Chapter Fifty-four

PAVEMENT DESIGN

BUREAU OF DESIGN AND ENVIRONMENT MANUAL

Chapter Fifty-four
PAVEMENT DESIGN

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Chapter Fifty-four
PAVEMENT DESIGN

54-1 GENERAL

54-1.01 Scope of Chapter

Following the AASHO Road Test Project, IDOT assessed the results and performed additional research to develop practical applications of the findings that would be applicable to all marked and all unmarked routes on the State highway system. In the late 1980's and early 1990's, mechanistic design concepts were investigated and developed into our first procedures using the actual stresses and strains from traffic to design our pavements. Since that time, extensive research, monitoring, and evaluation have occurred resulting in refinements in the late 2000s. As a result, the following structural pavement design methodologies were developed or updated for inclusion in Chapter 54:

- mechanistic design of rigid pavements,
- modified AASHTO design of rigid pavements,
- design procedures for unbonded concrete overlays,
- mechanistic design of flexible pavements,
- modified AASHTO design of flexible pavements,
- design procedures for hot-mix asphalt (HMA) overlay of rubblized PCC pavements, and
- design procedures for composite pavements.

A flowchart is presented in Chapter 54 (Figure 54-1.A) that will assist in determining the appropriate structural pavement design methodology, pavement type, and design criteria. In addition to providing an analytical approach to structural pavement design, Chapter 54 presents an analytical method for selecting the most economical pavement design that can be expected to meet structural design requirements. The pavement design submittal serves as documentation to substantiate the recommendation of pavement type and thickness.

The pavement design procedures outlined in this chapter are for pavements on the state system only. The procedures in this chapter are inappropriate for non-state agency pavements and/or parking lots. Designs for local agencies should be developed using Chapter 44 of the BLRS Manual.

54-1.02 Definitions

The following definitions are typically used in pavement design:

1. Base Course. The layer, or layers, of specified or selected material (e.g., HMA binder, cement aggregate mixture (CAM)) of designed thickness placed on a subbase or a subgrade to support the surface course.

2. Class I Roads and Streets. Roads and streets designed as a facility, or as part of a future facility, with four or more lanes, and all one-way streets with structural design traffic greater than 3,500 ADT.

3. Class II Roads and Streets. Roads and streets designed as a two-lane facility with structural design traffic greater than 2,000 ADT, and all one-way streets with structural design traffic less than 3,500 ADT.

4. Class III Roads and Streets. Roads and streets with structural design traffic between 750 ADT and 2,000 ADT.

5. Class IV Roads and Streets. Roads and streets with structural design traffic less than 750 ADT.

6. Composite Pavement. A pavement structure having an HMA surface overlaying a PCC slab of relatively high bending resistance that serves as the principal load distributing layer. The PCC slab may be either a newly constructed base course or an existing rigid or composite pavement that is to be resurfaced.

7. Composite Pavement Structural Number (SN_C). An index number derived from an analysis of traffic and roadbed soil conditions that is used to determine the thickness of HMA surface and PCC base course, if required, for a composite pavement.

8. Construction Year Traffic. Construction year traffic is the ADT for the year the facility is to be opened to traffic.

9. Continuously Reinforced Concrete Pavement (CRCP). A rigid pavement structure having continuous longitudinal reinforcement. The continuous reinforcement is achieved by overlapping the longitudinal steel reinforcing bars.

10. Design HMA Microstrain. The design HMA microstrain is the tensile strain at the bottom of the HMA pavement layer used in the design of mechanistic flexible pavements.

11. Design E_{HMA} (HMA Mixture Modulus). The design E_{HMA} is the HMA mixture modulus in the pavement that corresponds to the design pavement HMA mixture temperature. E_{HMA} is selected from Figure 54-5.D and is based on the relationship between the design HMA mixture temperature and the asphalt binder type (i.e., PGXX-22 or PGXX-28) used in the design of mechanistic flexible pavements.

12. Design Lane. The lane carrying the greatest number of SU and MU vehicles for which the pavement section thickness will be based.

13. Design Pavement HMA Mixture Temperature. The design temperature of the HMA mixture to be used in the design of mechanistic flexible pavements is based on the pavement's geographical location. See Figure 54-5.C.

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14. Design Period. The number of years that a pavement is to carry a specific traffic volume and retain a serviceability level at or above a designated minimum value.

15. Design Subgrade Support Rating (SSR). The design SSR is the rating of the subgrade support under mechanistically designed pavements. There are three classes of SSR: poor, fair, and granular. SSR is determined by the district geotechnical engineer and documented in the project soils report.

16. Equivalency Factor. A numerical factor that expresses the relationship of a given axle load to another axle load in terms of its effect on the serviceability of a pavement structure. In pavement design, all axle loads are equated in terms of an equivalent number of repetitions of an 18-kip, equivalent single-axle load (ESALs).

17. Existing Traffic. Existing traffic is the existing ADT of the facility.

18. Flexible Pavement. An HMA pavement structure which maintains intimate contact with and distributes loads to the subgrade which depends upon aggregate interlock, particle friction, and cohesion for stability.

19. Flexible Pavement Structural Number (SN_F). An index number derived from an analysis of traffic and roadbed soil conditions which may be converted to a flexible pavement thickness for modified AASHTO through the use of suitable factors related to the types and strengths of material being used within the pavement structure.

20. Full-Depth HMA Pavement. A flexible pavement structure that uses HMA throughout the entire thickness (binder course and surface course layers).

21. Future Traffic. Future traffic is the ADT of the facility at the end of the design period, typically 20 years.

22. Illinois Bearing Ratio (IBR). The IBR is a measure of the support provided by the roadbed soils or by unbound granular materials under modified AASHTO designed pavements or composite pavements. The IBR test procedure is a modification of the California Bearing Ratio (CBR) procedure and is a soaked laboratory test.

23. Improved Subgrade. A subgrade which has been modified with lime, by-product lime, cement, or other approved material or, alternatively, has been removed and replaced with aggregate.

24. Integral Curb and Gutter. A curb and gutter which is paved monolithically with the pavement. It is used to reduce edge stresses.

25. Jointed Plain Concrete Pavement (JPCP). A rigid pavement structure that uses doweled joints at 15 ft nominal intervals.

26. Jointed Reinforced Concrete Pavement (JRCP). A rigid pavement that uses distributed steel reinforcement and transverse contraction joints.

27. Limiting Strain Criterion Design Thickness. The full-depth HMA pavement thickness at which the tensile strain at the bottom of the HMA is reduced to such a level that fatigue is no longer a factor in the design. This thickness need not be exceeded.

28. Mechanistic Pavement Design. A structural pavement design procedure used to determine fatigue life based on actual conditions, including stresses, strains, and deflections. It can be used to suit any local condition and material.

29. Multiple-Unit (MU) Vehicles. MU vehicles include truck tractor semi-trailers, full trailer combination vehicles, and other similar combinations.

30. Passenger Vehicles (PV). PVs include automobiles, pickup trucks, vans, and other similar two-axle, four-tire vehicles.

31. Pavement Performance. The trend of pavement serviceability with respect to repetitive vehicular load applications.

32. Pavement Structure. The combination of subbase, base course, and surface course placed on a subgrade to support the traffic load and distribute it to the roadbed.

33. Present Serviceability Index (PSI). A number derived by formula for estimating the serviceability rating from measurements of certain physical features of the pavement.

34. Present Serviceability Rating (PSR). The mean value of the independent subjective ratings by members of a special panel for the AASHO Road Test as to the serviceability of a section of the highway.

35. Pumping. The ejection of foundation material through joints or cracks or along edges of rigid slabs due to vertical movements of the slab under traffic.

36. Ramp. A roadway that connects two or more legs at an interchange and includes at least one ramp terminal section. Roadways that connect two legs of freeway, and that are continuations of mainline lanes, or arise from tapers designed for high-speed operation are not considered ramps, but are considered part of the mainline for structural design.

37. Rigid Pavement. A pavement structure whose surface and principal load distributing component is a PCC slab of relatively high bending resistance (e.g., JPCP, CRCP, JRCP).

38. Roadbed. That portion of the highway within the side slopes that is graded and prepared as a foundation for the pavement structure and shoulders.

39. Serviceability. The ability of a pavement, at the time of observation, to serve automobile and truck traffic.

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40. Single-Axle Load. The total load transmitted by all wheels whose centers may be included between two parallel transverse vertical planes 40 in. apart, extending across the full width of the vehicle.

41. Single-Unit (SU) Vehicles. SU vehicles include two- or three-axle trucks and buses having six tires.

42. Structural Design Traffic. The ADT that is estimated for the year that represents one-half of the design period which is then classified into PV, SU, and MU vehicles and assigned to the design lane to determine a traffic factor.

43. Subbase. The layer, or layers, of specified or selected material (e.g., HMA, CAM) of designed thickness that is placed on the subgrade to support the base course or, in the case of rigid pavements, the PCC slab.

44. Subgrade. The top surface of a roadbed upon which the pavement structure and shoulders are constructed.

45. Surface Course. One or more layers of a pavement structure designed to accommodate the traffic load, the top layer of which resists skidding, traffic abrasion, and the disintegrating effects of climate. The top layer is sometimes called the “wearing course”.

46. Tandem-Axle Load. The total load transmitted to the road by two or more consecutive axles whose centers may be included between two parallel vertical planes spaced more than 40 in. but not more than 96 in. apart, extending across the full width of the vehicle.

47. Tied Curb and Gutter. A PCC curb and gutter, which is tied with reinforcing steel to the pavement. It is used to reduce edge stresses.

48. Tied Shoulder. A PCC stabilized shoulder tied with reinforcing steel to the pavement. It is used to reduce pavement edge stresses.

49. Time-Traffic Exposure Factor. A numerical factor applied to the pavement design term indicated by the AASHO Road Test pavement performance equations (rigid and flexible) to modify the equations to be more representative of the behavior of pavements serving under similar conditions but over periods of time more typical of regular service life.

50. Traffic Factor (TF). The total number of 18-kip equivalent single-axle load applications (ESALs) to the design lane anticipated during the design period, expressed in millions. It is used as an equivalency factor for mixed traffic loads.

51. Untied Shoulder. Any shoulder which does not provide edge support. The shoulder may consist of earth, aggregate, HMA, or other materials.

54-1.03 Pavement Design Methodologies

54-1.03(a) Mechanistic

Since the completion of the AASHO Road Test Project, the Department has developed many new highway materials and procedures to improve pavement construction. This effort has resulted in improved material usage, construction procedures, and pavement designs which, although common practice today, were neither envisioned nor included in design procedures at the time of the AASHO Road Test. Therefore, to supplement the AASHO Road Test Project and better address modern pavement design, mechanistically based structural pavement design procedures were developed using structural mechanical analysis, computer modeling, and actual performance and response of existing pavement sections.

Mechanistic pavement design procedures are applicable to JPCP designs with nominal 15-ft panels and full-depth HMA designs with HMA surface and binder. The procedures use the actual stresses, strains, and deflections experienced by the pavement to determine its expected fatigue life. Factors that are considered in mechanistic designs include:

- design HMA strain,
- design pavement HMA mixture temperature,
- design HMA mixture modulus (E_{HMA}),
- subgrade support ratio (SSR),
- design reliability of 95% (HMA and PCC),
- degree of PCC edge support,
- degree of PCC base erosion,
- PCC joint spacing, and
- PCC stresses.

See Section 54-1.02 for definitions of these factors.

An Excel spreadsheet which will perform the mechanistic pavement design calculations is available on the IDOT website: http://www.dot.il.gov/desenv/pdp.html .

54-1.03(b) Modified AASHTO

The modified AASHTO design procedures are based on the AASHO Road Test pavement performance equations, which correlate performance of test sections with pavement design, the magnitude and configuration of the axle load, and the number of axle-load applications. The AASHTO equations are necessarily limited to the following factors:

- physical environment of the Road Test Project,
- materials used in the test pavements,
- range of pavement thicknesses included in the experiments,
- axle loads used and number of axle-load applications experienced,
- specific times and rates of application of the test traffic,

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- construction techniques employed, and
- climatic cycles experienced during construction and testing of the experimental facility.

To apply the AASHTO equations in design, it is necessary to make certain assumptions and extrapolations based on experience and engineering judgment. In developing the design procedures, modifications were made to the AASHTO equations to reflect the effect of the following variations on pavement performance:

- mixed truck and passenger car traffic axle loadings when compared with controlled traffic axle loadings in the AASHO Road Test,

- pavements subjected to traffic over longer periods of time when compared to the two years of traffic in the AASHO Road Test, and

- variations in the support strengths of the roadbed soils.

Variations in climatic conditions as they exist from one part of the State to another and particularly between the extreme northern and extreme southern portions undoubtedly affect pavement performance. The relative effects of these variations on pavement performance, however, are not sufficiently distinguishable at the present time to be taken into account in pavement structural design. Therefore, in developing the modified AASHTO structural design procedures, climatic effects were considered only on a state-wide basis.

Initially, procedures were developed for design of rigid and flexible pavements. Continuation of this effort focused on design of HMA surfacing for both existing PCC pavements and new PCC base courses. This resulted in the composite design procedure, which is based on a modification of the rigid design methodology.

54-1.04 Selection of Design Methodology, Pavement Type, and Design Criteria

Figure 54-1.A presents a flowchart that will help to determine the appropriate structural pavement design methodology, pavement type, and design criteria. In general, the mechanistic structural pavement design methodology will be used for all projects involving new pavement construction, complete reconstruction of existing pavements, and widening greater than or equal to 6 ft. In addition to the above requirements, the following will apply:

1. New Construction/Reconstruction Projects. For new construction/reconstruction projects, the pavement selection will be based on an economic analysis using estimated costs of construction and life-cycle activities (see Section 54-7). This procedure uses a 45-year life cycle, a 3% discount rate, and a comparison of pavement types based on annualized costs.

   For reconstruction projects, supplemental designs such as unbonded concrete overlay and rubblizing with HMA overlay will be added to the comparison with mechanistic designs for new rigid and flexible pavements to determine the most appropriate and economical strategy.

If the economic analysis does not result in one design being more than 10% cheaper than the others, the pavement selection will be based upon an alternate pavement bidding process. The criteria and requirements shown below must be met/followed for use of the alternate pavement bidding process.

a. Project Criteria. The following criteria must be met for a project to be considered for the alternate pavement bidding process.

- The project length must be 2 lane-miles or more in length. Pavement projects less than 2 lane-miles in length may be developed for alternate bids with approval by BDE. A standard lane-mile is defined as pavement 12 ft wide and 1 mile in length. Full-depth paved shoulder widths that have the same pavement type as the mainline should be proportionally included when calculating the overall project length in lane-miles [compared to 12-ft lane width].

- Projects involving widening cannot be considered for the alternate pavement bidding process.

- Life cycle costs for both the rigid and flexible designs must be based on a 45-year analysis with equal pavement design life.

- Traffic and construction staging for all pavement designs are considered an equal cost and therefore not included in the analysis.

b. Project Requirements. The following requirements must be completed for preparing a project for the alternate pavement bidding process.

- Plans for all projects with alternate pavements should contain typical cross-sections and multiple sets of quantities. One set will contain all items that are common for all pavement designs. The remaining sets will contain those items exclusive to the a particular design. Typical sections for each alternate pavement design, including station limits and all side road connections, must be presented in the contract plans. All pay items for alternate pavements must be in square yards for the entire pavement surface.

- The profile grade should always be designed for the thickest pavement design. The Contractor is responsible for maintaining the profile grade shown on the plans for the selected pavement with no additional compensation.

- Projects using lime modification for the improved subgrade must be scheduled for letting such that the subgrade improvement and placement of the pavement is completed in the same construction season. If the letting date is too late in the year to meet this requirement, the project start date needs to be delayed until the following construction season.

- Crossroad structures should be designed to accommodate a minimum cover based on the thickest pavement design.

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- Cost adjustment special provisions which tend to benefit one alternative (e.g. Bituminous Materials Cost Adjustment) cannot be included in an alternate bidding project. Other cost adjustments which are of equal benefit (e.g. Fuel Cost Adjustment and Steel Cost Adjustment) should be included.

While alternate bidding is generally advantageous, circumstances occasionally arise which cause one pavement type to be preferred over the other. In such cases, the district can submit documention to BDE of the preferred pavement type and the reason(s) alternate bidding is not advantageous. The Pavement Selection Committee can then be convened to determine the final pavement design. Refer to Section 54-7.05 for more information on the Pavement Selection Committee.

2. Widening Projects Involving Resurfacing. For projects involving widening greater than or equal to 6 ft in width, where the existing pavement and widening are to be resurfaced, the following will apply.

- The thickness of resurfacing over the existing pavement should be determined first according to the Department's policy.

- The mechanistic pavement design procedure will then be used to determine the total thickness of pavement required for the full-depth HMA pavement widening. The thickness of the resurfacing will be subtracted from the total thickness to determine the thickness of the additional binder required.

- Procedures contained in Section 54-6 will be used to design the PCC base course thickness of the composite pavement alternative and procedures contained in Section 54-5 will be used to design a flexible widening with HMA. The thickness of the HMA surface and top lift of binder over the PCC base course or HMA binder course will be equal to that determined for the resurfacing of the existing pavement.

- The designer should review and consider the ability of the pavement to be matched in-kind. It is possible that the new pavement thickness is thinner than the original pavement and subsequent rehabilitations. The designer should review the cross-section to determine if drainage paths will be impacted or other factors exist that could lead to performance problems if a thinner section is placed next to a thicker section.

- The selection of the pavement type will then be based on first cost.

3. Widening Projects Not Involving Resurfacing. For projects involving widening greater than or equal to 6 ft in width, where the existing pavement and widening are not to be resurfaced, the following will apply:

- The mechanistic structural pavement design procedures will be used to determine the required thickness for both the rigid and full-depth HMA pavement alternatives.

- Procedures contained in Section 54-5 will be used to design a flexible widening with HMA binder course alternative.
- The designer should review and consider the ability of the pavement to be matched in-kind. It is possible that the new pavement thickness is thinner than the original pavement and subsequent rehabilitations. The designer should review the cross-section to determine if drainage paths will be impacted or other factors exist that could lead to performance problems if a thinner section is placed next to a thicker section.
- The selection of the pavement type will then be based on first cost.

4. Special Considerations of Mechanistic Designs. The design charts used in the mechanistic design methodology provide several options for both rigid and flexible pavement designs. The following special considerations apply to mechanistic designs:

   a. Rigid Pavement Designs. Unless prior approval to use untied shoulders has been granted by the BDE or if stabilized shoulders are not required, the rigid pavement design curve for the 12 ft lane with tied concrete shoulders will be used for determining the design thickness. If the rigid option is selected, plans will be prepared based on the thickness obtained with the 12 ft lane.

   b. Flexible Pavement Designs. For flexible pavement designs, the designer will be required to select the appropriate asphalt binder grade(s) (i.e., PGXX-22 or PGXX-28) for the lower binder, upper binder, and surface lifts. The grade(s) shall be selected according to Figure 53-4.R and will be included in the plans.

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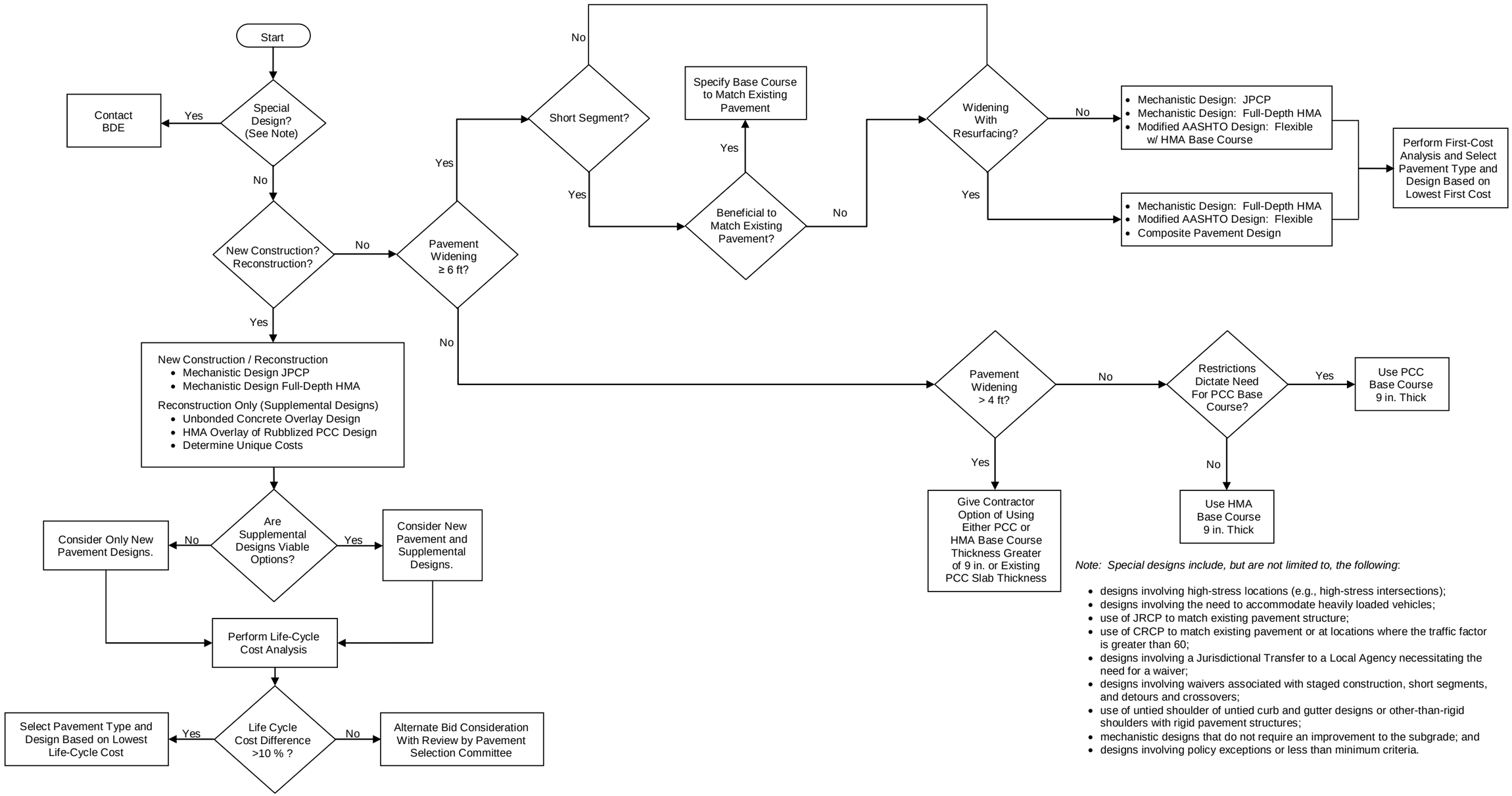

Note: Special designs include, but are not limited to, the following:

- designs involving high-stress locations (e.g., high-stress intersections);
- designs involving the need to accommodate heavily loaded vehicles;
- use of JRCP to match existing pavement structure;
- use of CRCP to match existing pavement or at locations where the traffic factor is greater than 60;
- designs involving a Jurisdictional Transfer to a Local Agency necessitating the need for a waiver;
- designs involving waivers associated with staged construction, short segments, and detours and crossovers;
- use of untied shoulder of untied curb and gutter designs or other-than-rigid shoulders with rigid pavement structures;
- mechanistic designs that do not require an improvement to the subgrade; and
- designs involving policy exceptions or less than minimum criteria.

FLOWCHART FOR SELECTION OF DESIGN METHODOLOGY, PAVEMENT TYPE, AND DESIGN CRITERIA

Figure 54-1.A

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54-1.05 Intersections

The type of pavement material selected for intersections will depend upon the existing pavement type and the volume and type of vehicles crossing or turning at the intersection. The following sections discuss the type and application of pavement materials typically used at intersections.

54-1.05(a) High-Stress Intersections

High-stress intersections are defined as those under stop control, either signal or signage, that have one or more of the following conditions:

- The approach grade on any stop-controlled leg of the intersection is greater than or equal to 3.5%.
- The design lane MU ADT is greater than or equal to 200 vehicles.
- The MU ADT for turning vehicles on any one lane of the intersection is greater than or equal to 200 vehicles. This also applies to sharp turning movements that are not under stop control.

Pavement types for high-stress intersections are limited to either of the following materials:

- PCC, or
- HMA $N_{design} \geq 90$ with polymer-modified binders.

Use these materials a minimum of 150 ft back from the location of the stop bar. The maximum length normally will be the length of the turn lane plus the taper. A greater length is permitted if a capacity study indicates a greater queue length.

If an existing intersection exhibits rutting and shoving of the HMA surface material, consider complete reconstruction rather than resurfacing the intersection.

54-1.05(b) Distress Treatment at Other Intersections

Intersections not meeting the criteria in Section 54-1.05(a) are not considered high-stress intersections. Nevertheless, they may exhibit, or have the potential to exhibit, pavement distress similar to that exhibited at high-stress intersections.

At intersections that do not meet high-stress criteria where the existing pavement type is bare PCC, PCC may be used if the improvement consists of minor widening without resurfacing.

Those intersections that are not considered high-stress but have an HMA pavement type exhibiting distress (e.g., rutting, shoving) should be investigated to determine the cause. Procedures outlined in Section 53-3.08 should be used in coordination with the District Bureau

of Materials to determine the suitability of the existing HMA pavement. If the existing material is found to be unsuitable, remove and replace the unstable material prior to resurfacing. If the investigation indicates a stable mixture but experience indicates a problem with flexible or composite pavements, consider obtaining an exception to the criteria presented in Section 54-1.05(a). Example conditions which may allow an exception include the following:

- MU ADT less than 200 if all are required to stop or if the approach speed is greater than 40 mph.
- MU ADT less than 200 if the majority are fully loaded at intersections near warehouse facilities, landfills, grain elevators, etc.
- High levels of SU vehicles that are primarily fully loaded hauling vehicles (e.g., grain trucks, concrete trucks, coal trucks). In this case, the designer should add the SU ADT to the MU ADT.
- Demonstrated and repeated history of pavement life significantly below 15 years with shoving of an HMA overlay related to tight turning movements.

The above exceptions also apply to intersections not exhibiting distress if the intersection is being constructed or reconstructed. Any exception request needs BDE approval. Requests to use PCC pavement for intersections that are not high-stress and do not require an exception will be submitted for approval through the normal pavement selection process.

54-1.05(c) Side Road Approaches

The following will apply to the placement of pavement material at side road approaches:

- For side road approaches that have a surface type lower than HMA, surface the approach with HMA to the right-of-way line or to 50 ft beyond the edge of the traveled way, whichever is less.
- For side road approaches that have a surface equal to or greater than HMA, surface the approach with HMA to at least the edge of shoulder.

54-1.06 Interchange Ramps

The following will apply to the structural pavement design of interchange ramps:

1. Design Methodology. Use the mechanistic structural pavement design methodology for interchange ramp projects, regardless of whether or not the ramps are to be newly constructed, reconstructed, or widened. The minimum traffic criteria are given in Figure 54-1.B. Other pavement design methodologies (e.g., modified AASHTO designs, composite pavement designs) are rarely used for interchange ramp designs and will require BDE approval.

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2. Ramp Pavement Type. Use a pavement type for interchange ramps that is the same as that of the contract being let for the mainline, except that ramps connected to entrance or exit ramp terminals shall be jointed PCC pavement if rigid pavement is used for the mainline.

3. Design Considerations. Typically, it is necessary to perform a structural pavement design for interchange ramps that is independent of that for the mainline pavement. Give adequate consideration to the requirements of high-stress locations (see Section 54-1.05(a)) and heavily loaded vehicles (see Section 54-2.01(e)) during design.

54-1.07 Rest Areas

Design the rest area exit and entrance ramps, roadways, shoulders, and parking areas for the greater of: 1) the actual projected mainline 2-way ADT in the design year using P=16% and S=M=25%, or 2) the interstate/freeway minimums assuming lane distribution of P=32% and S=M=45%. Use a pavement type for rest areas that is the same as that of the contract being let for the mainline, except use jointed PCC pavement if rigid pavement is used for the mainline. The concentration of heavy trucks braking on the ramps and inner roadways and the sharp turning maneuvers to enter parking stalls require these facilities to be considered high-stress locations. If HMA is used for the pavement material, it may require a different asphalt binder grade and/or special mixture design. See Sections 54-1.05(a) and 54-1.06 for additional information. For PCC pavements, use stabilized subbase for all routes with a design TF greater than 1.0. Design shoulders to the same thickness and material as the pavement as shown in Figure 16-1.H.

54-1.08 Weigh Stations

Design the weigh station ramps and detention parking area to provide the same structural capacity as the adjacent freeway. Weigh stations are, by definition, high-stress locations for pavement design purposes. When flexible or composite pavements are selected, apply modifications to the HMA mixtures to the driving lane pavements beginning 2,500 ft upstream of the exit ramp terminal stub, and downstream to a point 2,500 ft beyond the entrance ramp terminal stub, as well as throughout all pavements in the weigh stations and ramps. See Section 54-1.05(a) for additional information.

Crossroad	Design Traffic Criteria		Lane Distribution Criteria	Traffic Factor Equation	
				Rigid	Flexible
Interstate or Freeway	Greater of:	Actual Ramp Traffic Projection for Design Year	P = S = M = 100% (100% of Actual Ramp Traffic in Design Lane)	54-4.1	54-5.1
		Interstate/ Freeway Minimum Traffic (Figure 54-2C)	Same as used for Mainline Design Lane	54-4.1	54-5.1
Other State Marked Route	Greater of:	Actual Ramp Traffic Projection for Design Year	P = S = M = 100% (100% of Actual Ramp Traffic in Design Lane)	54-4.1	54-5.1
		Marked Route Minimum Design Traffic (Figure 54-2C)	Same as used for Crossroad Design Lane	54-4.2	54-5.2
Unmarked Route	Greater of:	Actual Ramp Traffic Projection for Design Year	P = S = M = 100% (100% of Actual Ramp Traffic in Design Lane)	54-4.1	54-5.1
		Marked Route Minimum Design Traffic (Figure 54-2C)	P = S = M = 50% (50% of Marked Route Minimum in Design Lane)	54-4.2	54-5.2

[1] *Use the highest traffic factor of any ramp at a given interchange for all ramps at that interchange.*

DESIGN TRAFFIC CRITERIA FOR INTERCHANGE RAMPS[1]

Figure 54-1.B

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54-2 BASIC DESIGN PARAMETERS (Mechanistic)

54-2.01 Development of Design Procedures

54-2.01(a) General

See Section 54-1.03(a) for a brief discussion of the mechanistic structural pavement design methodology.

