

5-3.0 PAVEMENT DESIGN

Pavement design is the process of determining the most cost-effective pavement structure for a roadway that will be functionally and structurally adequate to carry all anticipated loadings over its design life. Consideration of the traffic conditions, environmental conditions, availability of materials, and material properties are all required for an adequate design. In addition, the pavement must pose no undue problems to the construction and maintenance crews that will implement the design.

Pavement design should include a consideration of funding constraints and a thorough economic analysis to determine the pavement structure for a particular location. These analyses should accurately predict pavement performance, forecast future maintenance costs, and account for rehabilitation needs and treatments.

This section contains information on the Department's approach to determining pavement type and the subsequent design of the pavement structure that will best meet the conditions discussed above. Specifically, the basic requirements for a durable, long lasting and well performing pavement section are to:

- Provide a smooth, durable travel surface and structure that will withstand both the repeated wheel loads of vehicles and the effects of the environment;
- Provide a subgrade which is constructed with suitable, uniform, and stable soils whose engineering properties have been designed so as to ensure that the necessary support required by the pavement structural design is achieved during each of the seasons of the year;
- Provide adequate drainage capability for the removal of excess water from the roadway and the subsurface portions of the pavement, so that the structural integrity of the pavement remains intact; and
- Provide a surface with adequate friction characteristics.

5-3.01 PAVEMENT TYPES

A number of pavement types are currently used. The types of pavements constructed by Mn/DOT to meet the requirements described above are bituminous (asphaltic concrete), Portland cement concrete (PCC), and composite pavements. These pavement types are described below. A fourth pavement type, exposed aggregate, is used for many low volume roadways throughout the state. However, few of these are built or maintained by Mn/DOT.

5-3.01.01 BITUMINOUS

Bituminous pavements, also called flexible pavements, are designed so that traffic loadings are supported by several material layers. The materials used near the surface of the roadway, such as the bituminous or base, are designed to support more concentrated loads and are usually stiffer than the deeper layers, such as the subbase or subgrade, as a result.

There are two main types of bituminous pavements. The first is the conventional flexible pavement, which consists of a bituminous surfacing placed on an aggregate base and prepared subgrade. The second is a "deep-strength" bituminous pavement that consists of a bituminous surfacing placed on a granular subbase layer.

5-3.01.02 PORTLAND CEMENT CONCRETE

Portland Cement Concrete (PCC) pavements, also known as rigid pavements, are pavements that have a PCC wearing surface. This surface may be placed directly on the subgrade, but in most instances a base layer(s) is placed between the two. PCC pavements are classified into four types

based on the nature of the transverse joint spacing and reinforcement used to limit temperature cracking within the pavement. These are jointed plain concrete pavement (JPCP), jointed reinforced concrete pavement (JRCP), continuously reinforced concrete pavement (CRCP), and prestressed concrete pavement (PCP).

JPCPs are jointed, un-reinforced, PCC pavements made up of short slabs, typically no more than 20 feet long, with dowels placed in the transverse joints. The short joint spacing eliminates the likelihood of the formation of transverse cracks within a single slab and reduces warping and curling stresses. At the present time, Mn/DOT is constructing all JPCP pavements with a 15 ft uniform joint spacing.

JRCs are PCC pavements that are reinforced with steel bars or mesh to reduce the severity of transverse cracking between the joints. Slab lengths are typically between 20 and 40 feet, and all transverse joints are doweled. When last constructing JRCP pavements Mn/DOT used a 27 ft spacing, although none are being constructed at this time.)

CRCPs are PCC pavements that are continuously reinforced and have no transverse joints with the exception of construction joints. Enough reinforcement is provided so that the transverse temperature cracks which form are spaced between 1 and 8 ft and are less than 0.02 in wide. Mn/DOT began constructing CRCP pavements in 1963, but discontinued their use in 1970 due to poor performance.

PCPs are concrete prestressed slabs. Post-tensioning of reinforcement placed in ducts in the concrete slabs, the usual method of prestressing, exerts compressive forces on the pavement slabs and eliminates temperature cracks or keeps them tight when they do form. These pavements are constructed in slabs that range from 200 to 400 feet in length. They are considered experimental by most highway agencies and have not been used in Minnesota.

5-3.01.03 COMPOSITE

Pavements that are made up of a combination of PCC and bituminous layers are given the generic name "composite" pavements. These pavements are usually the result of rehabilitation (overlays). However, some new composite designs have been proposed.

5-3.02 PAVEMENT DESIGN CONSTRAINTS

The pavement design procedure involves the selection of a pavement type, initial structure, and lifecycle strategy that will be the most economic and adequate alternative for carrying projected traffic and environmental loadings. The full consideration of a number of constraints is required if the best solution is to be obtained: the most important of these are discussed below.

5-3.02.01 DESIGN PARAMETERS

There are a number of issues associated with obtaining the traffic condition, environmental condition, and material property information required by the design process.

There are two pieces of information needed to determine the traffic loading for a pavement design. First, there is the need to forecast the expected traffic volumes and vehicle classifications for the entire design life of a pavement. Second, accurate characterizations of the axle-load distributions are required. Traffic volume and classification forecasting techniques continue to improve, however, the inputs needed for these methods are difficult to obtain with accuracy. Therefore, large discrepancies between the predictions and the true loading states are not unusual. The axle-load distributions of truck traffic are also generally site specific and based on sparse weigh-station data.

Environmental effects also have a considerable effect on the performance of pavements. The designer should directly address soils which may have problematic water conditions and/or swell- and frost-susceptibility within the design, rather than trying to account for such environmental factors by increasing thickness or passing the issue on to construction personnel.

In addition, the characterization of pavement materials and their properties have a large effect on the predicted pavement design life. A proper characterization of material properties with respect to time, loading, and environmental variations will greatly increase the suitability of the design. These properties are particularly important in the mechanistic-empirical design procedures that are slowly replacing traditional methods.

Another issue often facing pavement design engineers is the availability of good quality materials for pavement construction. Increasingly, pavement engineers are being forced to use materials of less than desirable quality due to the limits placed on quality pits and other economic factors. In most instances, these less desirable materials are stabilized or treated in some manner to allow their effective use. Design procedures must take the availability of desired materials into account.

Aside from the issues discussed above, there is always the important matter of designing a pavement that is constructible and maintainable. The initial constructability of the pavement and the successful implementation of maintenance and rehabilitation treatments are important aspects that have to be considered in a successful pavement design process.

5-3.02.02 FUNDING

Pavement projects compete with projects in other government sectors for funds available to new construction, rehabilitation, and maintenance. It is, therefore, incumbent upon the pavement engineer to specify the economic benefits of a new pavement. Then, once funds have been acquired, it may be necessary to re-design portions of the project to account for a limited budget. The design engineer should always keep track of the funding discussion for his or her projects.

5-3.02.03 ECONOMIC ANALYSIS/PAVEMENT SELECTION

One of the most difficult aspects of pavement design is the economic analysis that determines the most cost-effective pavement to meet a project's needs. In such analyses, economic principles are used to compare alternatives to determine the most cost-effective combination of pavement type, initial structure, and maintenance and rehabilitation treatments for the analysis period. The road user costs associated with potential construction impacts should be considered as well.

One difficulty associated with this process is the continued action of private lobbyists on behalf of particular products, methods, and techniques. In particular, the concrete and asphalt lobbies exert political pressure that causes the Pavement Selection procedure to treat them as equally as possible. Care should be taken to ensure that this political pressure does not distract attention from the best alternative for a particular project.

Another difficulty encountered during this process is the variability of material costs, discount rates, salvage values, and other such factors. These values often have to be approximated as they are tied to events that may occur years in the future. However, by applying sound principles and analysis techniques it is possible to obtain results that will adequately answer the necessary questions in any particular case. In Minnesota, a comparison of the total annual costs of alternative strategies is used in such analyses.

5-3.03 ANALYSIS METHODS

Pavement analysis methods can be divided into three major groups. These are the empirical, mechanistic, and mechanistic-empirical analysis methods. The concepts behind these methods are described in this section with references to some existing pavement analysis methods.

5-3.03.01 EMPIRICAL

Empirical pavement analysis methods are experience-based methods that have been developed through the observation of existing pavements. For example, the AASHO Road Test in Illinois was the test section that many of the current empirical pavement design procedures are based upon. These methods are not based on the physical means by which materials behave when subjected to traffic and environmental loadings. Rather, they rely on what practical experience has shown about their performance over time.

Mn/DOT's empirical analysis procedures for flexible pavements were developed from research studies conducted by the Department, and the analysis method for rigid pavements is based on the 1981 Modified AASHTO analysis procedure for rigid pavement design. These procedures work well when using materials and loadings similar to the original test sections. However, as with all empirical methods, it is necessary to continually tweak and adapt these methods as new materials, loadings, and construction methods are introduced. The following paragraphs discuss these investigations and methods.

1. Mn/DOT's Investigations 183 and 195 (Flexible Pavements). The two types of flexible pavements that have been designed by the Department are asphaltic concrete pavement with aggregate bases and full-depth asphaltic concrete pavements. Research conducted by Mn/DOT in Investigation 183 forms the basis of the asphaltic concrete on aggregate base pavement analysis; and the analysis method for the design of full-depth asphaltic concrete pavements was based on the results of research conducted in Investigation 195. However, the poor performance of many full-depth pavements led to the discontinuation of that method, and a "deep strength" design is now used in its place.

Investigation 183 was designed to adapt the results of the AASHO Road Test to Minnesotan conditions and establish an analysis method for asphaltic concrete pavements. Data was collected from 58 test sections in Minnesota, which were selected to include a wide variety of soil types, pavement structures, and traffic conditions. Using this data, the effects of the various subgrade soils, local construction procedures, local paving materials, climatic conditions, and cumulative traffic loading on pavement performance were evaluated. The performance of the test sections, as determined by serviceability trends, was then related to the structure, subgrade R-value, and Benkleman beam deflections to allow the extension of the AASHO research results to the analysis and design of flexible pavements in Minnesota.

The research conducted in Investigation 195 employed data collected from 26 full-depth test sections using a variety of thicknesses and foundation soils. Benkleman beam deflection and pavement temperature measurements were taken on the test sections in the spring, summer, and fall. These data were used to determine the effects of temperature and seasonal variation on the deflections. They were also used to develop correction factors for application to measured deflections to allow their conversion to a standard peak season deflection at 26.7° C (80° F). These standard deflections were then compared to deflection measurements taken on flexible pavements with aggregate bases, and a relationship was developed between the thickness of full-depth pavements and the "granular equivalency" of comparable granular base pavements. This relationship was used to develop a design chart for full-depth pavements, which was the deflection equivalent of the granular base, flexible pavement, design chart then used by the Department. The structural design procedure for the two types of flexible pavements is based on the design-lane, cumulative, equivalent single-axle loads (ESALs) over a 20-year period. The subgrade strength is based on the design R-value.

Mn/DOT also uses an empirical procedure for rating flexible pavements for designation as 62-, 80-, and 89-kN (7-, 9-, and 10-ton) routes. These load ratings refer to springtime thaw restrictions and are based on deflection measurements. The procedures used in determining the springtime carrying capacity of a pavement are based on Investigation No. 603. A

computer program (TONN) was developed, which incorporated Investigation No. 603, and is used for estimating the springtime load carrying capacity. In addition, a procedure is available to determine if and when a particular roadway has the strength to withstand 89-kN (10-ton) loads without undue loss of useful pavement life ("Methods for Determining 10-Ton Routes on Flexible Pavements based on NDT"). It is likely that Mn/DOT will recommend only 10-ton designs in the future.

2. Modified AASHTO (Rigid Pavements). The rigid pavement design procedure used by the Department is an empirical method that is based on a combination of the modified 1981 AASHTO pavement design procedure and performance data from Minnesota. The structural design procedure is based on 35-year ESAL projections and the modulus of subgrade reaction, k . These values may be adjusted by empirical constants depending on whether the pavement edge is protected or unprotected.

5-3.03.02

MECHANISTIC

In recent years, there has been a move towards the use of mechanistic analysis procedures for the design of pavements. Mechanistic procedures are based on engineering mechanics principals, and involve the use of analytical methods to determine the actual stresses, strains, and deflections that occur in pavements from the application of traffic and environmental loads. By allowing proper characterization of all aspects of a pavement in terms of actual material properties, variability in the properties, traffic loading, environmental variations, and the physical aspects of the different types of pavements, mechanistic procedures seek to provide an exact representation of the pavement structure.

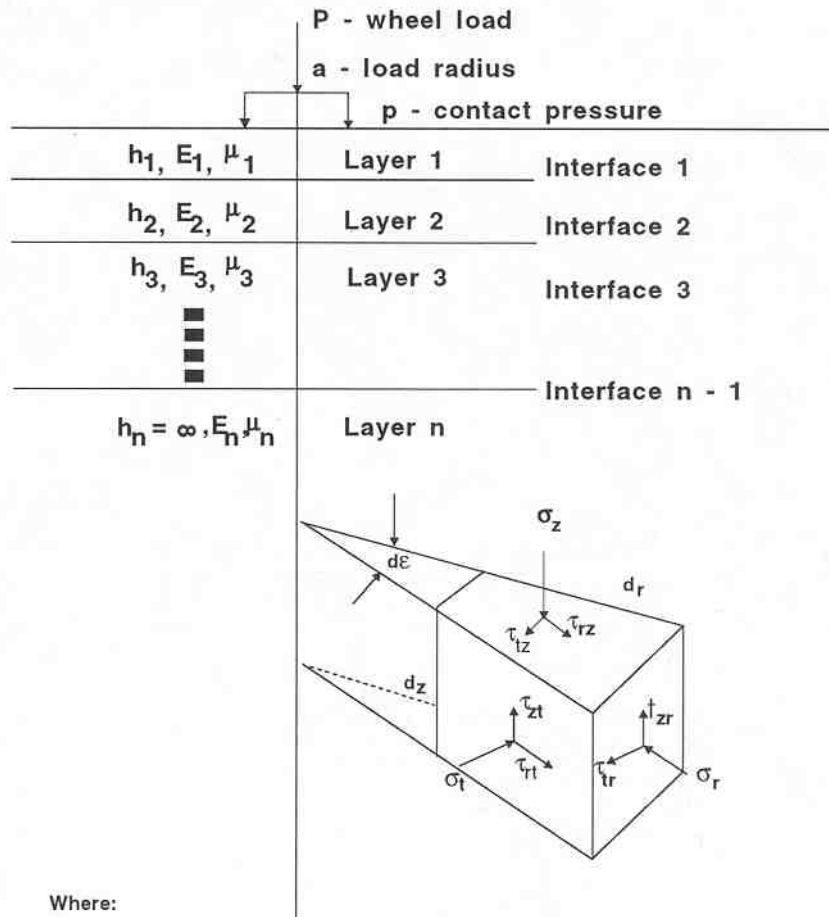
Another useful aspect of mechanistic analysis for pavement design is the use of damage and distress models in the analysis process to check designs. Damage models seek to predict the combination of traffic and environmental loading that would result in a failure, as determined by pre-defined distress failure criteria (fatigue cracking, rutting, faulting or thermal cracking), for a particular pavement structure. The distress model usually allows the prediction of the severity and extent of the various distress types in the pavement, as well. With this combination of detailing inputs and damage prediction models, mechanistic analysis procedures intend to closely simulate real pavement conditions.

Unfortunately, a purely mechanistic design procedure is somewhat hypothetical at this point in time. Despite recent advances in computing power, it is still not possible to model every reaction that takes place within pavement structures to the degree of accuracy needed for an accurate pavement design. Nonetheless, research is continuing in the field and a solution may be available at some point in the future. Elastic layer and finite element theory are the basis of the two major mechanistic analysis approaches currently used in the determination of structural response in pavements. These procedures are not yet design methods as they have yet to be complemented with the proper damage and distress models needed to predict pavement performance over time. Nonetheless, they are advanced enough in their current forms to provide insight into the performance of particular systems and serve as the basis for mechanistic-empirical designs.

1. Elastic Layer. The application of elastic layer theory to the analysis of pavements has primarily been in the area of flexible pavement design, as these pavements perform in a manner more suited to a multi-layer analysis. In this theory, the pavement is considered to be an axisymmetric multi-layered structure where each layer extends infinitely in the horizontal direction and the bottom layer extends infinitely downwards in the vertical direction as shown in Figure 5-3.1. Each layer is assumed to be homogeneous, isotropic, and linearly elastic in response to stress. The material properties of the layers are characterized by their modulus of elasticity and Poisson's ratio. It is assumed that there is full friction between the layers and that the load on the top layer of the pavement is applied through a circular area. Information on the stresses, strains, and deflections at any location from the axis of symmetry can be computed using this model, the Boussinesq equation, and current computer software. Elastic

layer analysis programs are now available that allow for a finite bottom layer, non-linear elastic material response, partial or no friction between layers, and multiple loads.

Figure 5-3.1. Idealized elastic multi-layer pavement structure.



Where:

h_n = Thickness of layer n.

E_n = Elastic Modulus of layer n.

μ_n = Poisson's Ratio of layer n.

s_i = Stress component in the i^{th} direction.

t_{ji} = Stress component in the i^{th} direction in the plane j .

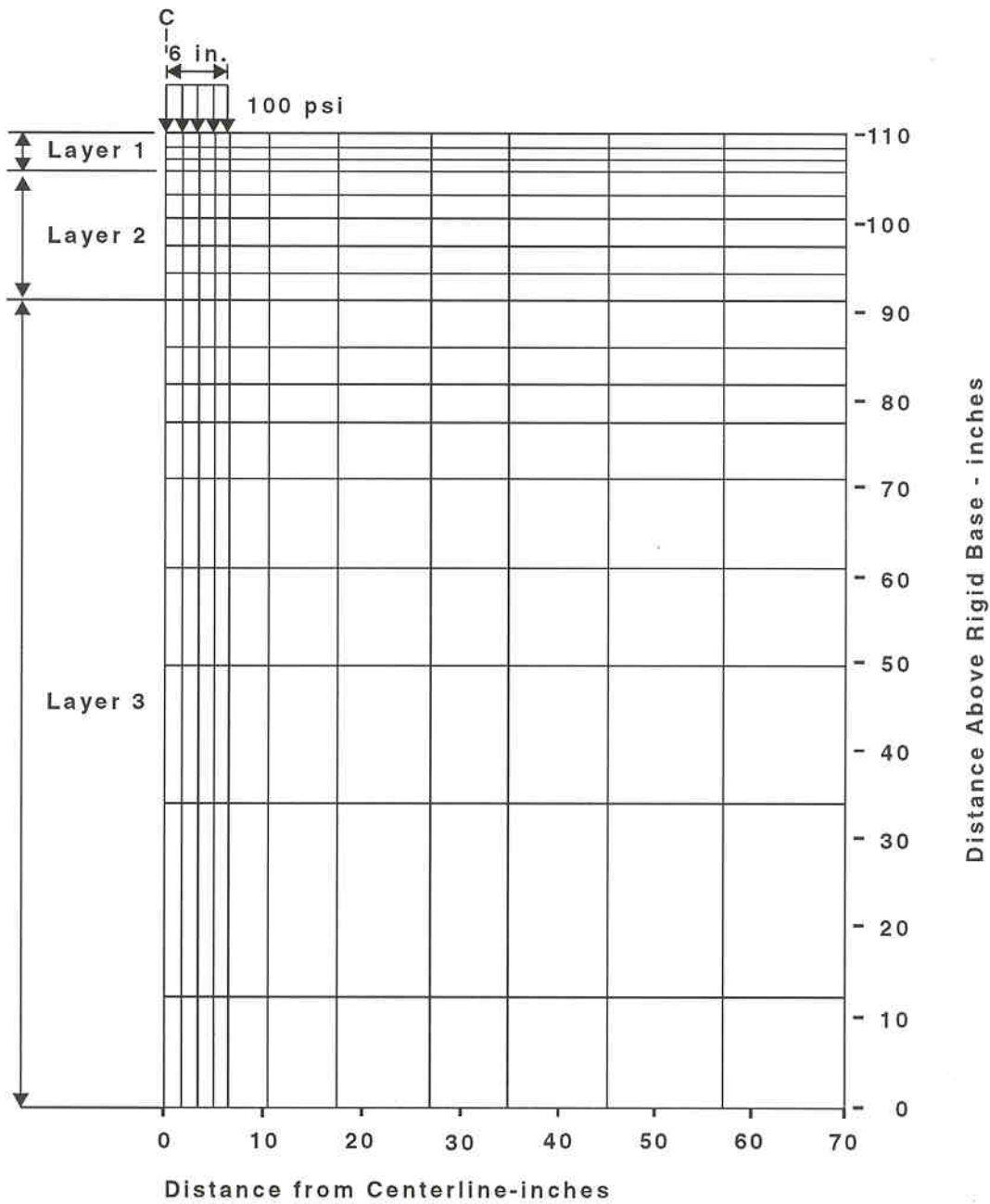
Multi-layered elastic analysis is not applicable to the solution of stresses, strains, and deflections at discontinuities in pavements and is, therefore, not suitable for solving rigid pavement problems (because of joints and cracks), except at the interior of slabs. There are also some limitations connected with the analysis of pavements on granular layers. Multi-layer elastic analysis programs sometimes yield solutions that show excessive tensile stress in the granular layers, which are known to have no tensile strength. Examples of elastic layer programs for pavement response analysis are the WESLEA, EVERSTRESS, ELSYM 5 and BISAR programs.

