CHAPTER II

LITERATURE REVIEW

2.1 Introduction

Available literatures include books, journal, papers pertaining to the design and calculation of flexible and rigid pavement will be aviewed and as a part of this study. Literature review was conducted specifically evaluate the issues relating to the objective of the study. Those issues were : a) Designing of flexible pavement and rigid pavement, b) Comparison of flexible pavement and rigid pavement. First of all will be described pavement design and follow with AASHTO 1993 method.

2.2 Pavement Design

As have been mentioned in Chapter 1, design method of both flexible and rigid pavement will use AASHTO 1993 design method. To conduct pavement design, AASHTO 1993 need or design requirements or inputs include, Design Variables, Performance Criteria, Material Properties for Structural Design, and Pavement Structural Characteristics as well as Reinforcement Variables for rigid pavement. Those five design requirements will be describe sequencely in the following subsection.

2.3 Design Requirements

Design requirements for the different initial pavement types that can be considered. This chapter discusses the preparation or selection of the inputs required pavement design.[2]

2.3.1 Design Variables

Design variables consist of Time Constrains, Traffic, Reliability, and Environment Impact.

2.3.1.1 Time Constrains

According to AASHTO 1993 design method this section involves the selection of performance and analysis period inputs which affect (or constrain) pavement design from the dimension of time. Consideration of these constraints required for both highway and low-volume road design. Time constraints permit the designer to select from strategies ranging from the initial structure lasting the entire analysis period (performance period equals the analysis period) to stage construction with an initial structure and planned overlays. [2]

Performance Period. This refers to the period of time that an initial pavement structure will last before it needs rehabilitation. It also refers to the performance time between rehabilitation operations. In the design procedures presented in this Guide, the performance period is equivalent to the time elapsed as a new reconstructed, or rehabilitated structure deteriorates from its initial serviceability to its terminal service ability.

For the performance period, the designer must select minimum and maximum bounds that are established by agency experience and policy. It is important to note that, in actual practice, the performance period can be significantly affected by the type and level of maintenance applied. The predicted performance inherent in this procedure is based on the maintenance practices at the AASHTO Road Test.

The minimum performance period is the shortest amount of time a given stage should last. For example, it may be desirable that the initial pavement structure last at least 10 years before some major rehabilitation operation is performed. The limit may be controlled by such factors as the public's perception of how long a "new" surface should last, the funds available for initial construction, life-cycle cost, and other engineering considerations.

The maximum performance periods the maximum practical amount of time that the user can expect from a given stage. For example, experience has shown in areas that pavements originally designed to last 20 years required some type of rehabilitation or facing within 15 years after initial construction. This limiting time period may be the result of PSI loss due to environmental factors, disintegration of surface, etc.

The selection of longer time periods than can be achieved in the field will result in unrealistic designs. Thus, if life-cycle costs are to be considered accurately, mportant to give some consideration to the t is maximum practical performance period of a given pavement type. [2]

Analysis Period. This refers to the period of time for which the analysis is to be conducted, i.e., the length of time that any design strategy must cover. The analysis period is analogous to the term design life used by designers in the past.

Because of the con- sideration of the maximum performance period, it may be necessary to consider and plan for stage construction an initial pavement structure followed by one or more rehabilitation operations to achieve the desired analysis period.

In the past, pavements were typically designed and analyzed for a 20 year performance period, since the original Interstate Highway 1956 required that traffic be considered through 1976.

It is now recommended that consideration be given to longer analysis periods, since these may be better suited for the evaluation of alternative long-term strategies based on life-cycle costs. [2]

Consideration should be given to extending the analysis period to include one rehabilitation. For high-volume urban freeways, longer analysis periods may be considered. Following are general guidelines :

Table 2.1 Traffic Volume

Highway conditions	Analysis period (years)
High volume urban	30 – 50
High volume rural	20 – 50
Low volume paved	15 – 25
Low volume	10 – 20

2.3.1.2 Traffic

The design procedures for both highways and low volume roads are all based on cumulative expected 8.16 ton equivalent single axle loads (ESAL) during the analysis period. The procedure for converting mixed traffic into these 8.16 ton ESAL units is presented of AASHTO 1993 Guide.

For any design situation in which the initial pavement structure is expected to last, the analysis period without any rehabilitation or resurfacing, all that is required is the total traffic over the analysis period. If, however, stage construction is considered. rehabilitation or resurfacing is anticipated (due to lack of initial funds, roadbed swelling, frost heave, etc.)

Then the user must prepare a graph of cumulative 8,16 ton ESAL traffic versus time, as illustrated in Figure 2.1 This will be used to separate the cumulative traffic into the periods (stages) during which it is encountered.