54-2.01(b) Design Period

The level of traffic and type of facility to be constructed affect the selection of the design period. Generally, it is desirable to design highway pavements to carry traffic without necessitating the need for major rehabilitation for a period of 20 years. However, it may be advantageous to design lesser roadways (e.g., frontage roads) for shorter periods. Use the following guidelines when selecting an appropriate design period:

1. New Construction/Reconstruction Projects. For new construction/reconstruction projects, see the following sections for the appropriate design period:

   - Rigid Pavement: Mechanistic Design — Section 54-4.01(e),
   - Rigid Pavement: Modified AASHTO Design — Section 54-4.02(b),
   - Flexible Pavement: Mechanistic Design — Section 54-5.01(e),
   - Flexible Pavement: Modified AASHTO Design — Section 54-5.02(b), or
   - Composite Pavement Design: Modified AASHTO Design — Section 54-6.02.

2. Widening Projects. Use the following guidelines when selecting the design period for widening projects:

   a. Widening Without Resurfacing. Use a design period of 20 years.

   b. Widening With Resurfacing. Use the Department's policy for the thickness of the resurfacing. Use a design period of 20 years for the total thickness of the widening.

54-2.01(c) Structural Design Traffic

Structural design traffic is an estimate, based on ADT, of the volume of PV, SU, and MU vehicles that will be in the design lane in the year that is one-half the design period from the established date of construction. For example, if the design period is 20 years, the structural design traffic will be projected for the year which is 10 years from the established date of construction. In all IDOT pavement design procedures, structural design traffic is used to calculate a traffic factor (i.e., a factor representing the anticipated traffic load in the design lane on the pavement structure). However, the procedures and equations used to calculate the traffic factor differ among pavement types (rigid pavements or flexible pavements). Use the following procedures to determine structural design traffic:

1. Estimate ADT of PV, SU, and MU Vehicles. Vehicular classification and traffic volume projections for structural design traffic are based on available traffic data (i.e., ADT). ADT and vehicular classification data for various roadway classes may be obtained from published IDOT traffic maps. Contact the district Programming Section for traffic data. If traffic data is unavailable or if published data is dated or does not appear to reflect known conditions or field observations (e.g., land uses, directional distributions), traffic volume and classification studies may be needed to establish a representative base of existing conditions. Factors that compound annual growth typically are used in traffic projections. Other methodologies may apply. It is important to consider any future land development or land use changes that may affect the volume or composition of traffic that will use the facility. If vehicular classification data is not available for Class III or Class IV facilities, use the percentages in Figure 54-2.A to estimate the number of PV, SU, and MU vehicles from ADT. Also, give consideration to the potential impacts of heavily loaded vehicles, especially in areas near mines, grain elevators, factories, and river ports. It may be necessary to specifically design for such vehicles (see Section 54-2.01(e)).

Facility Class	Percent of Total ADT		
	PV	SU	MU
Class III	88%	7%	5%
Class IV	88%	9%	3%

VEHICULAR CLASSIFICATION FOR STRUCTURAL DESIGN TRAFFIC
(Class III and Class IV Facilities)

Figure 54-2.A

2. Assign Traffic to Design Lane. Although the sum of the PV, SU, and MU vehicular volumes determined in Step 1 represents the total ADT that will be carried by the highway facility in the year of the projection, the structural design of the pavement will be based on the lane which carries the greatest number of SU and MU vehicles (i.e., the design lane). Use the distribution factors in Figure 54-2.B to estimate the number of PV, SU, and MU vehicles that will be in the design lane. Use the total two-way ADT for multilane facilities when calculating the structural design traffic as the distribution factors account for directional traffic and the percentage of vehicles in the design lane. For example, if the total projected ADT for a rural 4-lane facility determined in Step 1 includes 300 MU vehicles, the estimated number of MU vehicles in the design lane will be 135 (i.e., 300 • 0.45). Note that the design lane distribution factors in Figure 54-2.B are based on previous traffic studies under average conditions. Unusual traffic control or design features may influence lane usage (e.g., lane restrictions of commercial vehicles, relatively close interchange spacing). Give consideration to such factors before applying the distribution factors in Figure 54-2.B. Adjustments may be necessary. Contact BDE for additional guidance.

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Number of Facility Lanes	Percent of Total Vehicular Class Volume (ADT) in Design Lane					
	Rural			Urban		
	PV	SU	MU	PV	SU	MU
2 or 3*	50%	50%	50%	50%	50%	50%
4	32%	45%	45%	32%	45%	45%
≥ 6	20%	40%	40%	8%	37%	37%

** One-way roads and streets.*

DESIGN LANE DISTRIBUTION FACTORS FOR STRUCTURAL DESIGN TRAFFIC

Figure 54-2.B

3. Determine Actual Structural Design Traffic. Steps 1 and 2 above are used to estimate the actual structural design traffic. The actual structural design traffic is used to calculate an actual traffic factor, which is a number representing an estimate of the total ESALs that will be in the design lane on the pavement structure during the design period. Unless other minimum criteria apply (e.g., see Item 4), this traffic factor will be used to design the entire pavement structure. In all IDOT pavement design methodologies it is necessary to calculate an actual traffic factor. There are two different sets of traffic factor equations, one for rigid and composite pavements and one for flexible pavements. Section 54-4.01(g) applies to rigid mechanistic, rigid modified AASHTO, and composite pavement designs. Section 54-5.01(g) applies to flexible pavement designs using either the mechanistic or modified AASHTO methodologies.

4. Determine Minimum Structural Design Traffic. The concept of using minimum structural design traffic to determine a minimum traffic factor applies only to mechanistic pavement designs. The minimum traffic factor is a factor that represents a minimum threshold below which the Department will not permit lesser pavement designs and is based on a set of minimum SU and MU vehicular volumes which are obtained from Figure 54-2.C. Figure 54-2.C applies to mechanistic designs of both rigid and flexible pavements. To obtain the minimum structural design traffic, enter Figure 54-2.C and select the set of minimum SU and MU vehicular volumes for the type of facility being designed, regardless of actual design traffic. Use the procedures discussed in Step 2 to distribute the minimum volumes obtained from Figure 54-2.C and determine the number of SU and MU vehicles in the design lane. The SU and MU vehicles assigned to the design lane will be used to calculate the minimum traffic factor. The minimum traffic factor is calculated in the same manner as the actual traffic factor calculated in Step 3. The only difference is that minimum, not projected actual, traffic volumes are used. The greater of the two traffic factors, actual or minimum, will be used to perform the mechanistic design of the pavement structure. Note that mechanistic designs of flexible pavements for unmarked routes have an overriding absolute minimum traffic factor of 0.5. For example, if the actual traffic factor calculated in Step 3 is less than the calculated minimum traffic factor, and the calculated minimum traffic factor is less than 0.5, the absolute minimum traffic factor of 0.5 will be used in the mechanistic design of the flexible pavement.

Facility Type	PV ①	SU ①	MU ①
Class I Interstates and Freeways	0	500	1500
Class I Other Marked State Routes	0	250	750
Class II, III, and IV Marked State Routes	0	250	750
Class I, II, III, and IV Unmarked State Routes	Use Actual Volumes ②		

Notes:

① *The minimum vehicular class volumes presented in Figure 54-2.C are given as ADT (i.e., two-way traffic) and must be assigned to the design lane using the procedures described in Step 2 in Section 54-2.01(c).*

② *Mechanistic designs of flexible pavements have an overriding absolute minimum traffic factor of 0.5. See Step 4 in Section 54-2.01(c).*

MINIMUM VEHICULAR CLASS VOLUMES FOR STRUCTURAL DESIGN TRAFFIC (Mechanistic Design: Rigid and Flexible Pavements)

Figure 54-2.C

54-2.01(d) Mixed-Traffic Axle Loadings

To evaluate the effects of mixed-traffic axle loadings on pavement performance, a system was developed to convert these loadings into a traffic factor. The traffic factor represents the total number of 18-kip ESALs, expressed in millions, that a given pavement may be expected to carry throughout its entire service life.

In developing this system, equivalency factors for various groupings of single-axle and tandem-axle loadings were determined from the AASHO Road Test equations, statewide weigh survey data, and classification counts at weigh stations. The equivalency factor for any given single-axle or tandem-axle load expresses the number of 18-kip single-axle load applications that is equivalent in effect upon pavement performance to one application of the given axle load.

In determining the number of 18-kip ESALs that represents one application of each of the three classes of vehicles (i.e., PV, SU, and MU), consideration must be given to the differences in average axle weights of both SU and MU trucks operating on various highways (e.g., high volume major highways with heavy commercial truck traffic, low volume farm-to-market highways). Highways were divided into four general classifications to reflect these differences in average axle loads. Because rigid and flexible pavements respond differently to axle loadings, the equivalency factors reflect these differences. The 18-kip equivalencies per vehicular classification for Class I through Class IV roads and streets are presented in this chapter for each design methodology.

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In areas that include mines, grain elevators, factories, river ports, and landfills, the impacts of heavily loaded SU and MU vehicles may become significant. See Section 54-2.01(e) for additional information on designing for heavily loaded vehicles.

54-2.01(e) Heavily Loaded Vehicles and High Volume Truck Routes

The equivalency factors for SU and MU vehicles that are incorporated in the IDOT pavement design methodologies presented in this chapter assume that traffic on the roadway is a typical mix of fully loaded, partially loaded, and empty vehicles. See Section 54-2.01(d) for additional information on equivalency factors for mixed-traffic axle loadings. Highway sections in some areas of the State provide access to businesses that utilize fully loaded trucks. Using typical SU and MU equivalency factors often will result in an inadequately designed pavement section. It only requires an increase in gross vehicle weight of approximately 10% to double the resulting damage to the pavement. Failure to consider the effects of heavily loaded vehicles can reduce pavement life significantly, in some cases by as much as one-half. It is therefore important to seriously evaluate pavement designs that will accommodate heavily loaded vehicles.

Typically, operations that involve moving bulk commodities and hauling large quantities of materials to and from ports and other locations use heavily loaded vehicles. Such operations may include mining operations, grain terminals, factories, river ports, landfills, etc. In general, designs should be adjusted where heavily loaded vehicles comprise 10% or more of the design truck traffic. The designer should refer to the Bureau of Materials and Physical Research document *Pavement Technology Advisory PTA-D1: Designing for Heavily Loaded Vehicles* (http://www.dot.il.gov/materials/research/pdf/ptad1.pdf). If needed, contact the Bureau of Materials and Physical Research for assistance in analysis of traffic and pavement sections.

When determining the type of pavement to be constructed, first calculate the rigid traffic factor. Projects with a rigid traffic factor greater than 60 should typically use continuously reinforced concrete pavement as discussed in Section 54-4.02(a). If a district has a project with a rigid traffic factor greater than 60 and another pavement type is desired (e.g., match adjacent cross-sections, ease of construction, etc.), BDE must approve the design.

54-2.01(f) Roadbed Soils

Subsurface exploration is an essential part of the engineering survey for highway location and design. It includes soils investigation, sampling, testing, identification, and distribution with respect to the horizontal and vertical alignment of the highway. USDA county soils reports often are used in preliminary geotechnical investigations and in developing soil sampling surveys. Roadbed soil types, problem areas, recommendations, and corrective measures are compiled by the geotechnical engineer in the project geotechnical report, which will become a project design document for incorporation in the project plans and specifications. Consider the following guidelines when evaluating the need for subgrade improvements:

1. Importance of Subgrade Stability. Subgrade stability plays a critical role in the construction and performance of a pavement. Pavement performance is directly related

to the physical properties of the roadbed soils and the materials used in the pavement structure. Subgrade stability is a function of a soil's strength and its behavior under repeated traffic loadings. Both properties significantly influence pavement construction operations and the long-term performance of the pavement structure. The pavement subgrade should be sufficiently stable to:

- prevent excessive rutting and shoving during construction;
- provide uniform support for placement and compaction of pavement layers;
- minimize impacts of excessive volume change and frost;
- limit pavement resilient (i.e., rebound) deflections to acceptable limits; and
- restrict permanent deformation during pavement service life.

2. Department Policy. A subgrade that provides a stable working platform will minimize rutting and moisture related problems resulting in effective construction and a smoother pavement. Department policy was established to ensure that the pavement subgrade, in situ or improved, will provide a stable working platform for all pavement construction. Most Illinois soils are not capable of providing a subgrade that meets the criteria for a stable working platform. Although the negative effects of less satisfactory soils can, to some degree, be reduced by increasing pavement structure thickness, it usually is necessary to treat in situ soils to ensure adequate subgrade support for construction equipment operations and for the placement and compaction of pavement layers. At a minimum, it is required that a 12 in. improved subgrade layer be provided. Where in situ soils are found to be inadequate such that the 12 in. improved subgrade layer will not provide a stable working platform, the designer should include provisions to address the need for deeper treatment or removal and replacement per the *IDOT Subgrade Stability Manual* (http://www.dot.il.gov/bridges/brmanuals.html).

3. Design Considerations. Mechanistic structural pavement designs will provide structurally adequate pavements that are suited to local conditions and material selection. The mechanistic design methodology for both rigid and flexible pavement types assumes that a stable subgrade is present (in situ or improved) and that, if improved, the subgrade is constructed with the highest quality material. The subgrade, however, is not given additional credit during design nor is the cost of providing an improved subgrade included in the pavement selection process. If it is determined that an improved subgrade layer is needed to provide a stable working platform, no change will be made to the subgrade support rating that is used during design (see Item 4). For additional design guidance on providing a stable working platform, see the IDOT *Subgrade Stability Manual* (http://www.dot.il.gov/bridges/brmanuals.html). Provisions must be made for the entire thickness of the binder to be completed in a single construction season once the improved subgrade layer is in place. The Department has experienced repeated springtime pavement failures during construction of partially completed sections that have been exposed to winter conditions. Provisions for staging/completion may need to be used to prevent large sections of partially completed pavement being abandoned due to winter shutdown.

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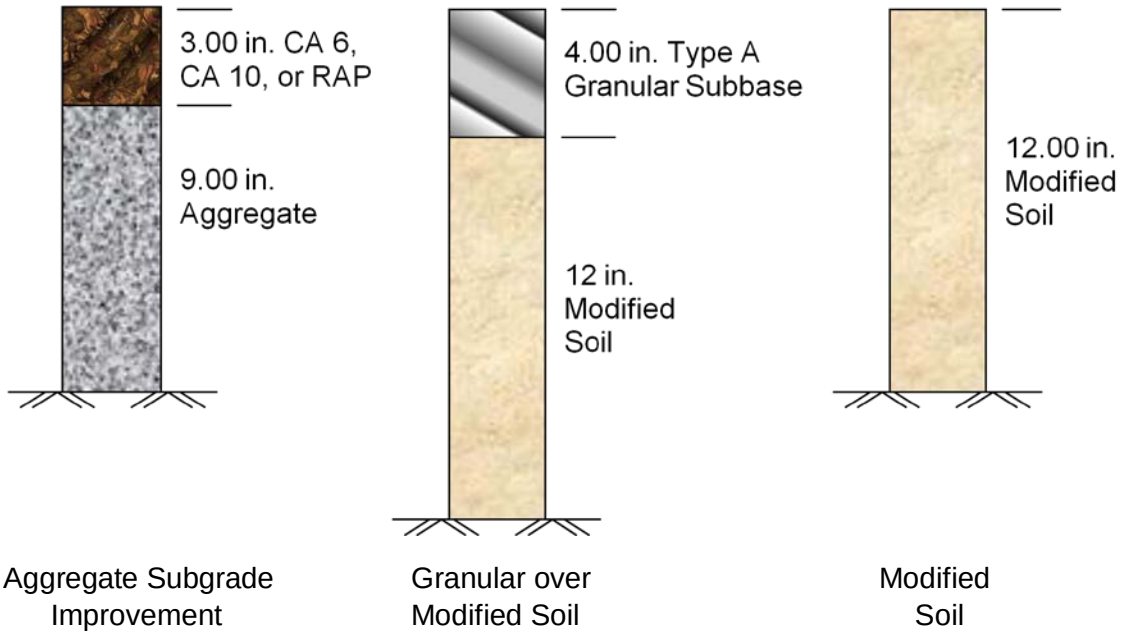
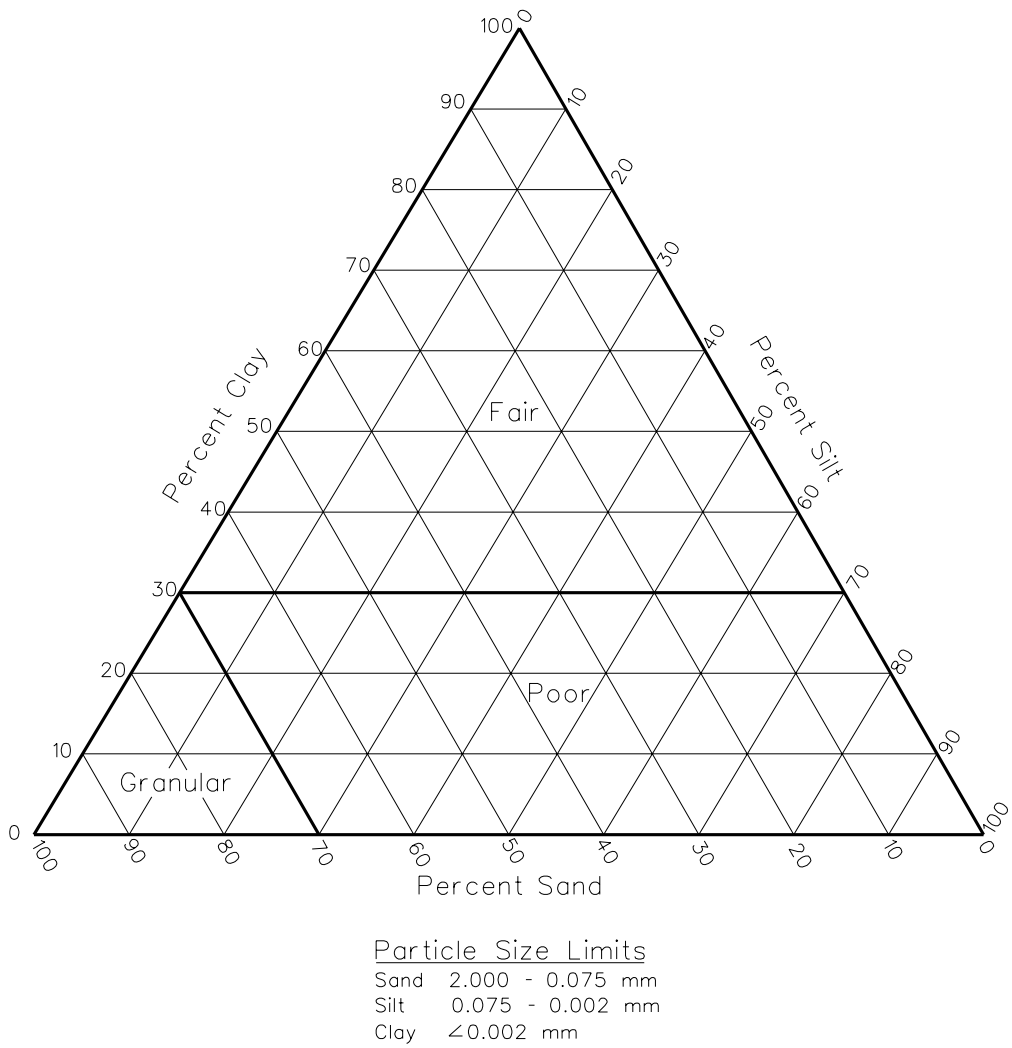


TREATMENT OPTIONS FOR IMPROVED SUBGRADE
(Mechanistic Design: Rigid and Flexible Pavements)

Figure 54-2.D

4. Soils Evaluation (Subgrade Support Rating — SSR). Mechanistic pavement design procedures require that subgrade support ratings be determined for the project's in situ soils. These SSR values will be used to design the pavement structure. SSRs are based on information provided in the project geotechnical report. There are three SSR categories (i.e., poor, fair, and granular). Based on the percent clay, silt, and sand found in the in situ soil, SSR values are obtained from Figure 54-2.E. Note that Figure 54-2.E already assumes a high-water table and appropriate frost penetration. All project soils will be evaluated by the district geotechnical engineer to determine and verify what, if any, subgrade treatment is needed. It also is the responsibility of the geotechnical engineer to provide the designer with SSRs and their application limits within the project. Each SSR in the soils report will represent the average soil conditions within the specified limits of the project. For small projects, in the absence of laboratory tests, SSR values may be estimated using USDA county soils reports.



SUBGRADE SUPPORT RATING (SSR)
(Mechanistic Design)

Figure 54-2.E

5. Policy Deviations. Typical Illinois soils have poor to fair SSR values and require an improved subgrade layer. Although infrequent, an in situ soil (e.g., granular) may be found at a project location that will provide a stable working platform without improvement. Where marginally adequate soils are found, the district geotechnical engineer may recommend something less than a 12 in. thick improved subgrade layer. On the other hand, the geotechnical report may recommend a thicker improved subgrade layer. Where an improved subgrade layer is needed to provide a stable working platform, any deviation from the 12 in. thickness criteria must be supported by the geotechnical engineer, documented in the project geotechnical report, and approved by the BDE. In such cases, ensure that supporting conclusions and recommendations are adequately documented in the pavement design submittal.

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6. Subgrade Improvement Alternatives. Where the in situ soil has an SSR value of either poor or fair, the subgrade shall be improved to the required depth using the allowable alternatives shown in Figure 54-2.D.

   Consider the following when selecting from these alternatives:

   a. Typical Treatment. Where required, lime modification typically is used for the improved subgrade layer.

   b. Non Lime-Reactive Soils. Where in situ soils are found to not react with lime or by-product lime (e.g., organic and sandy soils), cement or fly ash modification typically is used for the improved subgrade layer. Laboratory testing may be required to determine the ability of cement or fly ash to be used as a modifier.

   c. Urban Areas. For dust control in urban areas, granular material may be specified for the improved subgrade layer.

   d. Granular Material Availability. If granular material is readily available, the district may elect to specify aggregate subgrade improvement in lieu of typical treatments. This decision will be made on a case-by-case basis.

   e. Alternatives Analysis. For the subgrade treatments being considered, determine the appropriate construction parameters for the subgrade alternatives analysis. This may include the following:

      - required thickness of the improved subgrade layer;
      - percentage of lime, by-product lime, fly ash, or cement required; and/or
      - required depth of undercut and granular backfill.

      Base the decision for selecting the subgrade treatment on factors that include material availability, constructability, economics, permanence of treatment, and pavement performance benefits. Select the alternative for the improved subgrade layer that is best suited for the project.

7. Embankment Settlement. Underlying soils can settle under the weight of a newly constructed embankment resulting in differential settlement and/or embankment failures. The settlements are expensive to fix and can result in pavement dips. For all classes of roads and streets, construction should be sequenced to assure that earthwork is completed during one construction season and that paving is initiated the next. If this cannot be accomplished with some degree of certainty, include special provisions in the contract that will assure adequate embankment settlement prior to subsequent pavement layers being placed. Consult the district geotechnical engineer for guidance in preparing such special provisions.

54-2.02 Structural Design

To select the proper type and thickness of mainline pavement for a particular project, first determine the following:

- the volume and composition of traffic to be carried by the pavement,
- the length of time the pavement is to service this traffic,
- the strength characteristics of the subgrade soils and pavement materials, and
- the minimum quality of service to be provided by the pavement during its lifetime.

See Section 54-1.06 for information on pavement designs for interchange ramps. Pavement designs for rest areas and weigh stations are discussed in Sections 54-1.07 and 54-1.08, respectively.

54-2.03 Limitations and Requirements

54-2.03(a) General

The procedures that are presented in this chapter will allow the designer to select an economically optimal pavement design that is most capable of carrying the anticipated traffic. To ensure that the selected design is both practical and adequate, the following sections present policy limitations and requirements that must be considered.

54-2.03(b) Adherence to Specifications

The design procedures that are presented in this Chapter are based on the assumption that material requirements, mixture designs, and construction procedures and controls will be in accordance with current IDOT specifications and practices. To ensure satisfactory performance, the strengths of structural components that are assumed during design must be achieved during construction. These strengths should be shown on the cover sheet or typical cross-sections of the plans, along with the structural design traffic; the percentage breakdown of the structural design traffic for PV, SU, and MU vehicles; the percentage of these vehicles in the design lane; and the SSR or IBR values of the roadbed soils. See Section 63-4.05 for additional information on placing structural design traffic on plans.

54-2.03(c) Structural Design Traffic

The equations used to convert structural design traffic into 18-kip ESALs are based on an average distribution of vehicle types and axle loadings and are directly applicable to most roads and streets. However, cases will arise in which these equations should not be used, and a special analysis will be necessary. One such case would be that involving a highway adjacent to an industrial site where the commercial vehicles entering and leaving the site generally travel empty in one direction and fully loaded in the other. Contact the Bureau of Materials and Physical Research for assistance. It will be necessary for the district to furnish the structural

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design traffic and weight and classification count data in sufficient detail to permit a determination of the distribution of commercial vehicle types and the single-axle and tandem-axle loadings within each type. See Section 54-2.01(e) for additional information on designing for heavily loaded vehicles.

54-2.03(d) Terminal Service Level

At the end of the design period, the serviceability level of the pavement can be expected to have been reduced to a value of 2.5 for Class I roads and streets and to 2.0 for Class II, Class III, and Class IV roads and streets, and the pavement should be considered eligible for rehabilitation. The design period may or may not be the actual service life of the pavement. The actual service life may be longer or shorter than the design period depending upon the conditions under which the pavement actually serves and the conditions assumed for the design. Highly significant are the differences between the structural design traffic and the actual traffic carried by the pavement, and the difference between the design terminal serviceability level and the actual serviceability level at which the pavement is structurally upgraded.

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54-3 BASIC DESIGN PARAMETERS (Modified AASHTO)

54-3.01 Development of Design Procedures

54-3.01(a) General

See Section 54-1.03(b) for a brief description of the modified AASHTO design procedures.

54-3.01(b) Design Period

Section 54-2.01(b) applies to the modified AASHTO design procedures.

54-3.01(c) Structural Design Traffic

Section 54-2.01(c) applies to the modified AASHTO design procedures, except for the requirement of calculating a minimum traffic factor based on minimum structural design traffic.

54-3.01(d) Mixed-Traffic Axle Loadings

Section 54-2.01(d) applies to the modified AASHTO design procedures.

54-3.01(e) Heavily Loaded Vehicles

Section 54-2.01(e) applies to the modified AASHTO design procedures.

54-3.01(f) Roadbed Soils

Section 54-2.01(f) does not apply to the modified AASHTO design procedures. The following material specifically relates to the modified AASHTO design methodology.

An A-6 (9-13) type of roadbed soil was used throughout the entire embankment of the AASHO Road Test Project. Because only one soil type was taken into consideration in the AASHO Road Test, it was necessary to modify the AASHO Road Test equations so that pavement thicknesses could be developed for other soil types. The modification makes use of the Illinois Bearing Ratio (IBR) of the soil, which is the only soil support value normally determined by the Department for modified AASHTO designs. Other soil strength test procedures can be used provided that the test results can be directly correlated with those obtained by the IBR test procedure.

The IBR selected for use in design should represent a minimum value for the soil to be used. Preferably, laboratory tests should be made on four-day soaked samples of the soils to be used in construction. It is recommended that a soil survey be made prior to all construction; however, when test data are not available, use the values presented in Figure 54-3.A.

Soil Classification	IBR
A-1	20
A-2-4, A-2-5	15
A-2-6, A-2-7	12
A-3	10
A-4, A-5, A-6	3
A-7-5, A-7-6	2

SUGGESTED IBR VALUES FOR VARIOUS SOIL CLASSIFICATIONS

Figure 54-3.A

Pavement performance is directly related to the physical properties and the support capacity of the materials used in the pavement structure and of the roadbed soils. The effect of less satisfactory soils, to some degree, can be reduced by increasing the thickness of the pavement structure, but it may be necessary to take other steps to ensure adequate pavement performance. The problems that can be encountered because roadbed soils are subject to permanent deformation, excessive volume changes, excessive deflection and rebound, frost susceptibility, and non-uniform support from wide variations in soil types within the State should be recognized in the design stage. Corrective measures should be included in the plans and in the special provisions for any and all small isolated areas of unsatisfactory soils. If such areas contain soils that are unsatisfactory for roadbed construction, the soils should be either removed and replaced with satisfactory soils or granular material or improved in-place with a suitable stabilizing agent. If such soils are unsatisfactory only from the standpoint of having an IBR less than the minimum selected for design, consider the following treatments:

- remove and replace with soils or granular material at or above the minimum value,
- remove and replace with additional subbase material to a depth that will compensate for the deficiency in support strength, or
- improve the material in-place with a suitable stabilizing agent.

See the IDOT *Subgrade Stability Manual* (http://www.dot.il.gov/bridges/brmanuals.html) for further guidance.

54-3.02 Structural Design

Section 54-2.02 applies to the modified AASHTO design procedures.

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54-3.03 Limitations and Requirements

54-3.03(a) General

Section 54-2.03(a) applies to the modified AASHTO design procedures.

54-3.03(b) Adherence to Specifications

Section 54-2.03(b) applies to the modified AASHTO design procedures.

54-3.03(c) Structural Design Traffic

Section 54-2.03(c) applies to the modified AASHTO design procedures.