2. Finite Element. The finite element analysis method is applicable to the design of both rigid and flexible pavements and can take into consideration the effect of discontinuities (such as joints and cracks), design parameters (such as dowels and load transfer), and the stress sensitivity of materials.

A finite element analysis requires that the pavement structure be divided into small, rectangular elements that are connected at nodes, as shown in Figure 5-3.2. Each rectangle encompasses four triangular elements with their common node at the center eliminated. The continuous and non-linear variation of stress and strain in the real pavement are replaced by a constant stress and strain in each triangular element by assuming a linear variation in the displacement at the nodes. For each triangular element, a matrix of the stiffness of each node is set up relating the load and displacements. These stiffness matrices of all the elements are combined to obtain a symmetric banded matrix for the pavement structure. With known boundary displacement conditions, this matrix is modified, and the system of linear equations is solved to give the displacements at all nodes. From these nodal displacements, the stresses and strains of each triangular element are determined. Then, the values for the four triangular elements of each rectangle are combined to give the average stress and strain at the center. The smaller the elements of the finite element representation of the pavement, i.e., the finer the mesh used, the more accurate are the solutions obtained. However, this increased accuracy has to be balanced with the increased computer time and cost required to solve the larger system of linear equations.

There have been many enhancements to this elementary application of the finite element principle to allow the technique to be used for more complex and realistic problems. Finite element programs now exist which can take into account the effects of temperature variations in pavements as well as non-linear and inelastic materials. Three-dimensional finite element programs are also available, however, their use is limited by the very high computational effort that they require. Examples of finite element analysis programs available for the analysis of pavement structures include ILLIPAVE, ILLISLAB, and JSLAB.

Figure 5-3.2. Finite element configuration used for analysis of homogeneous and layered systems (from Dehlen, 1969).



5-3.03.03 MECHANISTIC-EMPIRICAL

The mechanistic and empirical design concepts mentioned previously can be combined to obtain procedures that incorporate mechanistic parameters and empirical performance models in the design process. These mechanistic-empirical procedures attempt to correlate mechanistic material responses (stress, strain, and deflection) with empirical damage models to serve as the basis for a pavement analysis and design procedure. These techniques allow for empirical analysis and design procedures that more closely take into account actual pavement conditions by relying on mechanistic values and principles. The Asphalt Institute Manual Series (MS-1) design method for flexible pavements and the Portland Cement Association (PCA) design procedure for rigid pavements (StreetPave) are two examples of design methods that incorporate mechanistic and empirical principles.

Another pavement design method that attempted to move towards the mechanistic-empirical approach is presented in the 1986 and 1993 AASHTO "Guide for Design of Pavement Structures." The AASHTO Road Test continues to provide the empirical background for this procedure, however, mechanistic-empirical principles such as resilient modulus (M_r) material characterization, seasonal material and moisture variation, and mechanistically determined coefficients of drainage and load transfer were incorporated into the process.

In 2006, AASHTO released preliminary versions of the "Mechanistic Empirical Design Guide", which utilizes far more mechanistic elements than the earlier mechanistic-empirical methods. In particular, the guide uses a large number of mechanistic input parameters, from resilient modulus to thermal conductivity, to quantify the damage caused by a large number of failure methods in the future. At the time this manual was last updated, the mechanistic engine of the guide was nearing completion, but the empirical damage correlations were in need of a significant amount of field verification before full implementation. Nonetheless, AASHTO expects to recommend the use of this guide in place of its older procedures within a few years.

Lastly, Mn/DOT has developed a mechanistic-empirical pavement design program, MnPAVE, which conforms with the traditional materials and practices used within the state. This program currently utilizes several different 'levels' of inputs to calculate the fatigue and rutting damage that a flexible roadway will experience over the life of the pavement.

The methods mentioned above briefly presented below to illustrate their various aspects.

1. Asphalt Institute MS-1 Design Method. The Asphalt Institute MS-1 method for the design of asphaltic concrete pavements is one of the design methods that can be considered a true mechanistic-empirical procedure. In this method, the pavement is considered to be a multi-layered elastic structure, with each layer characterized by a modulus value and a Poisson's ratio. The horizontal tensile strain at the bottom of the asphaltic concrete layer and the vertical compressive strain at the surface of the subgrade are the critical criteria used in the design procedure.

The object of the procedure is to provide a pavement structure which, for the given pavement materials, environmental characteristics, and traffic loading, would be adequate to resist fatigue failure and subgrade rutting resulting from the tensile strain and compressive strain, respectively. The mechanistic pavement analysis Chevron N-layer program, which can consider a wide range of material input parameters, environmental conditions, and traffic conditions, has also been incorporated in the computer program, DAMA, for determining the pavement layer thicknesses.

2. 1986/1993 AASHTO Design Method. The current AASHTO design method for pavement structures includes a number of mechanistically-derived improvements to the procedures which existed prior to the publication of the 1986/1993 AASHTO Guides. There are two design equations, one each for asphaltic concrete and PCC pavements. The previous design

procedures were empirical, for the most part, and based directly on the results obtained from the AASHTO Road Test. In some instances, such as in the case of rigid pavement design, these results were supplemented with then-existing theory and pavement design methods.

In the 1986/1993 guide, an attempt is made to incorporate some mechanistic principles. In the characterization of material properties for design, for example, mechanistic parameters, such as the resilient modulus, are used in place of empirical parameters, such as the CBR. The method of determining the damaging effect of seasonal variations on the pavement design also incorporates some mechanistic principles, as do the procedures for establishing the coefficients for drainage and load transfer.

3. AASHTO Mechanistic Empirical Pavement Design Guide. The Design Guide, which is currently in the beta testing phase of development, includes far more mechanistic inputs than previous mechanistic-empirical procedures. The climatic data for each project's location is loaded from one or more national weather stations in the area and has the capability to consider a wide variety of factors from the regular temperature and moisture conditions to the percentage of time the sun is shining. Likewise, material characterizations can be made using many different parameters, all of which are related back to resilient modulus for calculation. Finally, the pavement is analyzed in terms of pavement distresses particular to the surfacing material used and the results are stored in an Excel file. The large number of potential inputs allows this system to closely simulate the mechanistic system, however, they also require a very large amount of beta testing before they can be deemed accurate. This guide will most likely be useful to Mn/DOT in the future, although it is not ready for everyday use at this point.
4. Mn/DOT M-E Design Method (MnPAVE). The Department has been developing the MnPAVE design tool since 2000. This program predicts the life of a flexible pavement design using two damage criteria: subgrade rutting and asphalt fatigue. The severity of these distresses is predicted based upon material parameters, which can be entered in a variety of forms, and climatic conditions, which are based upon the county in which the work is to take place. A large number of local designs and other data have been used to calibrate the program, and it has reached a level of accuracy such that it is a recommended design tool for those designing pavements in Minnesota.

5-3.03.04 SUMMARY

The analysis procedures presented above comprise the bulk of the pavement design methods currently used by Mn/DOT. Most of these, especially in the area of flexible pavement design, are based on empirical principles. However, as the profession moves forward mechanistic principles will become more and more prevalent. The advantages associated with the use of these methods are many. For example, the true responses of the pavement materials are modeled only in the mechanistic models. This will improve the design for a particular material, as well as permit the use of a wider range of materials. The effects of physical characteristics, such as loss of subgrade support, widened lanes, and load transfer in rigid pavement design, can also be modeled more accurately in a mechanistic design. Lastly, any of the new improvements that are continually made in the field may be modeled using mechanistic principles, whereas past empirical methods would not be able to consider them. The acceptance of the need for such improvements is reflected in the mechanistic orientation of the design procedures presented in the subsequent sections of this manual.

5-3.04 MATERIAL PARAMETERS

Some of the most important inputs necessary for the analysis and design of pavement structures are the parameters that characterize the foundation soil and pavement surfacing materials. The proper characterization of these materials is essential to the development of a successful design

under any anticipated traffic or environmental loading. Typically, the material parameters used to characterize roadbed soils and pavement construction materials are either derived empirically or based on elementary physical reactions (mechanistic).

In the past, most of the material inputs for pavement design, such as the CBR and R-value, have been empirical in nature. Although these parameters have served pavement designers well, they are usually designed for a specific range of material types and properties, which may not exist in the vicinity of a project or may not be compatible with new developments. Consequently, attempts have been made to use mechanistic-based material parameters in the design of pavement structures. In this section, the material parameters used by the Department in the design of pavement structures are discussed. Although some of the empirical-based material parameters that have been used in the past are presented, emphasis is placed on the mechanistically-derived parameters used to characterize pavement and roadbed materials.

5-3.04.01 ROADBED SOILS

The parameters used by Mn/DOT to characterize roadbed soils for pavement design have included the modulus of subgrade reaction (k), resilient modulus (M_r), the R-value, and the soil support value. The important considerations that have to be taken into account in the use of these material parameters in pavement analysis and design are discussed below.

1. Resilient Modulus. The resilient modulus (M_r) has become widely accepted a good parameter for defining the stiffness of subgrade soils. It is defined as the ratio between the deviatoric stress and the recoverable strain of a soil specimen subject to cyclical impulse loadings in a confined test chamber. As a result, it characterizes the stiffness of subgrade soils under a moving wheel load and it is well adapted to mechanistic analysis.

However, the M_r is dependent on a number of factors that must be taken into account when determining the M_r value for design. To begin with, the resilient modulus value of a soil is highly dependent on the density (compaction) of the specimen as well as its moisture content. In addition, the magnitude of the applied stress affects the M_r of most soils. It is therefore necessary that the stress used in the determination of the M_r value be representative of the conditions expected in the field as well as consistent with other resilient modulus testing being performed throughout the industry.

For cohesive soils, the M_r varies inversely with the deviator stress. ($\Phi_D = \Phi_1 - \Phi_3$). A typical M_r response for fine-grained soils is shown in Figure 5-3.3. For triaxial conditions, the M_r of cohesionless or granular soils is also affected by the applied bulk stress — the sum of the principal stresses. ($\Phi = \Phi_1 + 2\Phi_3$) The relationship for such soils is shown in Figure 5-3.4. There is more than one resilient modulus test procedure being used at this time, including the LTPP Protocol 46 and NCHRP 1-37A, so designers should make certain that the tests that they are running conform with the design procedure that they are using.

Figure 5-3.3. Idealized resilient modulus curve for a fine-grained cohesive soil.

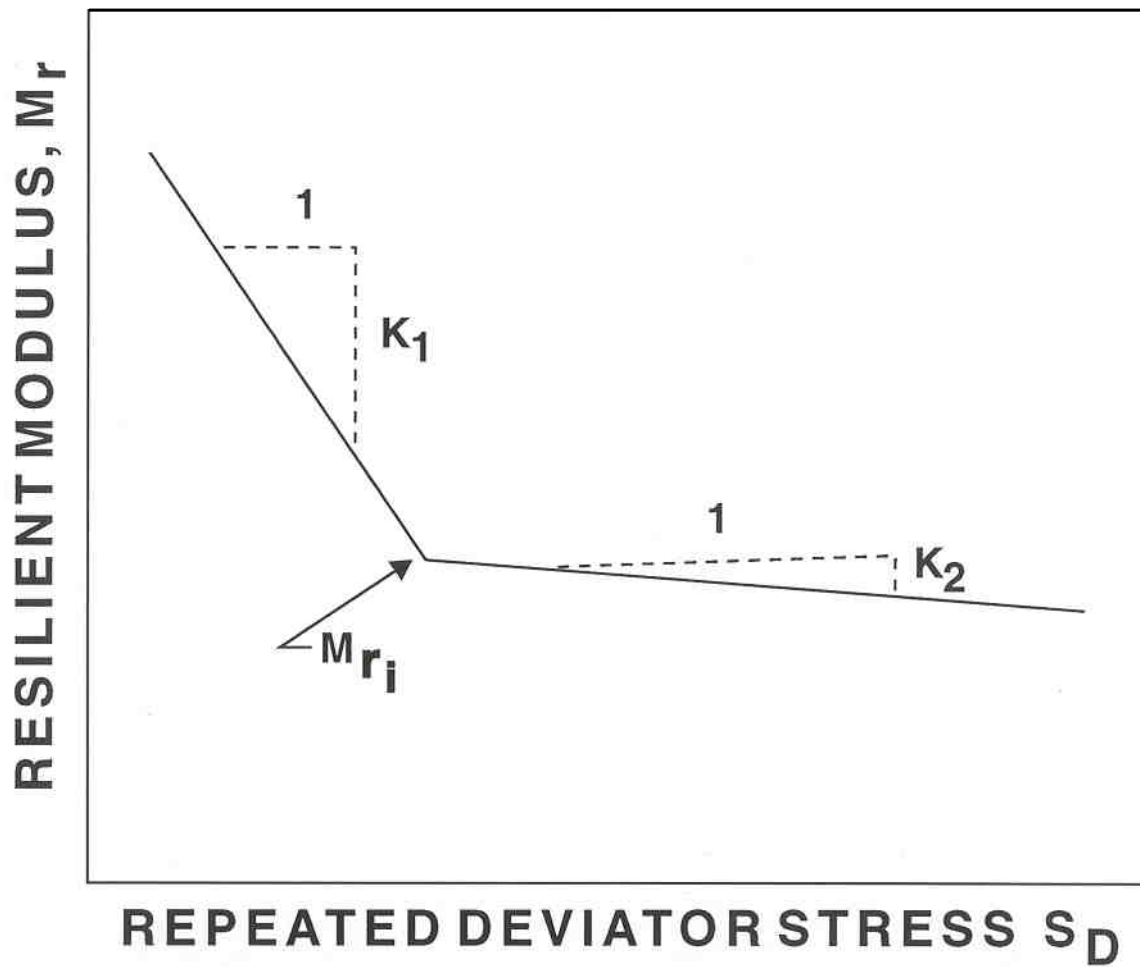
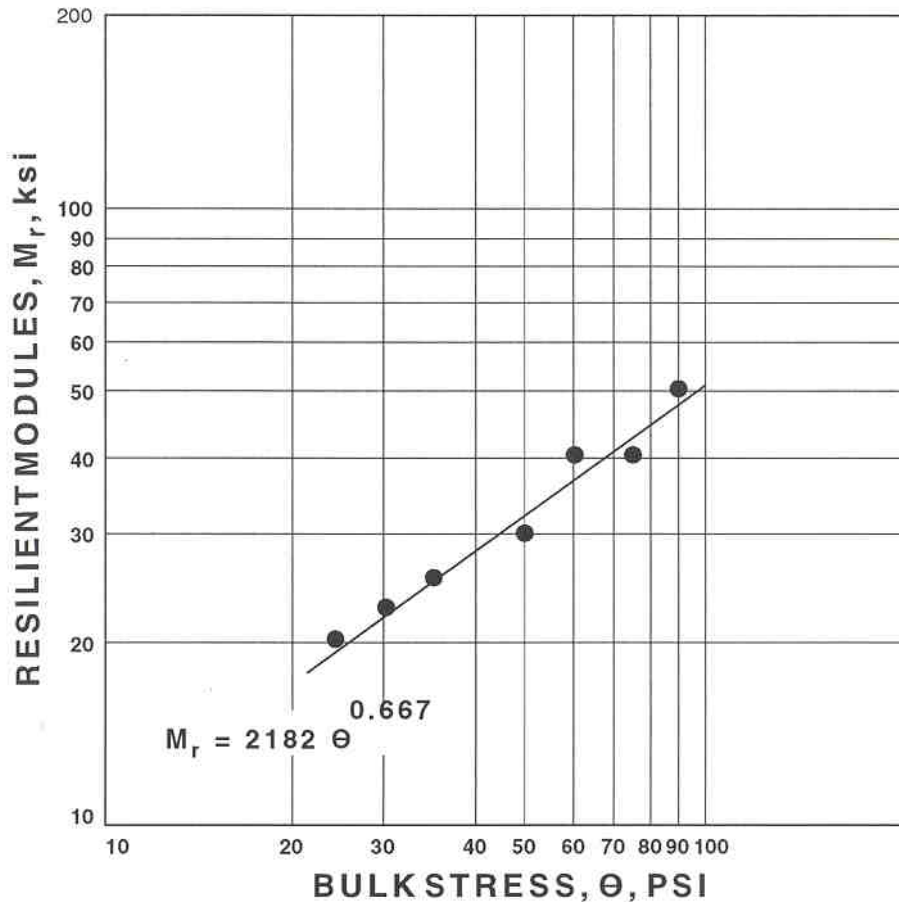


Figure 5-3.4. Resilient modulus - θ relation for a sandy gravel (AASHTO A-1-b (0)).



A discussion of the engineering significance of M_r and a brief summary of the test procedures for its determination are presented in Sections 3-2.03.04 and 4-2.06.05, respectively.

2. Modulus of Subgrade Reaction. The modulus of subgrade reaction (k) is used to characterize subgrade soils for rigid pavement design. The modulus of subgrade reaction is measured on the in-place soil on which a pavement is to be built. To determine the k value, a stress is applied to the subgrade through a 30-inch diameter plate, and the deflection is measured when the rate of deformation decreases to a predetermined constant. The modulus of subgrade reaction is calculated using Equation 5-3.1.

$$k = \frac{P}{d}$$

Eq. 5-3.1

where:

- k = modulus of subgrade reaction, psi/in.;
 p = stress, applied to the subgrade, psi; and
 d = deflection, measured at the center of the load plate, in.

Typically, large loads are required in order to obtain the stresses that will induce the kind of measurable deflection necessary for the determination of the k value. Consequently, although the concept of modulus of subgrade reaction has been in existence for some time, few direct measurements have been carried out in the field. As a result, k values are usually determined through correlations to other parameters, such as the R-value.

As part of Mn/DOT's Investigation 183, fractional plate bearing tests and Hveem Stabilometer R-value tests were conducted at 50 test sites. Regression analysis gave the following relationship (Equation 5-3.2), which is particularly suitable for Minnesota conditions.

$$k = 1.17 + 63 \sqrt{R} \quad \text{Eq. 5-3.2}$$

where:

- k = modulus of subgrade reaction, psi/in., and
 R = R-value of the subgrade.

This relationship should be used to convert R-values to equivalent k values for subgrades for Minnesotan projects. Some agencies routinely increase the k value when stiff base layers are used. This procedure is not recommended in Minnesota because it does not represent the actual support a pavement slab will experience.

3. R-Value. The engineering significance of the R-value is discussed in Section 3-2.03.04, and a summary of the test procedure for its determination is presented in Section 4-2.06.05.

For design purposes, the selection of the R-value is the responsibility of the District Soils and/or Materials Engineer. Its accurate determination is critical, because small changes in the value considerably influence the structural requirements of pavements.

Unfortunately, R-values are difficult to determine because of the limitations of the test itself, variations within a single embankment soil classification, variations in an embankment as constructed, and changes in environmental conditions. As a result, R-value data is commonly variable and some degree of judgment is necessary to arrive at a design value. In current practice, the mean R-value minus one standard deviation is often selected as the design value.