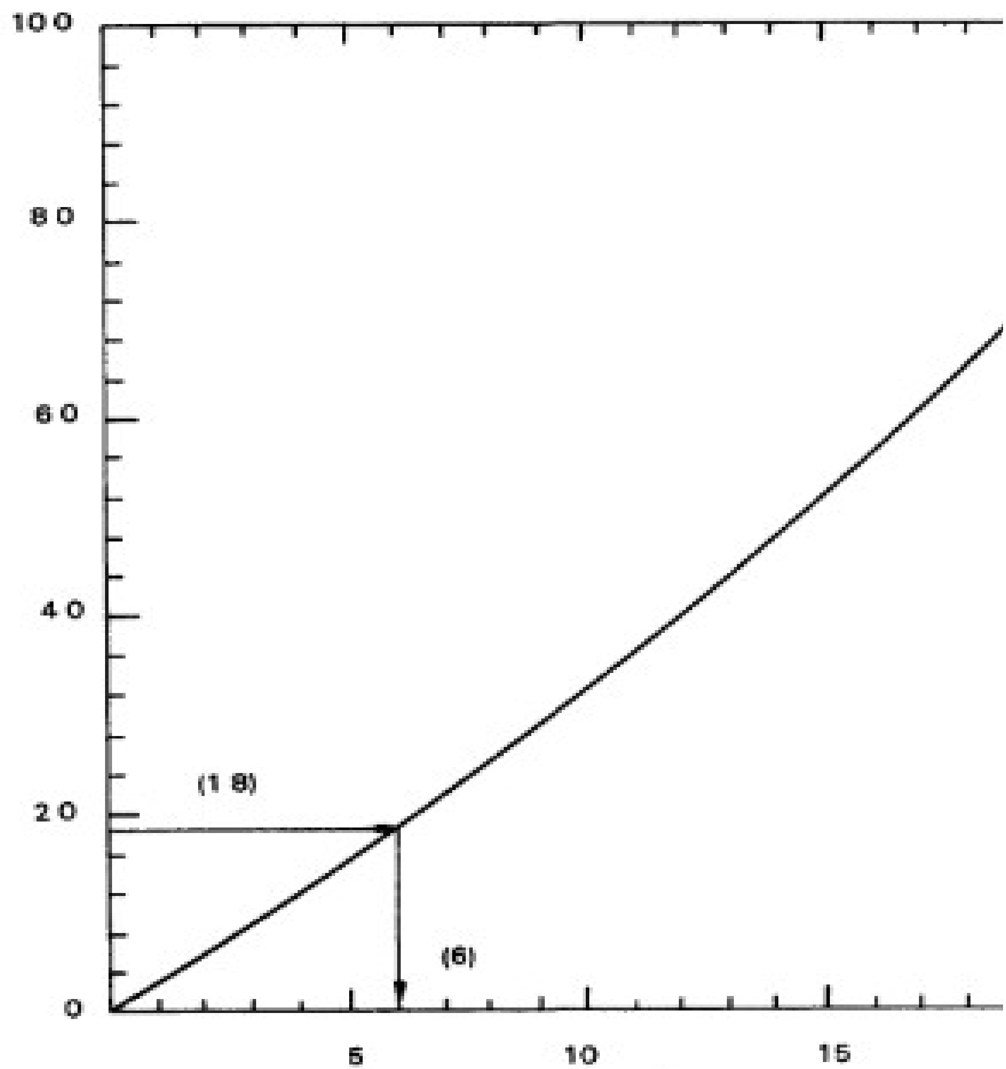

Figure 2.1 Example plot of cumulative 8,16 ton ESAL traffic versus time.

The predicted traffic furnished by the planning group is generally the cumulative 8,16 ton ESAL axle applications expected on the highway, whereas the designer requires the axle applications in the design lane. Thus, unless specifically furnished, the designer must factor the design traffic by direction and then by anes if more than two.

The following equation may be used to determine the traffic design lane:

$W_{8.16t} = D_D \times D_L \times w_{8.16t}$ . . . . .e.q. 2.1.

where

D_D = a directional distribution factor, ex- pressed as a ratio, that accounts for the distribution of ESALunits by direction, e.g., east-west, north-south, etc.,

D_L = a lane distribution factor, expressed as a ratio, that accounts for distribution of traffic when two or more lanes are available in one direction.

$w_{8.16t}$ = the cumulative two-directional 8.16 ton ESAL units predicted for a specific section of highway during the analysis period (from the planning group).

Although the D_D factor is generally 0.5 (50 percent) for most roadways, there are instances where more weight may be moving in one direction than the other. Thus, the side with heavier vehicles should be designed for a greater number of ESAL units. Experience has shown that D_D may vary from 0.3 to 0.7 depending on which direction is "loaded" and which is "unloaded". [2]

2.3.1.3 Reliability

Basically, it is a means of incorporating some degree of certainty into the design process to ensure that the various design alternatives will last the analysis period. The reliability design factor accounts for variations in both traffic prediction and the performance prediction, and therefore provides a predetermined level of assurance (R) that pavement sections will survive the period for which they were designed. Generally, as the volume of traffic, difficulty of diverting traffic, and public increases, the risk of not performing to expectations must be minimized. This is accomplished by selecting higher levels of reliability.

Note that the higher levels correspond to the facilities which receive the most use, while the lowest level 50 percent, corresponds to local roads. Design performance reliability is controlled through the use of a reliability factor (F_R) that is multiplied times the to design period traffic prediction $w_{8.16t}$ for design applications $W_{8.16t}$ for the design equation. is a a given reliability level (R), the reliability factor function of the overall standard deviation S that accounts for both chance variation in the traffic prediction and normal variation in pavement performance prediction for a given $W_{8.16t}$.

It is important to note that by treating design uncertainty as a separate factor, the designer should no longer use "conservative" estimates for all the other design input requirements. Rather than values, the designer should use his best estimate of the mean or average value for each input value. The selected level of reliability and overall standard devia- tion will account for the combined effect of the variation of all the design variables.

Application of the reliability concept requires the following steps:

(1) Define the functional classification of the facility and determine whether a rural or urban condition exists.

(2) Level the range in Table 2.2. The greater the value of reliability, the more pavement structure required.

(3) A standard deviation (So) should be selectedthat is representative of local condition.

Table 2.2 Suggested levels of reliability for various functional classifications.

Function Classification	Recommended Level of Reliability	
	Urban	Rural

Interstate and other freeways	85 - 99.9	80 - 99.9
Principal Arterials	85 – 99	75 - 95
Collectors	80 – 95	75 - 95
Local	50 -80	50 - 80

Values of SO developed at the AASHTO road test did not include traffic error. However, the performance prediction error developed at the road test 0.25 for rigid and 0.35 for flexible pavement. This corresponds to a total standard deviation for traffic of 0.35 and 0.45 for rigid and flexible pavement respectively.

2.4 Environmental Effects

The environment can affect pavement performance in several ways. Temperature and moisture changes can have an effect on the strength, durability, and load-carrying capacity of the pavement and roadbed materials. Another major environmental impact is the direct effect roadbed swelling, pavement blowups, frost heave, disintegration, etc., can have on loss of riding quality and serviceability. Additional effects, such as aging, drying, and overall material deterioration due to weathering, are considered in this Guide only in terms of their inherent influence on the pavement performance prediction models. The actual treatment of the effects of seasonal temperature and moisture changes on material properties is discussed in Section 2.3, "Material Properties for Structural Design. "This section only the criteria necessary for quantifying the input requirements for evaluating roadbed swelling and frost heave. If either of these can lead to a significant loss in serviceability or ride quality during the analysis period, then it (they) should be considered in the design analysis for all pavement structural types, except perhaps aggregate-surfaced roads. As service. ability-based models are developed for such factors as pavement blowups, then they may be added to the design procedure. [2]

2.4.1 Performance Criteria

This represents the userspecified set of boundary conditions within which a given pavement design alternative should perform serviceability.