54-3.03(d) Terminal Service Life

Section 54-2.03(d) applies to the modified AASHTO design procedures.

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54-4 STRUCTURAL DESIGN OF RIGID PAVEMENTS

54-4.01 Mechanistic

54-4.01(a) Limitations

Jointed plain concrete pavement (JPCP) thickness designs may be obtained for thicknesses up to 12 in. and for traffic factors up to approximately 100. For traffic factors above 60, CRCP should typically be used, see Section 54-4.02 for design procedures. The use of doweled joints will be required for pavement thicknesses that are 7 in. and greater on all Class I, Class II, and Class III roads and streets and Class IV marked roads and streets. Doweled joints will not be required for Class IV unmarked roads and streets. Recommended dowel diameters are given in Figure 54-4.A. If designs for traffic factors greater than 100 or for pavement thicknesses greater than 12 in. are desired, contact the BDE.

Pavement Thickness (T) (inches)	Nominal Dowel Diameter (inches)
≥ 8	1.5
$7 \leq T < 8$	1.25
< 7	1

RECOMMENDED DOWEL DIAMETERS

Figure 54-4.A

54-4.01(b) Application of Design Method

The mechanistic design procedures for rigid pavements enable the designer to determine the type and thickness of PCC pavement that are required to carry a specified volume and composition of traffic for a designated period of time while retaining minimum serviceability. Use the procedures presented in Section 54-4.01(j) to determine pavement type and thickness and provide a subbase in accordance with Section 54-4.01(h).

54-4.01(c) Edge Support Conditions

The mechanistic design methodology gives the designer the option of selecting from the following two-edge support conditions:

1. Tied Shoulder. The tied shoulder condition consists of a 12 ft paved lane that is tied with reinforcing steel to a PCC shoulder or curb and gutter.

2. Untied Shoulder. Any 12 ft pavement that is not positively tied with reinforcing steel to the shoulder or curb and gutter is considered an untied shoulder condition. An untied shoulder may consist of earth, aggregate, or HMA materials.

The selection of edge support has a pronounced effect on edge stresses and pavement thickness. Department policy dictates exclusive use of tied shoulders, unless otherwise approved by the BDE.

54-4.01(d) Joint Spacing Limitations

Joint spacing between panels may need to vary to accommodate adjacent pavements, drainage structures, etc. See the *Highway Standards* for typical designs. The mechanistic design procedures may be used to design pavement thickness for a nominal panel length of 15 ft, except Class IV unmarked roads and streets may use 12-ft joint spacing. The panel length may be adjusted ±3 ft to accommodate discontinuities in pavement cross-section (e.g., intersections, medians). If designs for longer panel sections are required to match existing pavements, contact BDE.

54-4.01(e) Design Period

The design period for all rigid pavements is typically 20 years.

54-4.01(f) Equivalency Factors

Section 54-2.01(d) describes the use of equivalency factors to convert mixed-traffic loadings to 18-kip ESAL applications. Equivalency factors for rigid pavements are presented in Figure 54-4.B. The factors in Figure 54-4.B were used to develop the traffic factor equations discussed in Section 54-4.01(g).

54-4.01(g) Traffic Factor

The traffic factor is the projected total 18-kip ESALs, expressed in millions, to be carried by the design lane during the design period. Based on the class of the facility, select the appropriate equation from Figure 54-4.C to calculate the traffic factor for rigid pavement designs. See Section 54-2.01(c) for information on structural design traffic.

Facility Class	18-kip ESAL Applications Per Vehicle		
	PV	SU	MU
Class I	0.0004	0.394	1.908
Class II	0.0004	0.372	1.554
Class III and IV	0.0004	0.355	1.541

EQUIVALENCY FACTORS
(Rigid Pavements)

Figure 54-4.B

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Facility Class	Traffic Factor Equation	Equation Number
Class I	$TF = DP\left[\frac{(0.15 \bullet P \bullet PV) + (143.81 \bullet S \bullet SU) + (696.42 \bullet M \bullet MU)}{1x10^6}\right]$	Equation 54-4.1
Class II	$TF = DP\left[\frac{(0.15 \bullet P \bullet PV) + (135.78 \bullet S \bullet SU) + (567.21 \bullet M \bullet MU)}{1x10^6}\right]$	Equation 54-4.2
Class III and Class IV	$TF = DP\left[\frac{(0.15 \bullet P \bullet PV) + (129.58 \bullet S \bullet SU) + (562.47 \bullet M \bullet MU)}{1x10^6}\right]$	Equation 54-4.3

where:

PV, SU, MU = structural design traffic expressed as the number of PV, SU, and MU vehicles.
P, S, M = percent of PV, SU, and MU in the design lane expressed as a decimal.
DP = design period — typically 20 years.

TRAFFIC FACTOR EQUATIONS
(Rigid Pavements)

Figure 54-4.C

54-4.01(h) Improved Subgrade and Subbase Type and Thickness

Thicknesses determined through the mechanistic design process assume that an adequate construction platform exists at the time of construction and it will perform adequately over the life of the pavement. The platform consists of an improved subgrade, and subbase when necessary. The requirements for the improved subgrade and subbase are as follows:

1. Improved Subgrade. The improved subgrade provides a stable construction platform for placement of the subsequent courses. All classes of roads and streets shall have an improved subgrade of the required thickness according to Section 54-2.01(f).

2. Subbase. The subbase serves two purposes. First, the subbase provides a separation layer between the rigid pavement and pumpable subgrade soils. Second, the subbase is there to resist erosion of the fine-graded soils during the service life of the pavement.

Treatment options for improved subgrade and requirements for subbase type and thickness are shown in Figure 54-4.D.

54-4.01(i) Designating Structural Information on Plans

See Section 63-4.05 for information on designating structural information on plans.

Facility Type	Subbase ①		Improved Subgrade Type ② ③
	Type	Minimum Thickness (inches)	
Class I			
Interstate / Freeway	HMA or PCC Stabilized	4	ASI, GM, or MS
Other Marked Routes	HMA or PCC Stabilized	4	ASI, GM, or MS
Unmarked Routes (TF ≥ 2.0)	HMA or PCC Stabilized	4	ASI, GM, or MS
Unmarked Routes (0.7 < TF < 2.0)	Not required	n/a	ASI, GM ④
Unmarked Routes (TF ≤ 0.7)	Not required	n/a	ASI, GM, or MS
Class II			
Marked Routes	HMA or PCC Stabilized	4	ASI, GM, or MS
Unmarked Routes (TF ≥ 2.0)	HMA or PCC Stabilized	4	ASI, GM, or MS
Unmarked Routes (0.7 < TF < 2.0)	Not required	n/a	ASI, GM ④
Unmarked Routes (TF ≤ 0.7)	Not required	n/a	ASI, GM, or MS
Class III			
Marked Routes	HMA or PCC Stabilized	4	ASI, GM, or MS
Unmarked Routes (TF ≥ 2.0)	HMA or PCC Stabilized	4	ASI, GM, or MS
Unmarked Routes (0.7 < TF < 2.0)	Not required	n/a	ASI, GM ④
Unmarked Routes (TF ≤ 0.7)	Not required	n/a	ASI, GM, or MS
Class IV			
Marked Routes	HMA or PCC Stabilized	4	ASI, GM, or MS
Unmarked Routes (TF ≥ 2.0)	HMA or PCC Stabilized	4	ASI, GM, or MS
Unmarked Routes (0.7 < TF < 2.0)	Not required	n/a	ASI, GM ④
Unmarked Routes (TF ≤ 0.7)	Not required	n/a	ASI, GM, or MS

Notes:

① *For urban sections containing curb and gutter and a storm sewer system, the designer may omit the stabilized subbase when an ASI or GM improved subgrade is used, regardless of the traffic factor.*

② *Improved Subgrade Types include:*

ASI - Aggregate Subgrade Improvement (mimimum of 12 in.)

GM – Granular over Modified Soil (4 in. CA 6 or CA 10 over 12 in. Modified Soil)

MS – Modified Soil (minimum of 12 in.)

③ *The minimum thickness of improved subgrade shall be according to Section 54-2.01(f).*

④ *Modified Soil may be used for the improved subgrade if a minimum 4-in. stabilized subbase is used.*

MINIMUM STRUCTURAL DESIGN REQUIREMENTS
(Rigid Pavement: Mechanistic Design)

Figure 54-4.D

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54-4.01(j) Thickness Design Procedure

For a mechanistic design of a rigid pavement, use the following steps to determine thickness:

1. Determine Traffic Factor. Use the following procedures to determine the traffic factor:

   a. Determine the facility class (e.g., Class I, II, III, or IV).

   b. Determine the actual structural design traffic as described in Section 54-2.01(c).

   c. Determine the minimum structural design traffic as described in Section 54-2.01(c) and Figure 54-2.C.

   d. Based on the facility class, select the appropriate traffic factor equation from Figure 54-4.C.

   e. Calculate the actual traffic factor.

   f. Calculate the minimum traffic factor.

   g. Compare the actual traffic factor to the minimum traffic factor and use the greater of the two as the traffic factor for design.

2. Determine the SSR. Determine the SSR as described in Section 54-2.01(f) (e.g., poor, fair, granular).

3. Determine the Edge Support. Determine the edge support condition to analyze as discussed in Section 54-4.01(c) (e.g., tied shoulder).

4. Determine the PCC Thickness. Use one of the following two procedures to determine the PCC thickness depending on the route class:

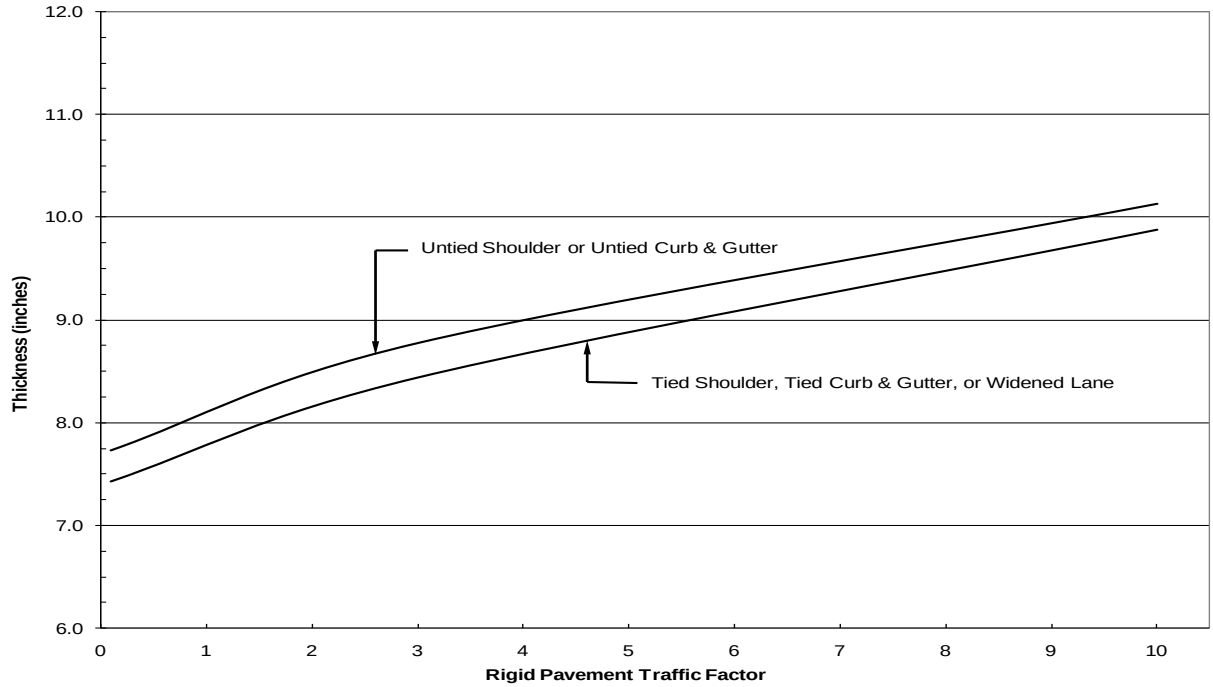
   a. For all Class I, II, III, and IV marked roads and streets, and for Class I, II, and III unmarked roads and streets, use the following procedures:

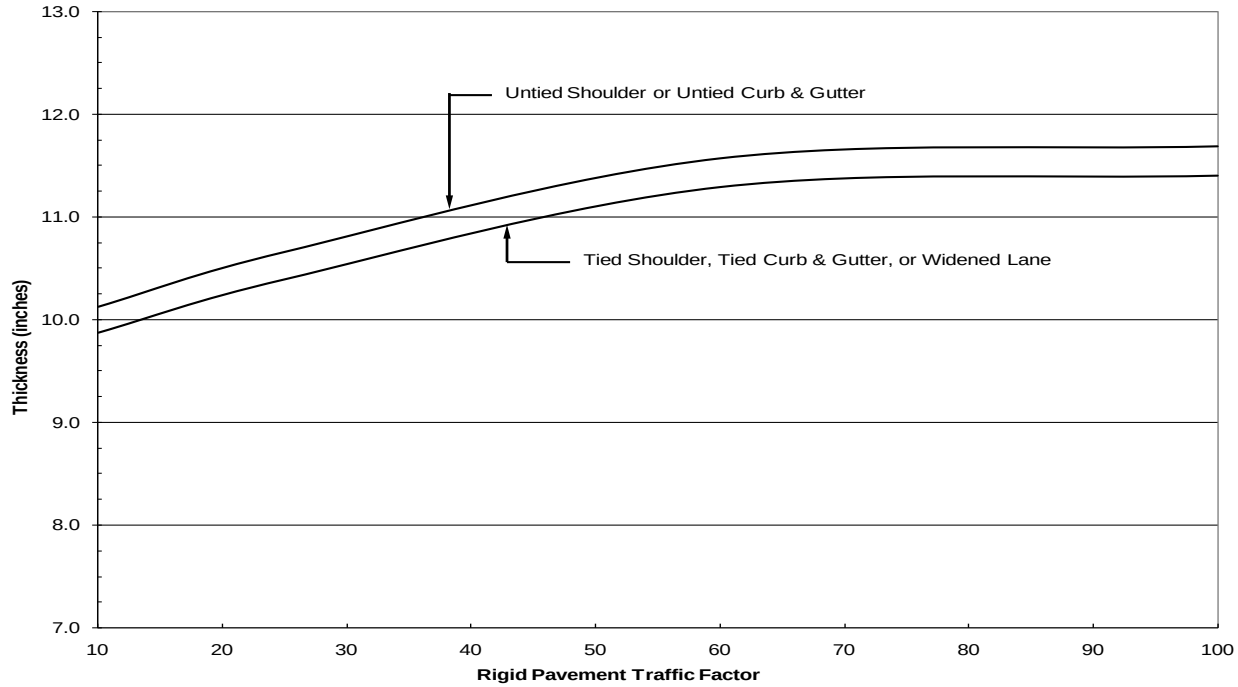
      (1) Based on the SSR (e.g., poor, fair, granular) determined in Step 2, select the appropriate rigid pavement mechanistic design chart from the following:

         - SSR = Poor, use Figure 54-4.E;
         - SSR = Fair, use Figure 54-4.F; or
         - SSR = Granular, use Figure 54-4.G.

      (2) Within the design chart, select the curve that represents the edge support condition, either tied or untied.

      (3) Enter the chart along the horizontal axis with the traffic factor determined in Step 1.





Note: Use of untied shoulder design requires BDE approval.

RIGID PAVEMENT DESIGN CHART
(Mechanistic Design: SSR = Poor)

Figure 54-4.E

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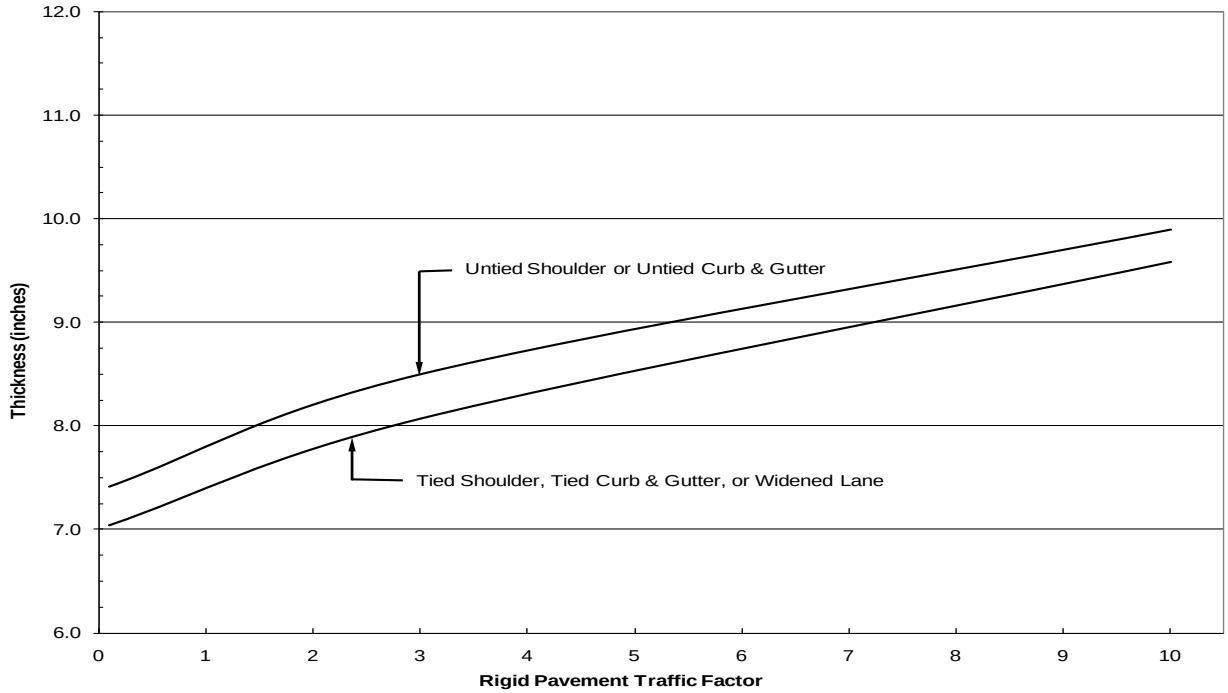
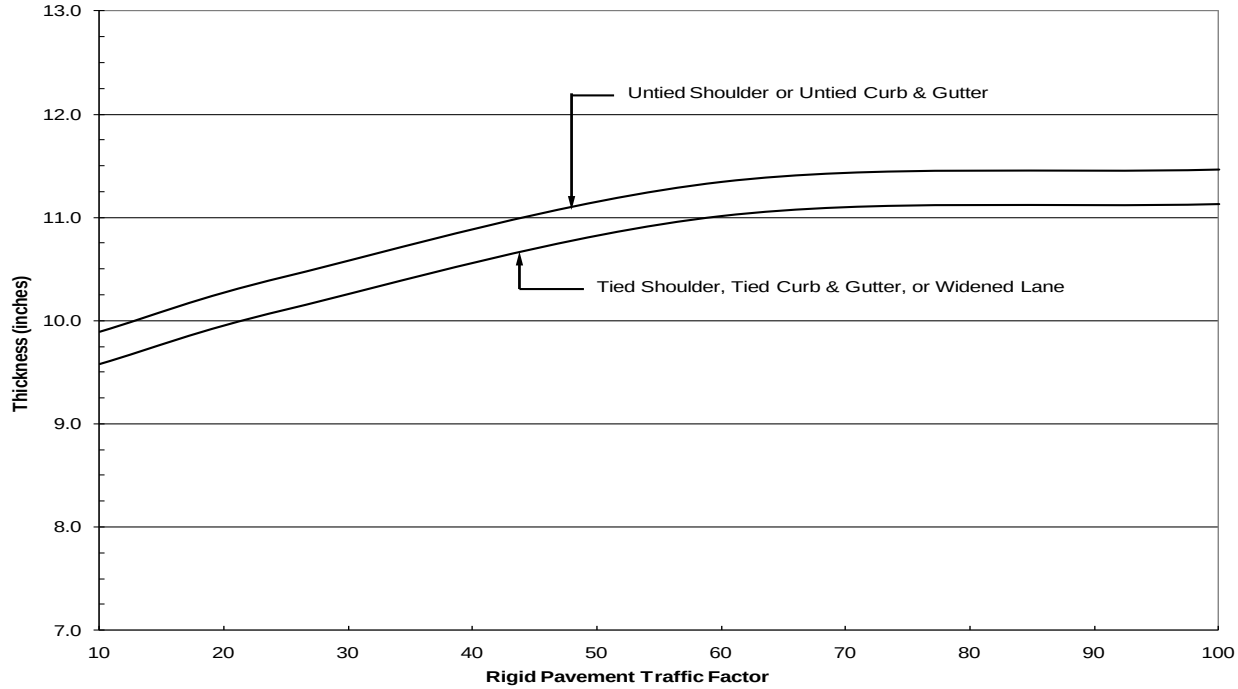
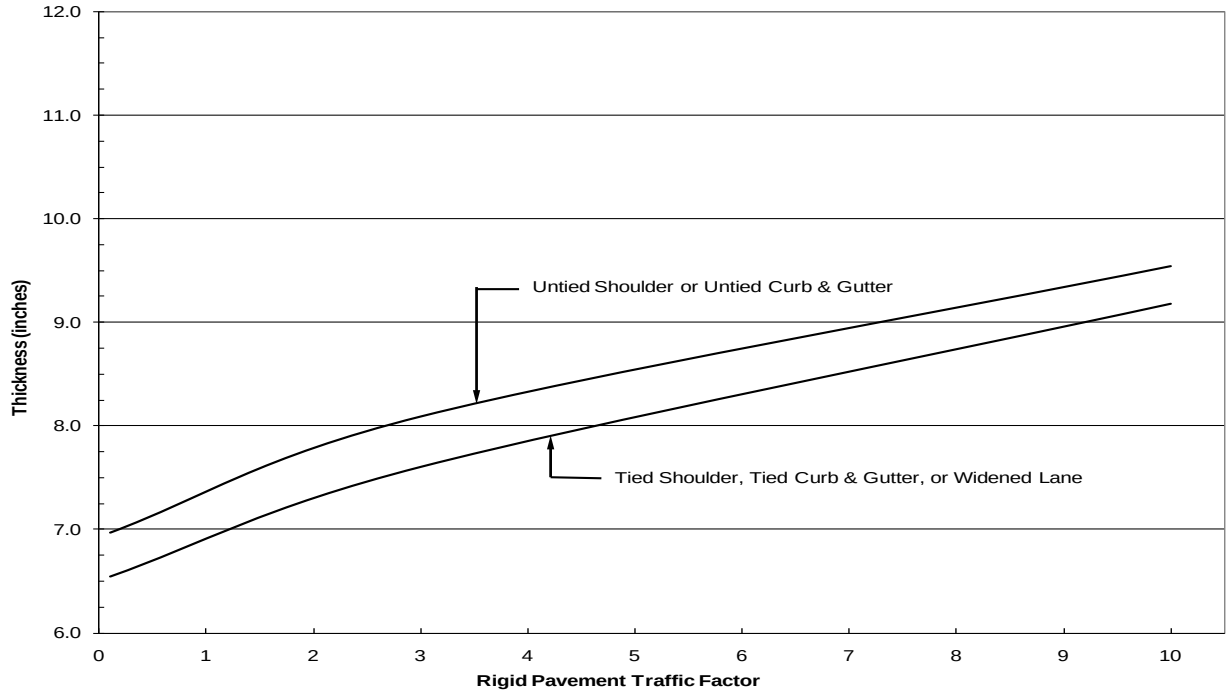


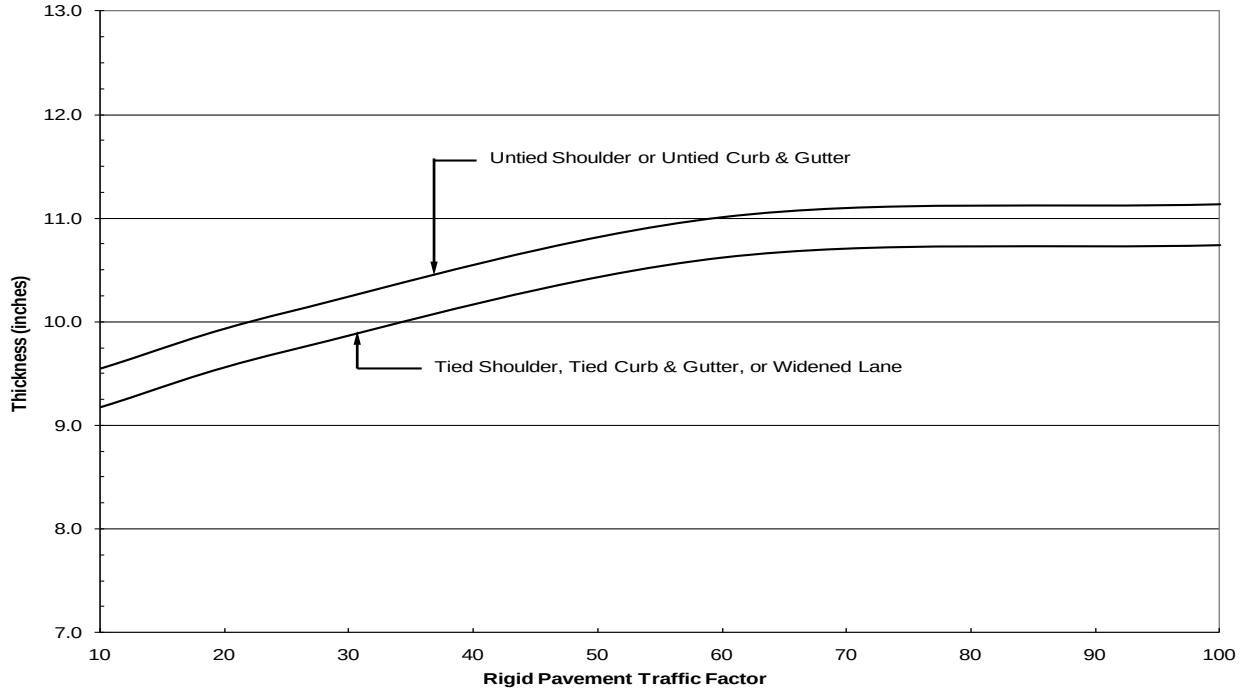


Note: Use of untied shoulder design requires BDE approval.

RIGID PAVEMENT DESIGN CHART
(Mechanistic Design: SSR = Fair)

Figure 54-4.F





Note: Use of untied shoulder design requires BDE approval.

RIGID PAVEMENT DESIGN CHART
(Mechanistic Design: SSR = Granular)

Figure 54-4.G

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(4) Move up vertically until the curve selected in Step a.(2) is intersected.

(5) From the point of intersection, move left horizontally until the vertical axis is intersected.

(6) Read the thickness from the chart's vertical axis.

(7) Round the thickness up to the nearest 0.25 in.

b. For Class IV Unmarked Routes, use the following procedures:

(1) Determine the thickness according to Figure 54-4.H using the standard case of untied shoulders, no subbase, poor subgrade (k=50 psi/in.), panel length of 12 ft, and no dowels.

Traffic Factor	Thickness
0.025	6.75
0.05	7.00
0.10	7.25
0.50	7.50
1.00	7.75
2.00	8.00

PCC PAVEMENT THICKNESS
CLASS IV UNMARKED ROADS AND STREETS

Figure 54-4.H

(2) Adjust the pavement thickness according to Figure 54-4.I to account for differences in the cross-section from the standard case.

Option Different from Standard Case	Thickness Adjustment (inches)
Tied Concrete Shoulders	- 0.25
4 in. Granular Subbase	none
4 in. Stabilized Subbase	- 0.25
15 ft Slab Length	+0.50
Fair Subgrade (k = 100 psi/in.)	- 0.25
Good Subgrade (k = 200 psi/in.)	- 0.50

THICKNESS ADJUSTMENTS FOR
CLASS IV UNMARKED ROADS AND STREETS

Figure 54-4.I

54-4.01(k) Shoulder Type/Design

For newly constructed or reconstructed rural projects, use a tied rigid shoulder with all mechanistic rigid pavement designs. The thickness of the shoulder will either match the pavement thickness or vary uniformly from the pavement thickness to a 6 in. minimum at the outside edge.

54-4.01(l) Design Example

See Section 54-9 for a design example.

54-4.01(m) Typical Sections

Figures 54-4.J through 54-4.L illustrate typical rural and urban pavement sections of rigid designs for various types of highway facilities.

54-4.01(n) Joint Placement

The proper placement of pavement joints greatly affects the overall performance of the pavement. Where joints are not properly designed, uncontrolled cracking can occur. Features that can affect joint spacing are turn lanes, drainage blockouts, medians, intersecting side streets, etc. This is especially true at large intersections. It is imperative that the designer and the construction engineer pay particular attention to joint placement in these areas.

Designers should provide jointing details in the plans, and construction engineers should make every effort to follow the joint layout. In many cases, it will be necessary to adjust the normal 15 ft spacing used by IDOT to accommodate other features. Joint spacing may be adjusted ±3 ft to match joints in intersecting pavements and/or to accommodate pavement discontinuities (e.g., drainage castings, median noses). In some cases, it also may be possible to adjust the location of pavement discontinuities so that they are located at a normal joint position. The American Concrete Pavement Association has published a bulletin entitled *Intersection Joint Layout*. Although this bulletin should not be construed as an IDOT policy or standard, it does contain helpful information on jointing intersections and can serve as a design reference.

The designer and construction engineer also must be aware of planned staged construction and its effect on joint layout. Where adjacent lanes are to be constructed in stages, it is important to plan the layout of the joints before any pavement is placed. Occasionally, new pavement is constructed adjacent to existing pavement that was designed using a different joint type and spacing. When this occurs, the designer must make provisions in the plans to match existing joints. A detailed joint survey prior to plan preparation can help eliminate problems during construction and throughout the life of the pavement.