Table 5-3.1 (a) in Section 5-3.05.03 establishes guidelines for the frequency of sampling for the determination of R-values as a function of the subgrade soil texture. For small projects, where it is impractical to obtain R-values, estimates for design can be obtained from Table 5-3.3 in Section 5-3.05.03, which gives the R-value as a function of the AASHTO subgrade soil type.

4. Soil Support Value. The use of a soil support value (S) for classifying subgrade soils was first introduced in the design procedures resulting from the AASHTO Road Test. This material characterization parameter cannot be obtained from a test and has to be established by correlations with other standard soil parameters, such as the CBR and R-value, using the AASHTO Road Test results.

At the Road Test, an S value of 3.0 was arbitrarily assigned to the subgrade soils of the test sections. By studying the performance of the pavement structures with sufficiently thick bases to minimize the effect of the silty clay subgrade soils, a soil support value of 10 was

selected to represent the support offered by a thick crushed-rock base on a subgrade that would provide good performance. The soil support value for all other subgrade soils is then determined on the scale established by these two points, based on the following relationship (Equation 5-3.3).

$$\log W_{t18} = \log N'_{t18} + 0.372 (S_i - S_r) \quad \text{Eq. 5-3.3}$$

where:

- W_{t18} = total load applications for any condition, i ;
- N'_{t18} = total load applications for Road Test conditions;
- S_i = soil support value for any condition, i ; and
- S_r = soil support value for Road Test conditions.

Because of the arbitrary nature of this scale, it is important to apply sound engineering judgment in the determination of the soil support value for a given subgrade soil. It is also important to remember the specific set of conditions under which the parameter was derived.

5-3.04.02 PAVEMENT MATERIALS

Pavement materials may be characterized in a number of ways. The Department uses several methods to accomplish this, however, the resilient modulus is once again the parameter most compatible with mechanistic empirical design procedures because it best expresses the stress-strain relationship of materials subjected to roadway loadings. A second parameter, the granular equivalent (G.E.), is also used by the Department to characterize the supporting strength provided by the bituminous and granular layers of flexible pavements. This parameter is designed to give a relative measure of stiffness to these layers in comparison to a selected, high-quality, aggregate base. Concrete materials make use of the modulus of rupture to characterize their strength.

1. Bituminous Resilient Modulus. There are a number of parameters designed to characterize the stiffness of the bituminous layers in pavements. However, the resilient modulus (M_r) has gained wide acceptance for use in pavement design. The diametral resilient modulus of bituminous mixtures is determined in a test that involves the repetitive dynamic loading of a cylindrical specimen. The M_r is calculated from the magnitude of the repeated load and the measured total deformation using Equation 5-3.4.

$$M_r = P(\mu + 0.2734)/\delta t \quad \text{Eq. 5-3.4}$$

where:

- P = magnitude of dynamic load, lb.;
- μ = Poisson's ratio (taken as 0.35 in most cases);
- δ = total deformation, in.; and
- t = thickness of specimen, in.

Since the M_r is influenced by temperature, it is determined at 40, 70, and 100° F. These values represent the general range experienced by bituminous mixtures in use.

2. Aggregate Base Modulus. Traditionally, Mn/DOT has used G.E. factors and R-values to characterize the strength and stiffness of aggregate base layers, as they are inputs the empirical pavement design procedure outlined in Section 5-3.03.01. However, M_r is the parameter used most often in the mechanistic characterization of granular base materials: the derivation and factors that influence the M_r of materials were discussed in Section 5-3.04.01. Mn/DOT, and the pavement industry as a whole, have been slowly moving towards using this parameter exclusively in new mechanistic-empirical design procedures, such as MnPAVE or the AASHTO M-E Design Guide.

Figure 5-3.5 shows the relationship between M_r and R-value for a number of common base and subbase materials.

Figure 5-3.5 Base and Subbase Resilient Modulus Values (modified from John Siekmeier, 2006).

Mn/DOT Material Classification	Granular Equivalent (G.E. Value)	Resilient Modulus by Season									
		Early Spring		Late Spring		Summer		Fall		Winter	
		MPa	ksi	MPa	ksi	MPa	ksi	MPa	ksi	MPa	ksi
CLASS 7 (Mn/DOT Spec. 3138)	1.0	62	9	124	18	207	30	228	33	350	50
CLASS 6 (Mn/DOT Spec. 3138)	1.0	71	10	141	20	236	34	259	38	350	50
CLASS 5 (Mn/DOT Spec. 3138)	1.0	62	9	124	18	207	30	228	33	350	50
CLASS 4 (Mn/DOT Spec. 3138)	0.75	58	8.3	115	16.7	192	27.8	211	30.6	350	50
CLASS 3 (Mn/DOT Spec. 3138)	0.75	58	8.3	115	16.7	192	27.8	211	30.6	350	50
SELECT GRANULAR (Mn/DOT Spec. 3149.2B2)	0.5	58	8.3	115	16.7	192	27.8	211	30.6	350	50
GRANULAR (Mn/DOT Spec. 3149.2B1)	NA	35	5.1	70	10.2	117	17.0	129	18.7	350	50

3. Modulus of Rupture. The modulus of rupture, or flexural tensile strength, is widely used to characterize Portland Cement Concrete (PCC). The modulus of rupture (f_r) is defined as the stress that will cause the extreme fibers of a test specimen to break in the conventional beam-breaking test (ASTM C 78, AASHTO T 97).

This flexural test involves third-point loading of a specimen, measuring 6 x 6 x 20 inches, cast from the design mix. Tests are performed on 7-, 28-, and 90-day cured specimens to determine the f_r over time. Mn/DOT performs this test to characterize the strength of concrete used in several structures, including pavements.

The modulus of rupture is generally calculated from the conventional flexural equation for a beam in bending (Equation 5-3.5).

$$f_r = \frac{PL}{bd^2} \quad \text{Eq. 5-3.5}$$

where:

$$\begin{aligned} f_r &= \text{modulus of rupture, psi} \\ P &= \text{maximum total load, lb.} \\ L &= \text{span length of beam, in.} \end{aligned}$$

- b = average specimen width, in.
 d = average specimen depth, in.

It should be noted that this equation is only valid if the beam breaks within the middle third of the specimen. If the beam fractures outside of the middle third, but less than five percent of the span length from either support, L in the equation is replaced by $3a$, where a is the average distance on the tension side of the beam between the line of fracture and the nearest support.

Center-point loading can also be used to determine the flexural strength of concrete, using specimens smaller than described above (AASHTO T 177, ASTM C 293). This method has been utilized by the Department in the past, although it is not currently in use. The result from the center-point load test has been found to be less reliable than the third-point loading test. Unlike the third-point loading test, where the specimen is subjected to pure moment in the middle third with zero shear, substantial stresses and shear forces develop in the test specimen with center-point loading. Studies by the PCA indicate that center-point loading, on the average, overestimates the modulus of rupture by about 75 pounds per square inch in comparison to the modulus determined by the third-point loading test.

Regardless of the test method used, the modulus of rupture has some degree of variance between individual tests. Mn/DOT uses a design modulus of rupture two standard deviations below the historical test results to account for this variation and add a degree of conservatism to the design. In practice, this analysis often arrives at a value close to 500 psi.

4. Granular Equivalency Factors. Granular equivalency (G.E.) factors provide a means of equating the structural performance of all bituminous and aggregate courses that make up a pavement structure. In this sense it is similar to the structural number concept used by AASHTO, however, the ratio between material factors varies slightly to account for local materials and conditions. The G.E. factors are also different in that they are normalized to the performance of Class 5 and 6 aggregate base materials. The G.E. concept is convenient for defining or rating pavement structures in similar "units" for the purposes of comparison.

The G.E. concept, which is expressed in either millimeters or inches, is the product of Mn/DOT's Investigation No. 183 (1969), which was discussed in Section 5-3.03.01. In this study, the total granular equivalent thickness of a pavement was defined by the subgrade soil R-value and the cumulative, 80kN (18-kip), equivalent single-axle load required to reduce the Present Serviceability Index (PSI) of a pavement to a terminal value of 2.5. The required G.E. for the various pavement materials was determined by assigning granular equivalent factors to them, on the basis of their contribution to the pavement strength, in comparison to the strength offered by a layer of Mn/DOT's Class 5 or 6 aggregate base. The equivalencies are shown in Table 5-3.3 (in Section 5-3.05.02). These equivalencies have since been used to determine the thickness of the various layers required in a flexible pavement.

The total G.E. thickness is defined by Equation 5-3.6

$$\text{G.E.} = a_1D_1 + a_2D_2 + a_3D_3 + \dots \quad \text{Eq. 5-3.6}$$

where:

- G.E. = total granular thickness determined from Figure 5-3.7.
 D_1 = thickness of bituminous mixture, mm (in.)
 D_2 = thickness of aggregate base course, mm (in.)
 D_3 = thickness of aggregate subbase course, mm (in.)
 a_1, a_2, a_3 = G.E. factors shown in Table 5-3.4

5-3.04.03 ENVIRONMENTAL CONSIDERATIONS

Traditional Mn/DOT pavement design methods do not directly consider the affect of environmental factors on pavements. Instead, the design engineer is expected to take measures to minimize or completely eliminate the detrimental effects of environmental variables on a case-by-case basis. No formal procedure exists, for example, for directly treating any detrimental moisture effects. One exception to this categorization is low temperature cracking, which is mitigated through the use of Superpave asphalt binders.

Moisture effects can be broken down into the effect of moisture on the subgrade soils and the effect of moisture on the pavement materials. In the case of the latter, measures are taken to prevent the moisture content of the materials used for the pavement layers from exceeding maximum limits. These include the use of materials that are at the correct moisture contents to construct the pavement, and ensuring that the pavement has a relatively waterproof surface.

Subgrade soils that may be subject to detrimental moisture effects are most commonly removed from the project and replaced by borrow materials. Problematic areas are identified during the geotechnical survey, and the designer uses this information to decide if any action is necessary. For example, where frost-susceptible material exists and the water table is high enough to result in detrimental frost penetration and frost heave, the frost-susceptible material is replaced with non-frost-susceptible material to the depth of frost penetration.

In cases where the ground water table is high and it is estimated that inflow rates are high enough to be detrimental to the pavement, subsurface drainage should be provided to ensure adequate outflow of water from below the pavement structure. These drains are recommended in several other situations as well, including locations where seepage is expected through the pavement surface and all high-volume roadways in general. Refer to section 5-4.03 for more details on subsurface drains.

5-3.04.04 EVALUATION OF SALVAGED MATERIALS

It is often necessary or desirable to use materials salvaged from old pavements in the construction new pavements. In some cases, these materials are used directly and are simply overlaid. Some materials, however, must be recycled or rejuvenated in some manner before they can be used. Examples of recycled materials include bituminous reclamation, rubbilized concrete, and cold-in-place (CIP) recycling. Each of these materials should be tested before and/or after placement in accordance with their specifications to make certain that they meet performance requirements. GE factors are assigned to each of these materials by the pavement designer and, if they do not meet quality standards during construction, the roadway might not be able to withstand its design loading. Currently, the GE factors for bituminous reclamation, rubbilized concrete, and CIP recycling are 1.0, 1.5, and 1.5, respectively. However, there has been discussion regarding the possibility of raising some of these factors owing to improved construction equipment and techniques. See Table 5.3.4.

5-3.05 NEW CONSTRUCTION OR MAJOR RECONSTRUCTION

Pavement design for new construction or major reconstruction in Minnesota consists of the selection of the pavement type and the determination of a pavement structure that will carry the projected traffic while maintaining an acceptable serviceability level throughout the analysis period. The procedures for selecting the pavement type and for the design of the various types of bituminous and PCC pavements in Minnesota are described in the following sections.

5-3.05.01 PAVEMENT SELECTION PROCESS

Pavement Selection refers to the process used by Mn/DOT to determine whether new construction or reconstruction projects will have an asphalt or concrete surface. Designs using both materials are proposed, and the alternative that is found to be the most cost-efficient over the lifetime of the facility is selected to be built. Mn/DOT has had a pavement selection process, in one form or another, since 1959. The entire process was reviewed in 1977 and 1978 and revisions made in 1983, 1995, 1997, and 2004. Past Mn/DOT reports dealing with this process are:

- “A Critical Review of the Pavement Selection Process”, July and October 1977
- “Report on the Method of Pavement Selection”, December 1978
- “Method of Pavement Selection, 1983 Edition”, January 1983.

In addition, Technical Memorandum No. 04-19-MAT-02, September 14, 2004 pertains to pavement selection and describe changes to the process.

All new pavement construction and reconstruction projects are required to go through the pavement selection process, and these are defined as any projects in which the subgrade is worked. Therefore, rehabilitation projects such as bituminous overlays, unbonded concrete overlays, cold-in-place recycling, full depth reclamation, and concrete rubblizing are not subject to the pavement selection process.

A pavement selection process may fall into one of three categories:

- District Process
- Informal Pavement Selection Process
- Formal Pavement Selection Process

The District Process involves short projects meeting either of the following criteria:

- Two-lane roadways: Projects less than 3.22 km (2 miles) long
- Multi-lane roadways: Projects less than 25,083 m² (30,000 square yards)

The project's length and/or size listed above are determined using only the driving lanes. No turn lanes, parking lanes, or ancillary lanes are computed in the above quantities.

The Informal Pavement Selection Process is a faster method vs. the time-consuming Formal process when the most economical pavement section is clearly indicated by traffic and subgrade soil strength data. An Informal Pavement Selection Process may be termed Informal Flexible or Informal Rigid, depending on the final surfacing determination. Informal Flexible projects are those with 20-year design lane BESALs (length-weighted Bituminous Equivalent Standard Axle Loads) of 7 million or less and a design subgrade soil R-value greater than 40. Bituminous surfacings should be used in these situations because they are able to take advantage of the significant subgrade strength. Informal Rigid projects are those where the 20-year design lane BESALs exceed 10 million. It is Mn/DOT's policy to construct Portland Cement Concrete (PCC) pavements in these high-volume situations so as to make maintenance and rehabilitation traffic disruptions as infrequent as possible.

The Formal Pavement Selection Process is required for all projects not falling into one of the categories listed above. Specially, it is required for all projects with 20-year design lane BESALs less than 7 million and design subgrade R-Values less than 40 as well as all projects with 20-year design lane BESALs between 7 to 10 million regardless of the design subgrade R-Value.

Informal Pavement Selection Procedure

Technical Memorandum 04-19-MAT-02 defines the detailed information and data that must accompany a surface determination request. For an Informal Pavement Selection request, the following minimum documentation must be included:

- Cover Letter signed by the District Engineer requesting the Informal Pavement Selection. The letter should include;
 - SP number
 - Roadway number and termini
 - County where project is located
 - Project length
 - Signature and date blocks for the Pavement Design Engineer and Pavement Engineer
 - Proposed Letting Date
- Traffic data, which should include the following 20-year data for Informal Flexible projects: one-way design lane BESALs, one-way design-lane AADT, one-way design-lane TST, and one-way DHV. For Informal Rigid projects, include all of the above with the addition of the 35-year CESALs. The traffic data must be approved and signed by the Traffic Forecast Engineer within one year of the submittal date.
- A description of the project in terms of termini, total mileage, number of lanes, use of any in-place pavement, and exceptions (if any).
- A preliminary Materials Recommendation Report delineating the major soil areas and potential borrow sites by textural classification, AASHTO soil type, and R value; listing any significant topographic features such as swamps, deep cuts, etc.; and identifying aggregate production sites (including commercial sources) in the area. It should also provide information on subcuts, subdrainage, etc. for projects to be built on existing subgrades.
- A life cycle cost analysis worksheet justifying the selection of the proposed design.

Any additional pertinent information, such as special features, condition and type of existing roadway surface, and recommendations.

Formal Pavement Selection Procedure

A Formal Pavement Selection request should follow the steps outlined in Technical Memorandum 04-19-MAT-02, which should be followed in the event that it conflicts with the steps that are summarized below.

- 1) The district sends a Pavement Selection request to the Pavement Management Engineer. The submittal packet should include the following:
 - a. Cover letter signed by the District Engineer
 - b. A general description of the project and its location
 - c. A map showing the existing and/or proposed roadway
 - d. Typical sections showing the proposed roadway width, number of lanes and shoulder configuration (not pavement thickness)
 - e. A preliminary Materials Design Recommendations report describing past history and providing information to be used in the pavement design, including laboratory test results substantiating the design subgrade soil R-Value
 - f. A signed traffic forecast showing the 20 and 35 year BESAL and CESAL estimates. The forecast must be signed by the State Traffic Forecast Engineer and dated within one year of the submittal date of the pavement selection packet

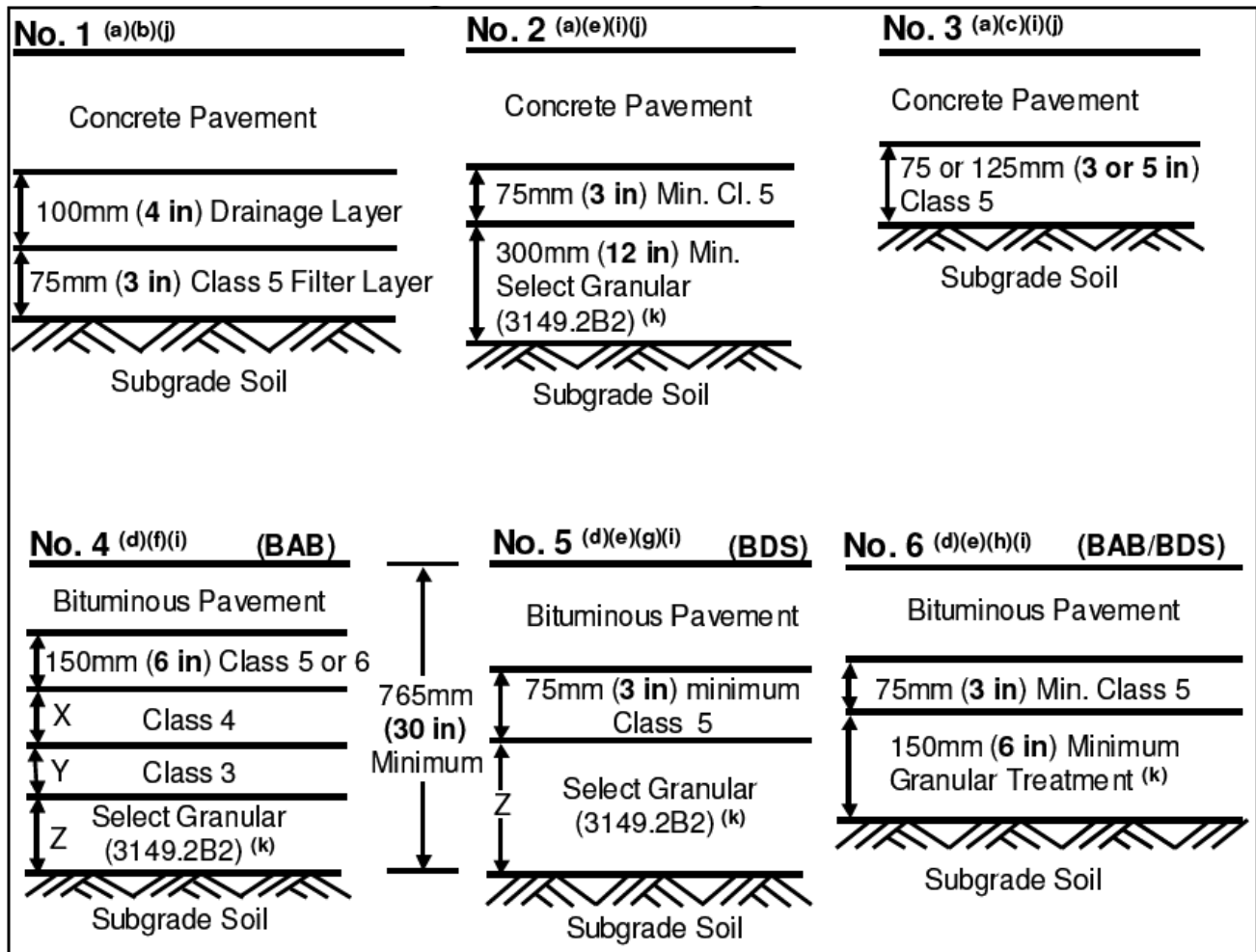
- 2) The project is logged in and the packet is sent to the Pavement Design Engineer who develops the standard pavement sections for comparison. The designs are done in accordance to Table 5-3.1 and Figure 5-3.6 and generally include the following:
 - a. Rigid – Open Graded Aggregate Base (OGAB)
 - b. Rigid – Select Granular Base
 - c. Flexible – Deep Strength Design (BDS)
 - d. Flexible – Aggregate Base Design (BAB)

Material choices should be made in the manner most likely to minimize costs without sacrificing the quality of the design. It is desirable for the design engineer to be in contact with district personnel and the Estimating Engineer for approximate pricing information during this process.