2.4.1.1 Serviceability

The serviceability of a pavement is defined as its ability to serve the type of traffic (automobiles and trucks) which use the facility. The primary measure of serviceability is the Present Serviceability Index (PSI, which ranges from 0 (impossible road) to 5 (perfect road). The basic design philosophy of this Guide is the serviceability-performance concept, which provides a means of designing a pavement based on a specific total traffic volume and a minimum level of service-ability desired at the end of the performance period. Selection of the lowest allowable PSI or terminal serviceability index (p.) is based on the lowest index that will be tolerated before rehabilitation, řesur- facing, or reconstruction becomes necessary. An index of 2,5 or higher is suggested for design of major highways and 2.0 for highways with lesser traffic volumes. One criterion for identifying a minimum level of serviceability may be established on the basis of public acceptance. Following are general guidelines for minimum levels of p, obtained from studies in connection with the AASHTO Road Test [3]:

Table 2.3 Serviceability Index

Terminal Serviceability Level	Percentage of People Stating Unacceptable
3	12
2.5	15
2	85

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

Since the time at which a given pavement structure reaches its terminal serviceability depends on traffic volume and the original or initial serviceability some consideration must also be given to the selection of $p_{o.}$ Its should be recognized that the p_o values observed at the AASHTO Road Test were 4.2 for flexible pavements and 4.5 for rigid pavements.Once p_0 and p_t are estabilished, the following equation should be applied to define the total change in serviceability index :

Δ PSI = $p_0 - p_t$ e.q.2.2

The equation is applicable to flexible, rigid, and aggregate surfaced roads.

2.4.1.2 Allowable Rutting

In this design guide, rutting is considered only as a performance criterion for aggregate surfaced roads. Although rutting is a problem with asphalt concrete surface pavements, no design model suitable for incorporation into this Guide is available at this time It is important to note that the rut depth failure predicted by the aggregate-surfaced road model does not refer to simple surface rutting (which can be corrected by normal blading operations), but to serious rutting associated with deformation of the pavement structure and roadbed support. The allowable rut depth for an aggregate-surfaced road is dependent on the average daily traffic. Typically, allowable rut depths range from 1.0 to 2.0 inches for aggregate surfaced roads. [2]

2.4.1.3 Aggregate Loss

For aggregate surfaced roads, an additional is the aggregate loss due to traffic and erosion. aggregate loss occurs, the pavement structure becomes thinner and the load carrying capacity is reduced reduction of the pavement structure thickness in the rate of surface deterioration. To treat aggregate loss in the procedure, it is necessary to estimate (1) the total thickness of a that will be lost during the design period, and (2) the minimum thickness of aggregate that is required to keep a maintainable working surface for the pavement structure.

2.4.2 Material Properties For Structural Design

As discussed previously in this Part and Part I, the basis for materials characterization in this Guide is elastic or resilient modulus. For roadbed materials laboratory resilient modulus tests AASHTO should be performed on representative samples in stress and moisture conditions simulating those of the primary moisture seasons. Alternatively, the seasonal resilient modulus values may be determined by correlations with soil properties, clay content, moisture, PI, etc. The purpose of identifying seasonal moduli is to quantify the relative damage a pavement is subjected to during each season of the year and treat it as part of the overall design. An effective roadbed soil resilient modulus is then established which is equivalent to the combined effect of all the seasonal modulus values. The seasonal moisture conditions for which the roadbed soil samples should be tested are those which result in significantly different resilient moduli. For example, in a climate which is not subjected to extended subfreezing temperatures, it would be important to test for differences between the wet (rainy) and dry seasons. It would probably not be necessary, however, to test for the difference between spring-wet and fall-wet, unless there is significant difference in the average rainfall during spring and fall. If operations make it difficult to test the roadbed soil for spring-thaw or winter-frozen conditions, then, for these extreme cases, practical values of resilient modulus of 20,000 to 50,000 psi may be used for frozen conditions, and for spring-thaw conditions, the retained modulus may be 20 to 30 percent of the normal modulus during the summer and fall periods. Two different procedures for determining the sea- sonal variation of the modulus are offered as guidelines. One method is to obtain a laboratory relationship between resilient modulus and moisture content. Then, with an estimate of the in situ moisture content of the soil beneath the pavement, the resilient modulus for each of the seasons may be estimated. An alternate procedure is to back calculate the resilient modulus for different seasons using the procedure described using deflections measured on in-service pavements. These may be used as adjustment factors to correct the resilient modulus for a reference condition. [2]

Besides defining the seasonal moduli, it is also necessary to separate the year into the various com- ponent time intervals during which the different moduli are effective. In making this breakdown, it is not necessary to specify a time interval of less than one-half month for any given season. If it is not possible to adequately estimate the season lengths, which provides criteria suggested for the design of low-volume roads. At this point, the length of the seasons and the seasonal roadbed resilient moduli are all that is required in terms of roadbed support for the design of rigid pavements and aggregate-surfaced roads. For the design of flexible pavements, however, the seasonal data must be translated into the effective roadbed soil resilient modulus described earlier. This is accomplished with the aid of the chart in Figure 2.3. The effective modulus is a weighted value that gives the equivalent annual damage obtained by treating each season independently in the performance equation and summing the damage. It is important to note, however, that the effective roadbed soil resilient modulus determined from this chart applies only to flexible pavements designed using the serviceability criteria. It is not necessarily applicable to other resilient modulus-based design procedures. [2]

Since a mean value of resilient modulus is used, design sections with coefficient of variations greater than 0.15 (within a season) should be subdivided into smaller sections. For example, if the mean value of resilient modulus is 10,000 psi, then approximately 99 percent of the data should be in a range of 5,500 to 14,500 psi.

The first step of this process is to enter the seasonal ive time periods. If the smallest moduli in their respect season is one-half month, then all seasons must be defined in terms of half months and each of the boxes must be filled. If the smallest season is one month, then all seasons must be defined in terms of whole months and only one box per month may be filled in.

The next step is to estimate the relative damage (u) values corresponding to each seasonal modulus. This is done using the vertical scale or the corresponding equation.

Next, the u_f, values should all be added together and divided by the number of seasonal increments (12 or 24) to determine the average relative damage. The effective roadbed soil resilient modulus (M_R), then, is the value corresponding to the average relative damage on the M_R. Again, it is emphasized that this effective Mp value should be used only for the design of flexible pavements based on serviceability criteria.

2.4.2.1 Effective Modulus of Subgrade Reaction

Like the effective roadbed soil resilient modulus for flexible pavement design, an effective modulus of subgrade reaction (k-value) will be developed for rigid pavement design. Since the k-value is directly proportional to roadbed soil resilient modulus, the season lengths and seasonal moduli developed in the previous section will be used as input to the estimation of an effective design k-value. But, because of the effects of subbase characteristics on the effective design k-value, its determination is included as a step in an iterative design procedure.

2.4.2.2 Pavement Layer Material Characterization

Although there are many types of material properties and laboratory test procedures for assessing the strength of pavement structural materials, one has been adopted as a basis for design in this Guide. If however, the user should have a better understanding of the "layer coefficients" that have traditionally been used in the original AASHTO flexible pavement design procedure, it is not essential that the elastic moduli of these materials be characterized. In general, layer coefficients derived from test roads or satellite sections are preferred.