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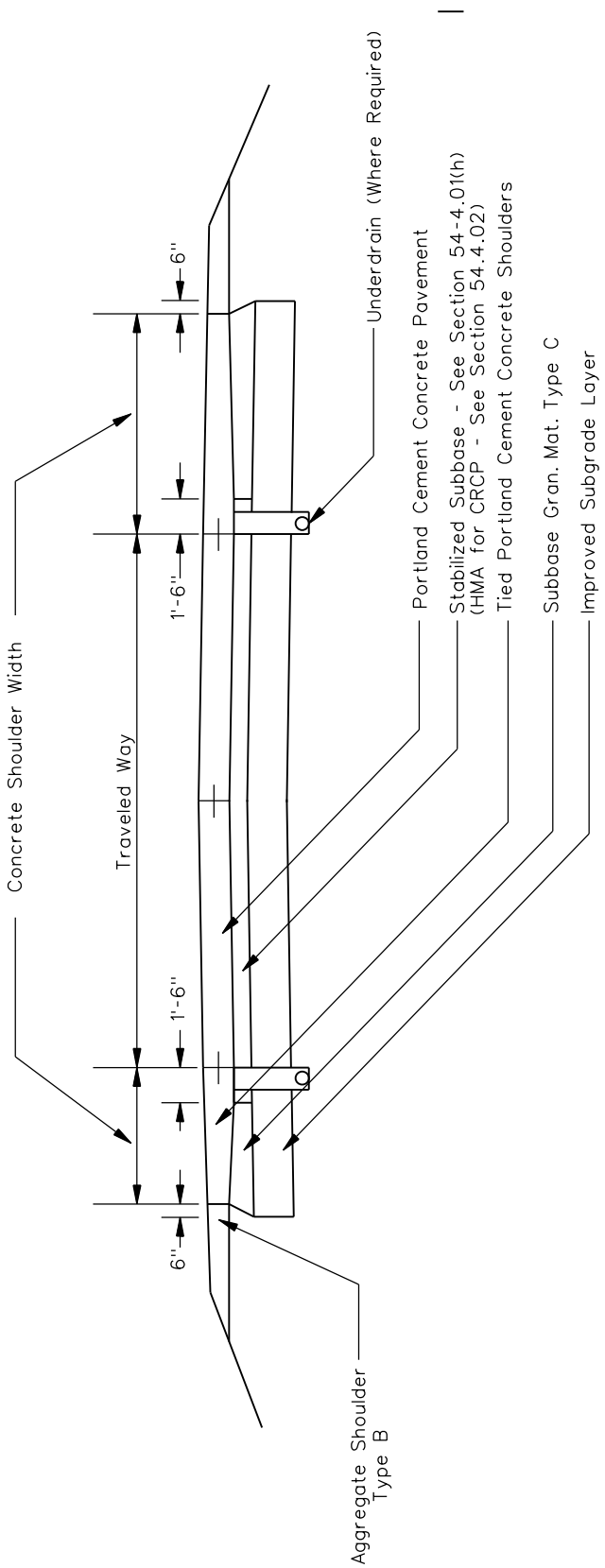
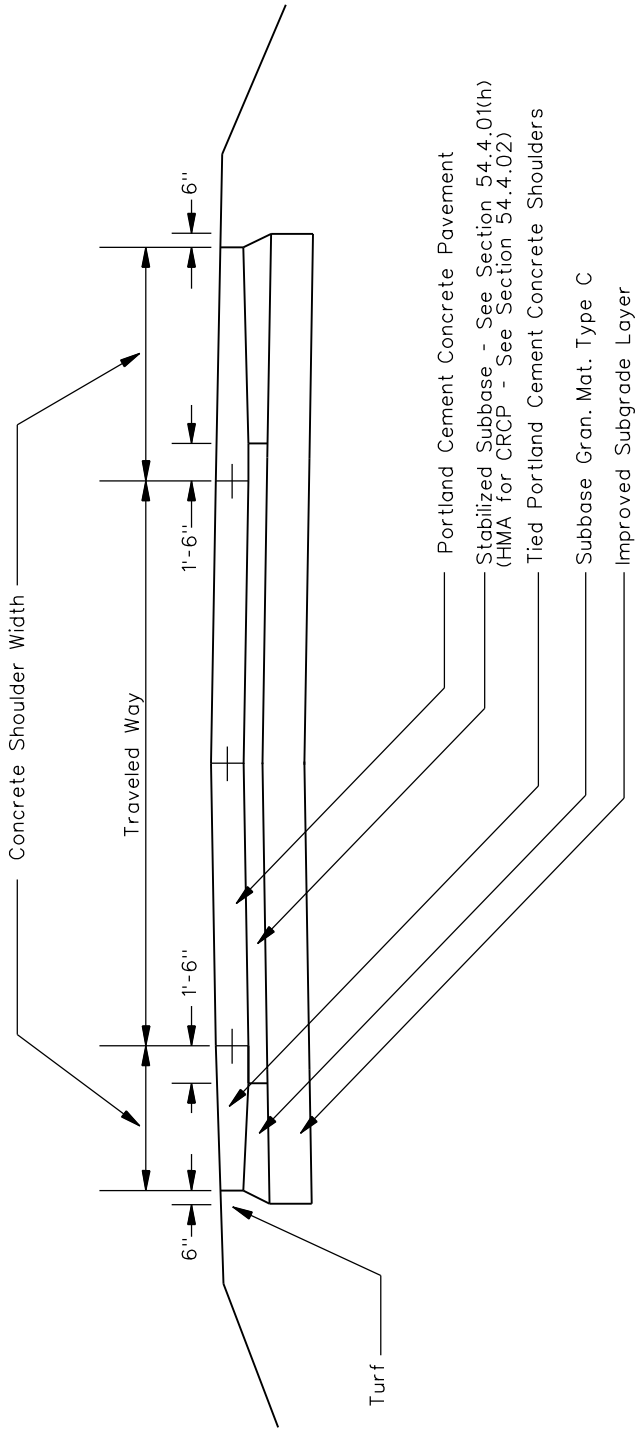


TYPICAL RURAL SECTION: RIGID DESIGN WITH TIED SHOULDER
(Trunk, Major Highways, Area Service Highways)

Figure 54-4.J



TYPICAL RURAL SECTION: RIGID DESIGN WITH TIED SHOULDER
(Collectors, Land Access Highways)

Figure 54-4.K

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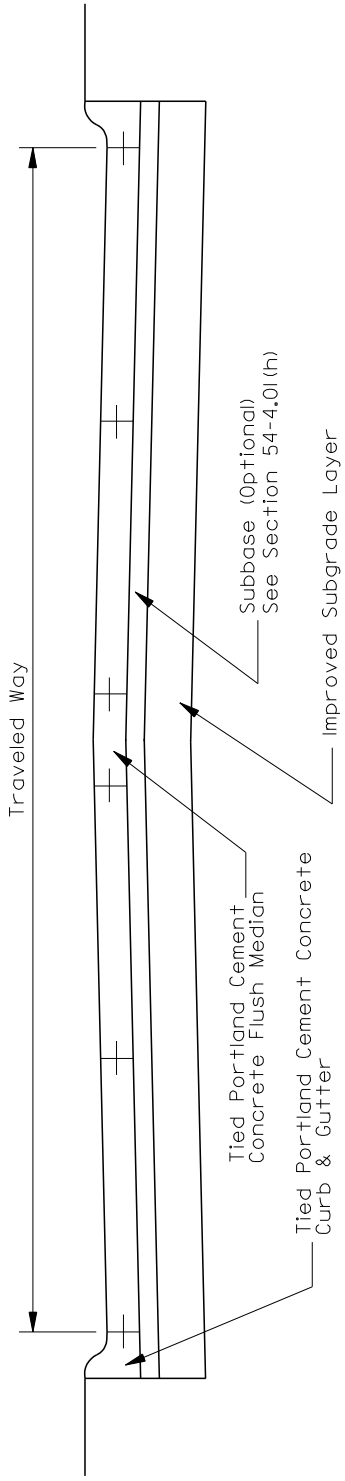

Note: Raised median with tied PCC curb and gutter may be use din lieu of a flush median.

TYPICAL URBAN SECTION: RIGID DESIGN WITH TIED CURB AND GUTTER
Figure 54-4.L

54-4.01(o) Surface Finish

The surface finish of a pavement provides skid resistance for the traveling public. The type of finish is dictated by the posted speed limit. As the posted speed limit increases, so does the need for higher skid resistance. The type of surface finish shall be indicated on the plans according to the following guidelines:

- Final finishing (of surfaces) on highways with posted speed limits in excess of 40 mph will be a Type A final finish as outlined in the *Standard Specifications for Road and Bridge Construction*.

- Final finishing (of surfaces) on highways with posted speed limits not exceeding 40 mph will be a Type A or Type B final finish as outlined in the *Standard Specifications for Road and Bridge Construction*.

54-4.02 <u>Modified AASHTO</u>

54-4.02(a) Application of Design Method

The modified AASHTO design procedures are used for JRCP and CRCP. These procedures provide a historical reference that enables the designer to determine the thickness of JRCP required where it is necessary to match existing JRCP which was constructed using these designs. These procedures are also used to design new or match existing CRCP. Use the procedures presented in Section 54-4.02(e) to determine pavement type and thickness and provide a subbase in accordance with Section 54-4.02(f). For JRCP, the maximum joint spacing will be 50 ft. CRCP designs should typically be used if the rigid traffic factor is greater than 60. CRCP designs for projects with rigid traffic factors greater than 60 will not require LCCA.

54-4.02(b) Design Period

Section 54-4.01(e) applies to modified AASHTO designs for rigid pavements.

54-4.02(c) Equivalency Factors

Section 54-4.01(f) applies to modified AASHTO designs for rigid pavements.

54-4.02(d) Traffic Factors

Section 54-4.01(g) applies to modified AASHTO designs for rigid pavements.

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54-4.02(e) Pavement Type and Thickness

For a modified AASHTO design of a rigid pavement, use the following steps to determine thickness:

1. Determine Traffic Factor. Use the following procedures to determine the traffic factor:

   a. Determine the facility class (e.g., Class I, II, III, or IV).

   b. Determine the actual structural design traffic as described in Section 54-3.01(c). Note that the minimum traffic as described in Section 54-2.01(c) does not apply to the modified AASHTO design.

   c. Based on the facility class, select the appropriate traffic factor equation from Figure 54-4.C.

   d. Calculate the actual traffic factor for design.

2. Determine the IBR. Determine the IBR as discussed in Section 54-3.01(f).

3. Determine the PCC Thickness. Use the following procedures to determine the PCC thickness:

   a. Based on the facility class determined in Step 1a, select the appropriate design nomograph from the following:

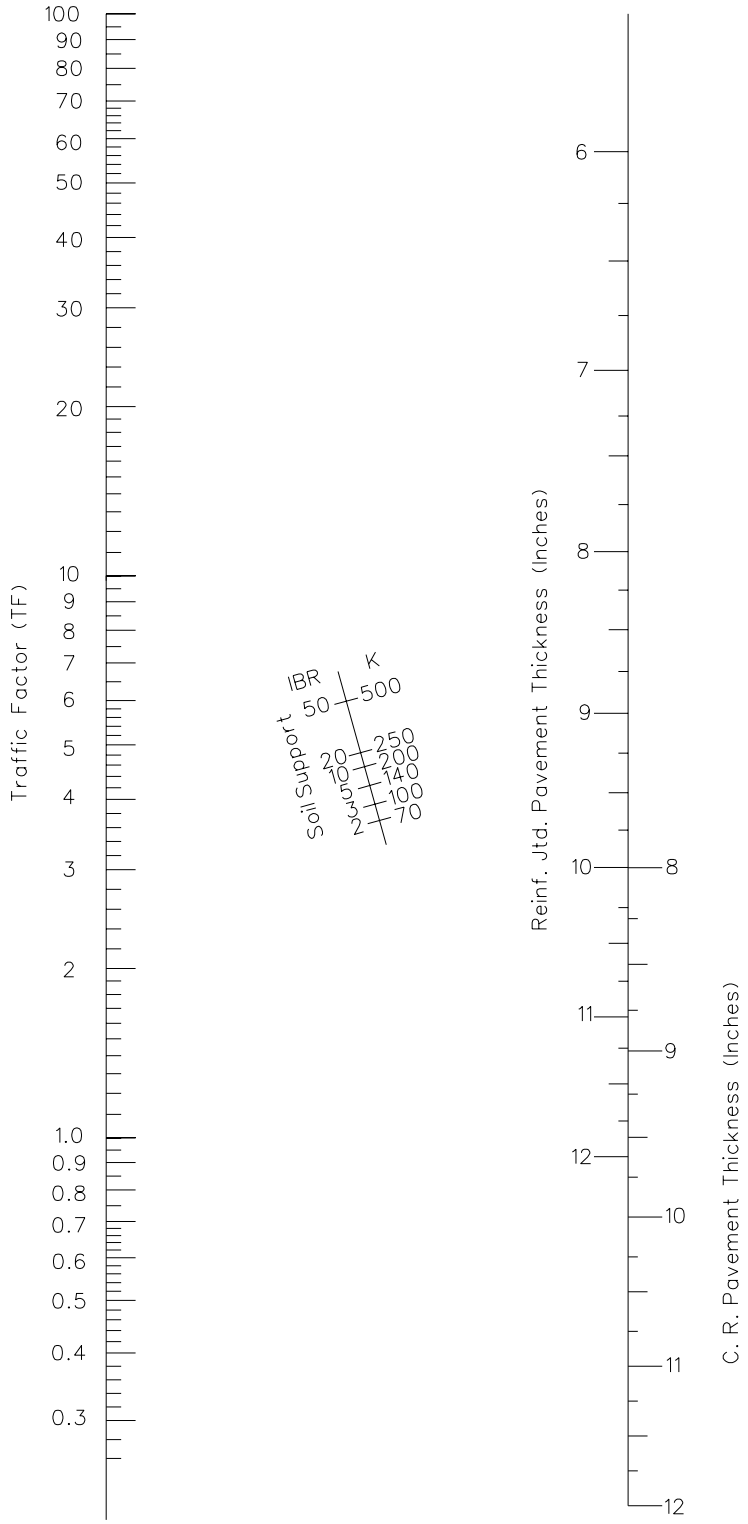
      - Class I Facilities; use Figure 54-4.M; or
      - Class II, III, and IV Facilities; use Figure 54-4.N.

   b. Within the nomograph, project a line from the traffic factor determined in Step 1 through the IBR of the roadbed soil determined in Step 2 and intersect the pavement type and thickness axis of the nomograph.

   c. Read the pavement type from the point of intersection on the nomograph (i.e., either JRCP or CRCP). Note that CRCP is used only if the traffic factor is greater than 60.

   d. Read the PCC thickness from the point of intersection on the nomograph and round the thickness up to the nearest 0.25 in.

   e. If design inputs produce values which exceed the chart's, the design's parameters should be forwarded to BDE for design.



RIGID PAVEMENT DESIGN NOMOGRAPH
(Modified AASHTO Design: Class I Facilities: JRCP and CRCP)

Figure 54-4.M

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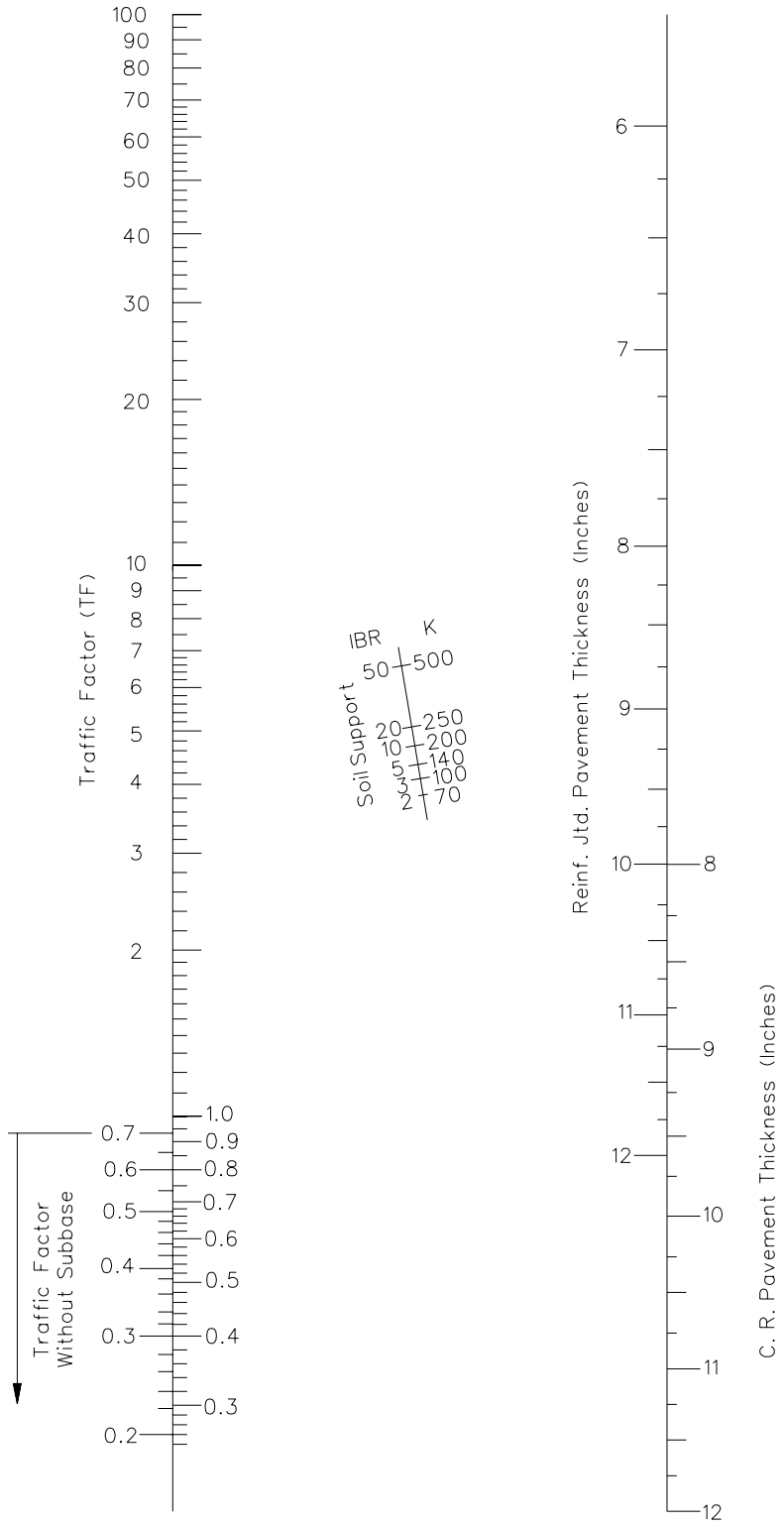

RIGID PAVEMENT DESIGN NOMOGRAPH
(Modified AASHTO Design: Class II, III, and IV Facilities: JRCP and CRCP)

Figure 54-4.N

54-4.02(f) Subbase Type and Thickness

Section 54-4.01(h) applies to modified AASHTO designs for rigid pavements, except the stabilized subbase shall be constructed of HMA for CRCP.

54-4.02(g) Minimum Structural Design Requirements

To ensure that practical and adequate designs are developed, the minimum structural design requirements presented in Figure 54-4.O have been established. These minimum criteria will govern in all cases where the calculated pavement thickness is less than the desired minimum.

54-4.02(h) Designating Structural Information on Plans

See Section 63-4.05 for information on designating structural information on plans.

54-4.02(i) Joints and Concrete Lug End Anchorages

The following guidelines should be used when CRCP is placed between structures that are in close proximity to each other.

Where the slab length of CRCP between bridges or other pavement types is less than 1,500 ft, use a doweled expansion joint. Where the slab length is between 1,500 ft and 2,000 ft, contact BDE. For sections of CRCP longer than 2,000 ft, use a lug system. A wide-flange beam terminal may be used in place of a lug system.

54-4.02(j) Design Example

See Section 54-9 for a design example.

54-4.02(k) Typical Sections

See Section 54-4.01(m) for typical rural and urban pavement sections of rigid designs for various types of highway facilities.

54-4.02(l) Surface Finish

See Section 54-4.01(o) for guidance on the type of surface finish that is required.

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Facility Type	PCC Pavement		Subbase⑤	
	Type①	Minimum②③ Thickness (inches)	Type	Minimum Thickness (inches)
Class I	JRCP④	10	Stabilized Granular Material	4
Class II	JRCP	8	Stabilized Granular Material	4
Class III State Primary All Others	JRCP	8 7	Stabilized Granular Material Granular Material⑥	4
Class IV	JRCP	6	Granular Material⑥	4

Notes:

① *JRCP should be used only to match existing pavement.*

② *For municipal streets having curb and gutter and storm sewer system, and for which maintenance responsibility is borne entirely by the Municipality, the minimum pavement thickness will be 6 in. In no case will the design thickness be less than that provided by Figures 54-4.M and 54-4.N.*

③ *Where the design thickness equals 6 in. or 7 in., standard reinforcement may be omitted at the option of the designer provided sawed transverse contraction joints are spaced no greater than 20 ft apart.*

④ *A structurally equivalent CRCP should typically be used if the traffic factor is greater than 60 as per Section 54-4.02 If CRCP is used, provide an HMA stabilized subbase.*

⑤ *A subbase will not be required for urban sections having curb and gutter and a storm sewer system. However, at the designer's option, a granular subbase may be used to serve as a working platform where poor soil conditions exist. The thickness of the subbase will be determined by the designer in accordance with this figure.*

⑥ *Pavement for Class III facilities (other than State primary highways) and for Class IV facilities may be designed with or without a subbase, at the designer's option, provided that the traffic factor is less than or equal to 0.7.*

MINIMUM STRUCTURAL DESIGN REQUIREMENTS
(Rigid Pavement: Modified AASHTO Design)
Figure 54-4.O

54-4.03 Unbonded Concrete Overlay

54-4.03(a) Limitations

Unbonded concrete overlays (UCOs) have a proven history in Illinois. Research in Illinois has shown that UCOs are a viable alternative to reconstruction when a pavement has deteriorated beyond the point that a standard patch and overlay will not perform. However, some pavements may not be the best candidates for UCOs. Pavements with existing HMA and/or undoweled PCC patches are not good candidates. The UCO is not able to bridge over unstable slabs or rocking or pumping patches. UCOs are not experimental; however, use of this type of pavement requires approval from BDE using economic justification.

54-4.03(b) Application of Design Method

A UCO consists of an existing concrete pavement, an interlayer, and a JPCP or CRCP overlay. The overlay relies on minimal structural contribution from the existing pavement. Essentially, the two layers function independently. The existing pavement acts as the subbase. The interlayer separates the two pavements. The interlayer retards reflective cracking in the overlay. HMA is an effective interlayer. A minimum HMA interlayer thickness of 4 in. is recommended.

54-4.03(c) Pavement Type and Thickness

When designing a UCO, use the same design parameters for design period, traffic factors, and pavement thickness as shown in Section 54-4.02 for rigid pavements. If using a CRCP, determine the UCO thickness by calculating the thickness of a new CRCP and subtracting 1 in.

Though several other States use JPCP pavements for UCOs, the Department does not have any experience with designing or constructing a UCO with that type of pavement. If the district is interested in using a JPCP for the UCO, the project will require assistance from BMPR for the design of the overlay and an experimental feature according to Construction Memorandum 02-2.

54-4.03(d) Special Considerations/Comments

Consider grade alignment over at-grade structures and vertical clearance between pavement and overhead structures when selecting this rehabilitation method. Long-term planning may be necessary to ensure that structures have sufficient clearance to accommodate a UCO. Due to the increase in pavement grade, side slopes must be modified. This increased slope may require variances from existing policy. Such variances must be approved by the BDE. Terminal treatments (e.g., lug systems, wide-flange beams, and special treatments that taper into existing sections) may need to be detailed to connect the adjacent pavement or bridge section to the overlay.

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Contact the BMPR for assistance in developing UCO designs (i.e., overlay and interlayer thickness requirements, UCO thickness, terminal treatments). The suitability of a UCO depends on many factors, and each set of conditions warrants an individualized design. Costs must be considered on a case-by-case basis. Rural sections without overhead structures are ideal locations for UCOs because vertical clearance will not become an issue. Contact the Engineer of Pavement Technology in the BMPR for additional information.

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54-5 STRUCTURAL DESIGN OF FLEXIBLE PAVEMENTS

54-5.01 Mechanistic

54-5.01(a) Limitations

Thickness designs may be obtained for traffic factors ranging from 0.5 to approximately 100 and for pavement thicknesses ranging from 6 in. to 18 in. However, the absolute minimum traffic factor of 0.5 will control the design thickness at the lower limits and the Limiting Strain Criterion Design Thickness will control the maximum design thickness. Limiting Strain Criterion Design thicknesses will only be allowed on those projects for which the design thickness calculated in Figures 54-5.F, 54-5.G, or 54-5.H exceeds the thickness shown in Figure 54-5.I. Approval for use of Limiting Strain Criterion designs is required from BDE.

54-5.01(b) Minimum Material Quality

The mechanistic full-depth HMA design procedures require, and are limited to, the use of HMA surface and binder courses with 4% air voids. HMA (4% voids, $N_{design} \geq 90$) mixtures will be specified for Interstates and freeways. HMA (4% voids, $N_{design} \leq 70$) mixtures may be specified for all other highway classifications. Any combination of surface course and binder course may be used to total the design HMA thickness. However, for the purpose of providing the most economical design, a surface course thickness of 2 in. should be used for new construction (see Section 54-5.01(i)). If there is any question as to the use of any HMA mixture in the procedure, contact the Bureau of Materials and Physical Research. HMA mixture design criteria must be met as outlined in Section 53-4.07.

54-5.01(c) Asphalt Binder Selection

The mechanistic design procedures give the designer the option of selecting an appropriate asphalt binder type within the limits of current IDOT policy. The designer should be aware that in northern Illinois the use of a softer asphalt binder will reduce the effects of thermal cracking. On lower volume roads, a softer asphalt binder may be desired to reduce weathering and raveling and improve durability. On high volume roads and in areas of slower moving or standing loads, a stiffer asphalt binder should be used. In some cases, it may be desirable to use one asphalt binder grade for the binder course and another asphalt binder grade for the surface course. Where more than one asphalt binder grade is used, the design thickness for the HMA binder course asphalt binder grade will be used. Information on design asphalt binder grade selection is provided in Section 53-4.07(c). The design asphalt binder grade will be provided by the district and will be noted on the plans.

Project location and traffic volume are the main factors affecting the performance of HMA pavements. Consult with the district materials engineer to determine the proper asphalt binder grade.

54-5.01(d) Application of Design Method

The mechanistic design procedures enable the designer to determine the material types and thicknesses for the various layers of a flexible pavement that are required to carry a specified volume and composition of traffic for a designated period of time while retaining a serviceability level at or above a selected minimum value. Use the procedures presented in Section 54-5.01(i) to determine the thickness design for full-depth HMA pavement.

54-5.01(e) Design Period

The design period for all Class I and Class II roads and streets and for Class III State primary highways is typically 20 years. Other Class III and all Class IV roads and streets may be designed for less than 20 years.

54-5.01(f) Equivalency Factors

Section 54-2.01(d) describes the use of equivalency factors to convert mixed-traffic loadings to 18-kip ESAL applications. Equivalency factors for flexible pavements are given in Figure 54-5.A. These equivalency factors have been used to develop the equations presented in Section 54-5.01(g).

Facility Class	18-kip ESAL Applications Per Vehicle		
	PV	SU	MU
Class I	0.0004	0.363	1.322
Class II	0.0004	0.307	1.056
Class III and IV	0.0004	0.299	1.053

EQUIVALENCY FACTORS
(Flexible Pavements)

Figure 54-5.A

54-5.01(g) Traffic Factor

The traffic factor is the projected total 18-kip ESALs, expressed in millions, to be carried by the design lane during the design period. Figure 54-5.B presents the traffic factor equations that should be used for flexible pavement designs.

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Facility Class	Traffic Factor Equation	Equation Number
Class I	$TF=DP\left[\frac{(0.15 \bullet P \bullet PV)+(132.50 \bullet S \bullet SU)+(482.53 \bullet M \bullet MU)}{1x10^6}\right]$	Equation 54-5.1
Class II	$TF=DP\left[\frac{(0.15 \bullet P \bullet PV)+(112.06 \bullet S \bullet SU)+(385.44 \bullet M \bullet MU)}{1x10^6}\right]$	Equation 54-5.2
Class III and Class IV	$TF=DP\left[\frac{(0.15 \bullet P \bullet PV)+(109.14 \bullet S \bullet SU)+(384.35 \bullet M \bullet MU)}{1x10^6}\right]$	Equation 54-5.3

where:
DP = design period in number of years.
PV, SU, MU = structural design traffic expressed as the number of PV, SU, and MU vehicles.
P, S, M = percent of PV, SU, and MU in the design lane expressed as a decimal.

TRAFFIC FACTOR EQUATIONS
(Flexible Pavements)

Figure 54-5.B

54-5.01(h) Improved Subgrade

The improved subgrade serves as a stable construction platform for placement of the subsequent layers of HMA pavement. All classes of roads and streets shall have an improved subgrade of the required thickness according to Section 54-2.01(f). Treatment options for improved subgrade are shown in Figure 54-2.D.

54-5.01(i) Thickness Design Process for Full-Depth HMA

For a mechanistic design of a flexible pavement, use the following steps to determine thickness:

1. Determine Traffic Factor. Use the following procedures to determine the traffic factor:

   a. Determine the facility class (e.g., Class I, II, III, or IV) and the design period (see Section 54-5.01(e)).

   b. Determine the actual structural design traffic as described in Section 54-2.01(c).

   c. Determine the minimum structural design traffic as described in Section 54-2.01(c) and shown in Figure 54-2.C.

   d. Based on the facility class, select the appropriate traffic factor equation from Figure 54-5.B.

e. Calculate the actual traffic factor.

f. Calculate the minimum traffic factor.

g. Compare the actual traffic factor to the minimum traffic factor and select the greater of the two. Compare the resulting traffic factor to 0.5. If less than 0.5, use 0.5 as the absolute minimum traffic factor for design.