Table 5-3.1 Pavement Selection Process and Design Options (Tech Memo No. 04-19-MAT-02, 2004)

20-Year Design Lane BESALs	Subgrade Soil R-Value	Process Type Design(s)	Description of Design(s)
1,000,000 or less	> 40	Informal – Flexible Design #6	Flexible – Aggregate Base (BAB) Flexible – Deep Strength (BDS)
	<= 40	Formal Design #3 & 6	Rigid – Aggregate Base Flexible – Aggregate Base (BAB) Flexible – Deep Strength (BDS)
1,000,001 to 7,000,000	> 40	Informal – Flexible Design #4 & 5	Flexible – Aggregate Base (BAB) Flexible – Deep Strength (BDS)
	<= 40	Formal Design #1, 2, 4, & 5	Rigid – Open Graded Base Rigid – Select Granular Flexible – Aggregate Base (BAB) Flexible – Deep Strength (BDS)
7,000,001 to 10,000,000	All Values	Formal Design #1, 2, 4, & 5	Rigid – Open Graded Base Rigid – Select Granular Flexible – Aggregate Base (BAB) Flexible – Deep Strength (BDS)
Over 10,000,000	All Values	Informal – Rigid Design #1 & 2	Rigid – Aggregate Base Rigid – Open Graded Base

Figure 5-3.6 Pavement Design Standards



- 3) Once the designs are complete, the packet is sent to the Estimating Engineer who prepares a detailed cost estimate for each design. Information is gathered from the districts as well as the Concrete Paving Association of Minnesota (CPAM) and the Minnesota Asphalt Pavement Association (MAPA) on available aggregate sources. For each pavement design, and Equivalent Uniform Annual Cost is calculated on a per mile basis and the Low Cost Option is determined. The analysis includes the following:
 - a. Bituminous Estimates
 - i. Current, percent discount rate
 - ii. 50 year analysis period (with the assumption of no residual value)
 - iii. A set schedule of mill & overlay, route & seal, etc. treatments are assumed depending on the traffic (ESAL) level
 - b. Concrete Estimates
 - i. Current, percent discount rate
 - ii. 50 year analysis period (last rehab assumed to have 33% residual life)
 - iii. Joint reseal after 17 years, minor CPR at 17 and 27 years, major CPR at 40 years
- 4) Once the estimate is finished, the Pavement Management Engineer sends a memo to the district identifying the Low Cost Option. The district is given two weeks to determine if they

concur. If the district concurs, the process is complete and MAPA and CPAM are notified of the Low Cost Option.

- 5) If the district does not agree the low cost option should be used, it an appeal to the Deputy Commissioner/Chief Engineer for a variance. The variance process begins by the District Engineer informing the Director of the Office of Materials, in writing, that the District is seeking a variance.

The Pavement Management Engineer will serve as secretary and be responsible for taking minutes of the Pavement Selection Committee meetings. The committee reviews the pertinent information and determines the final pavement type for the project. Once the committee has decided, MAPA and CPAM are notified and the process is complete.

Premium enhanced design options may be considered following the completion of the selection process. Enhanced design options include such items as Stone Matrix Asphalt (SMA) mixtures, premium dowel bars for PCC pavements, improved aggregate quality, and increased aggregate base and/or select granular material over the minimum thickness specified in Figure 5-3.6.

The cost of materials above the subgrade soil, as depicted in Figure 5-3.6, shall be included in the pavement selection economic analysis. Salvaged asphalt credit will not be given to any option in the pavement selection economic analysis. Yield losses from any concrete options will also not be included in the pavement selection economic analysis.

When the Formal Pavement Selection Process is used, adjacent projects can be combined for the pavement selection provided the following conditions are true:

- The combined length of the project is 25 miles or less, and
- The time between the earliest and latest letting date is five years or less.

Combining adjacent projects for pavement selection purposes accommodates longer projects that may be done in stages. This will result in a single pavement type for the combined length of the project.

5-3.05.02

PAVEMENT DESIGN STANDARDS

In the early 1990s, Mn/DOT and the local pavement industry had concerns regarding the performance and constructability of Mn/DOT's pavement designs. Therefore, a bituminous task force was established to study the issues. The findings and recommendations of this group were incorporated into new pavement design standards, which Mn/DOT released in 1995.

These standards revised or modified all previous bituminous and concrete pavement design practices and required that a certain thickness of frost-free materials (FFM) be incorporated into the various designs. The minimum depth of the FFM is a function of the 20-year design lane ESALs and varies between 30 and 36 inches for most bituminous designs. The exact thickness should be determined by consulting Technical Memorandum No 04-19-MAT-02. The FFM includes aggregate base (Mn/DOT's Specification 3138, Classes 3, 4, 5, 6, and 7) and select granular borrow (Mn/DOT's Specification 3149.2B2), which contains less than 12 percent passing the 0.075 (No. 200) sieve. These modifications were developed based on consultation with the FAA, Army Corps of Engineers, the AASHTO Guide, and European counterparts.

The pavement design standards for both concrete and bituminous pavements are contained in the Pavement Selection Technical Memorandum No. 04-19-MAT-02. These standards apply to all projects, whether or not they require submission for the Pavement Selection as described in Section 5-3.05.01.

These pavement designs are subject to change and will be evaluated periodically in terms of Mn/ROAD and Long Term Pavement Performance (LTPP) research findings and Mn/DOT's

objectives in designing and constructing economical pavements that will provide smooth ride and long-term performance and service life.

5-3.05.03 BITUMINOUS PAVEMENT DESIGN

Bituminous or flexible pavements may be designed as either bituminous pavements with an aggregate base (BAB) or as deep strength bituminous (BDS) pavements. In both cases, the structural requirements are determined as a function of the cumulative traffic load applications expected during the design period of the pavement, the subgrade soil strength, and the strength provided by each component of the completed pavement.

The design procedure consists of the determination of the total thickness of pavement components for given traffic and subgrade conditions. The procedures developed for the design of bituminous pavements with aggregate base and deep strength bituminous pavements are based on research conducted by the Department and reported in Mn/DOT's Investigations 183 and 195, respectively. (See Section 5-3.03.01)

The structural designs are based upon the cumulative damaging effect of traffic over a 20-year period, although it is expected that the pavement will be serviceable for a much longer period if regular maintenance and rehabilitation treatments are provided. These treatments extend bituminous pavement life by restoring ride quality, thickening the cross-section to enhance structural capacity, and improving the skid resistance characteristics.

Mn/DOT requires a number of steps to design a successful flexible pavement. To begin, a request for a traffic analysis should be submitted to the District Traffic Forecaster who will, in turn, request assistance or approval from Mn/DOT's Office of Transportation Data and Analysis (TDA). The information included in this report should be the one-way design lane ADT and the 20-year design lane ESALs. In situations where high design-lane AADT volumes are forecast, Design Hour Volume (DHV) estimates should be used to determine the correct one-way lane-volume values for design; it is customary to design arterial highways with a sufficient number of lanes to accommodate the forecast DHV for the design year. In addition, there may be a significant difference in the pavement loading in opposing directions on a multi-lane roadway. In such cases estimates of the cumulative ESALs in each direction should be provided. All of this data should be provided for each year of the design life of the roadway. The sources of the traffic data and the process of forecasting ESALs are discussed in Section 4-5.0.

The second piece of information required for a bituminous pavement design is an accurate soil characterization. Therefore, the District Materials and/or Soils Engineer, in conjunction with the Pavement Design Engineer, must determine a design R-value for the subgrade soil. This determination should be made during the project's geotechnical survey, and the results should be provided in the preliminary Materials Design Recommendations report. This report must be available for the pavement surface-type determination and should include, at a minimum, delineation of the major soil areas and potential borrow sites by texture, AASHTO soil type, and R-value; a listing of any significant topographic features such as springs, swamps, and deep cuts; and a listing of the potential aggregate production sites in the area, including commercial sources. The R-value selected for design should be the average value of the field samples measured in the lab minus one standard deviation. This design value must be representative of the soil in the upper 1.2 m (4 ft) of the proposed road grade. Lastly, the design R-value should be adjusted upwards if any select granular material (Mn/DOT's Specification 3149.2B2) is used on the project, as this material will be of a better quality than the in-place soil. The adjustment will be discussed later in the design sections of the manual.

Table 5-3.2 establishes the sampling frequency guidelines for stabilometer R-values as a function of major soil textural classifications. Tables 5-3.3 (a) and (b) lists typical R-values associated with AASHTO soil types. These tables are only to be used for small projects where it is impractical to obtain and test R-value samples.

Table 5-3.2. Stabilometer R-value sampling frequency guidelines.

<u>Major Soil Texture</u> *	<u>Recommended Minimum Sampling Rate</u>	<u>Minimum Number of Samples</u>
Sands	0 (assume a value of 70 or 75)***	0***
Clays, Clay Loams	1 every 2 miles	3**
Sandy Loams (nonplastic to slightly plastic)	3 per mile	5
Silt Loams	3 per mile	5
Silty Clay Loams	3 per mile	5
Plastic Sandy Loams	3 per mile	5
Sandy Clay Loams	3 per mile	5

* 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 0.075 (#200) sieve 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 representative of the upper 1.2 m (4 ft) 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 5-3.3(a). Stabilometer R-values by soil type, based on data collected by Mn/DOT through 1974.

<u>AASHTO Soil Type</u>	<u>Textural</u>	<u>Assumed R Value</u>	<u>Comments</u>
A-1-a	Sands, Gravels	75	Excellent confidence in using assumed value.
A-1-b	Sands, Sandy Loams (nonplastic)	70	If percent passing #200 sieve is 15 to 20%, 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 (nonplastic, 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 5-3.1 for sampling 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 5-3.2 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.

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's Specification 2105 and be capable of passing test rolling, Mn/DOT's Specification 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 5-3.3(b) Typical assumed R-values for granular subgrades

<u>Subgrade</u>	<u>Assumed R-Value</u>
Four feet of select granular	70
One foot of select granular and three feet of granular	60
Four feet of granular	50

Granular - (Mn/DOT Spec. 3149.2B1)
 Select Granular - (Mn/DOT Spec. 3149.2B2)
 (AASHTO Soils Types - A-1-a, A-1-b, and A-3)

1. Bituminous Pavement with Aggregate Base (BAB).

a. Pavement Design Overview–

The design of bituminous pavement with aggregate base is based on the concept of the Granular Equivalent (G.E.), which is the product of Mn/DOT's Investigation No. 183 (1969). In this study, the total granular equivalent thickness of a pavement was defined in terms of the subgrade design R-value and the 20-year design lane 80 kN (18-kip) bituminous ESALs (BESALs) required to reduce the Present Serviceability Index (PSI) of a pavement to a terminal value of 2.5. (A PSI of 2.5, by definition, is a ride quality condition at which trunk highways require a structural overlay to restore rideability and load support capacity.) G.E. per inch values are assigned to each of the pavement materials on the basis of their contribution to the pavement strength in comparison to the strength offered by a layer of Mn/DOT's 3138 Class 5, 6, or 7 base aggregate. Lastly, a pavement structure is designed with sufficient material thicknesses to meet or exceed a minimum G.E. threshold.

Figure 5-3.7 is the design chart for bituminous pavements with aggregate base. This chart is used to determine the required G.E. structural design thickness, expressed in inches, that will support the given 20-year BESALs on the design subgrade R-values. This value is converted into the thickness required for the various pavement structural components using the G.E. factors in Table 5-3.4 and/or a local program named 'FLEXPAVE'. The final pavement structural thickness must satisfy the total G.E. requirements, the minimum thicknesses shown in the design chart, and the overall minimum thickness shown in designs 4 (BAB) and 5 (BDS) Figure 5-3.6.

If the design chart results in a total converted thickness that is:

- i. Greater than the minimum pavement structural thickness (Figure 5-3.6 (4, 5)), then the design thickness should be in accordance to the design chart (Figure 5-3.7) and no further modifications are necessary.
- ii. Less than the minimum pavement structural thickness, then the structural thickness needs to be increased to the minimum thickness using Class 3 aggregate base, Select Granular subbase, or a combination of these materials. However, if the project's 20-year design lane BESALs are seven million or less and the in-place soils meet the requirements for granular material (3149.2B1, less than 20 percent passing the 0.075 mm (No. 200) sieve) the required "Z" thickness may be reduced to 300 mm (12 in).

In Figure 5-3.6, the type(s) of material selected for use in the “Z” thickness in designs (4, 5) and in the granular treatment shown in design (6) should be based on economic considerations and ease of construction. The District Materials and/or Soils Engineer will make this determination with the assistance of the Pavement Design Engineer.

b. Slow Traffic Modification

Additional thickness may be required at specific locations for flexible pavements (both BAB and BDS designs) under certain conditions. For sections that experience low speed traffic and/or high shear stresses due to stopping and turning movements (such as urban freeways, bus stops, intersections, weigh stations, etc.) elastic analyses recommend the following additional thicknesses:

<u>Predicted Percent of Slow Traffic</u>	<u>Increased Thickness</u>
1% - 3%	12.5 mm (0.5 in.)
± 3%	25.0 mm (1 in.)

Special mix designs should also be considered for such locations.

c. Design Procedure (BAB)

The design procedure for bituminous with aggregate base (BAB) pavements is based on Figure 5-3.7. The following are step-by-step instructions for using the chart.

- (1) The total structural requirement (in terms of G.E.) for a BAB pavement design is determined by extending a line from the design lane ESAL value to intercept the design R-value curve on Figure 5-3.7. From this intercept, extend the line left to the G.E. ordinate line to locate the G.E. value for these conditions. Note that the "traffic" line intersects two minimum requirement lines when projected to the top of the chart. These are the minimum bituminous and base material lines; they indicate the portions of the total G.E. that must be contributed by the bituminous and base layers, respectively. Bituminous base or Class 6 aggregate base may be used to satisfy minimum base requirements.
- (2) The portion of the G.E. that remains after determining the bituminous and base layer values should be accounted for using Class 3, Class 5, select granular, and/or other engineered subbase materials. Material selection should be based on economics and constructability.
- (3) The G.E. values determined above - minimum bituminous, minimum base, and additional materials (if any) - are converted into material thicknesses by dividing the values by the G.E. factors in Table 5-3.4. All fractions should be rounded upwards to the nearest full inch. Construction costs and limitations should be taken into account before accepting these thicknesses.
- (4) The total pavement thickness must equal or exceed the minimum thickness indicated in Figures 5-3.6 (4, 5). If the thickness resulting from the G.E. factor conversion is less than the minimum thickness, then the structural thickness must be increased to the minimum by adding “Z” thickness of Class 3, Class 5, and/or select granular borrow materials. If the total pavement thickness exceeds the minimum thickness, then the pavement structure should be designed in accordance with this thickness.

d. Design Problems

The following are two example design problems demonstrating the use of the bituminous pavement design chart for bituminous over aggregate base (BAB) designs. These designs are based upon Figure 5-3.7.

Problem #1

Determine the BAB design for a pavement given the following information:

20-year design-lane bituminous ESALs $\leq 700,000$

R-Value Test Results = 25, 17, 28, 22

Solution:

- (1) To make use of Figure 5-3.7, it is necessary that one 'design' R-value be used to represent all the foundation soil on the project. This value is determined by subtracting one standard deviation from the average of the test results.

$$\text{Average R-value} = (25+17+28+22)/4 = 23$$

$$\text{Standard Deviation} = 4.7 \text{ (round up to 5.0)}$$

$$\text{Design R-value} = 23 - 5 = \underline{18}$$

- (2) The granular equivalent intercepts are determined by entering the chart in Figure 5-3.7 at the ESAL value of 700,000 and moving down to intersect the correct design R-value line (in this example, 18) in the lower section of the chart. The points at which the traffic line intersects the minimum bituminous line, the minimum base line, and the design R-value line are denoted points B, C, and D respectively. A G.E. value corresponding to each of these points must be determined by projecting a line horizontally from each of these points to the 'y-axis'. In this example:

$$\text{Minimum Bituminous Intercept (Point B)} = 8.5 \text{ G.E.}$$

$$\text{Minimum Base Intercept (Point C)} = 14.5 \text{ G.E.}$$

$$\text{Design R-value Intercept (Point D)} = 25 \text{ G.E.}$$

- (3) Once the intercepts have been located it is possible to determine G.E. values and thicknesses for each of the material layers.

- (a) The minimum bituminous G.E. value for the pavement is the vertical distance between the top of the chart and the minimum bituminous intercept. Therefore, the minimum bituminous G.E. value is the same as the minimum bituminous intercept, 8.5 G.E. The G.E. factor for the bituminous layer can be found in Table 5-3.4, where it can be seen that all bituminous layers have a factor of 2.25. Lastly, the bituminous thickness is calculated by dividing the minimum bituminous G.E. value by the bituminous G.E. factor.

$$\text{Minimum bituminous thickness} = 8.5 / 2.25 = 3.8 \text{ inches (should be rounded to the nearest half-inch, in this case 4.0)}$$

- (b) The minimum base G.E. value for the pavement is the vertical distance between the minimum bituminous intercept and the minimum base intercept.

$$\text{Minimum base G.E. value} = 14.5 - 8.5 = 6.0 \text{ G.E.}$$

The G.E. factor for the base layer can be found in Table 5-3.4, where it can be seen that Class 5 and 6 base materials have a factor of 1.0. (Rubblized concrete, Cold-In-Place recycled materials, and other treated materials may be allowed larger G.E. factors) The base thickness is calculated by dividing the minimum base G.E. value by the base G.E. factor.

$$\text{Minimum base thickness} = 6.0 / 1.0 = 6.0 \text{ inches (should be rounded to the nearest inch)}$$

- (c) The additional material G.E. value for the pavement is the vertical distance between the minimum bituminous intercept and the design R-value intercept.

$$\text{Additional material G.E. value} = 25.0 - 14.5 = 10.5 \text{ G.E.}$$

The G.E. factor for the additional layer can be found in Table 5-3.4, where it can be seen that select granular subbase materials have a factor of 0.5. (Class 3 and 4 materials are allowed a larger G.E. factor of 0.75) The subbase thickness is calculated by dividing the minimum base G.E. value by the select granular G.E. factor.

$$\text{Minimum subbase thickness} = 10.5 / 0.5 = 21 \text{ inches (should be rounded to the nearest inch)}$$

- (d) Based on the above analysis, the completed design thickness and layer types in the pavement structure are:

<u>Thickness</u>	<u>Specification/Mixture Type</u>
4.0 in.	Bituminous
6.0 in.	Class 5
21.0 in.	Select Granular

The total thickness of this pavement is 31.0 inches, which exceeds the 30" minimum thickness required by Tech Memo 04-19-MAT-02. At this point the basic BAB design is complete, however the design should be checked for constructability, economic, environmental, and other concerns.

Figure 5-3.7. Bituminous Pavement Design Chart (Aggregate Base).