Elastic modulus is a fundamental engineering property of any paving or roadbed material. For those material types which are subject to significant permanent deformation under load, this property may not reflect the material's behavior under load. Thus, resilient modulus refers to the material's stress-strain behavior under normal pavement loading conditions. The strength of the material is important in addition to stiffness, and future mechanistic-based procedures may reflect strength as well as stiffness in the materials characterization procedures. In addition, stabilized base materials may be subject to cracking under certain conditions and the stiffness may not be an indicator for this distress type. It is important to note, that, although resilient modulus can apply to any type of material, the notation M_R as used in this Guide applies only to the roadbed soil. Different notations are used to express the moduli for subbase (EsB), base (Egs), asphalt concrete (EAc), and portland cement concrete (Ec). [2]

The procedure for estimating the resilient modulus of a particular pavement material depends on its type. Relatively low stiffness materials, such as natural soils, unbound granular layers, and even stabilized layers and asphalt concrete, should be tested using the resilient modulus test methods.

Although the testing apparatus for each of these types of materials is basically the same, there are some differences, such as the need for triaxial confinement for unbóund materials. Alternatively, the bound or higher stiffness materials, such as stabilized bases and asphalt concrete, may be tested using the repeated-load indirect tensile test.
This test still relies on the use of electronic gauges to measure small movements of the sample under load, but is less complex and easier to run than the triaxial resilient modulus test, because of the small displacements and brittle nature of the stiffest pavement materials, portland cement concrete and those base materials stabilized with a high cement content.

It is difficult to measure the modulus using the indirect tensile apparatus. Thus, it is recommended that the elastic modulus of such high-stiffness materials be determined according to the procedure described.

2.4.2.3 PCC Modulus of Repture

The modulus of rupture (flexural strength) of portland cement concrete is required only for the design of a rigid pavement. The modulus of rupture required by the design procedure is the mean value determined after 28 days using third-point loading. If standard agency practice dictates the use of center-point loading, then a correlation should be made between the two tests.

Because of the treatment of reliability in this Guide, it is strongly recommended that the normal construction specification for modulus of rupture (flexural strength) not be used as input, since it represents a value below which only a small percent of the distribution may lie. If it is desirable to use the construction specification, then some adjustment should be applied.

2.4.2.4 Layer Coefficients

This section describes a method for estimating the AASHTO structural layer coefficients (a, values) required for standard flexible pavement structural design. A value for this coefficient is assigned to each layer material in the pavement structure in order to convert actual layer thicknesses into structural number (SN).

This layer coefficient expresses the empirical relationship between SN and thickness and is a measure of the relative ability of the material to function as a structural component of the pavement. The following general equation for structural number reflects the relative impact of the layer coefficients (a) and thickness (D):

Although the elastic (resilient) modulus has been adopted as the standard material quality measure, it is still necessary to identify (corresponding) layer coefficients because of their treatment in the structural number design approach. Though there are correlations available to determine the modulus from tests such as the R-value, the procedure recommended is direct measurement using AASHTO Method (subbase and unbound granular materials) and for asphalt concrete and other stabilized materials. Research and field studies indicate many factors influence the layer coefficients, thus the agency's experience must be included in implementing the results from the procedures presented. For example, the layer coefficient may vary with thickness, under- lying support, position in the pavement structure, etc.

It should be noted that laboratory resilient modulus values can be obtained that are significantly different from what may exist for an in situ condition. For example, the presence of a very stiff unbound layer over a low stiffness layer may result in decompaction and a corresponding reduction of stiffness. As a guideline for successive layers of unbound materials, the ratio of resilient modulus of the upper layer to that of the lower layer should not exceed values that result in tensile stresses in unbound granular layers.

The discussion of how these coefficients are estimated is separated into five categories, depending on the type and function of the layer material. These are asphalt concrete, granular base, granular subbase, cement- treated, and bituminous base. Other materials such as lime, lime flyash, and cement flyash are acceptable materials, and each agency should develop charts. [2]

Asphalt Concrete Surface Course. Figure 2.2 provides a chart that may be used to estimate the structural layer coefficient of a dense-graded asphalt concrete surface course based on its elastic (resilient) modulus (EAC) at 68°F. Caution is recommended for modulus values above 450,000 psi. Although higher modulus asphalt concretes are stiffer and more resistant to bending, they are also more susceptible to thermal and fatigue cracking.

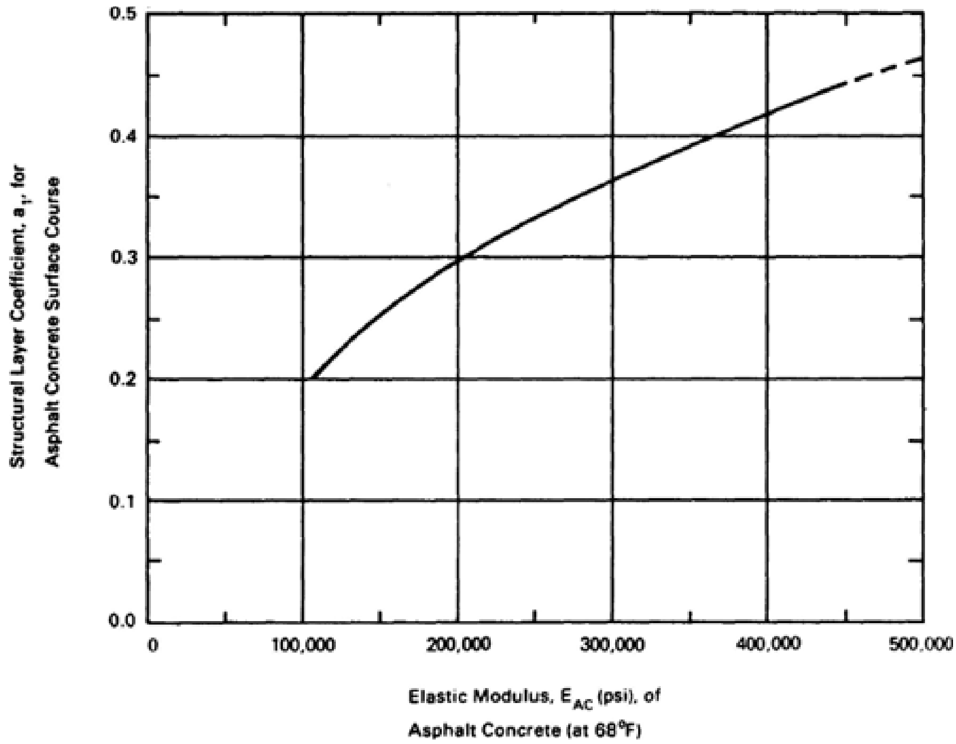

Figure 2.2 Chart to determine coefficient of surface layer [source:2]

Granular Base Layers. Figure 2.3 provides a chart that may be used to estimate a structural layer coefficient, a2, from one of four different laboratory test results on a granular base material, including base resilient modulus, $E_{Bs.}$

The following relationship may be used in lieu of figure 2.2 to estimate the layer coefficient $a_{2.}$

$a2 = 0.249(\log_{10} . E_{BS}) - 0.977$… . …………….. .e.q.2.3.