2. Determine the SSR. Determine the SSR as described in Section 54-2.01(f) (e.g., poor, fair, granular).

3. Determine the Asphalt Binder Grade. Determine the asphalt binder grade for the lower binder, upper binder, and surface lifts as discussed in Section 54-5.01(c).

4. Determine the HMA Mixture Temperature. Use the following steps to determine the HMA mixture temperature:

   a. On Figure 54-5.C, identify and mark the location where the pavement section will be constructed.

   b. Using the temperature contours on Figure 54-5.C, interpolate the temperature and round up to the nearest 0.5°F at the marked location, except that the minimum HMA mixture temperature will be 73°F.

5. Determine HMA Mixture Modulus (E_{HMA}). Use the following procedures to determine the HMA mixture modulus (E_{HMA}) for pavement design:

   a. Along the horizontal axis of Figure 54-5.D, locate the value of the HMA mixture temperature determined in Step 4.

   b. Move up vertically and intersect the curve that represents the asphalt binder grade for the lower binder lifts determined in Step 3.

   c. Move left horizontally from the point of intersection on the asphalt binder grade curve and intersect the vertical axis of the HMA mixture modulus (E_{HMA}).

   d. At the point of intersection with the vertical axis, read the HMA mixture modulus (E_{HMA}) and round the value to the nearest 10 ksi. This will be the HMA mixture modulus for use in the design of the flexible pavement thickness (see Figures 54-5.F, 54-5.G, or 54-5.H).

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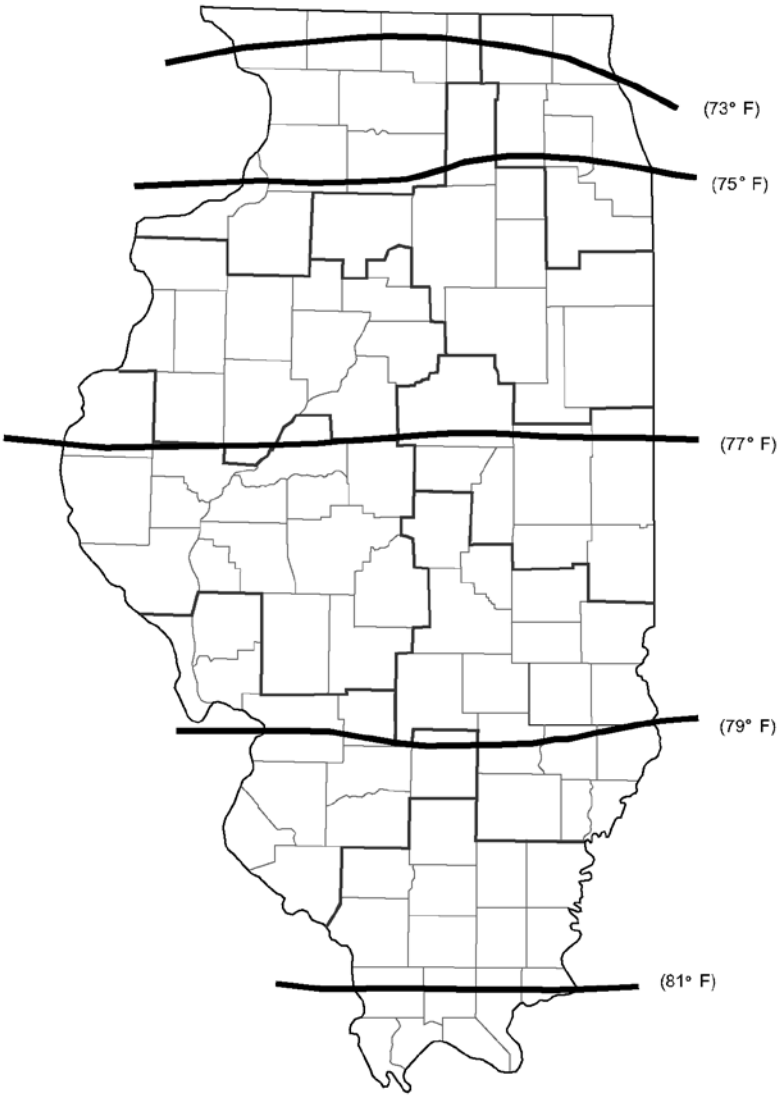
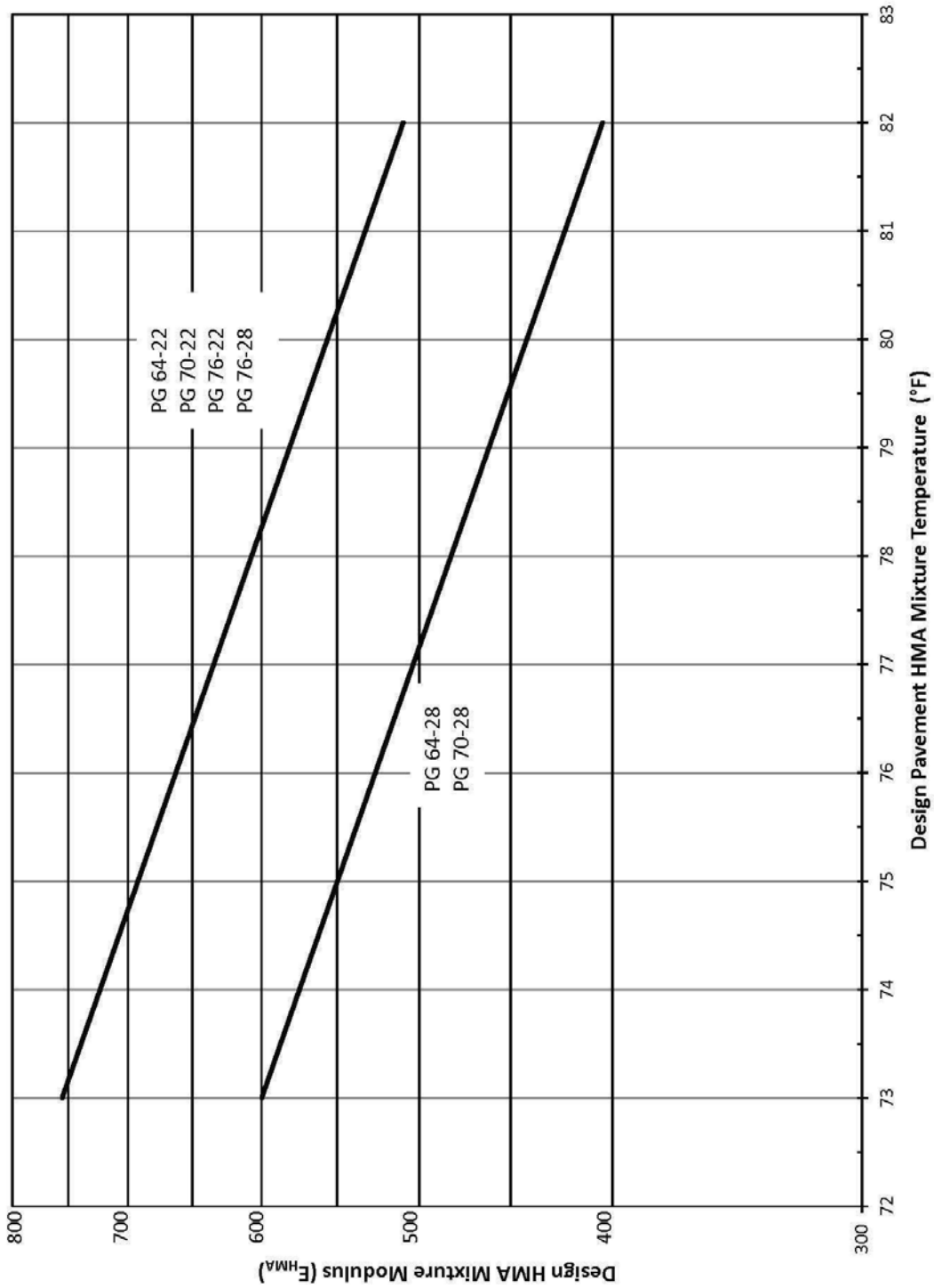


Note: The minimum design HMA mixture temperature will be 73°F.

HMA MIXTURE TEMPERATURE
(Mechanistic Design: Flexible Pavement)

Figure 54-5.C



HMA MIXTURE MODULUS (E_{HMA})
(Mechanistic Design: Flexible Pavement)

Figure 54-5.D

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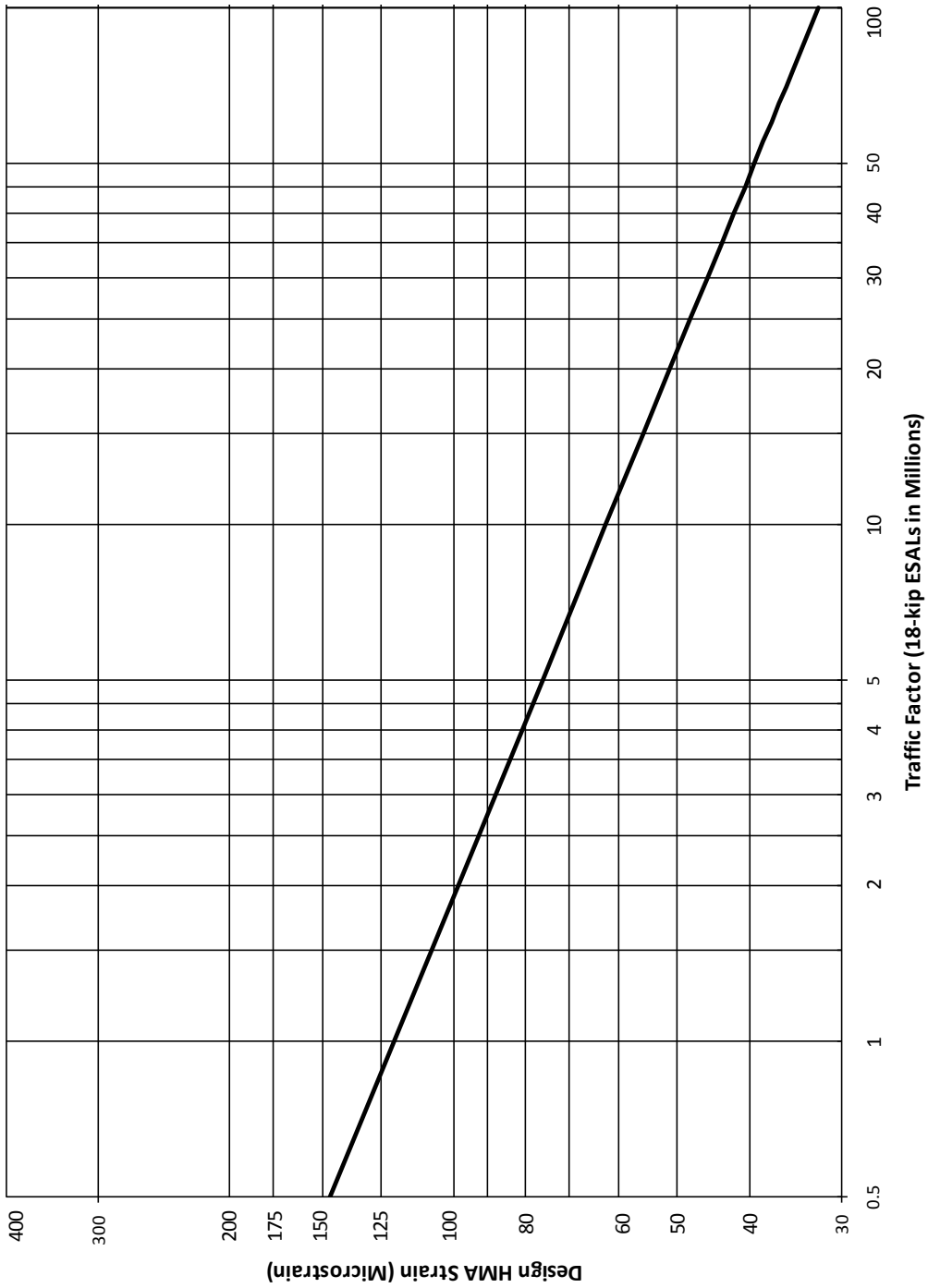
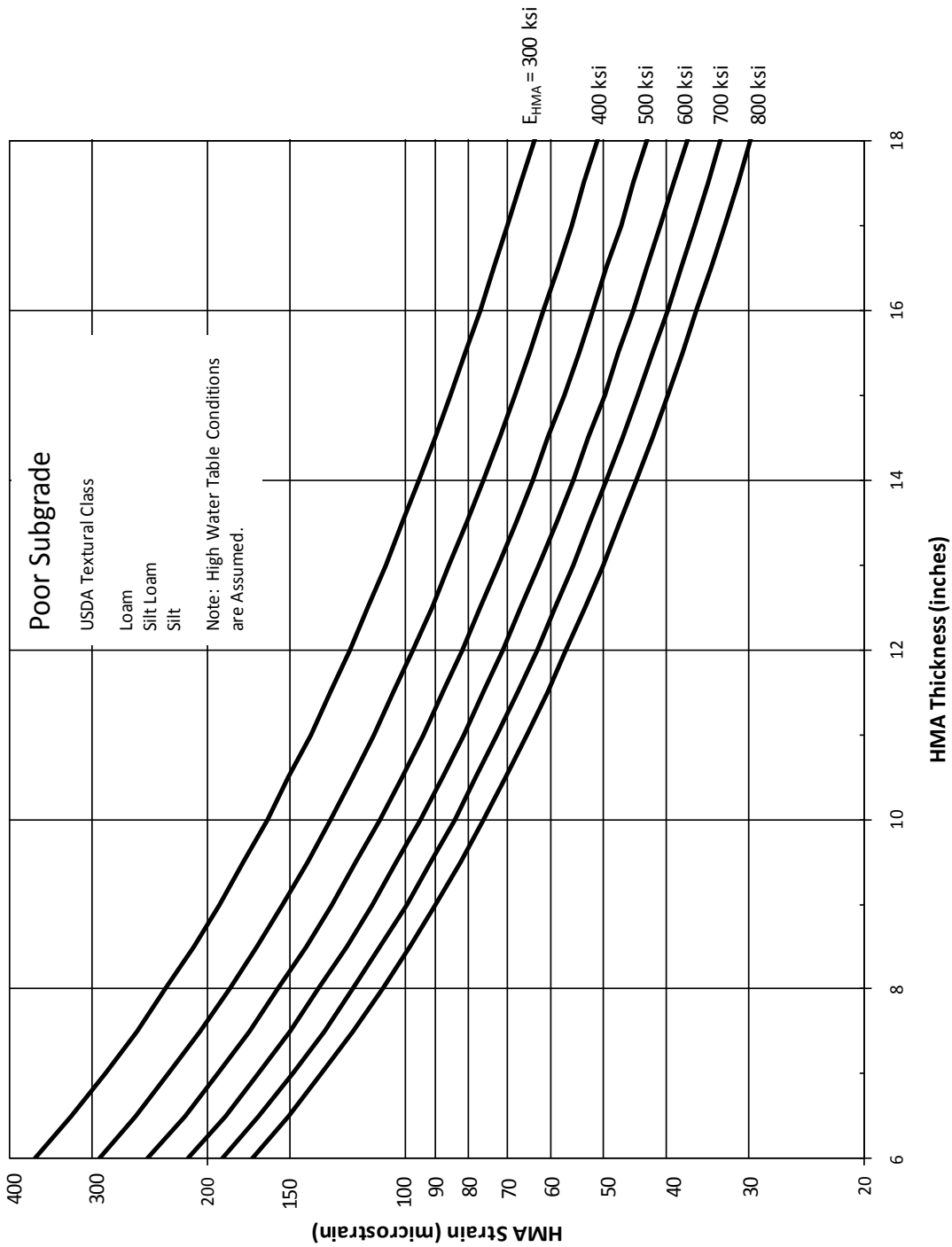


DESIGN HMA STRAIN
(Mechanistic Design: Flexible Pavement)

Figure 54-5.E



HMA THICKNESS DESIGN CHART
(Mechanistic Design: Flexible Pavement: SSR = Poor)

Figure 54-5.F

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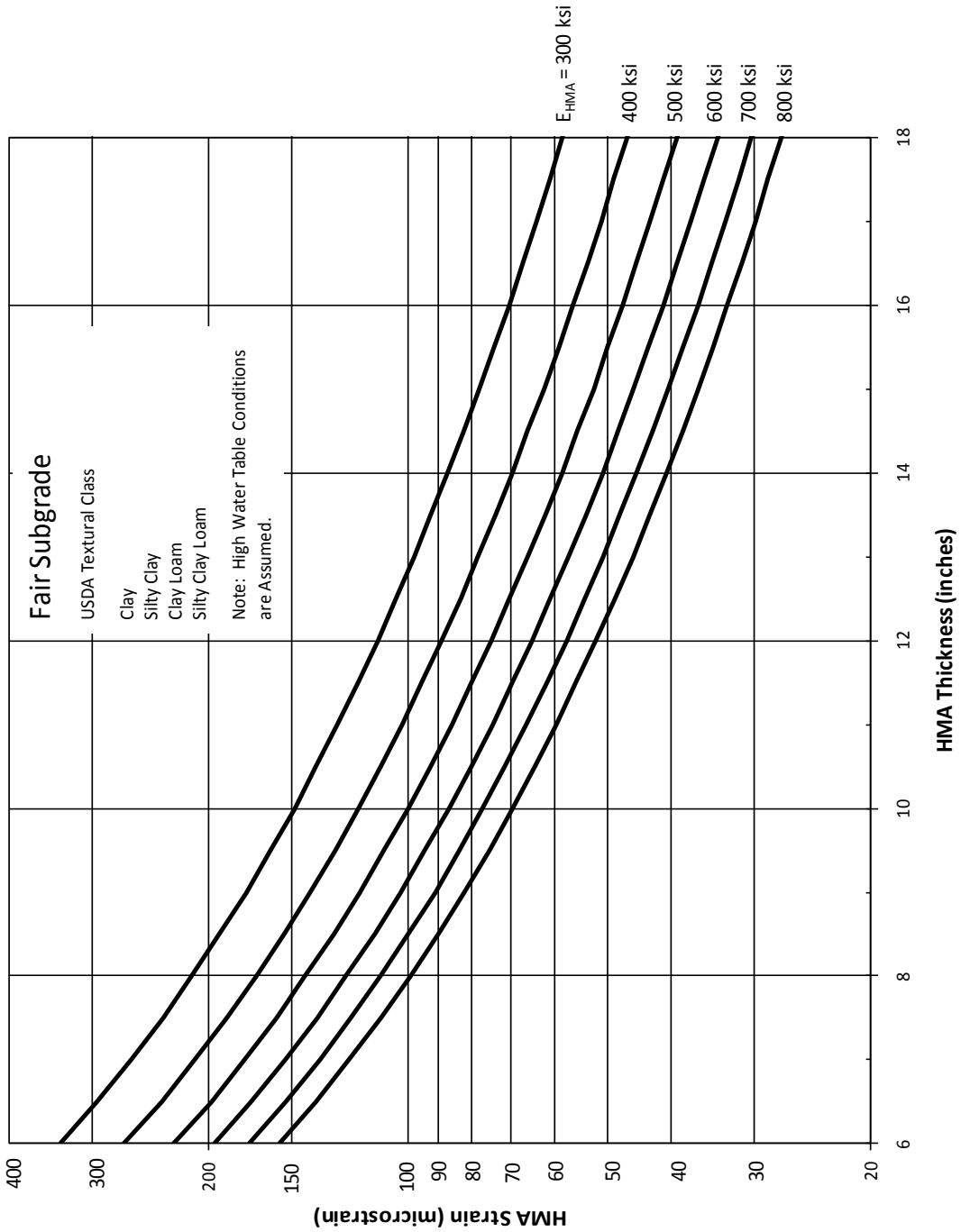
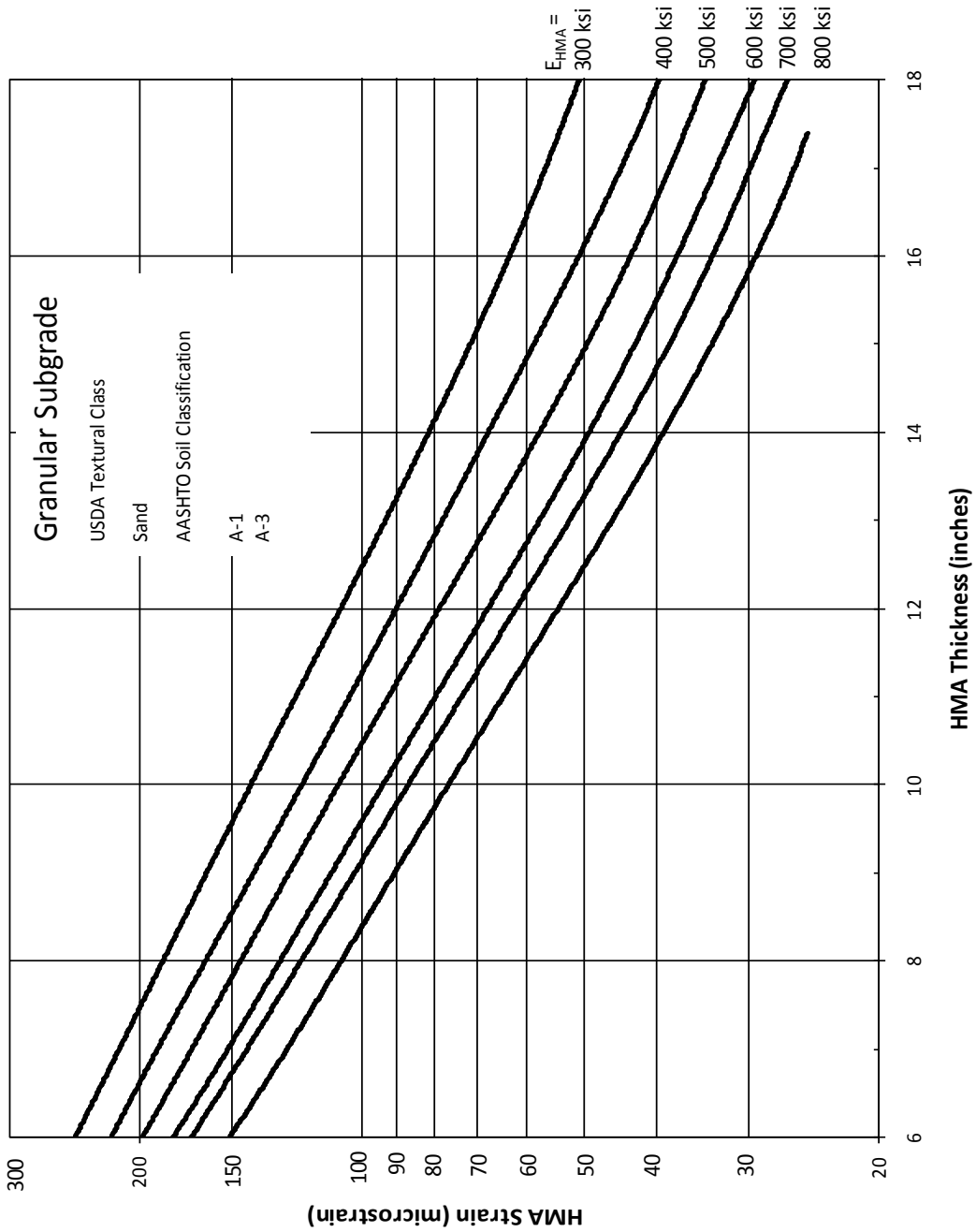


HMA THICKNESS DESIGN CHART
(Mechanistic Design: Flexible Pavement: SSR = Fair)

Figure 54-5.G



HMA THICKNESS DESIGN CHART
(Mechanistic Design: Flexible Pavement: SSR = Granular)

Figure 54-5.H

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6. Determine Design HMA Strain. Use the following procedures to determine the design HMA strain:

   a. Along the horizontal axis of Figure 54-5.E, locate the value of the traffic factor determined in Step 1.

   b. Move up vertically and intersect the curve.

   c. Move left horizontally from the point of intersection on the curve and intersect the vertical axis of the design HMA strain (i.e., microstrain).

   d. At the point of intersection with the vertical axis, read the design HMA strain and round the value to the nearest unit microstrain. This will be the HMA strain to use in the design of the flexible pavement thickness (see Figures 54-5.F, 54-5.G, or 54-5.H).

7. Determine Pavement Thickness. Use the following procedure to determine the pavement thickness:

   a. Using the SSR determined in Step 2 (e.g., poor, fair, granular), select from Figures 54-5.F, 54-5.G, or 54-5.H, the one chart that represents the SSR for use in design.

   b. On the chart selected in Step 7a, there is a set of six curves that represents HMA mixture moduli (E_{HMA}) from 300 ksi to 800 ksi. Using the value of E_{HMA} that was determined in Step 5, interpolate and draw a parallel curve that will represent the HMA mixture modulus for use in design.

   c. Along the left vertical axis of the selected chart, locate the value of the design HMA strain determined in Step 6.

   d. Move right horizontally and intersect the design E_{HMA} curve interpolated and drawn in Step 7b.

   e. Move down vertically and intersect the horizontal axis of the selected chart.

   f. Read the thickness from the point of intersection on the horizontal axis and round this value up to the nearest 0.25 in. This will be the total thickness for the pavement design.

   g. Use the following guidelines to determine the proper surface course thickness for full-depth pavements:

      - New or reconstructed pavements where the pavement selection is based on life-cycle costs will have a surface course thickness of 2 in.

      - Additional lanes, pavement widening, and short reconstruction segments where the pavement selection is based on first cost will have a surface

course thickness equal to the surface course thickness of the resurfacing if the adjacent pavement is being resurfaced. Otherwise, the surface course will be 2 in.

8. Limiting Strain Criterion Design Thickness Check. Use the following procedure to check the thickness determined in Step 7 against the limiting strain criterion design thickness for the project location.

   a. Determine the limiting strain criterion design thickness for the project location from Figure 54-5.I and round up to the nearest 0.25 in.

      If PG64-28 or PG70-28 is used in the lower binder lifts, add 1.00 in. of thickness to the value obtained from Figure 54-5.I and round up to the nearest 0.25 in.

   b. Compare the value obtained in Step 8a against the value obtained in Step 7 and select the lower value as the final design thickness.

9. Surface Friction Aggregate. See Section 53-4.07(d) for guidance on the selection of the appropriate surface friction aggregate.

   The use of an SMA surface course for projects with design TF greater than 10 is encouraged.

54-5.01(j) Use of Limiting Strain Criterion Design Cross-Section

Per Section 54-5.01(i), the surface course thickness for limiting strain criterion designs should be 2 in.

Limiting strain criterion designs should only need surface renewal throughout their design life. Material selection and attention to construction are critical to ensure that the design assumptions are met. Inadequate material selection and/or poor construction techniques will not result in a long-lived pavement. Limiting strain criterion designs must incorporate the following mixture characteristics and construction requirements in the initial cost of the full-depth HMA pavement when performing the life-cycle cost analysis; and, the designer must ensure the correct special provisions that achieve the following requirements are included.

1. Mix Characteristics.

   - Surface - The use of an SMA surface course is encouraged.
   - Moisture Damage - The use of hydrated lime slurry or hydrated lime on wetted aggregate is required for all mixtures in all lifts.

2. Construction Requirements.

   - Positive Dust Control - The use of positive dust control is required.

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- MTD - The use of a Material Transfer Device is required.
- Auger-Paver Mainframe Extensions - The use of mainframe extensions in addition to auger extensions is required.
- Tack Coats - The use of tack coats on all HMA lifts is required. Specify use of polymer tack coats on the top two lifts and non-polymer tack coats on all other lifts.
- Joint Construction - The use of echelon or full-width paving is required for all lifts. For situations where this is not possible, longitudinal joint sealant is required at a width of 9 in. on either side of the centerline joint prior to paving the surface lift.
- Density - The use of the proposed longitudinal joint density specification is required.

Potential increases to costs as a result of material selection and improved construction requirements must be factored into the life-cycle cost analysis discussed in Section 54-7.

54-5.01(k) Designating Structural Design Information on Plans

See Section 63-4.05 for information on designating structural design information on plans.

54-5.01(l) Shoulder Type/Design

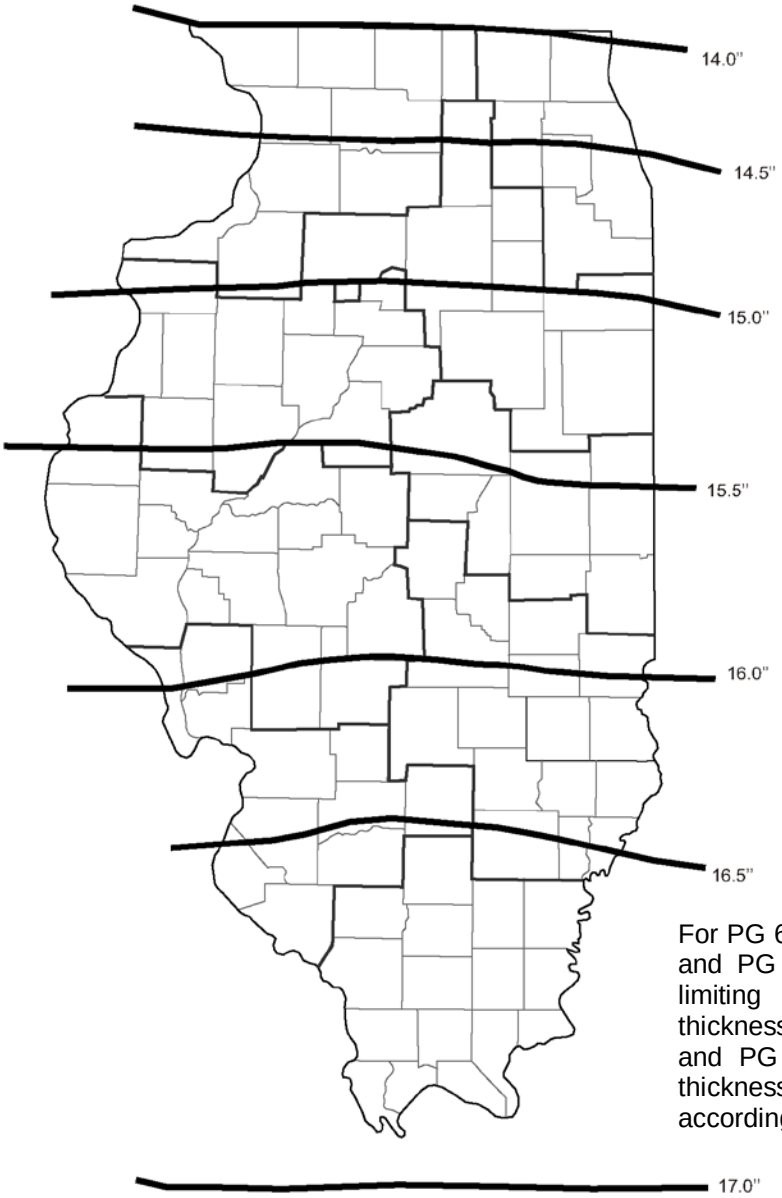
Use flexible shoulders with flexible pavement designs. The shoulder should be of constant thickness to a depth determined by the designer, but should not be less than 8 in. Give particular consideration to the need to provide a greater-than-minimum shoulder thickness along heavily traveled truck routes.