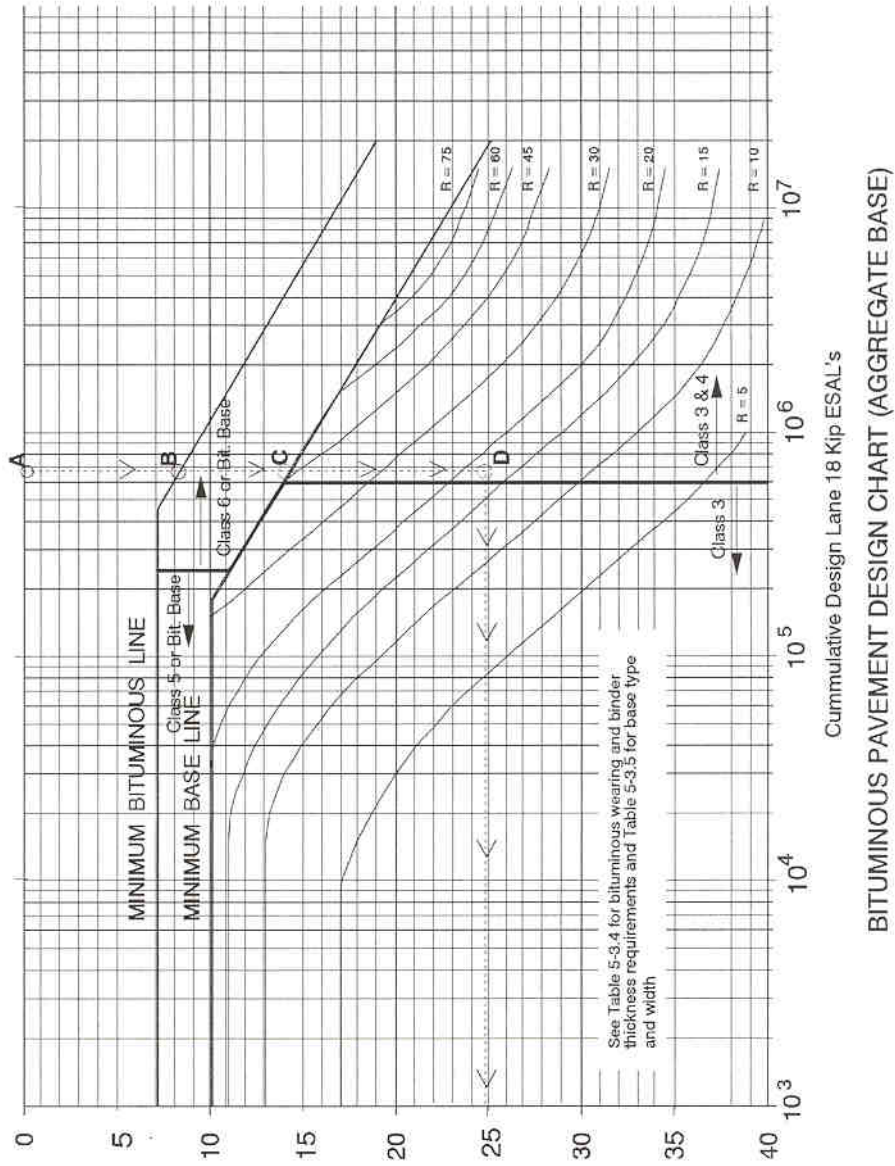


Table 5-3.4. Granular Equivalent (G.E.) factors.

Material	Specification	G.E. Factors
Bituminous Pavement	2350/2360	2.25
Cold-Inplace Recycling (CIR)	2331*	1.50
Pavement Breaking/Rubblized Concrete Pavement (2231*	1.50
Bituminous Pavement Reclamation (FDR)	2231*	1.00
Aggregate Base	3138 (Cl. 5, Cl. 6)	1.00
Aggregate Base	3138 (Cl. 3, Cl. 4)	0.75
Selected Granular Material**	3149.2B2	0.50

* By Special Provision

** When the subgrade is constructed with Select and/or Granular material (3149.2B) the District Materials and/or Soils Engineer may recommend that the upper portion of the subgrade be treated with stabilizing aggregate (3149.2C) at a rate of 85 kilograms (150 lb) per square meter (square yard) or more; or capped with 75 mm (3 in) of Cl 5 aggregate base (3138) to establish an uniform construction platform on which pavement layers can be properly placed and compacted.

2. Bituminous Deep Strength Pavement (BDS).

- (a) Pavement Design Practice - Bituminous deep strength pavement was formerly defined as a pavement structure in which bituminous mixtures are employed for all courses above the subgrade or improved subgrade. This type of pavement structure became a standard design alternative for flexible pavements in Minnesota in 1978. However, the full depth bituminous pavement structural design was modified in 1995 to address frost and other foundation concerns. (Refer to Section 5-3.05.02 for more information) This modification required that the pavement structure be constructed over 75 mm (3 in) of Class 5 aggregate base and enough select granular material to thicken the total structure to at least 30 inches.

This was a major pavement design change for Mn/DOT. Prior to 1995, it was allowable to construct full depth bituminous pavements on almost any subgrade, including those with fine contents in excess of 20%. However, the current standards require that the subgrade incorporate frost-free materials (meeting at least select granular material requirements) so long as the existing materials are not already frost-free. As a result of this subgrade design modification, Mn/DOT no longer constructs 'true' full depth bituminous pavements; the modified designs are known as deep strength bituminous pavements (BDS).

The design procedure for BDS pavements was derived from the BAB procedure, which was based on the results of Mn/DOT's Investigation 195. The BDS design procedure is, therefore, similar to the BAB procedure in the sense that both make use similar logarithmic charts.

The principal factors evaluated for BDS structural design are the traffic expected throughout the design period, the subgrade material characteristics, and a variety of environmental considerations. In practice, the pavement thickness is first determined

as a function of the 20-year, design-lane, 18 kip, bituminous ESALs (BESALs) and the subgrade design R-value, then modified as necessary to account for site specific environmental, material, or other factors. The ESAL values are obtained from forecasts using the same method described in Section 4-5.0.

(b) BDS Structural Thickness and Width Determination

The design chart shown in Figure 5-3.8 is used to determine the bituminous and select granular layer thicknesses for deep strength pavement structures illustrated in designs 5 (BDS) and 6 (BAB/BDS) of Figure 5-3.6. (This design thickness may also be determined using the Mn/DOT program FLEXPAVE) The chart makes use of the same inputs (cumulative ESALs and the adjusted design R-value) as the BAB chart, and the procedure for its use is similar. The bituminous thickness can be found by moving down from the correct ESAL value on the horizontal scale to the correct R-value curve, and then across the chart horizontally to read the necessary bituminous thickness. It should be noted that the output from this design chart is the bituminous thickness (not G.E.), so no conversion is necessary as is the case for bituminous pavement with aggregate base (BAB).

Another important difference between BDS and BAB pavement design is that the design R-value (test average minus one standard deviation) requires an adjustment for BDS designs if the existing subgrade soils do not meet the requirements of select granular material (3149.2B2), which is usually the case. This adjustment is necessary because the chart was created under the assumption that no engineered granular material would be added to the pavement structure and that the in-place soils would provide the entirety of the foundation support. Mn/DOT no longer follows this practice for the reasons listed above, and the granular materials that are added contribute to the structural support of the pavement structure.

The adjusted R-value is calculated using the BAB design chart in Figure 5-3.7. In this figure, the point at which the subgrade (test) R-value curve and expected traffic loading line intersect is located, the point is moved upwards by the G.E. value of the placed select granular material, and this new point is located at the adjusted R-value. This procedure often results in an iterative design process because the select granular layer thickness is usually the difference between the bituminous thickness (which is a function of R-value) and the required frost-free depth. (The 3" base layer is not included in the G.E. adjustment value as it is not designed to carry a structural load: it is included in the pavement solely as a construction platform.) Therefore, an R-value adjustment causes a change in the select granular layer thickness, which is a factor in the R-value determination. The iterative adjustment procedure is demonstrated in BDS example problem #1.

The thickness of the select granular material may be reduced to 12 inches if the 20-year design-lane BESALs are between one and seven million and the in-place subgrade soils meet the requirements for granular material (3149.2B1; less than 20 percent passing the 0.075 mm (No. 200) sieve). In this case, the R-value adjustment is based on the 12 inches of select granular material.

Additional thickness may be required in certain situations due to low speed traffic and/or high pavement stresses such as bus stops, intersections, and weigh stations. Guidelines for increased thicknesses are provided in the bituminous pavement with aggregate base (BAB) section.

The wearing and non-wearing courses should be designed at a width of 24 and 28 ft, respectively, for a two-lane roadway. For multi-lane roads, the non-wearing course(s) should extend 3 ft beyond the edge of the outside lane and 1 ft beyond the edge of the inside lane. The wearing course(s) should be designed as 12 ft lanes.

For urban design, the non-wearing courses should be placed in accordance with the shoulder structural design detail shown in the Road Design Manual.

The 3 inch Class 5 aggregate base in designs 5 and 6, Figure 5-3.6, should extend at least 18 inches beyond each edge of the bituminous non-wearing course width. The select granular subgrade width and drainage considerations are illustrated in Figure 5-3.7.

It should be noted that because of strip-loading conditions where deep strength pavement is used to widen existing PCC pavement, all chart-derived thickness requirements are to be increased by a factor of 1.2 for the widened section.

(c) Example Problems (BDS)

The following are two example design problems demonstrating the use of the bituminous pavement design chart for bituminous deep strength (BDS) designs. These designs are primarily based upon Figure 5-3.8, although Figure 5-3.7 is used to adjust the R-value.

Problem #1

Determine the BDS design for a pavement given the following information:

- 20-year design-lane bituminous ESALs = 700,000
- Existing subgrade soil design R-Value = 18
- Minimum structural thickness of 30 inches from Figure 5-3.6(5)

Solution:

(1) The first step in an iterative BDS pavement design is to determine the thickness of the bituminous layer by using Figure 5-3.8 (or Mn/DOT's FLEXPAVE program). The point at which the design R-value curve intersects the projected traffic level indicates the required bituminous thickness on the vertical axis. For this example, the R-value and traffic curves (18 and 700,000, respectively) intersect at a bituminous thickness of 9.5 inches.

(2) Secondly, the "Z" thickness of select granular material is determined. This is accomplished by subtracting the bituminous thickness determined above, as well as the required 3 inches of Class 5 material, from the minimum frost-free depth from Figure 5-3.6.

$$\text{"Z"} = 30.0 - 3.0 - 9.5 = 17.5 \text{ inches (rounds up to 18.0)}$$

(3) Next, the select granular Granular Equivalent (G.E.) must be calculated by multiplying the "Z" thickness by 0.5 (the select granular G.E. factor from Table 5-3.4).

$$\text{G.E.} = 18.0 \times 0.5 = 9.0$$

(4) At this point it is necessary to adjust the subgrade R-value as described in section (b). This adjustment is made by finding the intersection of the existing subgrade (test) R-value curve and the traffic line on the BAB (NOT BDS) design chart (Figure 5-3.7), which are 18 and 700,000 for this example. From this point, it is necessary to move a distance "up" the chart (towards the 0 G.E. line) equal to the G.E. value of the select granular subgrade; this distance should be measured using

the G.E. values on the vertical axis. Once at this new point has been located, a new R-value should be estimated from the surrounding R-value curves.

5) With an initial R-value of 18 and 700,000 projected ESALs, the first point on the BAB design chart is 'Point D' at 25 G.E. The second point is located 9.0 G.E. "above" this point on the 700,000 ESAL line at 16 G.E. (25-9). After measuring the vertical distance between the 30 and 45 R-value curves, it can be seen that the new point has an R-value near 41.

(6) After determining the adjusted R-value, the second iteration begins. A new bituminous thickness is determined using the original traffic projection (700,000 ESALs) and the adjusted R-value (41) using the same process as in step #1. The new thickness is 8.0 inches.

(7) Steps 2 through 4 are repeated to calculate a new (adjusted) R-value using the 8.0-inch bituminous thickness.

$$"Z" = 30.0 - 3.0 - 8.0 = 19.0 \text{ inches}$$

$$\text{G.E.} = 19.0 \times 0.5 = 9.5$$

$$\text{Adjusted R-value} = 43$$

Make certain to count upwards from the original (existing) subgrade R-value each time a new adjusted R-value is calculated.

(8) The third iteration begins: the new bituminous thickness is 7.9 inches. At this point it is clear that further iterations will make no difference once the thickness is rounded: the previous iteration resulted in a bituminous thickness reduction of 0.1 inch (8.0 to 7.9) and all future iterations will round to 8.0 inches. 8.0 inches should be used as the final bituminous thickness.

(9) The final pavement structure is as follows:

<u>Thickness</u>	<u>Material</u>
8.0 inches	Bituminous surfacing
3.0 inches	Class 3 aggregate base
19.0 inches	Select Granular

The total thickness of this structure is equal to the minimum structural thickness indicated in Figure 5-3.6(5), 30 inches. The widths of the various structural components should be as discussed in the previous section (b) Structural Thickness and Width Determination (BDS)

Figure 5-3.8. Bituminous Pavement Design Chart (Deep Strength bituminous).

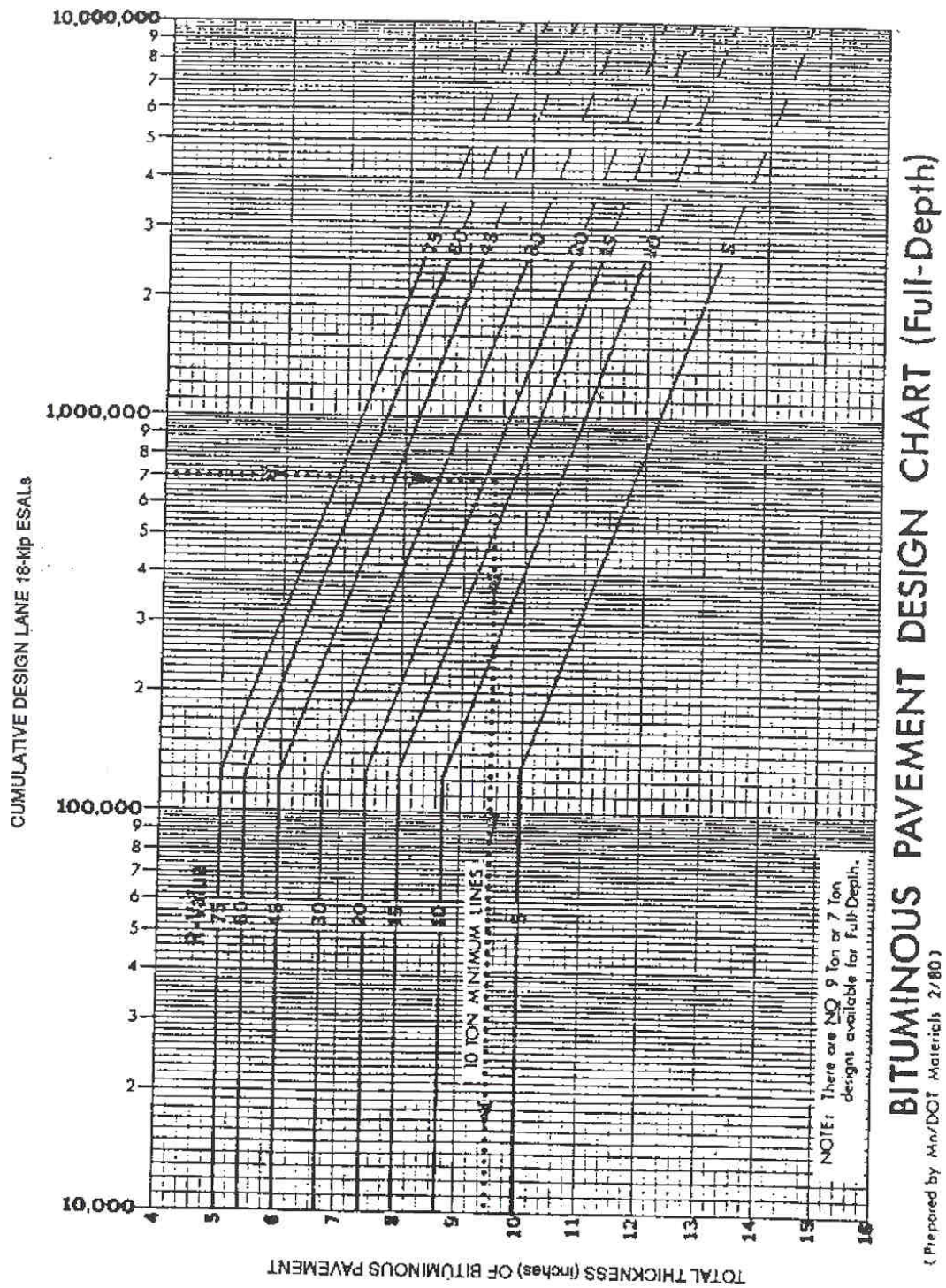


Table 5-3.6. Base type and width design

20-Year Design Lane ESALs	Base Type	Base Width ^c
Less than 250,000	Bituminous ^a	24 Feet ^f
	Class 5	Full Width
	Class 3 ^b	Full Width
250,000 to 600,000	Bituminous ^a	24 Feet ^f
	Class 6 ^c	30 Feet ^g
	Class 3 ^b	Full Width
More than 600,000	Bituminous ^a	24 Feet ^f
	Class 6 ^c	30 Feet ^g
	Class 4 ^d	Full Width
	Class 3 ^d	Full Width

- a. District Materials and/or Soils Engineer in conjunction with the Pavement Design Engineer may substitute bituminous base for all or a portion of Class 5 and/or Class 6.
- b. District Materials and/or Soils Engineer in conjunction with the Pavement 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 Pavement 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 should be 6 inches unless otherwise approved by the Pavement Design Engineer. If less than 6 inches of Class 3 and 4 are required, use all Class 4 unless otherwise approved by the Pavement Design Engineer.
- e. On urban sections, all bases are full width.
- f. If the total thickness of the bituminous base exceeds 3 inches, use width of 27 ft.
- g. Or 3 ft beyond each pavement edge.

5-3.05.04 PORTLAND CEMENT CONCRETE PAVEMENT DESIGN

Portland cement concrete (PCC) pavements (also known as “rigid”) consist of a slab placed on a base over a prepared subgrade, on an existing PCC pavement (unbonded and bonded overlays), or on an existing flexible pavement (“white topping”). This section of the manual deals primarily with the design of PCC pavements placed directly on base and prepared subgrade materials. The design procedure for unbonded overlays is described in Appendix D. The Pavement Design Unit (Office of Materials) should be contacted to determine the most current bonded overlay and white topping design procedures.

The primary goal of the PCC pavement design procedure is to determine slab and granular layer thicknesses that will provide a satisfactory ride quality for the smallest annual cost over a 35-year design life. However, several factors must be taken into account before making such a decision. Site-specific variables such as the traffic projections, soil and water conditions, and aggregate availability may have a large effect on the design. The same is true of design factors such as the

joint design, shoulder width, and material parameters. Lastly, economic factors such as the available funding and intended design life must be considered. As a result, the PCC pavement design procedure often becomes a process of finding the best possible balance for a number of competing interests.

In the past, Minnesota designed and constructed two types of PCC pavement. These were jointed plain non-reinforced concrete pavement (JPCP) and jointed reinforced concrete pavement (JRCP). After reviewing the cost and performance of both pavement designs, the decision was made to discontinue the construction of JRCP. Therefore, Mn/DOT is currently constructing only JPCP. The following are a number of issues to consider when designing JPCP.

1. **Pavement Lane Width (Protected vs. Non-protected Edge).** Concrete pavements have been found to be significantly more durable when the slab is widened past the minimum necessary for safe traffic flow. There are two reasons for this. First, much of the damage to a concrete pavement is caused by heavy vehicles driving on or near the edge of the slab. This configuration induces large edge stresses in the slab and may cause corner and edge cracking. A pavement with a properly designed widened edge will allow the vehicles to travel further from the edge of the slab and reduce these stresses. The second advantage to widened edge design is that surface runoff water is drained further from the wheelpaths before it (potentially) sinks into the shoulder joint. The presence of water in the base causes a number of problems for concrete pavements, and they perform better the further the water is displaced from the high stress areas.

In light of these advantages, Mn/DOT has designated the protected edge design as standard. "Protected" edge designs encompass the widened slabs mentioned above, standard-width slabs (12') with tied concrete shoulders, and, in most cases, urban designs with tied curb and gutter or integrant curb. The protected edge design for a widened two-lane roadway design has a total width of 27 ft with two 13.5 ft lanes. For a four-lane divided roadway, the lane widths are 13 and 14 ft for the inside and outside lanes, respectively. If a roadway has more than four lanes, then the standard 12' width may be used for the interior lanes of a protected edge design. In each of these cases the lanes are striped at the standard 12' width.