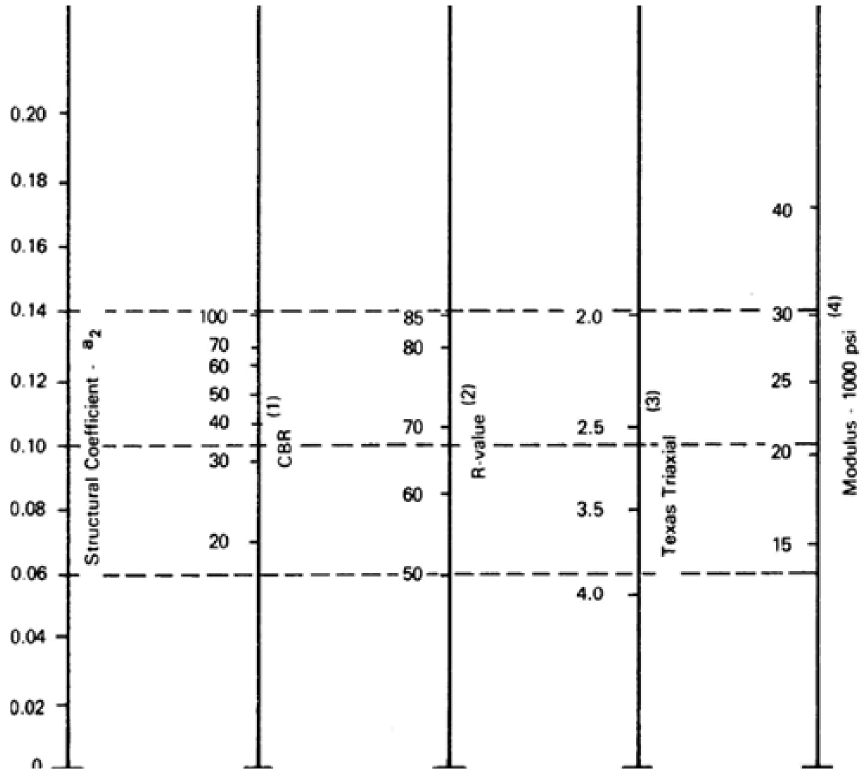

Figure 2.3 Chart above foundation layer coefficient a2

Granular Subbase Layers. Figure 2.3 provides a chart that may be used to estimate a structural layer coefficient, a_3 , from one of four different laboratory result on a granular subbase material, including subbase resilient modulus, $E_{BS.}$ [5]

a3 = 0.227 ($\log_{10} E_{BS}$) – 0.839 . . . . e.q.2.4.

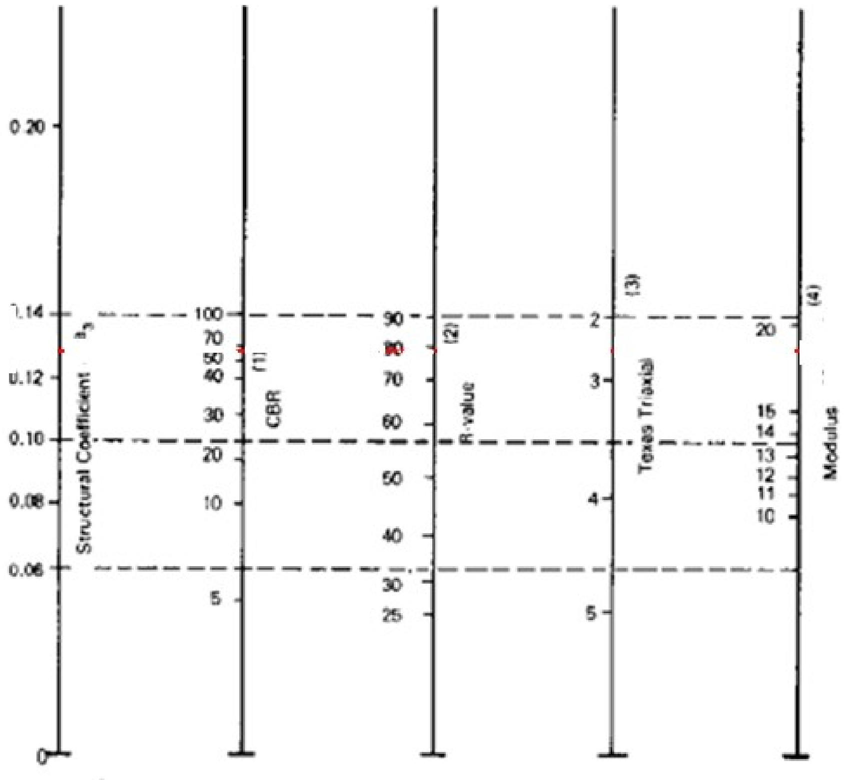

Figure 2.4 Chart above coefficient subgrade a_3 [source : 5]

2.4.3 Pavement Structural Characteristics

This refers to certain physical characteristics of the pavement structure which have an effect on its performance.

2.4.3.1 Drainage

This section describes the selection of inputs to treat the effects of certain levels of drainage on predicted pavements performance. Guidance is not provided here for any detailed drainage designs or contruction methods. Furthermore, criteria on the ability of various drainage methods to remove moisture from the pavement are not provided. it is up to the design engineer to identify what level(or quality) of drainage conditions.

Below are the general definitions corresponding to different drainage levels from the pavement structure [6]:

Tabel 2.4 Drainage levels from the pavemnet structure

Quality of Drainage	Water removed from the road surface within:
Excellent	2 hours
Good	1 day
Fair*	1 week*
Poor	1 month
Very Poor	water will not drain

For comparison purposes, the drainage conditions at the AASHTO Road test are considered to be fair, i.e.m free water was removed within 1 week.