54-5.01(m) Design Example

See Section 54-9 for a design example.

54-5.01(n) Typical Sections

Figures 54-5.J through 54-5.L illustrate typical rural and urban pavement sections of flexible designs for various types of highway facilities.



For PG 64-22, PG 70-22, PG 76-22, and PG 76-28 binder grades, read limiting strain criterion design thickness off of map. For PG 64-28 and PG 70-28 binder grades, the thickness should be corrected according to Section 54-5.01(i)8a.

Note. Thickness values based upon Mean Monthly Pavement Temperature at 4 in. depth correlated to July Mean Monthly Air Temperature, axle load of 20,000 lb, strain of 70 με, and E_{Ri} of 2 ksi.

MAXIMUM PAVEMENT THICKNESS
(Limiting Strain Criterion Design: Flexible Pavement)

Figure 54-5.I

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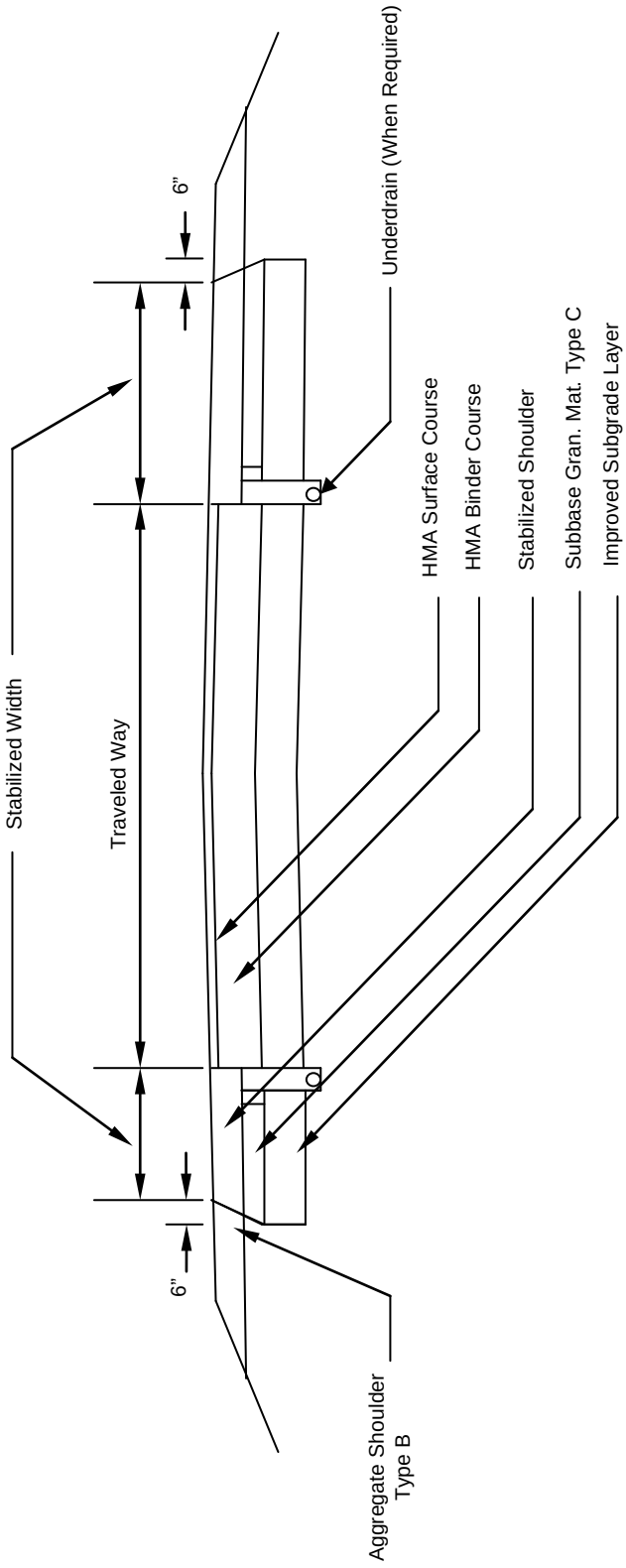
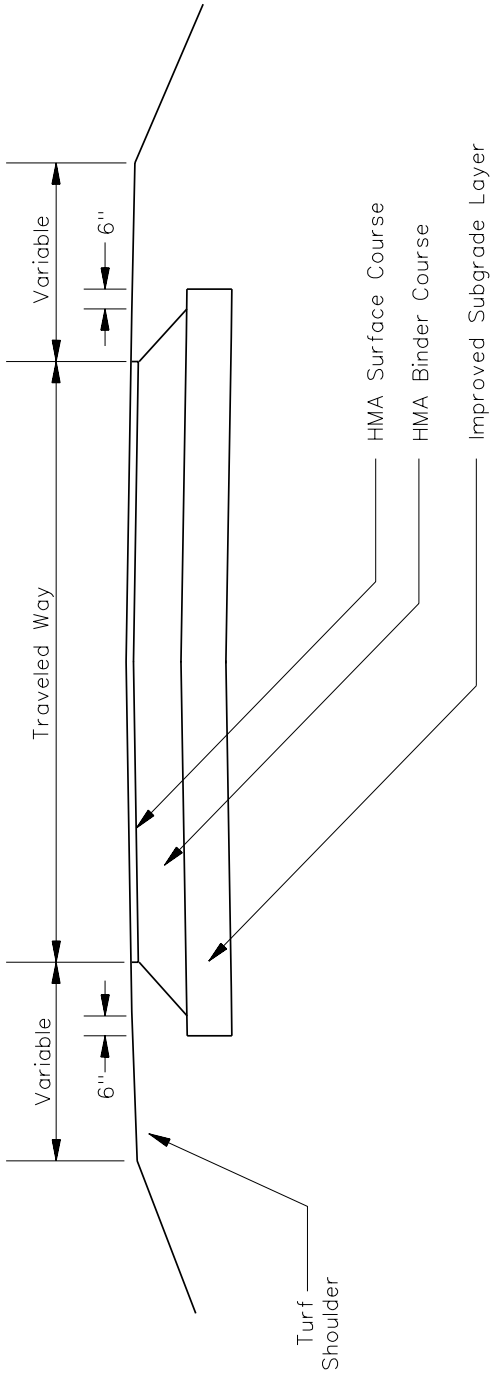


TYPICAL RURAL SECTION: FULL-DEPTH HMA DESIGN
(Trunk, Major Highways, Area Service Highways)

Figure 54-5.J



TYPICAL RURAL SECTION: FULL-DEPTH HMA DESIGN
(Collectors, Land Access Highways)

Figure 54-5.K

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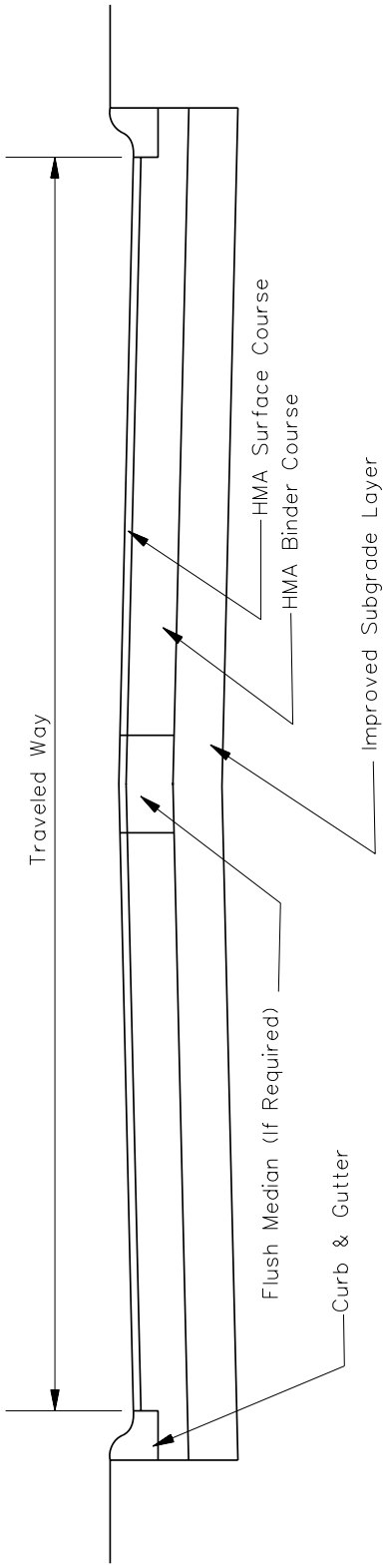

Note: Raised median with curb and gutter may be used in lieu of a flush median.

TYPICAL URBAN SECTION: FULL-DEPTH HMA DESIGN

Figure 54-5.L

54-5.02 Modified AASHTO

54-5.02(a) Application of Design Method

The modified AASHTO design procedure for flexible pavements provides a historical reference that enables the designer to determine the material types and thicknesses for the various layers of a flexible pavement, which are required to carry a specified volume and composition of traffic for a designated period of time while retaining a serviceability level at or above a selected minimum value. Application of this design method involves the following steps:

1. Determine Traffic Factor. Use the following procedures to determine the traffic factor:

    a. Determine the facility class (e.g., Class I, II, III, or IV) and the design period (see Section 54-5.02(b)).

    b. Determine the actual structural design traffic as described in Section 54-2.01(c), except the minimum traffic does not apply to the modified AASHTO design.

    c. Based on the facility class, select the appropriate traffic factor equation from Figure 54-5.B.

    d. Calculate the actual traffic factor for use in design.

2. Determine the IBR. Determine the IBR of the roadbed soil (see Section 54-3.01(f)).

3. Determine the Structural Number (SN_F). Determine the flexible pavement structural number (SN_F) using the appropriate design nomograph for the facility class (i.e., Figure 54-5.M for Class I facilities or Figure 54-5.N for Class II, III, and IV facilities). See Section 54-5.02(e). Check Figure 54-5.S for the minimum thickness, minimum structural number, and material requirements. See Section 54-5.02(h).

4. Determine Types and Thicknesses of Materials. Using Equation 54-5.4 in Section 54-5.02(f) and the appropriate strength coefficients obtained from Figure 54-5.O, select the types and thicknesses of materials which will satisfy the structural number at minimum cost. See Section 54-5.02(g). The various layers will be rounded up to the nearest 0.25 in. increment of thickness.

5. Compare with Minimum Criteria. Compare the selected design with the minimum requirements presented in Figure 54-5.S to ensure that the minimum design requirements have been met. See Section 54-5.02(h).

54-5.02(b) Design Period

Section 54-5.01(e) applies to modified AASHTO designs of flexible pavements.

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54-5.02(c) Equivalency Factors

Section 54-5.01(f) applies to modified AASHTO designs of flexible pavements.

54-5.02(d) Traffic Factors

Section 54-5.01(g) applies to modified AASHTO designs of flexible pavements.

54-5.02(e) Structural Number

Having calculated the traffic factor, only the IBR of the roadbed soil (see Section 54-3.01(f)) is needed to determine the structural number of the flexible pavement. The flexible pavement structural number (SN_F) is obtained by projecting a line through the traffic factor and the IBR on the appropriate design nomograph, either Figure 54-5.M for Class I facilities or Figure 54-5.N for Class II, III, and IV facilities. Check the structural number (SN_F) to see that it meets or exceeds the minimum structural numbers given to Note (1) of Figure 54-5.S for the type of facility being designed.

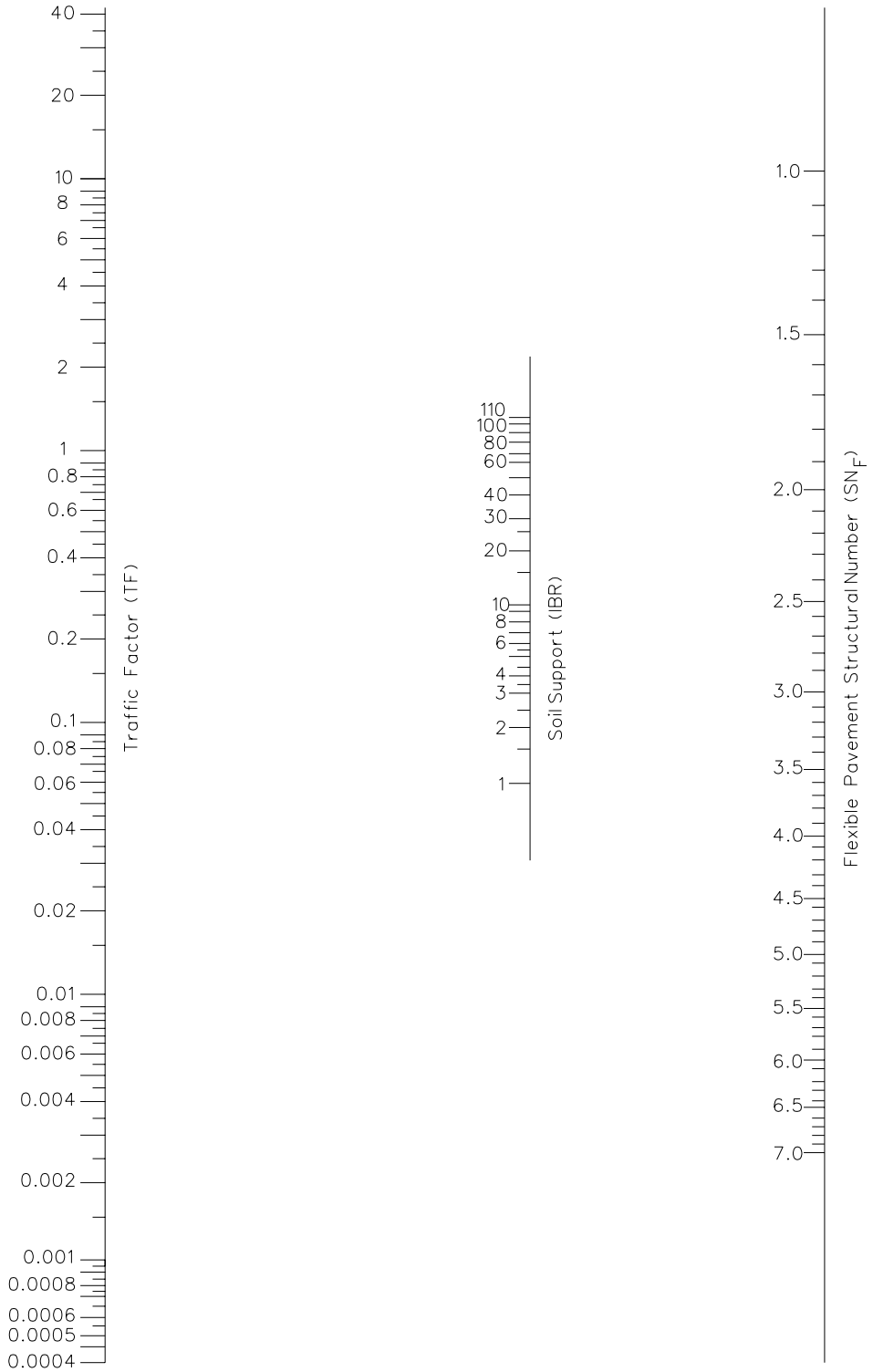
54-5.02(f) Structural Number Equation

The structural number (SN_F), an abstract number related to the strength required of the total pavement structure, is the summation of the layer thicknesses multiplied by their corresponding strength coefficients. Use the following equation to determine the structural number:

$$SN_F = a_1D_1 + a_2D_2 + a_3D_3 \qquad \text{Equation 54-5.4}$$

where:

SN_F	=	flexible pavement structural number
a_1, a_2, and a_3	=	coefficients of relative strength of the surface, base, and subbase materials, respectively
D_1, D_2, and D_3	=	thickness, in inches, of the surface, base, and subbase layers, respectively



FLEXIBLE PAVEMENT DESIGN NOMOGRAPH
(Modified AASHTO Design: Class I Facilities)

Figure 54-5.M

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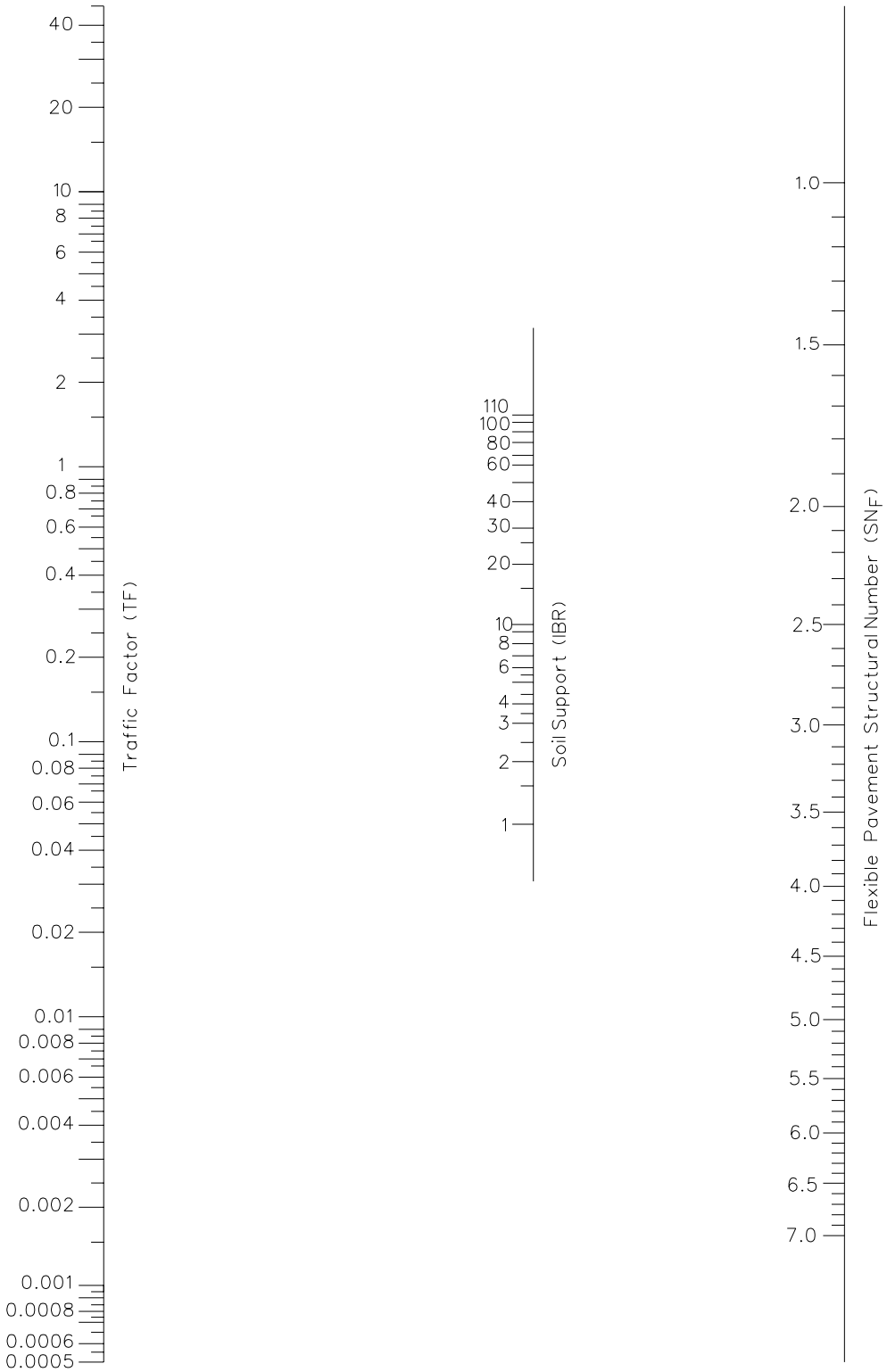

FLEXIBLE PAVEMENT DESIGN NOMOGRAPH
(Modified AASHTO Design: Class II, III, and IV Facilities)

Figure 54-5.N

STRUCTURAL MATERIALS	MINIMUM STRENGTH REQUIREMENTS			COEFFICIENTS ③		
	MS ①	IBR	CS ②	a_1	a_2	a_3
HMA Surface						
Road Mix (Class B)				0.20		
Plant Mix (Class B)						
Liquid Asphalt				0.22		
Asphalt Binder	900			0.30		
HMA Surface Course (4% voids)				0.40		
Base Course						
Aggregate, Type B						
Uncrushed		50			0.10	
Crushed		80			0.13	
Aggregate, Type A		80			0.13	
Waterbound Macadam		110			0.14	
Bituminous Stabilized Granular Material	300				0.16	
	400				0.18	
	800				0.23	
	1,000				0.25	
	1,200				0.27	
	1,500				0.30	
	1,700				0.33	
HMA Binder Course (4% voids)					0.33	
Pozzolanic, Type A			600		0.28	
Lime Stabilized Soil			150		0.11	
Select Soil Stabilized			300		0.15	
with Cement			500		0.20	
Cement Stabilized Granular Material			650		0.23	
			750		0.25	
			1,000		0.28	
Subbase						
Granular Material, Type B		30				0.11
Granular Material, Type A						
Uncrushed		50				0.12
Crushed		80				0.14
Lime Stabilized Soil			100			0.12

Notes:

① *Marshall Stability (MS) index or equivalent.*

② *Compressive strength (CS) in pounds per square inch (psi). For cement stabilized soils and granular materials, use the 7-day compressive strength that can be reasonably expected under field conditions. For lime stabilized soils, use the accelerated curing compressive strength at 120°F for 48 hours. For Pozzolanic, Type A, use the compressive strength after a 14-day curing period at 72°F.*

③ *For materials with strengths other than those shown, the coefficients may be determined from Figures 54-5.P, 54-5.Q, and 54-5.R. Other approved materials of similar strengths may be substituted for those presented in Figure 54-5.O.*

COEFFICIENTS FOR MATERIALS IN NEW FLEXIBLE PAVEMENT STRUCTURES (Modified AASHTO Design)

Figure 54-5.O

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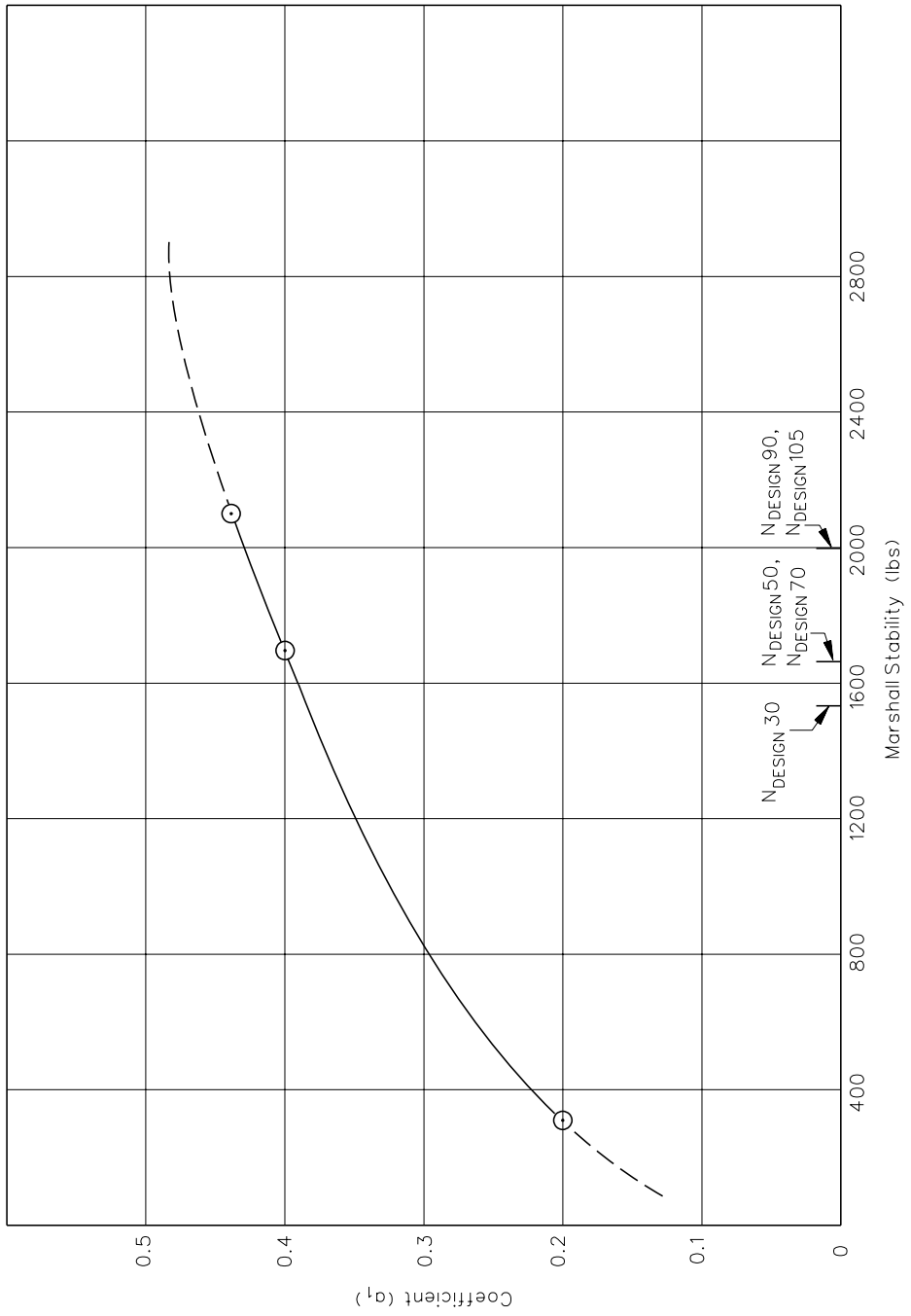
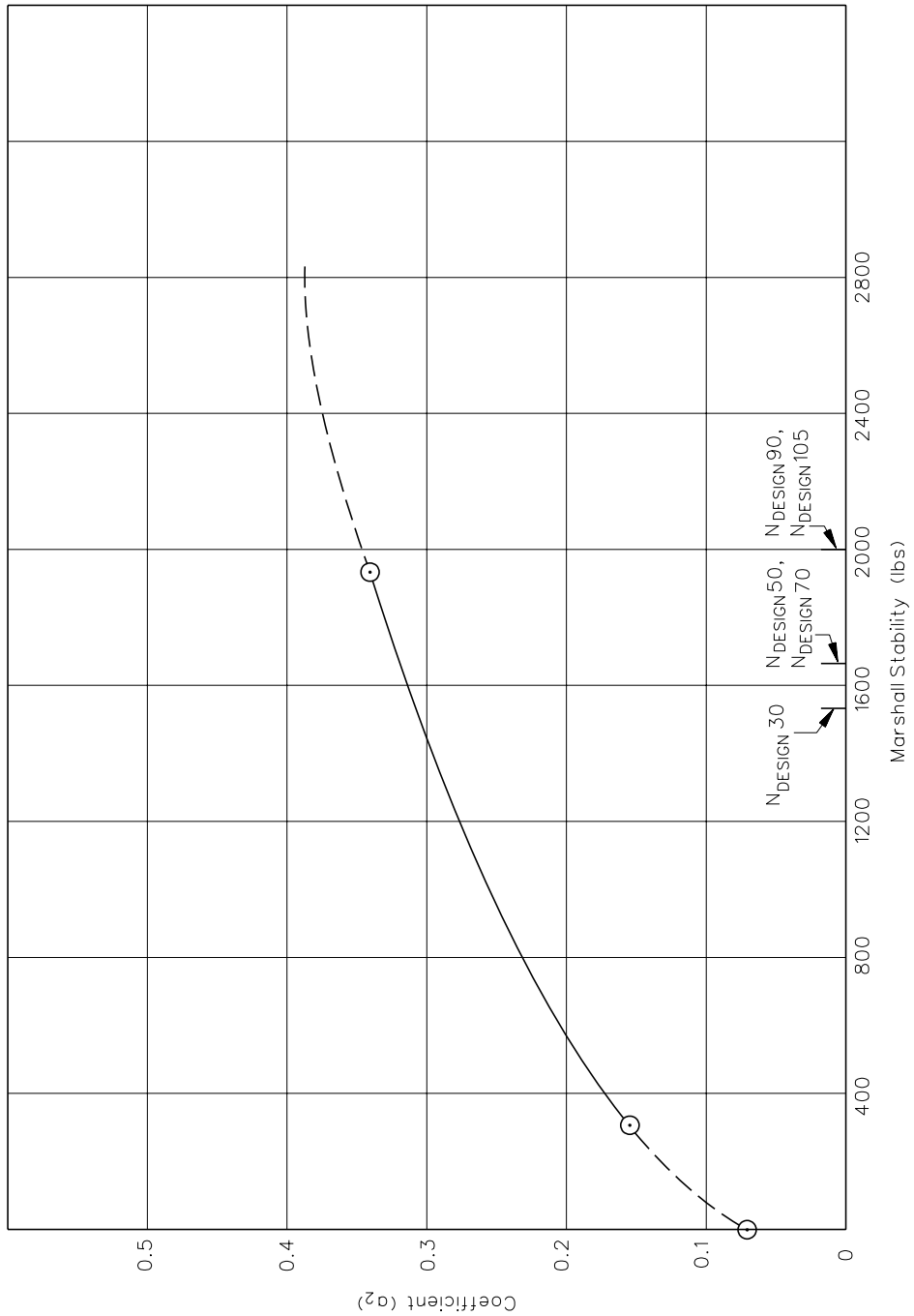


