. The relationships between AASHTO soil classes and soil factors are as shown below:

AASHTO SOIL CLASS	SOIL FACTOR (S.F.) %
A-1	50 - 75
A-2	50 - 75
A-3	50
A-4	100 - 130
A-5	130+
A-6	100
A-7-5	120
A-7-6	130

Environmental Considerations

Previously the environmental factors were considered when this procedure was developed by the Minnesota Department of Transportation for their use, however, it is not necessary to separately account for the environmental factors.

Traffic Considerations

Either the average daily traffic (ADT) or the heavy commercial average daily traffic (HCADT) can be utilized in this design procedure depending on the allowable spring axle load for the roadway being designed. The ADT is the only traffic parameter to apply to the design charts for a 7-ton design roadway. The HCADT is the only traffic parameter to apply to the design charts for 9-ton design roadway.

Failure Criteria

There are no failure criteria in this manual. However, a pavement is generally considered to have failed once the Present Serviceability Rating (PSR) has fallen below 2.5. At this level of serviceability there is generally some value remaining in the pavement.

Design Procedures

Once the soil factor and the traffic (ADT or HCADT) have been determined, the design granular equivalent thickness for the surface and base can be read from the design tables (Table V-1).

EXAMPLE

Given:

LOCATION : Carver County, Minnesota

ESAL = 150,000

Assumption:

Soil factor = 100

ADT = 750

HCADT = 140

Solution:

From Table V-1,

Road Design	Base G.E.	Surface G.E.	Total G.E.
7 Ton	12.0*	3.0	15.0
9 Ton	11.5	6.0	17.5

* Unit is inch.

Table V-1. Required Gravel Equivalency (G.E.) for Various Soil Factors (S.F.) [18].

7 Ton : Less than 400 ADT

S.F.	Base G.E.	Surface G.E.	Total G.E.
50 -	4.25	3.0	7.25
75 -	6.38	3.0	9.38
100 -	8.5	3.0	11.5
110 -	9.4	3.0	12.4
120 -	10.2	3.0	13.2
130 -	11.0	3.0	14.0

7 Ton : 400 - 1000 ADT

S.F.	Base G.E.	Surface G.E.	Total G.E.
50 -	6.0	3.0	9.0
75 -	9.0	3.0	12.0
100 -	12.0	3.0	15.0
110 -	13.2	3.0	16.2
120 -	14.4	3.0	17.4
130 -	15.6	3.0	18.6

Table V-1. Required Gravel Equivalency (G.E.) for Various Soil Factors (S.F.) [18] (Continued).

9 Ton : Less than 150 HCADT

S.F.	Base G.E.	Surface G.E.	Total G.E.
50 -	4.25	6.0	10.25
75 -	7.9	6.0	13.9
100 -	11.5	6.0	17.5
110 -	13.0	6.0	19.0
120 -	14.5	6.0	20.5
130 -	16.0	6.0	22.0

9 Ton : 150 - 300 HCADT

S.F.	Base G.E.	Surface G.E.	Total G.E.
50 -	7.0	7.0	14.0
75 -	10.5	7.0	17.5
100 -	14.0	7.0	21.0
110 -	15.4	7.0	22.4
120 -	16.8	7.0	23.8
130 -	18.2	7.0	25.2

Table V-1. Required Gravel Equivalency (G.E.) for Various Soil Factors (S.F.) [18] (Continued).

9 Ton : 300 - 600 HCADT

S.F.	Base G.E.	Surface G.E.	Total G.E.
50 -	9.0	7.0	16.0
75 -	13.5	7.0	20.5
100 -	18.0	7.0	25.0
110 -	19.8	7.0	26.8
120 -	21.6	7.0	28.6
130 -	23.4	7.0	30.4

9 Ton : 600 - 1100 HCADT

S.F.	Base G.E.	Surface G.E.	Total G.E.
50 -	10.5	8.0	18.5
75 -	15.7	8.0	23.7
100 -	21.0	8.0	29.0
110 -	23.1	8.0	31.1
120 -	25.2	8.0	33.2
130 -	27.3	8.0	35.3

Table V-1. Required Gravel Equivalency (G.E.) for Various Soil Factors (S.F.) [18] (Continued).