Flexible Pavements. The treatment for the expected level of drainage for a flexible pavement is through the use of modified layer coefficients. The factor for modifying the layer coefficient is referred to as an m, value and has been integrated into the structural number (SN) equation along with layer coefficient (a) and thickness (D); thus:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \qquad \text{e.q.2.5.}$$

The possible effect of drainage on the asphalt. concrete surface course is not considered. The conversion of the structural number into actual pavement layer thicknesses. Obviously, the latter is dependent on the average yearly rainfall and the prevailing drainage conditions. As a basis for comparison, the m, value for conditions at the AASHTO road test is 1.0, regardless of the type of material.
Finally, it is also important to note that these values apply only to the effects of drainage on untreated base and subbase layers. Although improved drainage is

certainly beneficial to stabilized or treated materials, the effects on performance of flexible pavements are not as profound as those quantified.

Rigid Pavements. The treatment for the expected level of drainage for a rigid pavement is through the use of a drainage coefficient, C, in the performance equation. It has an effect similar to that of the load transfer coefficient. As a basis for comparison, the value for Cd for conditions at the AASHTO Road Test is 1.0. As before, the latter is dependent on the average yearly rainfall and the prevailing drainage conditions. [2]

Figure 2.5 Typical section for rigid or flexible pavement structure

2.5 Pavement Material

Materials used for construction of the pavement structure can be divided into two general classes; (1) those for flexible pavements and (2) those for rigid pavements. Materials used for composite pavements include those for roadbed preparation, for a subbase, and for a Portland cement concrete slab with an asphalt concrete wearing surface.

2.5.1 Prepared Roadbed

The prepared roadbed is a layer of compacted roadbed soil or select borrow material which has been compacted to a specified density.

2.5.1.1 Subbase Course

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

When roadbed soils are of relatively poor quality and the design procedure indicates that a substantial thickness of pavement is required, several alternate designs should be prepared for structural sections with and without subbase .the selection of an alternate may then be made on the basis of availability and relative costs of materials suitable for base and subbase . [7]

Because lower quality materials may be used in the lower layers of a flexible pavement structure, the use of a subbase course is often the most economical solution for construction of pavements over poor roadbed soils.

Although no specific quality requirements for subbase material are presented in this guide, the AASHTO Construction Manual for Highway Construction can be used as a guide . Many different materials have been use successfully for subbase. Local experience can be used as the basis for selection. For use in this design procedure, subbase material, if present, requires the use of a layer coefficient (a3), in order to convert its actual thickness to a structural number (SN). Special consideration must be given to determining the minimum thickness of base and surfacing required over a given subbase material. [2]

Untreated aggregate subbase should be compacted to 95 percent of maximum laboratory density, or higher, based on AASTHO Test, Method D, or the equivalent in addition to the major function as a structural portion of the pavement, subbase courses may have additional secondary function, such as :

1. Preventing the intrusion of fine-grained roadbed soils into base course-relatively dense-graded materials must be specified if the subbase is intended to serve this purpose.
2. Minimize the damaging effects of frost action-materials not susceptible to detrimental frost action must be specified if the subbase is intended for this purpose.
3. Preventing the accumulation of free water within or below the pavement structure a relatively free-draining material may be specified for the subbase if this is the intention. Provisions must also be made for collecting and removing the accumulated water from the subbase if this layer is to be included as part of the drainage system.
4. Providing a working platform for construction equipment-important when roadbed soil cannot provide the necessary support.

2.5.1.2 Base Course

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

When utilizing pozzolonic stabilized bases under a relatively thin asphaltic wearing surface, it can usually be expected that uncontrolled transverse reflection cracks will occur in the surface in a relatively short period of time, 1 to 3 years sawed and sealed joints through the asphalt concrete into the base may be utilized to minimize the adverse effects on appearance and to provide for better future sealing operations.

Joint spacing may very from 20 to 40 feet depending on local experience with past uncontrolled crack-spacing problems. Although no specific quality requirements for base courses are presented in Guide, the specifications included in AASHTO. “Graded Aggregate Material for Bases or Subbase for Highways and Airports”, are often used materials varying in gradation and quality from these specifications have been used in certain areas and have provided satisfactory performance .additional requirements for quality of base materials, based on test procedures used by the constructing agency, may also be included in materials or construction specifications. Untreated aggregate base should be compacted to at least 95 percent of maximum laboratory density based on AASHTO, or the equivalent. A wide variety of materials unsuitable for use as untreated base course have given satisfactory performance when improved by addition of a stabilizing admixture, such as Portland cement, asphalt, or lime Consideration should be given to the use of such treated materials for base course whenever they are economically feasible, particularly when suitable untreated materials are in short supply. Economic advantages may result not only from the use of low-cost aggregates but also from possible reduction in the total thickness of the pavement structure that may result from the use of treated materials.

Careful study is required in the selection of the type and amount of admixture to be used for optimum performance and economy. For use in this design procedure, base material must be represented by a layer coefficient (a_2) in order that its actual thickness may be converted to a structural number. [2]

2.5.1.3 Drainage Layer

A number of agencies are now considering or constructing pavement with a drainage course, or layer, as shown in figure 2.6. Figure 2.6 illustrates one configuration. The cross section shown in figure 2.6 is illustrative only the location of the longitudinal drain with respect to the traveled way can very depending on designer preference and local experience. Also, this figure does not show the collector system and outlet requirements for a total drainage design.

The designer should give some consideration to the preferred construction sequence when specifying a drainage system, excavation and installation after the travel lane paving has been completed local practice should be followed ; however the designer should be aware that special provisions to the specifications may be necessary additional information concerning.

Tables 2.5, 2.6 and 2.7 provide some background information for estimating the permeability of various types of material.

Table 2.5 provides general relationships between coarse-graded unstabilized materials and their coefficients of permeability.

Table 2.6 provides guidelines for the gradation of asphalt-treated permeable material. At least one state agency has reported the same gradation for porous concrete used a drainage layer.

Table 2.7 summarizes information relative to the permeability of graded aggregates as a function of the percent passing No 200 mesh sieve.

The approximate coefficient of permeability of the asphalt-treated permeable material is 3,000 feet or more per day when treated with 2 percent asphalt and 8,000 feet per day with no asphalt.

Specifications, for both design and construction, of drainage courses are under development, hence, material requirements should be referenced to the latest guide specifications of AASHTO or the approprite state agency responsible for developing statewide criteria and requirements. Information in table 2.5, 2.6, and 2.7 provides some guidelines for estimating permeability.