Mn/DOT requires the use of rumble strips for the protected edge design, which are constructed on the edge of the widened section or the tied concrete shoulder. The use of these strips lessens the number of vehicles encroaching onto the widened section or the tied concrete shoulder, thereby minimizing edge stresses and reducing run-off-the-road accidents. The rumble strip design is illustrated in Figure 7-4.03A of the Road Design Manual, while Mn/DOT's policy on rumble strips is discussed in Section 4-4.02.

2. **Slab Thickness.** Concrete pavement thickness design in Minnesota is based on the 1981 AASHTO Interim Guide procedure, although a modification has been added to adapt the nationally-calibrated equation to local conditions. Using this procedure, the concrete slab thickness is determined using the cumulative 35-year design lane concrete ESALs and the modulus of subgrade reaction (k). A discussion on the determination of concrete ESALs (CESALs) is presented in Section 4-5.0. The modulus of subgrade reaction and its derivation from R-values are discussed in Section 5-3.04.01.

Equation 5-3.6, which was developed from the AASHTO Road Test, calculates the number of ESALs that a pavement can withstand before it falls to a given serviceability level. This equation forms the backbone of Mn/DOT's concrete pavement design, and it may be solved iteratively using the program "Rigid PAVE".

$$\log W_t = 7.35 \log (D + 1) - 0.06 + \frac{G_t}{1 + \frac{1.624 \times 10^7}{(D + 1)^{8.46}}}$$

$$(4.22 - 0.32p_t) \log \left[\left(\frac{S_c}{215.63J} \right) \left(\frac{D^{0.75} - 1.132}{D^{0.75} - \frac{18.42}{(E/k)^{0.25}}} \right) \right] \quad \text{Eq. 5-3.6}$$

where:

W_t	=	number of applications of 18-kip, equivalent, single-axle ESAL loads required to reduce serviceability to p_t
D	=	slab thickness, in.
G_t	=	the logarithm of the ratio of the loss in serviceability at time t to the potential loss taken to a point where $p_t = 1.5$, or $\log [(4.5 - p_t) / (4.5 - 1.5)]$
p_t	=	serviceability at end of time t (terminal serviceability)
S_c	=	modulus of rupture, psi (third-point loading)
J	=	load transfer coefficient ($J = 2.6$ for protected edge designs and 3.2 for other applications)
E	=	Young's modulus of elasticity of concrete, psi
k	=	modulus of subgrade reaction, psi/in

Mn/DOT's standard values for these inputs are as follows:

- Serviceability Factor (p_t). The serviceability factor is a measure of a pavement's current functionality as measured by a number of factors related to smoothness. A rating of 5.0 is perfectly smooth new construction, and the value declines as the pavement ages. Mn/DOT uses a terminal serviceability factor of 2.5 in this program for both urban and rural designs.
- Modulus of Subgrade Reaction (k). The mean modulus of subgrade reaction is the gross value from a field plate bearing test. Unfortunately, these tests are impractical to run in design situations, so agencies must rely on relationships to convert other soil values to the k value. Mn/DOT has developed a relationship between a soil's R-value and the k value that is presented in Equation 5-3.2 in Section 5-3.04.01. This equation has been incorporated into the computer program Rigid PAVE, therefore, R-value is the only required soil input. It is not recommended that the k value be modified in cases where non-standard base or subbase layers are utilized (as would have been the case for bituminous deep strength designs).

It should be noted that Mn/DOT's k value relationship significantly differs from relationships used by other agencies. Caution should be exercised when comparing designs with multiple programs and/or processes.

- Modulus of Rupture (S_c). The mean modulus of rupture (flexural strength) used in the analysis is from 28-day, third-point loading beam tests in the field. The test method is discussed in Section 5-3.04.02, Item No. 3. Recent 28-day flexural strength data shows that the average third-point loading strength value is approximately 650 psi. However, Mn/DOT chooses to introduce a safety factor to the design at this point and divides this

value by (approximately) 1.33. Therefore, the design modulus of rupture used in Rigid PAVE is 500 psi. Once again, care should be taken when using this value in programs from other sources.

- d. Young's Modulus of Elasticity (E). The static modulus of elasticity test for concrete (ASTM C469) is rarely conducted for pavement design. A value of 4,200,000 psi is recommended at the present time for use in the concrete pavement design equations.
 - e. Cumulative ESALs (W_i). The 35-year design lane ESALs, previously determined in Section 4-5.0, are modified to account for local weather conditions and protected edge conditions as follows:
 - An adjustment factor of 0.93 is applied to the cumulative ESALs to account for the difference in severity and duration of winter conditions between Ottawa, Illinois (AASHTO Road Test) and the state of Minnesota
 - Non-protected edge design makes use of a load transfer coefficient (J) of 3.2, which does not induce a modification to the input ESAL value. Protected edge design (which is the standard in Minnesota) allows the pavement to withstand many more loadings, as mentioned above, and is reflected in the design by using a J-factor of 2.6. This factor effectively modifies the input ESAL value by a factor of 0.5: this adjustment is automatically made within Mn/DOT's RigidPave program.
3. Base, Subbase, and Subgrade. The materials placed beneath the slab have a large effect on its overall performance. The base normally consists of one or more compacted layers of aggregate placed between the subgrade and the slab. A correctly-designed base provides uniform and stable support for the pavement slab, prevents the pumping of fine-grained material at slab joints, reduces cracking and faulting, contributes to pavement drainage, and serves as a stable working platform for slab construction. Furthermore, a base layer may be designed to be permeable, which may assist in mitigating problematic moisture conditions that would develop in a dense graded base. Mn/DOT Specifications 2105 and 2211 detail the construction techniques for the preparation of paving grades.

As discussed in Section 5-3.05.02, Mn/DOT developed new bituminous and concrete pavement design standards in 1995, which are illustrated in designs 1, 2, and 3 of Figure 5-3.6. The primary change in these standards occurs in design 2, which requires that pavements that are designed to withstand more than one-million ESALs in 20 years shall utilize at least a 12-inch select granular layer over the existing subgrade soils if they do not meet the requirements of select granular material (Mn/DOT Specification 3149.2B2). The purpose of this layer is to better control frost action, drainage, and the other previously mentioned attributes for high-volume roadways.

The base normally extends 3 ft beyond the outside edges of the concrete pavement structure to accommodate construction operations. Refer to Chapter 7 of the Road Design Manual for specific details.

The following are particular base design recommendations to supplement the information in Figure 5-3.6:

- Table 5-3.7 outlines the cases in which design 2 (the Permeable Aggregate Base (PAB) drainage system) should be given consideration. When using PAB, the slab thickness may be reduced 1/2 inch below the standard design thickness. However, in no case should the thickness be less than 7 inches.
- In design 3, use 3-inches and 5-inches of Class 5 when constructing over granular (less than 20 percent passing the No. 200 sieve) and non-granular subgrade soils, respectively.
- In design 2, the design R-Value (average minus one standard deviation) for the existing subgrade soils should be adjusted to account for the presence of the select granular layer

unless the embankment soils meet the requirements of select granular material, in which case no select granular material is required. The 3 inches of aggregate base are not included in this adjustment as they are only designed to serve as a construction platform. The procedure for making this adjustment is the same as the one described in Section 5-3.05.03 for Bituminous Deep Strength pavements.

- Subsurface drainage should be considered for designs 2 and 3 if the subgrade soils consist of non-granular material (greater than 20 percent passing the No. 200 sieve). Specifically, subcut drains and pavement edge drains should be considered for designs 2 and 3, respectively. These drains should be designed in accordance with the guidelines provided in Figure 5-3.7 and Section 5-4.03. A more extensive discussion of pavement drainage considerations is included in section 5-4.0.

Table 5-3.7 Concrete Pavement Permeable Aggregate Base (PAB) Application Guidelines

Subgrade Soil	Plastic / Non-Granular				Granular (D)			
	VH	H	M	L	VH	H	M	L
Interstate	R	R	NA	NA	R	R/AR	NA	NA
Non-Interstate	R	R	R	AR	R	R/AR	NR	NR

Legend:	Traffic Level:	35-yr Design Lane CESALs
AR = As Recommended (A)	VH = (Very High)	> 30 million
NA = Not Applicable (B)	H = (High)	9 – 30 million
NR = Not Recommended	M = (Medium)	3 - million
R = Recommended	L = (Low)	< 3 million
R/AR = (C)		

NOTES:

(A) AR – As Recommended. District Soils/Materials Engineer should consider:

1. Past pavement performance and experience
2. Types of distress (D-cracking, etc.)
3. Anticipated paving aggregate quality
4. Availability of materials
5. Grade-line modification needed to improve grade-line / drainage with respect to the in-place water table
6. Cost differential and anticipated increase in service life through the use of permeable type bases

(B) NA – Not Applicable. Applies to interstate traffic levels M and L. (Interstate has only VH and H traffic levels)

(C) R/AR – R (Recommended) if granular material has between 12 and 20 percent passing the No. 200 sieve per Mn/DOT 3149.2A. AR (As Recommended) if granular material has 12 percent or less passing the No. 200 sieve per Mn/DOT 3149.2B.

(D) A granular subgrade is defined as having 20 percent or less passing the No. 200 sieve in its uppermost 3 ft.

4. Concrete Joints. Joints are placed in rigid pavements primarily for the purpose of controlling transverse and longitudinal cracking. The inclusive of joints relieves stresses in the pavement that result from shrinkage of the concrete and differential temperature and moisture conditions between the top and bottom of the slab. Furthermore, stresses induced by vehicle loads affect the pavement structure in a number of ways and must be taken into account in joint design. Note that all joint layouts should be reviewed by the Concrete Unit (Office of Materials).
 - a. Transverse Contraction Joint Design (C). Properly designed transverse contraction joints will control transverse cracking in the pavement by relieving shrinkage, thermal, traffic

and other stresses to a great extent. Mn/DOT currently constructs all JPCP designs with transverse joints spaced uniformly at 15 feet and a perpendicular orientation to the roadway centerline. The primary exception to this standard is for urban design sections in which the curb and gutter joints should align with the pavement joints. Mn/DOT experimented with longer joint spacings and angled cuts in the past, however, the standard design was found to provide a more reliable performance.

Mn/DOT requires that all transverse contraction joints utilize dowel bars to provide load transfer across the joint. This practice provides a number of benefits. First, the potential for differential deflection across the joint is reduced, which reduces the likelihood of erosion, pumping, and subsequent faulting of the joint. Second, tensile stresses near the top corner of the slab are reduced, which nearly eliminates corner breaks, diagonal cracks, and perhaps even some transverse cracks in situations where severe warping or negative curling is present in the slab.

These dowel bars are generally 15 inches in length, and are placed mid-depth in the pavement at 12-inch spacing. However, the bars should not be placed within 6 inches of the pavement edge or a longitudinal joint. Mn/DOT currently requires the use of corrosion-resistant, epoxy-coated bars, although the performance of other bar types is being investigated. The following table contains the dowel bar sizing requirements.

Table 5-3.8 Concrete Pavement Dowel Bar Sizes

Pavement Thickness Inches (in)	Dowel Bar Diameter Inches (in)
$t < 10.5$	1.25
$10.5 \leq t < 13$	1.50
$13 \leq t < 14$	1.75

Accurate dowel placement and good concrete consolidation around the dowel are very important to good performance. The bars should be placed parallel to the top of the subgrade with no more than a 0.2-inch vertical tilt and parallel to the pavement centerline with no more than a 0.2-inch horizontal tilt within the length of the bar.

Details showing the dowel bar assembly are shown in Standard Plate No. 1103. Note that the dowel bar placement distance varies at the pavement edge depending on the lane width (from 6 in. for a 12 ft lane to 32 in. for a 14ft lane).

The transverse contraction joint is designated as C4E-D. The joints are sealed with preformed elastic-type seals (namely, neoprene compression seals) or hot pour sealants (Mn/DOT 2725) when the concrete mix is produced with Class B type aggregate (Mn/DOT 3137), which includes carbonates, shylite and schist. Neoprene seals are used exclusively in the Metro area, although silicone sealant (Mn/DOT 3722) is used elsewhere.

Mn/DOT's Standard Plans Manual shows transverse contraction joint design details for concrete mainline pavement and ramps in Figures 5-297.217 and 5-297.219. Figure 5-297.221 (1 of 2) depicts joint class designations and design details of the joint sealant reservoirs and sealants.

- b. Longitudinal Joint Design (L). Longitudinal joints are used to prevent longitudinal cracking induced by thermal and moisture gradients in the slab as well as shrinkage stresses resulting from friction between the slab and base. The slab thickness, base stiffness and friction, subgrade stiffness, and lane and shoulder widths all affect the potential for longitudinal cracking. Past experience has shown that longitudinal cracking

will occur when longitudinal joint spacing is greater than 15 ft for conventional slab thicknesses (6 to 10 in.). Longitudinal joints should coincide with pavement traffic lane lines whenever possible to improve traffic separations.

Longitudinal joint configurations are usually referred to by abbreviations. The most common configurations are be butt-joint (L3), mechanically-formed or keyed (L2K), or sawed-groove (L1). Deformed steel tie bars are often necessary to facilitate load transfer across longitudinal joints, maintain aggregate interlock (I1 joint), and keep adjacent lanes and tapers from separating and faulting. In these situations, the letter 'T' is added to the joint class designation (i.e.L1T). The tie bars size should be in accordance with Table 5-3.9, and the entirety of Mn/DOT's longitudinal joint design layout practices are illustrated in Figure 5-3.11.

Table 5-3.9 Concrete Pavement Tie Bar Sizes

Panel Length		Pavement Thickness		Spacing		Length		Size No.*
meters	feet	mm	inches	mm	inches	mm	inches	
4.6	15	$t \leq 250$	$t \leq 10$	760	30	760	30	13
4.6	15	$t > 250$	$t > 10$	760	30	915	36	16

* = metric size, no SI customary unit size given.

Figure 5-3.11 Concrete Pavement Longitudinal Joint Layout Design – Class Designation

No. Lanes	Lane Configuration											
	pvmt edge		L1T	pvmt edge		Normal Lane Width						
2	shoulder	inside	+	outside	shoulder	Lane	meters	feet				
						inside	4.0	13.0				
						interior	3.6	12.0				
						outside	4.2	14.0				
3	shoulder	inside	+	middle	+	outside	shoulder					
4	shoulder	inside	+	middle (1)	+	middle (2)	+	outside	shoulder			
5	shoulder	inside	+	middle (1)	+	middle (2)	+	middle (3)	+	outside	shoulder	

Notes:

1. The joint layout may change in the field depending on the construction stages, concrete paving sequence, and paving equipment width capabilities.
2. Refer to supplemental steel guidelines for the middle slab (2) in the 5-lane configuration.

Mn/DOT’s Standard Plate Manual offers additional keyed joint design (L2K) details in plate number 1141. Furthermore, Mn/DOT’s Standard Plans Manual shows longitudinal joint tie bar placement design details for concrete mainline pavements and ramps in Figures 5-297.217 and 5-297.219. Figure 5-297.221 (2 of 2) depicts joint class designations and joint sealant design details. The following are a list of recommendations from these sources:

- A total roadway width exceeding 51 ft or 4 lanes (including a concrete shoulder) should be tied together for any extended length.
- If there are more than five lanes the outside three lanes should always be tied together and the L3 joint should be kept, if possible, at least two lanes away from a pavement edge to prevent lane separation.

- Tie bars are placed in the slab at mid-depth.
 - Tie bars should not be placed within 18 in. of a doweled transverse contraction joint.
 - Joints may be sealed with a hot-pour elastic joint sealer (Mn/DOT Specification 3725). An 'H' is added to the joint designation when this material is used. (i.e. L1TH, L2KTH, etc.)
 - Supplemental panel reinforcement is required when the slab exceeds 15 ft in width without a longitudinal joint, as well as in the middle lane when the tied pavement width exceeds four lanes (or 51 ft). This reinforcement is necessary to forestall the formation of irregular longitudinal cracks: refer to Item No. 5 in the Supplemental Steel Guidelines.
 - All steel tie bars must be epoxy-coated (Mn/DOT Specification 3301) and meet Grade 60 for AASHTO M-31 or M-53.
 - The use of longitudinal joint inserts is prohibited.
- c. Expansion Joint Design (E). Expansion joints are used to prevent thermal compressive stresses from damaging the pavement slab by the transmission of excessive forces to adjacent structures. Expansion joints are normally used for concrete pavement surfaces or bases adjacent to fixed structures, curbs and walls, crossroads, and crossovers. In addition, unusual conditions, such as cold weather construction or the use of materials with a high coefficient of expansion, may require special consideration.
- Mn/DOT's Standard Plan, Figure 5-297.221 (1 of 2) provides details for the various types of expansion joints used by the Department.
- The width of an expansion joint is typically 0.5 in. or more. In some cases, it may be desirable to place two or three expansion joints in a row to protect a structure. Filler material (Mn/DOT Specification 3702) is commonly placed 1 in. below the slab surface to allow space for sealing material.
 - Load transfer is provided most commonly by dowel bars, which are fabricated with a cap on one end that creates a void in the slab to accommodate the movement of the dowels as the joint closes.
- d. Construction Joint Design. Construction joints are used to temporarily bound a slab at the end of a day's paving, and are also used to divide the pavement into suitable increments for construction. So much as possible, construction joints are eliminated by making them coincide with the other joint types. Refer to Item No. 7, Concrete Pavement Structure Appurtenances, Item b for discussion on the use of various types of headers where concrete pavements terminate.
5. Supplemental Steel Reinforcement Design. Concrete pavements often crack in certain situations due to material shrinkage and foundation soil movement. These cracks can be prevented or minimized by the placement of supplemental steel in the slab. This steel will hold any cracks that may form tightly together and maintain the pavement as an integral structural unit. Situations where supplemental steel should be considered include concrete slabs constructed over a culvert, wide pavement slabs without longitudinal joints, a number of lanes tied together, and storm sewer construction. The steel reinforcement guidelines for these situations are as follows:
- Concrete Slabs Over Culverts. Refer to Standard Plate 1070L.
 - ✓ This design should be used when the fill height (H_f) is less than 10 ft regardless of the pipe size.

- ✓ Use No. 13 bars for pavement thickness less than 12 in. and No. 16 bars for thickness 12 in. or greater. Bar placement in the slab should be at mid-depth plus or minus 1 in.
- ✓ Culvert pipes are assumed to be open on one or both ends. A closed pipe would be unlikely to have the cold air flow that could cause heaving problems and, therefore, it would reduce the need for reinforced panels as long as the pipe is 4 ft or greater below grade.
- Concrete Mainline Pavement. Refer to Standard Plans Figure 5-297.217.
 - ✓ Place in panels where pavement widths exceed 15 ft without a longitudinal joint and in the middle lanes of a tied pavement with more than 4 lanes.
 - ✓ Use No. 13 bars for pavement thickness 10 in. or less and No. 16 bars for thickness greater than 10 in. Bar placement in the slab should be at mid-depth plus or minus 1 in.
- Concrete Ramp Pavement. Refer to Standard Plans Figure 5-297.219.
 - ✓ Place in panels where pavement slab width exceeds 15 ft without longitudinal joints. Use the same bar size as previously mentioned for mainline pavement.
- Storm Sewers and Water Mains. There are no specific standards for these situations, but the following are recommended:
 - ✓ If the fill around a pipe is properly compacted, supplemental steel should not be needed. Supplemental steel should be considered if the compaction quality is a concern and the diameter of the pipe is 24 in. or larger.
 - ✓ Supplemental steel should be used regardless of the pipe size if the fill height (H_f) is less than 4 ft.

The Pavement Design and Concrete Units (Office of Materials) should be contacted for all supplemental steel reinforcement applications.

6. Parking Lot Design. Concrete surfacing for parking lots should be designed in accordance with the report "Guide for Design and Construction of Concrete Parking Lots," by ACI Committee 330.
7. Concrete Pavement Structure Appurtenances.
 - a. Pavement End Anchors. An end anchor is used to arrest the transmission of pavement forces induced by either the ingress of incompressible materials, thermal (expansion and contraction) forces, and/or gravitational activity acting on jointed pavements that are constructed on greater than four percent grades. These anchors are designed to prevent the transmission of expansive forces from the pavement to an abutting bridge structure. The anchorage system develops its working capacity through the development of the passive bearing and shear resistance forces of the underlying subgrade soils.