9 Ton : More than 1100 HCADT

S.F.	Base G.E.	Surface G.E.	Total G.E.
50	12.3	8.0	20.3
75	18.4	8.0	26.4
100	24.5	8.0	32.5
110	27.0	8.0	35.0
120	29.4	8.0	37.4
130	31.8	8.0	39.8

MUNICIPALITIES

Design Method

Two major pavement design methods are used in Minnesota; one is the MN/DOT Road Design Manual and the other is the State Aid Manual. The flexible pavement design procedure shown in the MN/DOT manual is usually the first choice. The design developed by this procedure then checked against the Flexible Pavement Design Procedure using soil factors as outlined in the State Aid Manual. Sometimes this procedure is compared to the Asphalt Institute "Thickness Design - Asphalt Pavement for Highways and Streets MS - 1" or AASHTO Interim Guide for Design of Pavement Structures 1972.

Traffic

Most of the municipalities of Minnesota use the traffic counts that are provided by MN/DOT every 4 years. The traffic is usually expressed as ESALs. The number and distribution of trucks and other units are based on the standard distribution model (Table V- 13) in the MN/DOT Road Design Manual or State Aid Manual. If this distribution does not seem appropriate, a traffic count is taken to determine actual distribution. This distribution and traffic projection are then used with the average sigma N-18 factors shown in MN/DOT Road Design Manual (Table V-15) or 1972 AASHTO Interim Guide Tables to determine the estimated ESAL's. Many counties have distribution spreads from actual county traffic and intersection counts that they use in special cases. Some municipalities use HCADT (heavy commercial average daily traffic) instead of ESAL's.

Soil Properties

To determine the soil classification the sieve analysis and Atterberg limit tests are used. If the roadway is a new construction or reconstruction, typically soil samples are taken from the borrow areas to determine the design R-value for the pavement design. Soil factors can be used instead of R-value. The typical soil factor of each municipality is listed in Table V-2.

Load Restrictions

Load restrictions are based on state law. Normally they begin in late March and last until mid May. However, the Road Rater and field observations are used to determine for placement and removal of road restrictions by some counties. Roads are typically posted for the tonnage which appears appropriate from the design. Field observation is used to determine if weight restrictions should be increased or decreased. The deflection testing can be performed to determine limits.

Failure

Failure of the pavement is said to occur when there is a frost boil, severe alligator or cracking, wide cracks (1.5 - 2 inches) or ruts (more than 1.5 inch deep). Rideability is said to fail the test when citizens' complaints get too numerous. Pavement failures are usually related to poor subgrade, weak base or poor drainage.

Rehabilitation

The sequence of rehabilitation is: 1) pothole patch, 2) spot seals, 3) short overlays, 4) major road overlay, and 5) final complete removal and replacement including regrading. Sealcoating is used to treat minor reflective cracking. A thin overlay is used to repair rutting or medium cracking and also to add strength to the pavement section. A thick overlay is used to repair more serious problems. Total pavement removal and replacement is used if repair is not feasible.

For longer segments of roadway, the local agencies have to do some analysis of the cost benefits of an overlay versus reconstruction and where the project fits in the construction program. Subgrade corrections, sub-surface drains, additional gravel base, gradation and compaction are also considered.

Examples of Local Practices

Mille Lacs County

- Design Method: MN/DOT Road Design Manual and State Aid Manual
- Traffic: Traffic counts from the state are used and adjusted to the current year. They use ESAL for the road and design on 9 ton roads.
- Soil Properties: Soil borings are taken, and past experience is used in some instances. R-values are determined from borings by testing firms.
- Load Restrictions: Based on state law. They have been monitoring thawing degree-days for the last several years and it seems to work well to determine when they should put on restrictions.
- Failure: Determined by field observation.

- Rehabilitation: Pothole patch, spot seals, short overlays, major road overlay and complete removal and replacement including regrading are considered.

Wabasha County

- Design Method: MN/DOT Road Design Manual or AASHTO Interim Guide For Design of Pavement Structures 1972.
- Traffic: Traffic counts from MN/DOT are used (ADT only). ESAL factors from the MN/DOT Road Design Manual or 1972 AASHTO Guide tables are used.
- Soil Properties: To determine the soil classification, the sieve analysis and Atterberg limit tests are used.
- Load Restrictions: Based on the state law which specifies March 20th to May 15th as a guideline and adjusted from that depending upon visual observations.
- Failure: Failure is determined by visual observation.
- Rehabilitation: They consider complete regrading of the roadway with a stronger base course, stripping of the surface pavement and adding base material or geotextiles before repaving. Overlays are also considered.

Stearns County

- Design Method: State Aid Manual
- Traffic: Traffic counts from MN/DOT are used. The traffic for the most recent 12 years is used to project traffic for a design life of 20 years based on the "least-squares" method.