Note. The following Marshall stabilities can be assumed:

- *1,500 lb for HMA N_{Design}30 mixes,*
- *1,700 lb for HMA N_{Design}50 and N_{Design}70 mixes, and*
- *2,000 lb for HMA N_{Design}90 and N_{Design}105 mixes.*

COEFFICIENTS FOR HMA SURFACE COURSE MATERIALS (Modified AASHTO Design)

Figure 54-5.P



Note. The following Marshall stabilities can be assumed:

- *1500 lb for HMA N_{Design}30 mixes,*
- *1700 lb for HMA N_{Design}50 and N_{Design} 70 mixes, and*
- *2000 lb for HMA N_{Design} 90 and N_{Design} 105 mixes.*

COEFFICIENTS FOR HMA STABILIZED GRANULAR MATERIALS (Modified AASHTO Design)

Figure 54-5.Q

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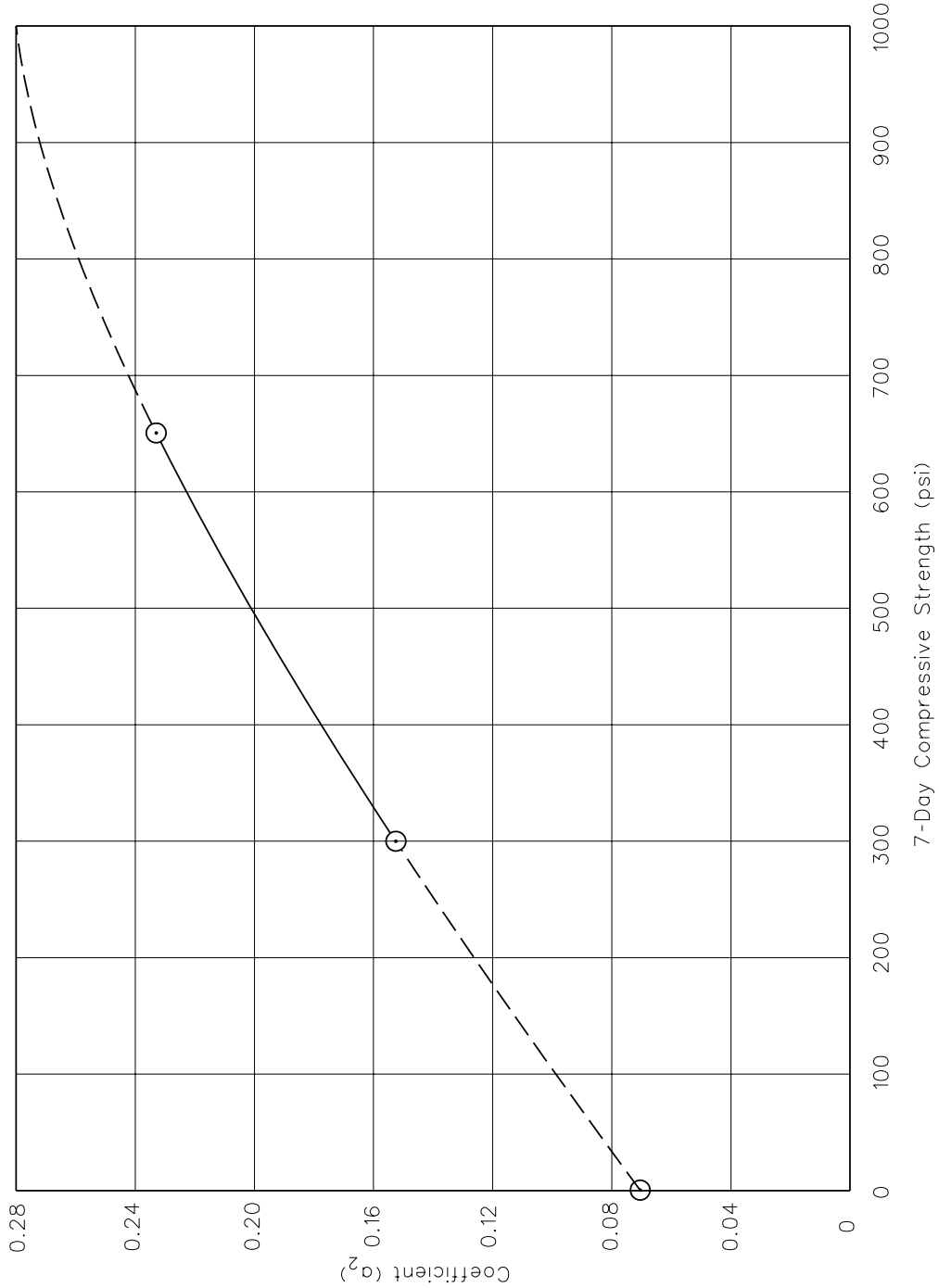

COEFFICIENTS FOR CEMENT STABILIZED GRANULAR MATERIALS
(Modified AASHTO Design)

Figure 54-5.R

STRUCTURAL NUMBER (D_t) ①		MINIMUM THICKNESS (inches)			MINIMUM MATERIAL ⑤		
From	To	Surface	Base ②	Subbase ③ (optional)	Surface	Base ⑥	Subbase ⑧ (optional)
1.00	1.99	2	8	4 ④	Class B Road Mix	Aggregate, Type B ⑦	Granular Material, Type B ④
			6			Stabilized Granular Material (MS_{min} = 300 or CS_{min} = 300)	
2.00	2.49	2	8	4 ④	Class B Plant Mix (asphalt binder)	Aggregate, Type B ⑦	Granular Material, Type B ④
			6			Stabilized Granular Material (MS_{min} = 300 or CS_{min} = 300)	
2.50	2.99	⑨	9	4	Class B Plant Mix (asphalt binder)	Aggregate, Type A	Granular Material, Type B
			7			Stabilized Granular Material (MS_{min} = 300 or CS_{min} = 300)	
3.00	3.49	⑨	11	4	HMA (4% voids)	Aggregate, Type A	Granular Material, Type B
			8			Stabilized Granular Material (MS_{min} = 400 or CS_{min} = 400)	
3.50	3.99	⑨	8	4	HMA (4% voids)	Stabilized Granular Material (MS_{min} = 800 or CS_{min} = 650)	Granular Material, Type B
4.00	4.49	⑨	8	4	HMA (4% voids)	Stabilized Granular Material (MS_{min} = 1,000 or CS_{min} = 750)	Granular Material, Type B
4.50	4.99	⑨	9	4	HMA (4% voids)	Stabilized Granular Material (MS_{min} = 1,200 or CS_{min} = 1,000)	Granular Material, Type B
			9			Pozzolanic, Type A	
5.00	5.99	⑨	10	4	HMA (4% voids)	Stabilized Granular Material (MS_{min} = 1,500)	Granular Material, Type A
≥ 6.00		⑨	12	4	HMA (4% voids)	Stabilized Granular Material (MS_{min} = 1,700)	Granular Material, Type A

Notes:

① *The minimum allowable structural number for Interstates and freeways will be 5.6; for multi-lane State primary highways, 5.0; and for two-lane State primary highways, 4.0.*

② *Where bituminous stabilized granular material with a strength greater than the minimum required above is used, a reduction in the minimum required thickness, up to a maximum of 1 in., will be allowed.*

③ *The minimum thickness of a lime stabilized soil subbase will be 6 in.*

④ *If an uncrushed Aggregate Base Course, Type B is used, a subbase will not be used.*

⑤ *Other approved materials having equal or greater strengths may be substituted for those listed above.*

⑥ *MS = Marshall Stability (lb) or equivalent, CS = 7-day compressive strength (psi) that can be reasonably expected under field conditions.*

⑦ *Lime stabilized soil may be used, provided the minimum thickness is not less than 8 in.*

⑧ *The use of a granular subbase is not mandatory.*

⑨ *Use Policy Resurfacing Thickness (see Chapter 53), unless prior BDE approval is received.*

MINIMUM THICKNESS AND MATERIAL REQUIREMENTS FOR FLEXIBLE PAVEMENTS
(Modified AASHTO Design)
Figure 54-5.S

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54-5.02(g) Trial Designs

The structural number equation discussed in Section 54-5.02(f) allows the designer to develop trial designs using various combinations of materials. By applying current bid prices or labor and material quotes to the trial designs, it is possible to select the most economical alternative.

Certain practical restrictions limit the designer's freedom in developing trial designs. A flexible pavement consists of a two or three layer structure, including a surface course and base course or a surface course, base course, and subbase course. Each layer must have sufficient strength and thickness to sustain the load imposed upon it and to distribute it over a sufficient area so as not to exceed the structural strength of the underlying layer. Thus, the composition of the pavement structure must be such that the strength characteristics of the surface course material are higher than those of the base or subbase, and the strength characteristics of the base course materials are higher than those of the subbase.

The above guidelines must be considered when selecting the materials to be used in the pavement structure. For example, if two granular materials having different strength characteristics are selected for use, the higher strength material must be used as the base course and the lower strength material as the subbase. If only one material is to be used for both subbase and base courses, the pavement structure then must be considered as a two-layer system consisting only of a surface course and a base course.

Values of coefficients for various materials that are typically used in flexible pavement structures are presented in Figure 54-5.O. These coefficients are consistent with the minimum strength values that can be expected throughout the State and may be used in determining the structural design of flexible pavements using modified AASHTO procedures for any class of road or street.

54-5.02(h) Minimum Thickness and Material Requirements

To ensure practical and adequate designs, the minimum design requirements presented in Figure 54-5.S have been established. Surface, base, and subbase materials having strength characteristics greater than those shown for the various structural number groups may be selected, but in no case may an inferior quality of surface, base, or subbase material be selected. Note that increasing the quality of material will reduce the required layer thickness and will tend to increase the level of performance that can be expected by the pavement during its design life, provided the minimum thickness requirements contained in Figure 54-5.S are met. Figure 54-5.S also serves as an aid in developing trial designs (see Section 54-5.02(g)). Thicknesses for two layers of the pavement first may be selected from the minimums in Figure 54-5.S and then the third layer thickness computed.

54-5.02(i) Surface Friction Aggregate

See Section 53-4.07(d) for guidance on the selection of the appropriate surface friction aggregate.

54-5.02(j) Designating Structural Design Information on Plans

See Section 63-4.05 for information on designating structural design information on plans.

54-5.02(k) Design Example

See Section 54-9 for a design example.

54-5.02(l) Typical Sections

Because the modified AASHTO design procedure for flexible pavements allows for a variety of solutions, no typical sections are provided.

54-5.03 HMA Overlay of Rubblized PCC Pavement

54-5.03(a) Application of Design Method

The following procedures are to be used to determine the appropriate HMA overlay thickness to be placed over rubblized PCC pavement. Rubblization is a reconstruction alternative in which the existing PCC pavement is broken (in-place) into small pieces and compacted to create a uniform base for the new HMA overlay.

These guidelines encompass the evaluation of an existing pavement structure to determine if the section can support the rubblizing construction process, and design and construction steps needed to successfully use this option. The use of rubblizing requires close attention to subgrade support. This technique requires sufficient thickness of the rubblized pavement and subbase structure to protect the subgrade during construction operations.

54-5.03(b) Review of the Existing Pavement Structure

The selection of rubblization with an HMA overlay as a viable reconstruction alternative should be the result of a thorough review of the existing pavement structure and other design issues. A thorough investigation of the existing pavement and subsurface should be conducted. The purpose of the investigation is to determine if the pavement section can be successfully rubblized. It is essential that only constructible sections be selected for this reconstruction alternative. This requires adequate support from the subgrade, subbase, and rubblized pavement section for each of the various construction activities. If conditions exist that would result in extensive removal and replacement of the existing pavement, or the subgrade is weak and would result in severe construction problems, the designer should consider other options.

1. Preliminary Soils Review. Before ordering an extensive subgrade investigation, the designer should contact the district's geotechnical engineer to discuss the proposed rubblizing section. From the typical pavement sections, soil maps, and typical

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Immediate Bearing Values (IBVs) of soils in the area, the designer and geotechnical engineer should determine if the rubblized section will protect the subgrade, as outlined in the Department's *Subgrade Stability Manual*.

If the rubblized pavement will not provide adequate cover for potentially soft subgrades, rubblizing should not be considered as an option. Rubblizing destroys the slab action of the PCC pavement; and if an unstable subgrade is encountered during construction, the pavement section may require expensive change orders to reconstruct.

If it appears that the pavement can be rubblized, then a detailed pavement and subsurface investigation is needed to verify constructability of the pavement.

2. Detailed Pavement and Subsurface Investigation. After passing a preliminary review, a detailed pavement and subsurface investigation should be conducted and a report prepared to specifically address the following points:

- HMA overlay thickness (if present);
- subbase condition and thickness (if present);
- subgrade IBV from Dynamic Cone Penetrometer (DCP) tests;
- subgrade soil samples (if needed for further evaluation);
- survey of existing drainage conditions;
- all shoulders' ability to carry traffic while under construction;
- identification of locations where pavement removal and replacement, or alternative rehabilitation is recommended; and
- subgrade stability during rubblization.

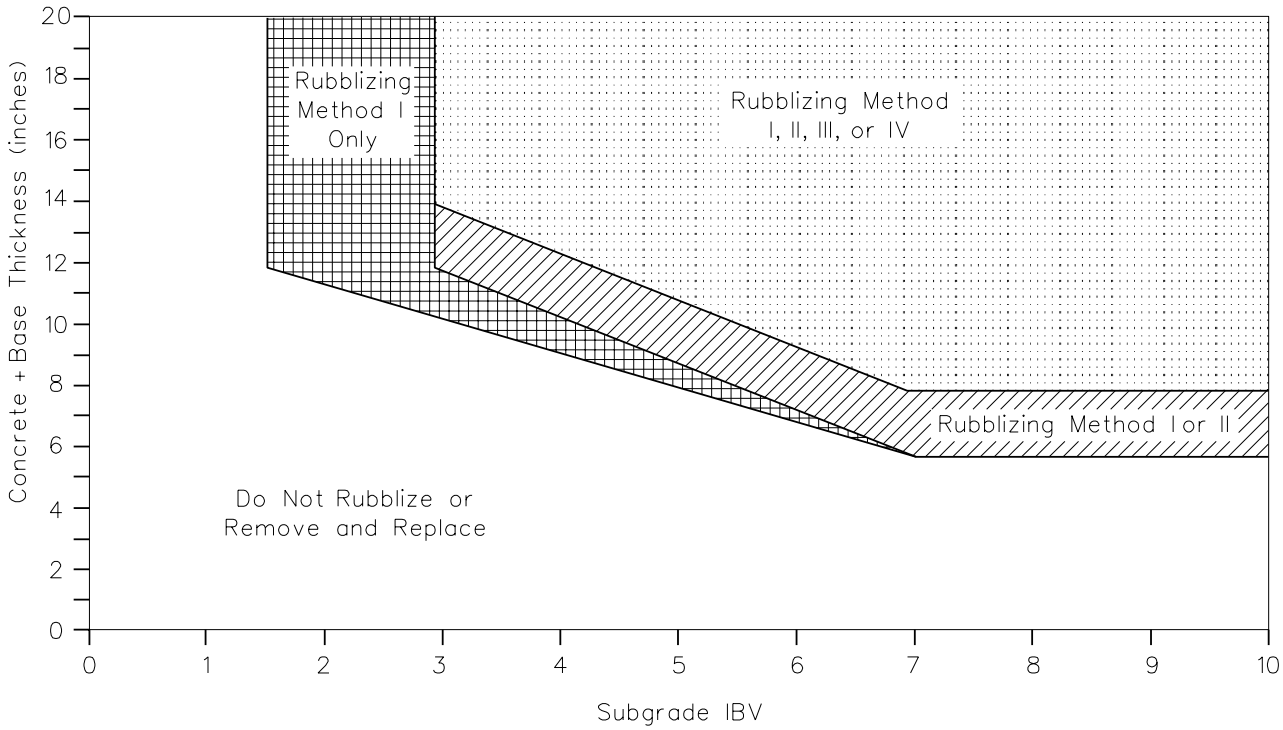
The district's geotechnical engineer should develop a coring, DCP, and soil sampling plan for the section. If the total thickness of existing concrete and base (i.e. stabilized subbase and/or granular layers) exceeds 12 in. and rubblizing Method I will be used, then DCP testing is optional. In general, a minimum of two cores per lane-mile should be taken. Core locations should be in representative cut and fill locations, and staggered between lanes. Additional coring and testing may be needed to define limits of weak subgrade areas.

The condition of any recovered stabilized material should be noted as being sound (intact and like new), slightly deteriorated (20% or less unsound or missing material), or deteriorated (more than 20% unsound or missing material). The overall condition of the subbase should be reported as a percentage of cores in each of these groups (i.e., 60% – sound, 30% – slightly deteriorated, and 10% – unsound).

After the core is removed, the DCP should be run in the hole for subgrade IBV. It is preferable to record single blow increments, to a depth of approximately 30 in. below the bottom of the pavement. If a granular base exists, the DCP may be driven through it and the depth determined from the change in IBV. A 6-lb to 8-lb soil sample should be taken and stored in an air-tight container for later testing if required. Forms BC 334 and BBS 2640 shall be used for documentation.

After the field survey is complete, typical IBVs should be developed, along with cross section data and condition of each layer. The data from each test location should be presented in table form including depth, penetration, and calculated IBV.

For the 12 in. of subgrade directly below the pavement, additional analysis is required. The top of the subgrade is broken into two layers, from 0 in. to 6 in. and 6 in. to 12 in. The average IBV is determined for each layer and plotted on Figure 54-5.T, using the pavement cross-section information. Once the data is plotted, a determination should be made as to what type of rubblizing method should be specified.



SUBGRADE RUBBLIZING GUIDE

Figure 54-5.T

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For very limited areas of very soft subgrades, the designer may remove and replace the pavement, omit rubblizing, or perform a cracking and seating operation so the pavement can bridge weak subgrade areas where undercutting is not cost-effective. These areas should be identified on the plans. If it is found that several short or a few substantial segments of the project require rubblizing omissions, or removal and replacement of the pavement, then rubblization is not a viable alternative.

The pavement and subsurface report should include the following:

- existing typical pavement section(s),
- core soundness and condition,
- summarized results of subsurface investigation,
- data plotted on Subgrade Rubblizing Guide (Figure 54-5.T),
- number and locations of transitions to meet mainline structures,
- clearances for overheads,
- utilities and culverts,
- location of any buildings or structures within 50 ft of the rubblization, and
- location and condition of underdrains.

54-5.03(c) Design Issues

The following design issues must be considered before the project can be submitted for review and approval:

1. Equipment Selection. A pavement breaker and self-propelled rollers are the major equipment necessary to rubblize a PCC pavement. The pavement breaker should be selected to meet the project's needs with respect to traffic control, staging, and subgrade support limitations. The following equipment characteristics should be considered when making a decision on breaker selection:

   a. Method I — Multi-Head Breaker (MHB). The MHB is a self-propelled unit with multiple drop-hammers mounted at the rear of the machine. The hammers are set in one or two rows, and strike the pavement approximately every 4.5 in. The hammers have variable drop heights and variable cycling speeds.

      The equipment has the ability to break pavement up to 13 ft wide, in one pass. The rate of production depends on the type of base/subbase material, and is approximately 1.0 lane-mile per day.

      The Z-pattern steel grid roller, a vibratory roller with a grid pattern, must be used in conjunction with the MHB to complete the breaking process. A Z-pattern grid is attached transversely to the drum surface. This roller further breaks flat and elongated material into more uniform pieces. The vibratory roller is self-propelled, with a minimum gross weight of 10 tons.

Method I should be specified if there is any question of the rubblized section's ability to support construction equipment. The rubblized section and subgrade still must be able to support compaction equipment and loaded trucks without rutting or dislodging the rubblized PCC pavement.

The MHB should be specified if the roadway is to remain open to traffic and encroachment into the adjacent lane cannot be accommodated. Encroachment of the MHB into the adjacent lane is similar to the rolling operation of HMA paving.

The paving operation may work directly behind the breaking operation, in such a manner that the lane may be rubblized and overlaid for opening to traffic at the end of the day.

Caution should be used if buildings are within 50 ft of the rubblizing operation, especially in an urban setting. Buildings that may be sensitive to vibration should be identified in the project report, with an alternative method of localized pavement breaking recommended. Alternative breaking methods (e.g., skid steer mounted jack hammer) should be considered or pavement rubblizing omitted near vibration sensitive buildings.

Underground utilities and drainage structures must be identified for protection. An omission in the breaking operation may be required over utilities and drainage structures. These omitted areas shall be broken with an alternative breaking method.

b. Method II — Resonant Frequency Breaker with High Flotation Tires. This method uses a resonant frequency breaker with tires, which have pressures below 60 psi. This allows operation on pavement sections that are thinner or have soft subgrades.

A resonant frequency breaker is a self-propelled unit that uses high frequency, low amplitude impacts with a shoe force of 2,000 lb to fracture the PCC pavement. The shoe, or hammer, is located at the end of a pedestal, which is attached to a beam and counter weight. The breaking principle is that low amplitude, high frequency resonant energy is delivered to the concrete slab, resulting in high tension at the top. This causes the slab to fracture on a shear plane, inclined at about 35 degrees from the pavement surface. The shoe, beam size, operating frequency, loading pressure, and speed of the machine can all be varied.

The breaking begins at the centerline and proceeds to the outside edge of the pavement. The breaking pattern is approximately 8 in. wide, and requires 18 to 20 passes to break a 12 ft lane width. The rate of production depends on the type of base/subbase material, and is about 1.0 lane-mile per day.

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The resonant breaker has very heavy wheel loads of 20,000 lb. The broken pavement, shoulder, and subgrade must be adequate to support multiple passes of the equipment. The resonant breaker encroaches 3 ft to 5 ft into the adjacent lane to rubblize pavement near the centerline. The pavement section/shoulder must be structurally adequate for traffic to be moved 7 ft to 8 ft from the centerline and onto the shoulder. The use of the resonant breaker is best suited on roads that can be closed to traffic and support the breaker's weight.

The resonant breaker produces limited vibrations. Caution should be used with vibration sensitive buildings that are within 10 ft of the rubblizing operation.

Utilities or culverts within 6 in. of the PCC pavement bottom need to be protected, as described in Method I.

c. Method III — Resonant Frequency Breaker. This is the same basic machine as in Method II. However, it does not utilize the high flotation tires. This results in limiting usage as shown in Figure 54-5.T.

d. Method IV — Breaking Device Not Specified. This method can be specified if Methods I, II, and III could be used without restrictions to subgrade support, traffic, staging, or structures as noted above.

2. Drainage Considerations. The Department's longitudinal underdrain policy (see Chapter 53) should be followed. Installation of new underdrains is strongly recommended. At a minimum, sag areas of vertical curves must be addressed. French drains, which are capable of draining the entire depth of the section, are acceptable for isolated areas. Existing underdrains that will remain in place shall be thoroughly investigated to ensure that they are functioning properly. For sections where underdrains will not be installed, the designer should consider limiting the amount of time the rubblized pavement may be left without an overlay, to minimize delays from rain saturation.

3. Priming. The rubblized surface should be overlaid without priming. Priming adds an extra step and curing period, which delays construction with no benefit to the finished product.

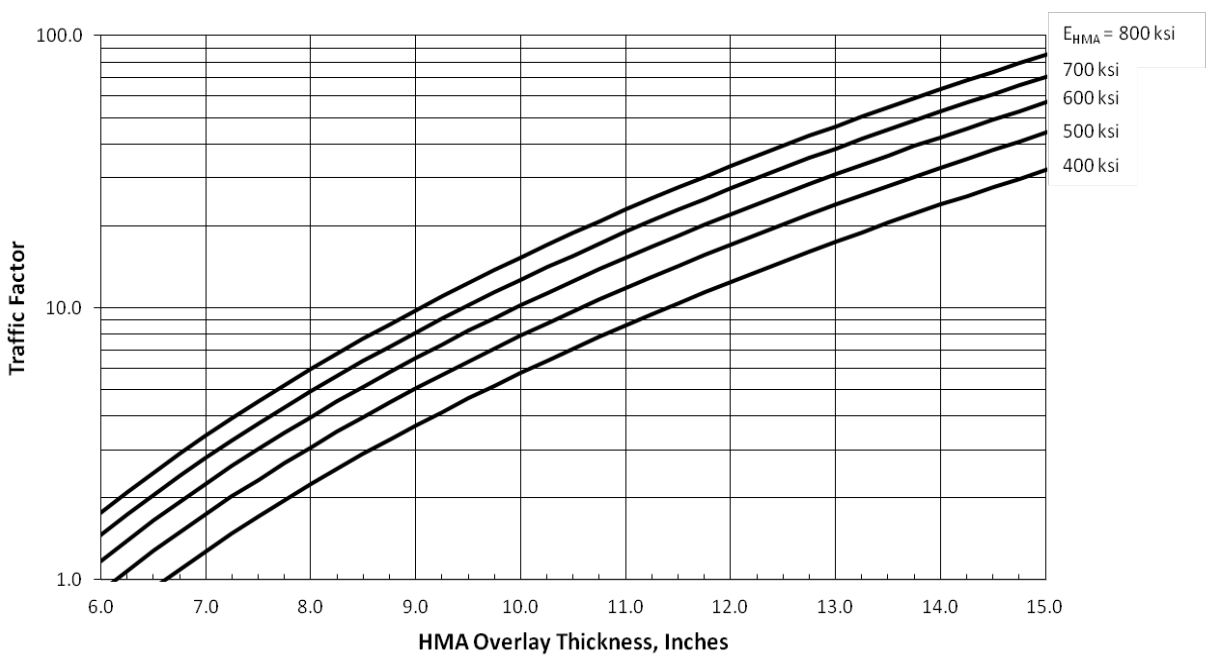
4. HMA Overlay Thickness Design. Use the following procedure to determine the HMA overlay thickness.

a. Overlay Thickness Design Based on Actual Traffic. The designer should determine the required Traffic Factor (TF) needed for the design period (see Section 54-5.01(g)) using a recommended design period of 20 years. Design periods less than 10 years should not be considered. Asphalt binder for the HMA overlay is selected according to Section 53-4.07(c) using the requirements for a full-depth HMA pavement (Figure 53-4.R). The thickness of the HMA overlay needed on top of the rubblized section is determined using Figure 54-5.U. All

designs are rounded up to the next 0.25 in. The design thickness, as a function of HMA mixture modulus and traffic factor, is determined as follows:

- Determine the HMA mixture modulus (E_{HMA}) using the procedure described in Section 54-5.01(i)5.
- Using the TF and E_{HMA} determined above, obtain the design thickness using Figure 54-5.U. An overall maximum thickness of 15 in. applies regardless of TF.
- Determine the limiting strain criterion design thickness for the project location from Figure 54-5.V and round up to the nearest 0.25 in. If PG 64-28 or PG 70-28 is used in the lower binder lifts, add 1.00 in. of thickness to the value obtained from Figure 54-5.V. Compare the values obtained from Figures 54-5.U and 54-5.V and select the lower value as the final design thickness.

b. Minimum HMA Overlay and Lift Thicknesses. The minimum HMA overlay thickness for rubblized pavement is 6 in. The first lift of the overlay should be 3 in. to 4 in. This thickness allows good compaction on and minimizes dislodging of the rubblized base. The surface lift should be 2 in. For pavement overlays that are 7 in. or less, surface lifts of 1.5 in. are allowable. Contact the Bureau of Materials and Physical Research if first lifts less than 3 in. are desired.