The end anchor is used near the terminal end of a rigid pavement that is constructed on a grade that abuts a bridge structure. These anchors are used in conjunction with a bridge approach panel, concrete sill, and pressure relief joint. The system details are illustrated in Figures 5-297.230M and 5-297.235 of Mn/DOT's Standard Plans Manual. "Lug" anchors have been found to be ineffective in Minnesota and are no longer used.

The end anchors are intended only for downgrades abutting bridge structures and should not be used in other situations (i.e. on-ramps, abutting downgrade pavements) unless recommended by the Pavement Design and Concrete Units (Office of Materials).

- b. Pavement Headers. Pavement headers are installed at all locations where concrete pavements terminate. There are four types of headers: permanent headers, terminal headers, construction headers, and railroad crossing headers.

The purpose of the "permanent header" is to resist the normal expansion and contraction of the concrete pavements due to temperature fluctuation, protect the joints adjacent to the ends of the concrete pavement from excessive movement, and prevent joint deterioration. The headers are placed at the beginning and end of concrete pavement jobs where the new pavement is placed adjacent to a bituminous pavement.

If the concrete pavement is placed adjacent to an existing concrete pavement, a "terminal header" should be used. In this method, the new concrete pavement is doweled into the existing concrete pavement to maintain a smooth ride and prevent faulting between slabs.

The "construction header" is used at the end of a day's paving. This header is a non-moving joint used to tie the concrete pavements together to maintain ride. See Mn/DOT's Standard Plate No. 1150 for details on permanent, terminal, and construction headers.

The last header type is a "railroad crossing header", which is designed to protect a railroad by resisting all pavement movement. See Mn/DOT's Standard Plate No. 1210 for details.

- c. Approach Panels. Approach Panels are reinforced slabs used between all bridges and adjoining pavements that are designed to carry the traffic load from the pavement to the bridge. When a bridge is built within a concrete pavement section, a special header/sleeper slab is designed to transfer the load from the pavement to the approach panel. This slab resists the pavement movement due to temperature changes and has an expansion device built in as a safety factor to resist possible encroachment from the concrete pavement to the approach panel and the bridge. The details for the bridge approach panels for concrete and bituminous mainline pavements are shown Mn/DOT's Standard Plans Manual, Figures 5-297.223 through 5-297.229. The criterion for the use of the bridge approach panels is discussed in Section 7-2.04.04 of the Road Design Manual.
8. Typical Cross Sections. Cross sectional design elements include pavement and shoulder cross-slope, lane and shoulder width, side slope and curb. Refer to Chapters 4 and 7 of Mn/DOT's Road Design Manual.
 9. Shoulder Designs. Refer to Section 5-3.08.
 10. Ramp and Loop Designs. Refer to Section 5-3.09.

5-3.05.05 DRAINAGE CONSIDERATIONS

The importance of quality pavement drainage has become increasingly emphasized in recent years. The sustained saturation of a pavement structure has the potential to negatively affect performance in a number of ways. In rigid pavements, common problems include pumping of the base, non-uniform support due to void creation, and deterioration of susceptible aggregates and mortar in concrete mixtures. Flexible pavements suffer from pumping of base fines, stripping, a loss of base and subgrade support, and accelerated fatigue or alligator cracking of the surface. Both types of pavements may be affected by soil swell and frost heave, as well.

Typically, the water that is of concern to pavement designers comes from two sources. Surface water from precipitation can sink into defects in the pavement and shoulder surface. Subsurface water can enter the pavement structure via a naturally high ground water table or other natural drainage paths that may have been disrupted by the placement of the roadway.

Three measures should be taken to reduce the damaging effects of water to pavements. First, water should be prevented from entering the pavement structure. Second, drainage structures should be incorporated into the system to ensure that the water that does enter the pavement is promptly removed. Third, pavements should be designed to an adequate degree to withstand the expected effect of water, traffic loads, and other external factors. The specific measures that may be considered to prevent water infiltration are discussed in Section 5-4.0. Ditch design, which is of prime importance in all cases, is discussed separately in Section 4-6.03 of the Road Design Manual.

Experience has shown that attempts to solve drainage problems through pavement design alone, typically by increasing thicknesses and using special materials, frequently fail to perform at an acceptable level. In many cases this failure is the result of our limited capacity to accurately characterize the effects of moisture on pavements. Therefore, the inclusion of beneficial drainage elements is often the preferred solution to potential moisture problems.

5-3.05.06 CONSTRUCTION CONSIDERATIONS

One aspect of pavement design that is often not given due consideration during a project's design phase is the issue of constructability. By and large, the best way to avoid constructability concerns is to involve construction personnel early in the design process. However, there are several steps that a designer can take on his or her own to prevent future problems. This section of the manual addresses two construction issues that should be considered during every pavement design: the handling of construction traffic and the scheduling of project activities.

1. **Traffic.** Traffic that has been diverted by construction projects can cause damage to the pavement under construction as well as existing structures if not handled properly. Therefore, the design engineer should give consideration to the traffic level, composition, and route during the design phase of the project. A construction engineer should be contacted to determine if a particular design is advisable in their locality. Furthermore, several general guidelines can be followed to ensure that the construction traffic itself does not damage the pavement.

It is desirable for pavement construction to proceed towards the material sources being used on the project. This practice prevents the haul trucks from driving on the newly constructed sections of a pavement to the greatest extent possible. A plan detailing the path of the construction traffic should be drawn up before the start of construction to ensure smooth operations and prevent problems.

Advanced planning of the channeling of construction traffic and adherence to such plans will also considerably improve the economy of the construction and increase on-site safety. Personnel awareness of the established patterns of construction traffic will reduce the occurrence of accidents.

2. **Scheduling.** Scheduling techniques designed to keep construction projects moving safely and on-schedule are recommended for most projects and are offered in most construction management texts. These techniques range from the use of simple bar charts showing the expected progress of activities with time to the use of the critical path method (CPM), which shows the dynamic relationship between activities. The exact technique to use depends on the size and complexity of a project.

Horizontal time bar (Gantt) charts are a simple method that displays the planned start and completion times of activities, and their relative timing. Secondary bars, color coding, and other such methods are often placed adjacent to the bars to show the progress of activities during construction. A disadvantage of bar charts is that they often fail to show the interrelationships between activities clearly.

Triangular bar charts are another method that have been used to schedule projects. These charts are similar to the Gantt charts, however, they also show the percentage of work completed on a vertical scale. There are obvious advantages to being able to keep track of the rate at which the various activities are carried out. Highway construction activities are often repetitive in nature: the same activities are often performed over and over for successive sections of the pavement. A good record of the rate of construction will allow beneficial changes to be made during the construction of subsequent sections. Other names given to this method are line-of-balance (LOB), vertical production method (VPM), and linear schedule method (LSM).

Relatively advanced scheduling techniques, such as CPM and the Project Evaluation and Review Technique (PERT), can also be used for highway projects. However, because these methods take more time to use than the simple bar charts, they are often reserved for specific activities within a project or for large projects where the scheduling efficiencies outweigh the time requirement. CPM and PERT are similar techniques that show activity durations and earliest start times, the relationship between an activity and its predecessors and successors, and the exact sequence in which activities must be performed for proper completion of the project. The major difference between the two techniques is in the way in which the duration of activities is specified. In PERT, a “most likely” time, a minimum time, and a maximum time of duration are specified for each activity, while CPM requires only a single estimate of the expected duration.

5-3.06 REHABILITATION AND PREVENTATIVE MAINTENANCE

All pavements eventually require additional scheduled work for them to maintain their functionality. There are, in general terms, three types of work that may be considered for aging roadways. The first is to wait until the pavement is damaged beyond repair and schedule its complete reconstruction. This approach has the advantage of completely removing all existing problem areas within the pavement and starting from scratch; however, it is much more expensive than other options and is usually not economically sustainable from a systematic point of view. The second option is to provide pavements with preventative maintenance when they are still in relatively good condition. Appropriately scheduled preventative maintenance has the capability to significantly extend pavement lives at a relatively low cost, and Mn/DOT is moving towards scheduling more of these projects. Thirdly, pavements may be provided with rehabilitation work if they have begun to display distresses but are still capable of supporting traffic. This section of this manual deals with the factors that come into play when deciding if rehabilitation is appropriate, as well as which rehabilitation methods to use, for both bituminous and concrete pavements.

5-3.06.01 DISTRESSED PAVEMENTS

There are a variety of pavement conditions that may serve to trigger concrete or bituminous pavement rehabilitation. The most common of these is poor ride quality; however, deteriorating surface friction, structural weakening, unprotected underlying materials, and other distresses may come to influence the decision-making process as well. Concrete pavement rehabilitation may include surface texture planing, joint resealing, the installation of subsurface drains, partial-depth repairs, full-depth repairs, and several varieties of overlays. Bituminous pavement rehabilitation may include crack sealing, pothole patching, surface texture improvements, the installation of subsurface drains, cold in-place recycling, full-depth reclamation, and overlays.

The choice between the rehabilitation and reconstruction of a distressed pavement is dependent on the extent to which the current serviceability of the road has been degraded, the cost of the proposed solutions, and the need to upgrade the geometric and safety characteristics of the roadway. In most cases, rehabilitation is the most desirable solution as it usually brings about an acceptable improvement in performance for a reasonable cost. Reconstruction becomes the required alternative either when a pavement has deteriorated to the point where its replacement is the only cost-effective method for improving performance or there is a need to enhance the geometric or safety characteristics of a pavement.

On the systematic level, each District should have a pavement rehabilitation plan in place that outlines a preventative maintenance schedule that will forestall rehabilitation projects for as long as possible and provides criteria for determining which routes are the most in need of rehabilitation or reconstruction. In the long run, it has been proven that regularly scheduled preventative maintenance projects greatly decrease the total amount of money needed to keep a District's roadways performing at an acceptable level.

5-3.06.02 BITUMINOUS REHABILITATION AND MAINTENANCE

The rehabilitation and maintenance of bituminous pavements can be divided into four general categories: surface treatments, crack seals, overlays, and recycling/reclamation. The methodology for deciding which of these options is appropriate for a particular situation will be discussed in section 5-3.06.04. In general terms, however, surface treatments are used to provide a better seal for the surface, to enhance the frictional features of the surface, to slow pavement deterioration, or to address other distresses related to the surface of the pavement. These treatments are often viewed as preventative maintenance rather than pure rehabilitation. Overlays are used in situations where surface treatments would not be adequate to address the observed surface distresses or in situations where the structural capacity of the pavement needs to be increased.

Recycling and reclamation activities should be used in situations where the entire bituminous layer is compromised (such as severe cracking) but the underlying support is deemed to be adequate.

1. **Surface Treatments.** Surface treatments are a common rehabilitation and preventative maintenance measure for flexible pavements. This category of treatments includes a large number and variety of products; however, chip seals tend to be the most common. Surface treatments are used to waterproof the pavement surface, improve friction and drainage characteristics, slow weathering, seal cracks, and provide a smoother wearing surface. On the whole, these treatments are best suited for use as preventative maintenance activities: few have the capability to repair significant distresses (although they may cover them well enough to increase the ride quality in the short term). These treatments are commonly used throughout the state.

Skin patching, which is the practice of patching flexible pavements with bituminous mixtures, is sometimes considered to be a surface treatment and has been used as a successful rehabilitation measure for the following cases: the repair of potholes, the replacement of fatigue-cracked asphaltic concrete surfaces, and spot leveling to prepare a surface for overlay or to improve ride quality. Cold mixes, which are made up of a combination of aggregates and either cutbacks or asphalt emulsion binders, may occasionally be used for such patching in the winter months when hot mix is not readily available. However, hot plant-mixed asphaltic concrete is inherently more suitable for patching of flexible pavements and should be used wherever possible. In addition, cold mixes are not suitable for pavements that are expected to be overlaid in the near future.

2. **Crack Seals.** Crack sealing is a common rehabilitation and preventative maintenance measure that is used to prevent water from seeping into a pavement structure through cracks in its surface. All cracks in bituminous pavements should be sealed as soon as is practical to prevent deterioration around their edges. Therefore, the present target is to seal all cracks within the first four years of a pavement's life.

Crack sealing is a multi-step process that includes the routing or sawing of a reservoir with the proper shape factor to hold the sealant. This reservoir is cleaned with compressed air and/or heated compressed air (heat lance) and then filled with a hot-pour polymer-modified asphalt. (Mn/DOT 3723 or 3725). Reservoir shapes and overband configurations vary depending upon the specific project. Present practice is to use a 1/2 to 3/4 inch deep by 1-1/4 inch wide reservoir with a 1/16-inch thick overband extending 1 to 1-1/4 inches on either side of the reservoir.

Crack filling is a process that should be considered when a pavement has deteriorated to the extent that sealing operations will be unable to make a functional seal. In this process, cracks should be blown clean with compressed air and filled with an asphalt-crumb rubber blend (Mn/DOT 3719) or a hot-pour polymer-modified asphalt (Mn/DOT 3723 or 3725). The practice of crack filling should be looked upon as a stopgap procedure that merely slows the pavement degradation while a more permanent fix is under consideration. This process will not prevent the ingress of water through the cracks; however, it does retard the oxidation of the crack edges by coating them with asphalt.

3. **Overlays.** Overlays are a common preventative maintenance and rehabilitation technique that may improve the ride quality, frictional characteristics, or structural capacity of a roadway at a reasonable cost. Many overlays are considered to be preventative maintenance in that they are not intended to increase the structural capacity of the pavement. In these situations the reasons the overlay is required should be clearly defined and the causes of the problems or distresses warranting the overlay should be thoroughly investigated. These causes must be eliminated before placement of the surfacing in order to prevent or diminish the recurrence of the distresses. Where the pavement ride is poor because of multiple cracking due to age, temperature, or mix deficiencies, consideration should be given to milling, removal, and the

recycling or replacement of the existing bituminous surfacing. It should be noted, however, that these functional overlays are temporary fixes; existing cracks will eventually work their way through the overlay.

In other situations, overlays are rehabilitation techniques designed to increase the structural capacity of the roadway. In these cases, consideration of the past performance of the roadway, its age, the level of traffic, the results of pavement condition surveys and non-destructive load testing, and the known properties/characteristics of the underlying soils and pavement materials is essential in the determination of effective rehabilitation measures. Borings, cores, and even test pits may be necessary to determine the extent of deterioration of the layers of the pavement. Two types of overlays will be considered here: bituminous over bituminous, and bituminous over concrete.

- a. Bituminous Over Bituminous (BOB). The thickness of a bituminous overlay over an existing bituminous pavement is determined from a measure of the current structural capacity of the pavement. This capacity is determined either using field deflection measurements or by Mn/DOT's bituminous design chart.

In the case where field deflection data is used for a road of less than 9 or 10-ton capacity, the allowable deflection for the desired tonnage is determined as required by the results of the Investigation 603 study (as modified by Technical Memorandum No. 82-26-R-1, "Determination of Allowable Spring Load Limits," July 15, 1982). Where the road has a capacity greater than 9- or 10-tons, the following modified AASHO Road Test equation (Equation 5-3.7) is used to determine the allowable deflection for the sum of past and projected 18-kip ESALs.

$$d_s = 10 \left[\frac{11.06 - \log \Sigma ESAL}{3.25} \right] \quad \text{Eq. 5-3.7}$$

where

$\Sigma ESAL$ = sum of past and projected 18-kip axle loads carried to a PSI of 2.5
 d_s = allowable spring deflection, in.

If the results indicate that the pavement is strong with the exception of areas near transverse cracks, consideration should be given to special crack rehabilitation. The required overlay thickness is determined using either the Investigation 603 relationship (in which one inch of overlay on plastic soils reduces deflection by about 11 percent) or the following Equation 5-3.8 from Mn/DOT's Investigation 183.

$$\log(d_s + 2s) = 2.728 - 0.0175(G.E.) - 0.525 \log R \quad \text{Eq. 5-3.8}$$

where

$(d_s + 2s)$ = the average peak spring Benkelman beam deflection plus two standard deviations at an 80° F mat temperature
 G.E. = total granular equivalency
 R = R-value

The overlay thickness is based on the difference between the G.E. derived from the Inv. No. 183 design chart (Figure 5-3.7) and the reduced G.E. for the existing pavement structure. This calculation should make use of the design R-value and the projected ESALs. The reduced G.E. is normally taken to be between two-thirds and three-quarters of the G.E. computed for the existing pavement in order to add a degree of conservatism to the design. It is assumed that the G.E. factors of the in-place pavement structure are the

same as the G.E. factors of the original design unless available samples indicate otherwise.

- b. Bituminous Over Concrete (BOC). Bituminous over concrete (BOC) pavements are designed using the full-depth bituminous design chart (Figure 5-3.8), the 15 or 20 year projected traffic (depending on the desired design life), and the R-value of the underlying soils. The total thickness of pavement is determined by adding the effective concrete thickness of the existing pavement, which usually assumed to be two-thirds to three-quarters of the original thickness, to the proposed overlay thickness. Extremely deteriorated concrete pavements should assume lower effective thickness fractions. In either case, it is the policy of Mn/DOT to provide a minimum bituminous overlay of 4 to 4½ inches on D-cracked concrete pavements and 3 to 3½ inches on all other concrete pavements.

Mn/DOT does not construct a large number of BOC overlays because of one common problem: reflection cracking. Possible techniques for minimizing the severity of reflection cracking in the overlays are as follows:

- Sawing and sealing joints in the bituminous overlay at the locations of joints and cracks in the underlying concrete slab. However, if the existing joints are badly deteriorated, the panels are badly cracked, or the cracks are actively working it may be more effective to rout and seal the overlay after the cracks develop.
- Cracking and seating the concrete slab before placement of the bituminous overlay. This technique is used to reduce the size of the concrete slab pieces to minimize the differential movements at existing cracks and joints, thereby minimizing the occurrence and severity of reflection cracks.
- Increasing the bituminous thickness. Reflection cracks will take more time to migrate through a thicker overlay than an overlay of standard thickness.

Rubblizing the concrete slab, (2231, Pavement Breaking), before placement of the bituminous overlay can eliminate the reflective cracking. This technique uses a machine that breaks the concrete in place, and results in the complete destruction of the slab section and reduces the size of the concrete pieces to a maximum of 12 inches. This process results in a layer of material similar in appearance to unbound base layers; however, its strength is several times that of a standard aggregate base.

If the existing concrete pavement is in relatively good structural condition the saw and seal technique should be given first consideration.

4. Reclamation/Recycling. Full-depth recycling (FDR) and cold in-place recycling (CIR) are rehabilitation techniques used in situations where the cracking or distresses observed in a pavement are too widespread or severe to be solved with an overlay, but the underlying base layers are still relatively sound. The FDR process involves the pulverization of the entire bituminous layer. This material is mixed with a small amount of the in-place base material, then placed back on the roadway to serve as an improved base layer for a new roadway surface. This technique eliminates all distresses that originated in the bituminous layer and is a good solution for roads that can tolerate a grade raise.

The CIR process involves the reclamation of the upper 2-4 inches of the bituminous surface such that only 1 inch of the asphalt is left in place. The recycled material is mixed with new asphalt, laid back down, and it provides a uniform base that may be overlaid with new material. This technique mitigates reflective cracking problems associated with a straight overlay, and it is a good rehabilitation technique for low volume roadways.

Mn/DOT's Office of Pavement Design should be contacted for design information relating to full-depth reclamation or cold in-place recycling.

5-3.06.03 CONCRETE REHABILITATION AND MAINTENANCE

The rehabilitation and maintenance of concrete pavements can be divided into three general categories: milling, concrete pavement rehabilitation (CPR), and overlays. The methodology for deciding which of these options is appropriate for a particular situation will be discussed in section 5-3.06.04.