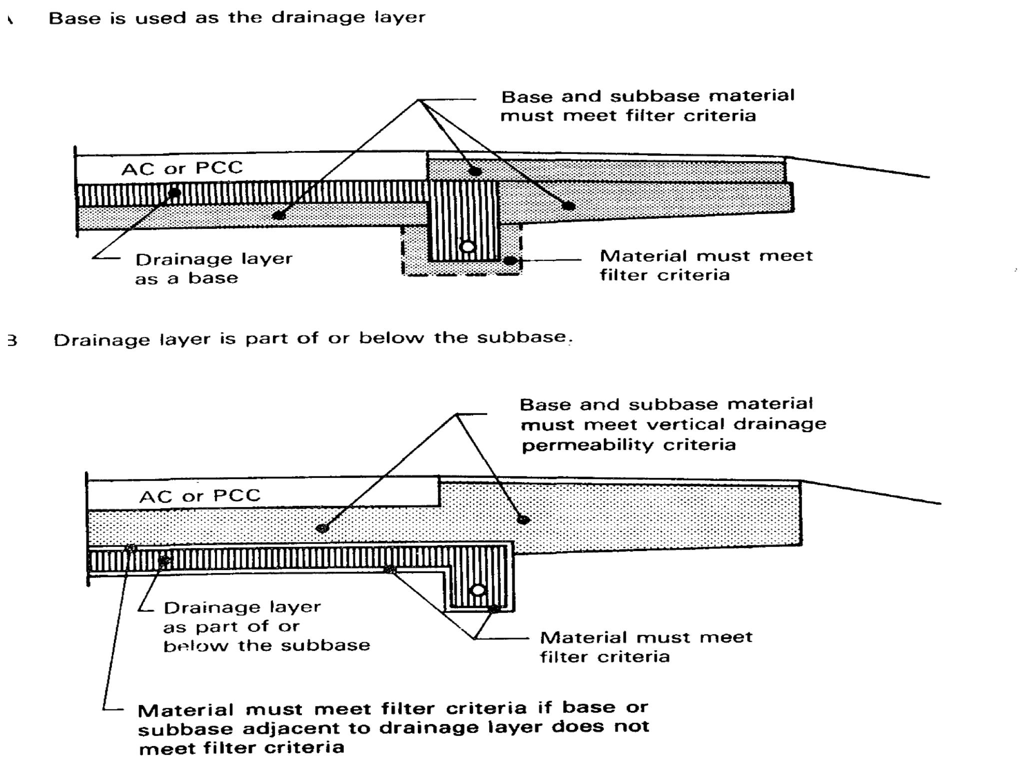

Figure 2.6 Example of drainage layer in pavement structure [source:2]

Table 2.5 Permeability of graded aggregates

Percent Passing	Sample Number

	1	2	3	4	5	6
3/4 inch sieve	100	100	100	100	100	100
1/2 inch sieve	85	84	83	81.5	79.5	75
3/8 inch sieve	77.5	76	74	72.5	69.5	63
No 4 sieve	58.5	56	52.5	49	43.5	32
No 8 sieve	42.5	39	34	29.5	22	58
No 10 sieve	39	35	30	25	17	0
No 20 sieve	26.5	22	15.5	9.8	0	0
No 40 sieve	18.5	13.3	6.3	0	0	0
No 60 sieve	13	7.5	0	0	0	0
No 140 sieve	6	0	0	0	0	0
No 200 sieve	0	0	0	0	0	0
Dry density (pcf)	121	117	115	111	104	101
Coefficient of permeability (ft per day)	10	110	320	1000	2600	300 0

Table 2.6 Gradation for asphalt treated permeable layer

Sleve Size	Percent Passing
1"	100
3/4"	90 – 100
3/8"	30 - 50
No 4	0 – 5
No 8	0 – 2

Table 2.7 Effect of percentage passing 200 mesh sieve on coefficient of permeability of dense graded aggregate, feet per day

Types of Fines	Percent Passing No. 200 Sieve			
	0	5	10	15
Silica or limestone	10	0,07	0,08	0,03
Silt	10	0,08	0,001	0,0002
Clay	10	0,01	0,0005	0,00009

2.5.1.4 Filter Material

The drainage layer and the collector system must be prevented from clogging if the system is to remain functioning for a long period of time. This is accomplished by means of a filter between the drain and the adjacent material. The filter material, which is made from select aggregates or fabrics, must meet three general requirements (1) it must prevent finer material, usually the subgrade, from piping or migrating into the drainage layer and clogging it, (2) it must be permeable enough to carry water without any resistance, and (3) it must be strong enough to carry the loads applied and, for aggregate, to distribute live loads to the subgrade.

2.5.1.5 Surface Course

The surface course of a flexible structure consists of a mixture of mineral aggregates and bintuminous materials placed as the upper course and usually constructed on a base course .in addition to its major function as a structural portion of the pavement, it must also be designed to resist the abrasive forces of traffic, to reduce the amount of surface water penetrating the pavement, to provide skid-resistance surface, and to provide a smooth and uniform riding surface.

The success of a surface course depends to a degree on obtaining a mixture with the optimum gradation of aggregate and percent of bintuminous binder to be durable and to resist fracture and raveling without becoming unstable under expected traffic and climatic conditions. The use of a laboratory design procedure is essential to ensure that a mixture will be satisfactory.

Although dense-graded aggregates with a maximum size of about 1 inch are most commonly specified for surface course for highways, a wide variety of other gradations, from sands to coarse, open-graded mixtures, have been used and have provided satisfactory performance for specific conditions. Surface courses are usually prepared by hot plant mixing with an asphalt cement, but satisfactory performance has also been obtained by cold plant mixing, or even mixing, in-place, with liquid asphalt or asphalt emulsions hot plant mixing, e g, asphalt concrete, are recommended for use on all moderate to heavily trafficked highways.

Construction specifications usually require that a bintuminous material be applied on untreated aggregate base courses as a prime coat, and on treated base courses and between layers of the surface course to serve as a tack coat.

No specific quality requirements for surface courses are presented in this Guide. It is recognized that each agrncy will prepare specifications that are based on performance, local constructions practices, and the most economical use of local materials ASTM Specification D 3515 provides some guidelines for designing asphalt concrete paving mixes.

It is particularly important that surface courses be properly compacted during construction. Improperly compacted surface courses are more likely to exhibit a variety of types of distress that tend to reduce the life and overall level of performance of the pavement.

Types of distress that are often related to insufficient compaction during construction include rutting resulting from further densification under traffic, structural failure resulting from excess infiltration of surface water through the surface course, and cracking or raveling of the surface course resulting from embrittlement of the bituminous binder by exposure to air and water in the mixture.

Specific criteria for compaction must be established by each highway agency based on local experience. Theoretical maximum densities of 92 percent or more are sometimes specified for dense-graded mixes.