- Soil Properties: The types of material tests and the frequency with which they are administered are the same as those prescribed by MN/DOT.
- Load Restriction: Determined by monitoring climatic conditions and consulting with MN/DOT and neighboring counties.
- Failure: Visual inspection
- Rehabilitation: Seal coat, various thickness of overlay and total reconstruction are considered.

Carver County

- Design Method: MN/DOT Road Design Manual and State Aid Manual.
- Traffic: Traffic is determined by semi-annual traffic counts taken on the county road system.
- Soil Properties: If the soil factor method is used, a typical soil factor of 100 is used. Soil samples are taken to determine the design R-value for the pavement design.
- Load Restrictions: The determination for placement and removal of Load restrictions is made from field observation.
- Failure: The field observation is used to determine the pavement failure. Pavement rutting and flushing are considered pavement failures and may or may not be corrected depending on the circumstances.
- Rehabilitation: Sealcoating, thin overlay, thick overlay and reconstruction are considered.

Goodhue County

- Design Method: MN/DOT Road Design Manual and State Aid Manual.
- Traffic: HCADT is used instead of ESAL's. They estimate HCADT to be 3% of the ADT on their roads.
- Soil Properties: Gradations for aggregate base construction with ordinary compaction method (MN/DOT) is used. The county runs gradation tests and uses a portable nuclear density testing device to spot check compaction requirements.
- Load Restrictions: They use MN/DOT District 6 at Rochester.
- Failure: They usually use Surface Condition Rating to monitor pavements. This information is used to set up 5 and 10 year plans for grading and paving.
- Rehabilitation: Pavement on roads which are scheduled for regrading receive the minimum maintenance required to reach the year of reconstruction. Other roads are scheduled for seal coats every 4 or 5 years and overlays at about 18 years of age.

Table V-2. Typical Soil Factor of various municipalities [18].

Municipality	Soil Factor	Municipality	Soil Factor
Albert Lea	100	Alexandria	100
Andover	50	Anoka	50
Apple Valley	100	Arden Hills	75
Austin	100	Bemidji	75
Blaine	50	Bloomington	50
Brainerd	75	Brooklyn Center	50
Brooklyn Park	50	Burnsville	100
Champlin	50	Chanhassen	100
Chaska	100	Chisholm	130
Cloquet	100	Columbia Heights	100
Coon Rapids	50	Cottage Grove	100
Crookston	130	Crystal	75
Detroit Lakes	75	Duluth	130
Eagan	100	East Bethel	50
East Grand Forks	130	Eden Prairie	75
Edina	130	Elk River	75
Eveleth	130	Fairmont	100
Falcon Heights	100	Fairbault	100
Fergus Falls	100	Fridley	50
Golden Valley	100	Grand Rapids	75
Ham Lake	50	Hastings	75
Hermantown	130	Hibbing	130

Table V-2. Typical Soil Factor of various municipalities [18] (Continued).

Municipality	Soil Factor	Municipality	Soil Factor
Hopkins	100	Hutchinson	100
International Falls	130	Inver Grove Hights	100
Lake Elmo	100	Lakeville	100
Lino Lakes	100	Litchfield	100
Little Canada	100	Little Falls	75
Mankato	100	Maple Grove	100
Maplewood	100	Marshall	100
Mendota Heights	100	Minneapolis	100
Minnetonka	100	Montevideo	100
Moorhead	130	Morris	130
Mound	100	Mounds View	75
New Brighton	100	New Hope	75
New Ulm	100	Northfield	100
North Mankato	100	North St. Paul	100
Oakdale	100	Orono	100
Owatonna	100	Plymouth	100
Prior Lake	100	Ramsey	50
Red Wing	100	Redwood Falls	130
Richfield	50	Robbinsdale	75
Rochester	100	Rosemount	100
Roseville	100	St. Anthony	100

Table V-2. Typical Soil Factor of various municipalities [18] (Continued).

Municipality	Soil Factor	Municipality	Soil Factor
St. Cloud	75	St. Louis Park	75
St. Paul	100	St. Peter	100
Savage	100	Sauk Rapids	75
Shakopee	100	Shoreview	75
South St. Paul	100	Spring Lake Park	50
Stillwater	100	Thief River Falls	130
Vadnais Heights	100	Virginia	130
Waseca	100	West St. Paul	100
White Bear Lake	100	Willmar	100
Winona	50	Woodbury	100
Worthington	100		

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