1. Milling. Milling is a preventative maintenance measure commonly used to increase the ride quality index and frictional characteristics of a concrete roadway or to prepare it for an overlay. This procedure, which is also known as diamond grinding, makes use of a milling machine to evenly chip off the surface of the pavement. The resulting surface is level and lightly grooved, which makes it a good solution for faulted and/or excessively smooth pavements. The grooved nature of the finished surface is also ideal for providing a good bond to future wearing courses.
2. Concrete Pavement Rehabilitation (CPR). Concrete pavement rehabilitation (CPR) is a catch-all phrase designed to capture several maintenance and rehabilitation activities. Mn/DOT generally divides this categorization into 'Minor' CPR, which is primarily a preventative maintenance activity, and 'Major' CPR, which is a rehabilitation activity including full-depth repairs.
 - a. Minor CPR. Minor CPR is a term that may encompass a number of different activities. In most cases, it includes Type B crack repair on approximately 5% of the transverse joints and joint resealing on all joints. However, it may also include a small number of full-depth (Type C) crack repairs (no more than 5% of transverse joints) and a surface milling depending on the condition of the pavement.

Type B repairs are partial-depth repairs that include the removal and replacement of spalled concrete with high-strength concrete. The removal of the spalled areas is accomplished by either sawing the concrete to aid the jackhammer operations or by milling. This type of repair is used at spalled joints, spalled mid-panel cracks, delaminated mid-panel spalls, and spalled pavement edges.

The sawing method for the removal of spalled concrete, while approved, is not commonly used and seems to have less success than the milling method. This method involves using a saw to cut alongside the spalled area to a minimum depth of two inches. The concrete in the affected area is then removed using an air hammer of a capacity not to exceed 14kg (30 pounds) and the saw cut edges are knocked off at approximately 45 degrees to maximize the bonding surface.

Most contractors in Minnesota make use of a milling machine to remove spalled concrete, especially for full-width partial-depth repairs at joints. This procedure makes use of a roto-mill that creates a semi-circular cross-section at least two inches in depth which bonds well to the future concrete patch. In most cases, it is cheaper to mill the entire joint than just the spalled area if more than 40% of the joint is spalled.

Regardless of the removal technique used, all spalled concrete should be removed to a minimum depth of two inches and a maximum depth of one-half the pavement thickness or the top of the dowel bars, whichever is less. If only the tops of the dowel bars are exposed, they should be coated with MC-250 oil or an approved equal. If the outside edges of the dowel bars are exposed, however, a spacer must be placed to ensure joint movement in the longitudinal direction. Heavily corroded (or "necked-down") dowel

bars should be burned off. If more than three dowels are removed in one joint, a full-depth repair should be performed to restore the joint's load transfer capabilities.

Undercutting refers to the deterioration of the concrete beneath the dowel bars. If this situation is encountered, a trained inspector should determine whether the severity is high enough to warrant full depth repairs or whether the joint will perform adequately as is. If the joints are left in place, it is better to leave the debris below the dowels as it will allow for some movement of the joint.

The working surface of the patch is cleaned by sand and air blasting after the removal of the spalled area and the dowel inspection. A bonding grout is applied to the surface prior to the placement of the high strength concrete; the grout should have a creamy consistency and be brushed over the entire patch area. If the grout sets and whitens before the concrete has been placed, the area will need to be sand and air blasted a second time before the grout is re-applied.

Mn/DOT uses a high-strength concrete (grade 3U18) to fill the patch. This mix is placed in the repair area, vibrated into place, and finished to the proper grade and texture. In all partial-depth repairs the final finishing or troweling operation should be directed toward the in-place concrete surface to prevent the patch from pulling away.

When a Type B repair is placed over a joint, the joint should be re-established using an angle iron cutter bar at the original failure plane and later saw cutting to the required dimensions. When a Type B repair is placed over a transverse crack, it is imperative that the original crack be re-established in the exact location with the use of a flexible insert at the time of concrete placement.

The membrane cure should be applied to the finished surface as soon as possible due to the high cement content of grade 3U18 concrete and the minimum depth of the repairs. There is a high likelihood of shrinkage if the cure is not placed on time.

Minor CPR also includes joint and crack resealing. This common procedure involves saw cutting the joint or crack, sand and air blasting, and the re-application of the silicone seal.

b. Major CPR. Major CPR

Major CPR encompasses many of the same activities as minor CPR. The difference is that major CPR also includes a significant percentage (about 20% of transverse joints) of Type C full-depth panel repairs. Type C repairs are localized repairs where the removal of the deteriorated concrete at a joint, crack, mid-panel spot area, or entire panel is necessary. These problem areas are removed in entirety and replaced with a new concrete patch. Common causes for Type C repairs include subsoil problems, excessive thermal expansion, construction errors, and/or the natural aging of old concrete.

The first step in the Type C repair process is to remove the existing problem areas after sawing through the entire panel around them. This area should be no smaller than 3 ft by 3 ft and the subgrade should be left undisturbed to the greatest extent possible during the removal process.

The second step is to prepare the reinforcement for the future concrete patch. To do this, holes are drilled in the exposed sides of the adjacent panels that are large enough to accommodate No. 8 tie steel on 12-inch centers. The tie steel should be 18 inches long and extend 9 inches into the in-place pavement at a 20-degree skew (normal to the joint) to prevent the new concrete from separating from the adjacent slab. This steel is set into

the holes using a non-shrink grout that covers the entire bar and cavity to ensure a good bond. Failures of the tied joint often develop if the grout bond is inadequate.

A full-depth repair at a joint involves the removal and replacement of the old concrete and dowel bar assembly. One side of the resulting repair patches is tied into the adjacent panel while the other side is doweled. The tied end should be on the downstream side of the patch, and it is constructed using No. 8 rebar as described above. A straight, epoxy-coated dowel bar is installed at 12-inch centers parallel to the centerline joint on the upstream side of the patch that is 1 inch in diameter and 18 inches long. It is imperative that the dowel bars are not misaligned by more than one-eighth of an inch or the joint will work poorly and possibly spall.

Lastly, it is important to note that a four-inch wide transverse relief cut must be made in adjoining lanes prior to the pavement removal if traffic is to be maintained on them during the Type C repair. These cuts help to relieve undue stress in the pavement and can prevent concrete blow-ups. This phenomenon can occur when incompressibles enter the joints while the pavement is under compression.

3. Overlays. Concrete overlays can be placed in a number of different configurations depending on local conditions and the desired outcomes. In each of these situations, however, the overlay is designed to be structurally equivalent to a new or reconstructed pavement with a standard 35-year life. There are currently three concrete overlay configurations:
 - a. Bonded Overlays. Fully bonded overlays are designed to ensure a complete bond with the existing concrete pavement surface. Therefore, this overlay is the structural equivalent of increasing the existing slab thickness.
 - b. Unbonded Overlays. Unbonded overlays are constructed by placing a separation layer (asphalt or permeable aggregate base) over the existing pavement, then paving as normal. Unbonded overlays are useful in situations where there are concerns about existing distresses propagating into the overlay.
 - c. Concrete over Bituminous (COB or “Whitetopping”). Whitetopping overlays are those in which a concrete overlay is placed directly over an existing asphalt pavement.

Unbonded overlay design is discussed in Appendix D of this manual. The whitetopping and bonded overlay design procedures, however, are currently under modification by the Pavement Design unit. Therefore, it is recommended that designers contact the Pavement unit before attempting either of these designs.

5-3.06.04

REHABILITATION AND MAINTENANCE SELECTION

It is of great importance that the appropriate rehabilitation or maintenance activity be applied to roadways at the proper time in order to maximize the service life of the road for the minimum amount of money. Unfortunately, it is very difficult to determine the exact time at which a particular treatment should be applied to a roadway due to the large number of factors involved such as serviceability rating, safety ratings, the condition of other roads in a District, project cost, material availability, political factors, and more. Therefore, it is recommended that each District use their own criteria to establish a preventative maintenance and rehabilitation schedule for their roads that rates potential activities on a cost per lane-mile-year basis.

To assist with this process, the Pavement Management unit created maintenance and rehabilitation decision trees that provide an indication as towards which activities should be performed at which times. These trees are included as Figures 5-3.12 and 5-3.13.

An economic analysis should be performed for all projects, and the procedure and results should be documented in the Materials Design Recommendations Report.

5-3.07 WIDENINGS

Mn/DOT has a number of guidelines for widening existing concrete pavements, which is often required before overlaying the pavement.

Full-depth bituminous widenings are normally designed to provide a 14 ft base on either side of centerline (e.g. 20 to 22 ft wide pavements widened 8 to 6 ft, respectively). The thickness of the widening is determined from Figure 5-3.8 using the 15 to 20-year cumulative traffic and increased by 20 percent to compensate for edge effects.

Aggregate base widenings should be the same width as indicated for the full-depth bituminous widenings. The thickness is determined from Figure 5-3.7 using the 15 to 20-year cumulative traffic and again increased by 20 percent to compensate for edge effects. However, this design is not recommended if the potential for a severe frost differential exists. Such a potential may exist if the resulting slab thickness is appreciably thicker than the in-place slab.

Concrete widenings of existing concrete pavements should provide a lane width of at least 13.5 ft and have the same thickness as the outside edge of the existing pavement. Concrete widenings are tied to the in-place slab.

It is essential to promote lateral drainage of widened pavements when traffic levels rise in excess of one-million ESALs. This is accomplished using improved ditches, subsurface drains, and/or the replacement of the in-place shoulder with clean material.

It is recommended that a drainage layer of permeable asphalt-stabilized or open-graded base be placed beneath bituminous and concrete widenings. This system should include a geotextile fabric to provide separation between the base and subgrade soil. This layer is designed to drain the infiltrated surface water to a pavement edge drain system: this concept is illustrated in Figure 5-4.2C.

5-3.08 SHOULDERS

Shoulders are provided for the storage of stopped vehicles, bus and emergency vehicle use, a recovery zone between lanes and the ditch/roadside appurtenances, flexibility during future roadway projects, and the lateral support of base and surface courses. It can be shown that the stresses, strains, and deflections at the edge of a pavement without a shoulder far exceed those of a similar pavement with edge support: this is especially true for concrete pavements. Shoulders, therefore, increase pavement life and are recommended in most situations.

The four types of shoulders used in Minnesota are aggregate, bituminous, concrete, and composite shoulders. A composite shoulder is an aggregate shoulder that has been partially paved with bituminous material near the traveled lanes. Shoulder design is influenced by the following factors: mainline pavement type, traffic, environment, safety, planned maintenance, and thickness considerations.

5-3.08.01 GEOMETRIC CONSIDERATIONS

A shoulder's width should be enough to allow a stopped vehicle to clear the driving lane by at least two feet according to Department standards. The exact width and surfacing type is dependent upon the roadway's traffic volume (ADT), design speed, cross-section (rural or urban), functional class, and the scope of work. Section 4-4.01 of Mn/DOT's Road Design Manual provides general information and design requirements.

Aggregate shoulders' width is measured from the mainline/shoulder interface to its point of intersection with the cut or fill slope (P.I.). Bituminous and concrete shoulders are widened an additional 1½ ft to ensure stability and provide support to the edge of the shoulder. The total

width from the edge of the traveled lane to the shoulder P.I. is considered to be the usable shoulder width for rural designs (i.e. sections without curbs). The distance from the edge of the travel lane to the face of curb is the useable width for urban designs.

Two-lane roadways are normally constructed with the full shoulder width on both sides. Four-lane divided roadways are normally designed with a 10 ft paved shoulder on the right and a 4 ft paved shoulder on the left. Six to eight-lane divided roadways are designed with 10 ft paved shoulders on both sides. These shoulders should be continuous and without variation in width or elevation with respect to the mainline pavement. Chapters 4 and 7 of Mn/DOT's Road Design Manual should be referred to for specific design details.

Rumble strips should be placed on all rural highways projects that involve bituminous shoulder construction or reconstruction and where the speed limit is 50 mi/hr or greater. These strips are designed to reduce the number of run-off-the-road (ROR) accidents by grabbing a driver's attention as they drift from their lane. Section 4-4.02 of Mn/DOT's Road Design Manual includes more information regarding rumble strip design criteria and design details.

Public transit buses are permitted to travel on designated shoulders of congested roadways during peak periods. Section 4-4.03 of the Road Design Manual includes criteria to be considered for bus shoulder design.

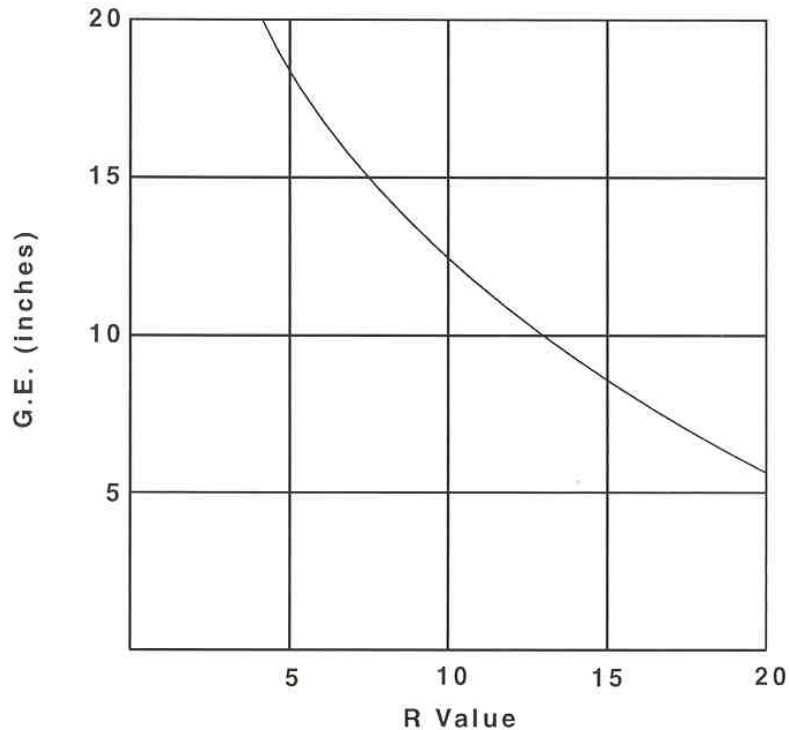
5-3.08.02 STRUCTURAL DESIGN

Standard shoulder designs for both rigid and flexible pavements are outlined in Chapter 7 of Mn/DOT's Road Design Manual. The bituminous shoulder G.E. thickness listed in this manual should be evaluated relative to the minimum G.E. thickness from Figure 5-3.10. Figure 5-3.10 makes use of the same design R-value as was used in the mainline design (mean minus one standard deviation). However, in situations where traffic volumes are low to moderate and there is a large amount of scatter in the measured R-values (they have a large standard deviation), a design R-value that is somewhat greater than the mainline design R-value may be used to avoid an overly conservative shoulder design.

The shoulder is lacking in structural capacity if the minimum G.E. requirement from Figure 5-3.10 is greater than the G.E. value obtained through the standard design process. If this occurs, the shoulder should be re-designed using the G.E. value determined Figure 5-3.10. This re-design commonly involves the use of larger G.E. materials (i.e. Class 5 instead of Class 3), the modification of the subgrade cross slope beneath the shoulder, and/or the use of bituminous base in place of some of the aggregate base. These alternatives allow the design of adequate shoulders without excessive cost.

Consideration should be given to making a shoulder's thickness equal to the driving lane thickness in locations where the shoulder is expected to be used as a driving lane during periods of congestion and/or during future construction projects.

Figure 5-3.10 Minimum shoulder design.



NOTE: Based on an 0.150 inch spring Benkelman Beam Deflection (average plus two standard deviations) for a 10-ton axle load with edge effect.

Aggregate shoulder design is important, however, it is not as critical as the designs for bituminous or concrete shoulders. Bituminous and concrete shoulders require extensive repairs upon failure, while the aggregate shoulder can be maintained relatively easily by the addition of more aggregate. Therefore, the R-value used to design an aggregate shoulder may be somewhat greater than the mainline design R-value. The District Soils and/or Materials Engineer should determine the R-value used in design.

5-3.09 RAMPS AND LOOPS

The pavement materials used in the construction of ramps and loops, acceleration lanes, deceleration lanes, escape lanes, auxiliary lanes, directional connections, and gore areas adjacent to the through lanes should be the same as those used for the mainline roadways in most cases.

Section 6-3.0 of Mn/DOT's Road Design Manual outlines the design process for urban and rural ramps and loops, as well as containing several typical cross sections.

5-3.09.01 STRUCTURAL DESIGN

1. Concrete Ramps and Loops

The following are several points to keep in mind when designing concrete ramps and loops:

- The slab thickness should be determined using Section 5-3.05.04 of this manual. The traffic input should be the number of concrete ESALs (CESALs) expected to be applied to the loop or ramp over the design period.
- The joint and dowel design should be based on the procedures described in Section 5-3.05.04. The transverse joints in the ramp pavement should coincide with the joints in the mainline pavement up to the gore point. The transverse joints should be placed perpendicular to centerline and spaced uniformly at 15 ft.
- Supplemental reinforcement should be placed in all panels with widths between 15 and 22 ft. However, it is preferable to construct a longitudinal joint rather than pave panels of this width in most situations. Figure 5-297.219 of Mn/DOT's Standard Plans Manual contains details regarding supplemental steel reinforcement design.
- Transverse Contraction joints for ramps and loops should be designed in accordance with Figure 5-297.221 of Mn/DOT's Standard Plans Manual.
- Concrete pavement sections of ramps with storage area should be designed in accordance with the detailing shown in Chapter 7 of Mn/DOT's Road Design Manual.
- There are a number of differences between urban and rural ramp design. The normal rural ramp width is 16 feet for the majority of its length. However, it is tapered outwards to two 12-foot lanes before the crossroad intersection. Urban loops are typically 18 feet wide. The center joint should be an LIT for rural design; an L2KT is used for urban design. Urban curbs should be D 424 and are carried through the tapered section.
- In tapered sections the joint can be terminated at the point where the taper width equals 10 feet or extended to a point 4 ft from the end of the taper as noted in Chapter 7 of Mn/DOT's Road Design Manual. This area shall contain supplement reinforcement.
- The concrete pavement on ramps and loops intersecting a bituminous crossroad or street should extend to the edge of the crossroad's through lane. Pavement widths on ramps and loops should be designed in 1 ft increments from back-to-back of integrant curbs. Pavement widths on auxiliary lanes with integrant curb may be designed in feet and inches; a 12-foot lane with a B6 Integrant Curb, for example, would have a total pavement width of 12ft 8in. Every effort should be made to provide for uniform ramp and loop widths to facilitate construction and minimize costs.

2. Bituminous Ramps and Loops

The pavement thickness should be determined using Section 5-3.05.03 of this manual. The traffic input should be the number of bituminous ESALs expected to be applied to the loop or ramp over the design period.

5-3.10

BITUMINOUS MIXTURE COMPACTION GUIDELINES

Compaction is the final stage in the construction of a bituminous pavement. The compaction effort plays an important role in determining future pavement performance due to its effect on the strength of the material and the texture of the mat. Therefore, it is important that the proper method of compaction is used for a given project to ensure the maximum pavement service life.

The specifications for bituminous mixture are subject to change, so the latest version of the standard specification should always be reviewed .

Mn/DOT allows construction engineers to choose between several different compaction control methods. These methods are described in detail in the Standard and Supplemental Specifications section of the Bituminous Manual. The following is a general description of the methods:

1. Maximum Density (2360.6B)

Maximum density allows the contractor some discretion in their rolling pattern. Some requirements are given in 2360.6. Mn/DOT will determine the density obtained by the contractor via cores cut into the pavement and tested in the lab. The bulk density of the core sample is compared to the maximum density of the material as determined by lab testing, and reported as a percentage of that maximum density. The contractor is then paid an incentive or disincentive based on the percentage density achieved.

The specification requires the bituminous pavement compaction is always measured according to the maximum density specification, unless the contract states that ordinary compaction is to be used.

2. Ordinary Compaction (2360.6C)

Ordinary compaction may be used for thin lifts (less than 1 ½ inches) and small or irregular shaped areas. A control strip is established for the contractor to demonstrate a pattern the compaction equipment is to follow. This referred to as the rolling pattern. As the compaction proceeds, the contractor uses his/her own nuclear density testing device to document the increase in density of the rolled bituminous. When the density no longer increases with additional passes, or when the density decrease by more than 2%, the rolling is stopped. The contractor shall then follow the rolling pattern that was used, on the rest of the bituminous that is to be compacted by ordinary compaction. New control strips are required when the JMF is changed, material source change, 10 days production have been accepted, or other reasons the original control strip may not be considered representative any longer.