2.5.1.6 Subbase

The subbase of a rigid pavement structure consist of one or more compacted layers of granular or stabilized material placed between the subgrade and the rigid slab for the following purposes :

1. To provide uniform, stable, and permanent support
2. To increase the modulus of subgrade reaction(k)
3. To minimize the damaging effects of frost action
4. To prevent pumping of finc-grained soils at joints, cracks, and edges of the rigid slab
5. To provide a working platform for construction equipment

If the roadbed soils are of a quality equal to that of a subbase, or in cases where design traffic is less than 1,000,000 8,16 ton an additional subbase layer may not be needed.

A number of different types ofsubbase have been used successfully. These include graded granular materials stabilized with suitable admixtures. Local experience may also provide useful criteria for the selection of subbase type. The prevention of water accumulations on or in roadbed soils or subbases is essential if satisfactory performance of the pavement structure is to be attained. It is recommended that the subbase layer be carried 1 to 3 feet beyond the paved roadway eidth or to the inslope if required for drainage.

Problem with the erosion of the subbase material under the pavement slab at joints and at the pavement edge have led some designers to use a lean concrete or porous layer is encouraged it should be noted that design criteria for such materials are still in the development stage and the designer should review the literature orcontact agency personnel familiar with current requirements.

2.5.2 Pavement Slab

The basic materials in the pavement slab are Portland cement concrete, reinforcing steel, load transfers device, and joint sealing materials. Quality control on the project to ensure yhet the materials conform to AASHTO or the agency specifications will minimize distrees resulting from distortion or disintregation.

2.5.2.1 Portland Cement Concrete

The mix design and material spesifications for the concrete should be accordance with, or equivalent to, to requirements of the AASHTO guide specifications for highway construction and the standard specifications for transportations materials. Under the given conditions of a specific projects, the minimum cement factor should be determined on the basis of laboratory tests and prior experience of strength and durability. [2]

2.5.2.2 Longitudinal Joint

Longitudinal joints are needed to form cracks at the desired location so that they may be keyed, butted, or tied joints, or combinations there of longitudinal joints should be sawed or formed to a minimum depth of one-fourth of the slab thickness. Timing of the sawcutting is critical to the crack formation of the desired location. The maximum recommended longitudinal spacing is 16 feet.

2.5.2.3 Load Transfer Devices

Mechanical load-transfer devices for the transverse joints should possess the following attributes :

1. They should be simple in desin, be practical to install, and permit complete encasement by the concrete

2. They should properly distribute the load stresses without overstressing the concrete st its contact with the device
3. They should offer little restraint to longitudinal movement of the joint at anytime
4. They should be mechanically stable under the wheel load weights and frequencies that will prevail in practice
5. They should be resistant to corrosion when used in those geographic location where corrosive elements are a problem (Various types of coatings are often used to minimize corrosion)

A commonly use load-transfer device is the plain, round steel dowel conforming to AASHTO Designation M31-Grade 60 or higher. Specific design requirements for these relative to diameter, length, and spacing are provided in part II. Although round dowels are the most commonly used, other mechanical devices that have proven satisfactory in field installation may also be used.

Consideration may also be given to omitting load-transfer devices from transverses weakened plane joints in plain jointed concrete pavement when supported on a treated permeable base.

2.5.2.5 Tie Bars

Tie bars either deformed steel bars or connectors, are designed to hold the faces of abutting slabs in firm contact. Tie bars are designed to withstand the maximum tensile forces required to overcome subgrade dreag. They are not designed to act as load-transfer devices. Deformed bars should be fabricated from billet or axle steel of grade 40 conforming to AASHTO. Specific recommendation onbar sizes, lengths, and spacings for different pavement conditions.

Other approved connectors may also be used, the tensile strength of such connectors should be equal to that of te deformed bar that would be required. The spacing of these connectors should conform to the same requirements given for deformed tie bars. Consideration should be given to the use of corrosion-resistant materials or coatings for both tie bars and dowels where salts are to be applied to to surface of the pavement. [2]

2.6 General Overlay

The general overlay presented here in is applicable to all types of overlay place on any type of pavement structure. This also implicit in this approach is the remaining pavement life concept which considers both the damage within the existing pavement as well as desired level of damage (design terminal serviceability level) within the overlaid pavement.

2.7 Development of Design Input Factor

There are seventh step in this design overlay procedure. They are :

1. Analysis Unit Delineation
2. Traffic Analysis
3. Material an Eviromental Study
4. Effective Structural Capacity Analysis (SC_{xeff})
5. Future Overlay Structural Capacity Analysis (SC_y)
6. Remaining Life Factor Determinan (F_{RL})
7. Overlay Design Analysis

Analysis Unit Delination. The objective is to determine boundaries along the project length that subdivide the rehabilitation project into statiscally homogenous pavement units processing uniform pavement cross section, subgrade (foundation) support, construction histories, and subsequent pavement condition.

Traffic Analysis. The purpose of the traffic analysis step is to determine the cumulative 18 KSAL repetitions along a pavement was originally opened to traffic thought the end of the anticipated overlay period.

Materials and Environment Study. Design values for the layer materials used in the bilitation process may be categorized into three Design major groups: 1) Existing pavement layer properties (2) Existing pavement subgrade (foundation) properties (3) Design properties of overlay layers (including the use of reeyeled materials). As with the overlay procedure for new pavement design-(Part Il), the primary material property of concern for all three categories listed above is the stic modulus.

Effective Structural Capacity Analysis. In an overlay analysis is to estimate effective (in situ) structural capacity of the pavement to be overlaid. Information regarding material properties derived in the previous step is used to arrive at this parameter. The two alternative methods described in Step 3 result in uniquely different approaches to the problem, though both will result irn identical structural capacity evaluations.

Future Overlay Structural Capacity Analysis (SC_y). In an overlay design analysis is to determine the future overlay structural capacity (SC_y). The major objective of this step is simply to determine the total structural capacity of a required to carry y repetitions in the overlay period to a terminal serviceability of P_{t2}, using the same existing subgrade (foundation) support for the design value. The analysis assumes that the new pavement isting pavement (SC_{xeff}) does not exist over the foundation. Consequently, this step in the overlay process is simply a new pavement design for either a flexible system or rigid system.

Remaining Life Factor Determinan (F_{RL}). in the overlay design determination of the remaining life factor, $F_{RL.}$ F_{RL} is an adjustment factor applied to the effective capacity parameter (SN_{xeff} or D_{xeff}) to reflect a assessment of the weighted effective capacit the overlay period. This factor is dependent remaining life value of the existing pavement overlay (R_{Lx}).