HMA OVERLAY THICKNESS FOR RUBBLIZED PAVEMENTS

Figure 54-5.U

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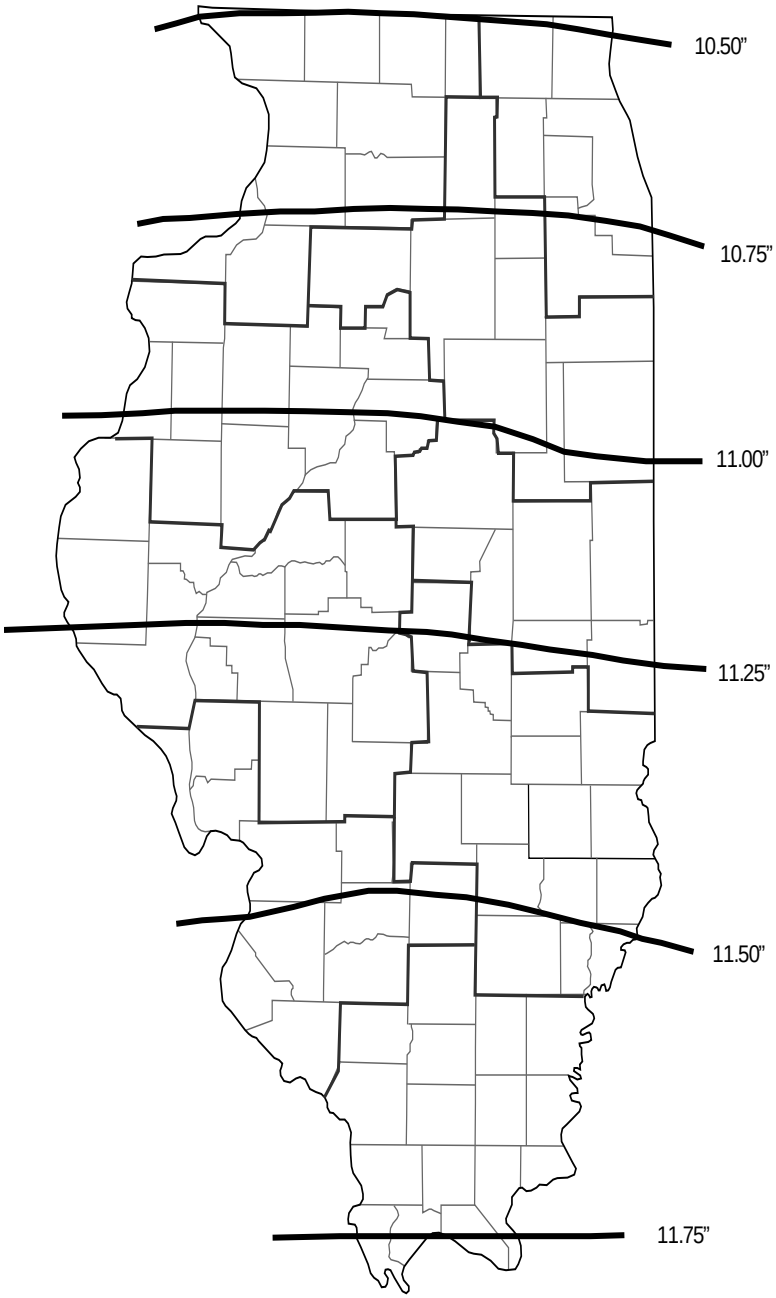

For PG 64-22, PG 70-22, PG 76-22, and PG 76-28 binder grades, read limiting strain criterion design thickness off of map. For PG 64-28 and PG 70-28 binder grades, the thickness should be corrected according to Section 54-5.03(c)4a.

Note. Thickness values based upon Mean Monthly Pavement Temperature at 4 in. depth correlated to July Mean Monthly Air Temperature, axle load of 20,000 lb, strain of 70 με.

MAXIMUM PAVEMENT THICKNESS
(Limiting Strain Criterion Design: HMA Overlay of Rubblized PCC Pavement)

Figure 54-5.V

5. Traffic Control. Traffic may be maintained during much of the construction process. The road may be used after the installation of underdrains and the milling of any existing HMA overlay. The safety of open trenches, lane to lane drop-offs, high shoulders, and the condition of the exposed pavement surface should be considered when determining if the road can be reopened to traffic.

   No traffic (including unnecessary construction traffic) should be allowed on the fractured pavement surface once the breaking operation begins. All HMA binder lifts should be paved before traffic is allowed onto the section. If staging requires that the pavement be opened to traffic before all the binder layers are in place, contact the Bureau of Materials and Physical Research to review the structural impacts.

   Edge differentials in elevation of rubblized pavements can be substantially greater than standard overlays, and may require additional traffic control measures. The designer should evaluate the overall design and traffic staging to determine if any additional traffic control may be required. The designer should also evaluate differentials in elevation if milling to bare pavement is needed.

6. Specification of Material Transfer Devices (MTDs). The use of MTDs on the rubblized base must be evaluated on a case-by-case basis, due to the weights and axle configurations of the equipment. Contact the Bureau of Materials and Physical Research to perform an analysis.

7. Construction Sequence. The general sequence of construction should be as follows:

   - Install underdrains or French drains, as required.
   - Remove any existing HMA overlay to the staged width.
   - Remove and replace any existing unsound HMA repair materials.
   - Rubblize the pavement.
   - Compact the broken pavement.
   - Pave the binder lifts of the HMA overlay.
   - Allow traffic on sections that have adequate thickness, as shown on the plans (if needed).
   - Pave the surface of the HMA overlay.

8. Other Design Issues. Any HMA material on the pavement from pothole patching may be left in place. If there are any full-depth HMA patches in the section, soundness of the patch material should be determined. HMA patches should be rated in the same manner as subbase in Section 54-5.03(b). Visually indeterminate patches may be investigated with a limited coring program. If an HMA patch is unsound, the material should be removed. When traffic is maintained during the patching operation, the

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replacement material should be a Class C or D patch. If concrete is the replacement material, it shall be rubblized.

If the unsound patch is greater than 10 sq ft, HMA binder mixture shall be used. When the road is closed to traffic and the unsound patch is less than or equal to 10 sq ft, the replacement material may otherwise be aggregate. The aggregate shall be a Class D Quality (or better) crushed stone, crushed slag, crushed concrete, or crushed gravel meeting a CA 6 or CA 10 gradation; according to Section 1004 of the *Standard Specifications*.

Partial-depth HMA patches may be left in place during rubblization. If partial-depth patches prevent proper breaking of the PCC pavement, a skid steer loader (with a jack hammer attachment or similar device) may be used to complete breaking in these areas.

The rubblizing process will increase the pavement width 1 in. to 3 in. per 2-lane width, and encroach slightly into the underdrain trench. This has not caused performance problems with sand trench and pipe type underdrains to date. If the resonant breaker is used, the driving of heavy wheel loads directly over the underdrain trench should be avoided as much as possible. Wheel loads directly over the underdrain trench are of less concern if the existing shoulder is in sound condition.

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54-6 STRUCTURAL DESIGN OF COMPOSITE PAVEMENTS

54-6.01 Application of Design Method

The design procedures for composite pavements enable the designer to select:

- the thickness of HMA surface needed to structurally rehabilitate an existing rigid or composite pavement, or
- the thickness of both HMA surface and PCC base course for a new composite pavement.

The resulting composite pavement will be capable of carrying a specified volume and composition of traffic for a designated period of time while retaining a serviceability level at or above a selected minimum value. The composite design method assumes that the existing rigid or composite pavement has reached the end of its design life and is in need of structural rehabilitation. If the existing pavement has not reached the end of its design life, as may be the case when a resurfacing is being designed in conjunction with a lane addition, higher strength coefficients than those discussed in Section 54-6.06 may be appropriate. Such cases should be referred to the BDE or the Bureau of Materials and Physical Research. Application of the composite design method involves the following steps:

1. Determine Traffic Factor. Use the following procedures to determine the traffic factor:

   a. Determine the facility class (e.g., Class I, II, III, or IV).

   b. Determine the actual structural design traffic as described in Section 54-2.01(c).

   c. Based on the facility class, select the appropriate traffic factor equation from Figure 54-4.C.

   d. Calculate the actual traffic factor to use in design.

2. Determine the IBR. Determine the IBR of the roadbed soil (see Section 54-3.01(f)).

3. Determine the Structural Number (SN_C). Determine the composite pavement structural number (SN_C) using the appropriate design nomograph for the facility class (i.e., Figure 54-6.A for Class I facilities or Figure 54-6.B for Class II, III, and IV facilities).

4. Determine Thickness. Select the appropriate equation from Section 54-6.06 as follows:

   - first resurfacing, use Equation 54-6.1;
   - second resurfacing, use Equation 54-6.2; or
   - new composite pavement, use Equation 54-6.3.

   Using the appropriate equation and Figure 54-6.C, calculate the thickness of surface and base course, if applicable. Round the thickness(es) up to the nearest 0.25 in. For pavements that are in need of a third resurfacing, see Section 54-6.06.

5. Compare with Minimum Criteria. Compare the calculated thickness(es) with the minimum requirements presented in Figure 54-6.D. Use the larger of the values for design.

54-6.02 Design Period

The design period for all composite pavements is typically 20 years. See Section 54-3.01(b).

54-6.03 Equivalency Factors

Section 54-3.01(d) describes the use of equivalency factors to convert mixed-traffic loadings to 18-kip ESAL applications. Because the main structural layer of a composite pavement is a rigid slab, the equivalency factors are the same as for rigid pavement (see Section 54-4.02(c)).

These equivalency factors have been used to develop the traffic factor equations discussed in Section 54-6.04.

54-6.04 Traffic Factors

The traffic factor is the projected total 18-kip ESALs, expressed in millions, to be carried by the design lane during the design period. Because the main structural layer of a composite pavement is a rigid slab, the equivalency factors are the same as for rigid pavement (see Section 54-6.03). The traffic factor equations discussed in Section 54-4.02(d) also apply to composite pavement designs.

54-6.05 Composite Pavement Structural Number

Having calculated the traffic factor, only the IBR of the roadbed soil (see Section 54-3.01(f)) is needed to determine the composite pavement structural number. The composite pavement structural number (SN_C) is obtained by projecting a line through the traffic factor and the IBR of the roadbed soil on the appropriate design nomograph, either Figure 54-6.A for Class I facilities or Figure 54-6.B for Class II, III, and IV facilities.

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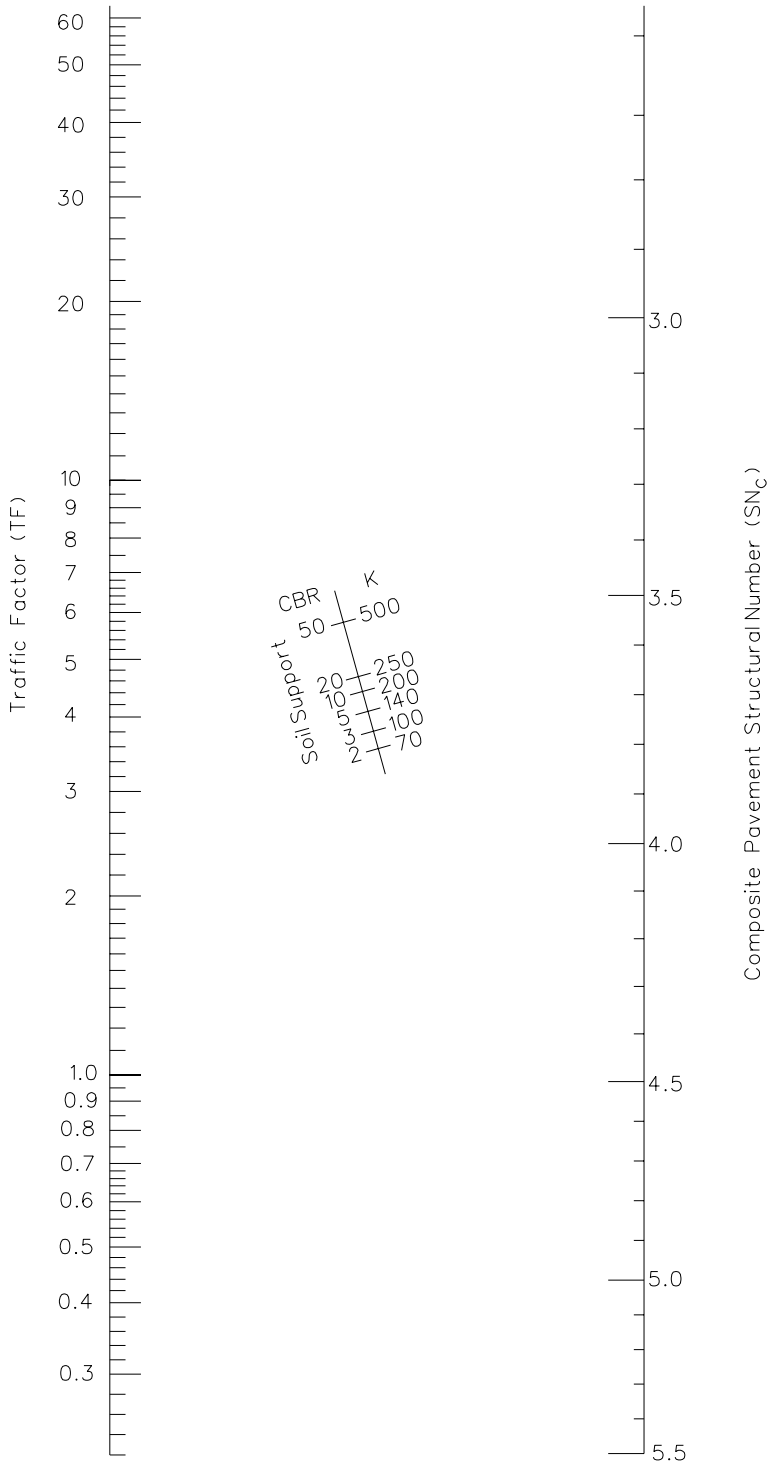
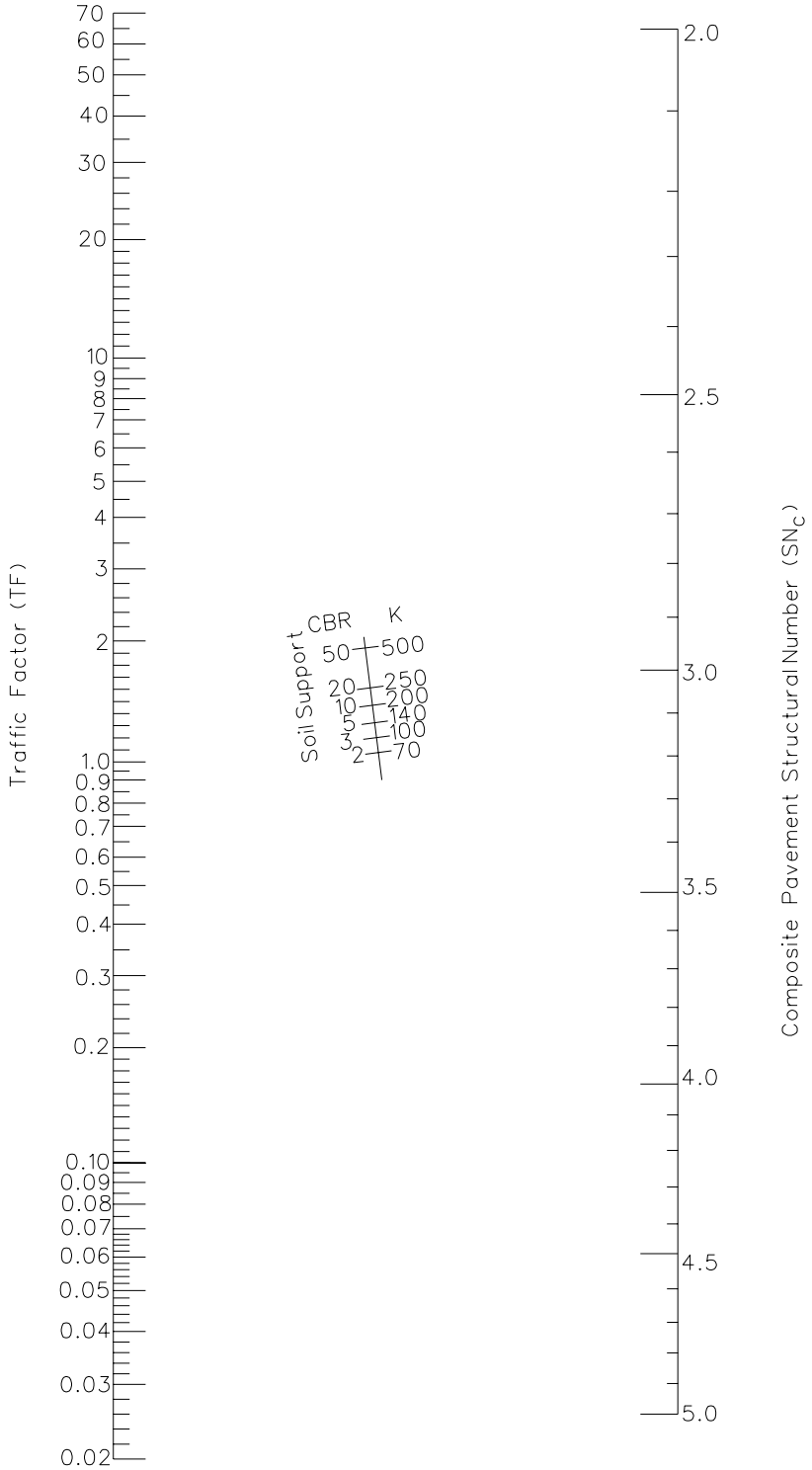


COMPOSITE PAVEMENT DESIGN NOMOGRAPH
(Class I Facilities)

Figure 54-6.A



COMPOSITE PAVEMENT DESIGN NOMOGRAPH
(Class II, III, and IV Facilities)

Figure 54-6.B

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Pavement Cross Section (inches)	Equivalent Thickness (D_C) (inches)
10 – 10 – 8 – 10 – 10	9.75
9 – 9 – 7 – 9 – 9	8.75
9 – 7 – 9	7.71
9 – 9 – 6.5 – 9 – 9	8.50
9 – 6 – 9	7.06
8 – 8 – 6 – 8 – 8	7.75
7 – 8 – 7	7.00

EQUIVALENT THICKNESS (D_C) FOR EXISTING NON-UNIFORM PCC PAVEMENTS
(Composite Pavement Design)
Figure 54-6.C

Facility Class	HMA Surface Course	PCC Base Course Minimum Thickness (inches)
Class I	Use Policy Thickness unless otherwise approved by BDE. See Chapter 53.	8
Class II		7
Class II State Primary All Others		 7 6
Class IV		6

MINIMUM DESIGN REQUIREMENTS FOR COMPOSITE PAVEMENTS
(Composite Pavement Design)

Figure 54-6.D

54-6.06 Thickness Design Equations

The composite pavement structural number (SN_C), an abstract number related to the strength required of the total pavement structure, is a summation of layer thicknesses multiplied by their corresponding strength coefficients. Three design equations incorporate the incorporate the composite pavement structural number as follows:

1. First Resurfacing. For the initial HMA surfacing over an existing rigid pavement, use the following equation:

$$D_S = \frac{SN_C - 0.26D_C}{0.40} \qquad \text{Equation 54-6.1}$$

2. Second Resurfacing. For a second HMA surfacing over an existing resurfaced rigid pavement, use the following equation:

$$D_S = \frac{SN_C - 0.25D_E - 0.17D_C}{0.40} \qquad \text{Equation 54-6.2}$$

3. New Composite Pavement. For the design of a new composite pavement, use the following equation:

$$SN_C = 0.40\ D_S + 0.33\ D_B \qquad \text{Equation 54-6.3}$$

where:

SN_C = composite pavement structural number
D_S = thickness of HMA policy overlay (inches)
D_C = equivalent thickness of existing PCC slab (inches)
D_E = thickness of existing HMA surface (inches)
D_B = thickness of new PCC base course (inches)

Note that the above equations do not include provisions for a third resurfacing. Pavements that are in need of a third resurfacing for structural reasons often are badly deteriorated and may no longer be functioning as a rigid pavement. Contact BDE or the Bureau of Materials and Physical Research for guidance in selecting the appropriate strength coefficients for such pavements.

In the case of an existing JRCP/JPCP of uniform thickness, the equivalent thickness of the PCC slab (D_C) is the actual slab thickness. For a CRCP, D_C is the slab thickness multiplied by 1.25. Figure 54-6.C presents the equivalent thickness (D_C) of the non-uniform PCC pavements formerly constructed by the Department.

Use Equation 54-6.3 to develop designs for totally new composite pavements composed of an HMA surface and a PCC base course. The application of this pavement design procedure is restricted as follows:

- to changes in horizontal or vertical alignment for short segments of rural pavement,
- to lane additions,
- to reconstruction of short segments of urban pavement, and
- as an option to flexible base materials.

Equation 54-6.3 requires determination of two unknowns (i.e., the surface and the base course thicknesses). To develop a design, it becomes necessary, therefore, to assume the thickness of one pavement component and compute the required thickness of the other. In most cases, it will be best to initially assume the surface course thickness. The surface course thickness selected should be the standard policy resurfacing thickness or the thickness of the resurfacing being placed on the adjacent pavement.

54-6.07 Minimum Design Requirements

The composite design procedures are used to analyze PCC slabs that are surfaced with high-type HMA and are therefore limited to HMA surfacing materials that meet the requirements of the *Standard Specifications* for HMA. To ensure practical and adequate designs, adhere to the minimum criteria presented in Figure 54-6.D for composite pavements.

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54-6.08 Designating Structural Design Information on Plans

See Section 63-4.05 for information on designating structural design information on plans.

54-6.09 Design Example

See Section 54-9 for a design example.

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54-7 PAVEMENT SELECTION ANALYSIS

54-7.01 Introduction

The life-cycle activities for mechanistically designed pavements that are presented in this section were developed by a panel of Department experts who have experience in the areas of design, construction, materials, and maintenance of Illinois pavements. The expert group established rehabilitation, patching, and maintenance strategies for a 45-year analysis period for typical rigid and flexible pavements.

A framework for data collection has been established to gather actual data on maintenance activities and costs. As these data are collected, appropriate modifications to the life-cycle strategies will be made where needed.

54-7.02 Selection Basis

The selection of pavement design alternatives is based on the following criteria:

1. Widening Projects. Pavement design alternatives for widening projects are evaluated based on a first-cost analysis. The alternative with the lowest first cost is selected for construction.

2. New Construction/Reconstruction Projects. Pavement design alternatives for new or reconstructed pavements are evaluated based on a life-cycle cost analysis. The analysis will consider the following alternatives:

   a. New Construction Projects. The analysis for a new pavement shall consider mechanistic designs for rigid and flexible pavements. If the difference in life-cycle costs between alternatives is greater than 10%, the alternative with the lowest life-cycle cost is selected for construction.

      If the difference in life-cycle costs is 10% or less, the selection will be based upon the alternate pavement bidding process described in Section 54-1.04. However if the project does not fit the criteria for alternate pavement bidding, or one pavement type is preferrable, the project will be referred to the Pavement Selection Committee. If the Committee agrees alternate pavement bidding is not appropriate, the Committee will select the pavement type.

   b. Reconstruction Projects. The analysis for a reconstructed pavement shall consider new pavement mechanistic designs for rigid and flexible pavements; as well as supplemental pavement designs for unbonded JPC/CRC overlay and HMA overlay of rubblized PCC pavement. When developing the supplemental designs, the designer shall review the criteria provided in Sections 54-4.03 and 54-5.03 to determine which supplemental designs are viable options.

      When comparing the new pavements designs to the viable supplemental designs, the life-cycle cost analysis must include costs that are unique to this

type of comparison. For example, the cost of removing the existing pavement must be added to the new pavement alternatives and the cost of preparing the existing pavement must be added to the supplemental designs. If the difference in life-cycle cost between one alternative and the others is greater than 10%, then that alternative with the lowest life-cycle cost is selected for construction.

If the difference in life-cycle costs is 10% or less, the selection will be based upon the alternate pavement bidding process described in Section 54-1.04 with only those alternatives within the 10% being taken forward to bidding. However if the project does not fit the criteria for alternate pavement bidding, or one pavement type is preferrable, the project will be referred to the Pavement Selection Committee. If the Committee agrees alternate pavement bidding is not appropriate, the Committee will select the pavement type.

3. Waivers. Although the guidelines presented in Item 1 and Item 2 will apply in most cases, a waiver based on issues related to policy, Local Agency requests, or constructibility may need to be considered. Such cases will be referred to the BDE for approval.

54-7.03 Life-Cycle Activities

Figures 54-7.A through 54-7.C present the maintenance and rehabilitation activities during 45 years of service. Figure 54-7.A illustrates the activities for mechanistically designed JPCPs and unbonded JPC overlays. Figure 54-7.B illustrates the activities for CRCPs and unbonded CRC overlays. Figure 54-7.C illustrates the activities for mechanistically designed full-depth HMA pavements and HMA overlays of rubblized PCC pavements.

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ACTIVITY 1 — YEAR 10 • 0.10% Class B Pavement Patching
ACTIVITY 2 — YEAR 15 • 0.20% Class B Pavement Patching
ACTIVITY 3 — YEAR 20 • 2.0% Class B Pavement Patching • 0.50% Class C Shoulder Patching • 100% Longitudinal Shoulder Joint Routing & Sealing • 100% Centerline Joint Routing & Sealing
ACTIVITY 4 — YEAR 25 • 3.0% Class B Pavement Patching • 1.0% Class C Shoulder Patching
ACTIVITY 5 — YEAR 30 • 4.0% Class B Pavement Patching • 1.5% Class C Shoulder Patching • Policy HMA Overlay of Pavement and Shoulder (see Chapter 53-4.04 for thickness)
ACTIVITY 6 — YEAR 35 • 100% Longitudinal Shoulder Joint Routing & Sealing • 100% Centerline Joint Routing & Sealing • 50% Random Crack Routing & Sealing (see Note) • 40% Reflective Transverse Crack Routing & Sealing • 0.10% Partial-Depth Pavement Patching (Mill & Fill Surface - Interstates; Mill & Fill 2.50 in. - Non-Interstates)
ACTIVITY 7 —YEAR 40 • 0.50% Class B Pavement Patching • 100% Longitudinal Shoulder Joint Routing & Sealing • 100% Centerline Joint Routing & Sealing • 60% Reflective Transverse Crack Routing & Sealing • 50% Random Crack Routing & Sealing (see Note) • 0.50% Partial-Depth Pavement Patching (Mill & Fill Surface - Interstates; Mill & Fill 2.50 in. - Non-Interstates)

Note: For random crack routing and sealing, assume 100 ft/station/lane.

MAINTENANCE AND REHABILITATION ACTIVITY SCHEDULE
JOINTED PLAIN CONCRETE PAVEMENT
AND UNBONDED JOINTED PLAIN CONCRETE OVERLAY

Figure 54-7.A

ACTIVITY 1 — YEAR 10 • 0.10% Class A Pavement Patching
ACTIVITY 2 — YEAR 15 • 0.20% Class A Pavement Patching
ACTIVITY 3 — YEAR 20 • 0.50% Class A Pavement Patching • 100% Longitudinal Shoulder Joint Routing & Sealing • 100% Centerline Joint Routing & Sealing
ACTIVITY 4 — YEAR 25 • 0.75% Class A Pavement Patching • 0.50% Class C Shoulder Patching
ACTIVITY 5 — YEAR 30 • 3.0% Class A Pavement Patching • 1.0% Class C Shoulder Patching • Policy HMA Overlay of Pavement and Shoulder (see Chapter 53-4.04 for thickness)
ACTIVITY 6 — YEAR 35 • 100% Longitudinal Shoulder Joint Routing & Sealing • 100% Centerline Joint Routing & Sealing • 50% Random Crack Routing & Sealing (see Note) • 0.10% Partial-Depth Pavement Patching (Mill & Fill Surface)
ACTIVITY 7 —YEAR 40 • 100% Longitudinal Shoulder Joint Routing & Sealing • 100% Centerline Joint Routing & Sealing • 50% Random Crack Routing & Sealing (see Note) • 0.50% Class A Pavement Patching • 0.50% Partial-Depth Patching (Mill & Fill Surface)

Note: For random crack routing and sealing, assume 100 ft/station/lane.

MAINTENANCE AND REHABILITATION ACTIVITY SCHEDULE
CONTINUOUSLY REINFORCED CONCRETE PAVEMENT
AND UNBONDED CONTINUOUSLY REINFORCED CONCRETE OVERLAY

Figure 54-7.B

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ACTIVITY 1 — YEAR 5

- 100% Longitudinal Shoulder Joint Routing & Sealing
- 100% Centerline Joint Routing & Sealing
- 50% Random/Thermal Crack Routing & Sealing (see Note)
- 0.10% Partial-Depth Pavement Patching (Mill & Fill Surface)

ACTIVITY 2 — YEAR 10

- 100% Longitudinal Shoulder Joint Routing & Sealing
- 100% Centerline Joint Routing & Sealing
- 50% Random/Thermal Crack Routing & Sealing (see Note)
- 0.50% Partial-Depth Pavement Patching (Mill & Fill Surface)

ACTIVITY 3 — YEAR 15

- 2.00 in. Milling - Pavement & Shoulder
- 1.0% Partial-Depth Pavement Patching (Mill & Fill Additional 2.00 in.)
- 2.00 in. HMA Overlay - Pavement & Shoulder

ACTIVITY 4 — YEAR 20

- 100% Longitudinal Shoulder Joint Routing & Sealing
- 100% Centerline Joint Routing & Sealing
- 50% Random/Thermal Crack Routing & Sealing (see Note)
- 0.10% Partial-Depth Pavement Patching (Mill & Fill Surface)

ACTIVITY 5 — YEAR 25

- 100% Longitudinal Shoulder Joint Routing & Sealing
- 100% Centerline Joint Routing & Sealing
- 50% Random/Thermal Crack Routing & Sealing (see Note)
- 0.50% Partial-Depth Pavement Patching (Mill & Fill Surface)

ACTIVITY 6 — YEAR 30

Interstate Standard Design:

- 2.00 in. Milling - Pavement Only
- 2.0% Partial-Depth Pavement Patching (Mill & Fill Additional 2.00 in.)
- 1.0% Partial-Depth Shoulder Patching (Mill & Fill Surface)
- 3.75 in. HMA Overlay Pavement
- 1.75 in. HMA Overlay Shoulder

Other State Maintained Route Standard Design:

- 2.00 in. Milling - Pavement & Shoulder
- 2.0% Partial-Depth Pavement Patching (Mill & Fill Additional 2.00 in.)
- 1.0% Partial-Depth Shoulder Patching (Mill & Fill Additional 2.00 in.)
- 2.25 in. HMA Overlay Pavement & Shoulder

All Limiting Strain Criterion Designs:

- 2.00 in. Milling - Pavement & Shoulder
- 2.0% Partial-Depth Pavement Patching (Mill & Fill Additional 2.00 in.)
- 1.0% Partial-Depth Shoulder Patching (Mill & Fill Additional 2.00 in.)
- 2.00 in. HMA Overlay - Pavement & Shoulder

MAINTENANCE AND REHABILITATION ACTIVITY SCHEDULE
FULL-DEPTH HMA PAVEMENT
AND HMA OVERLAY OF RUBBLIZED PCC PAVEMENT

Figure 54-7.C

ACTIVITY 7 — YEAR 35
• 100% Longitudinal Shoulder Joint Routing & Sealing • 100% Centerline Joint Routing & Sealing • 50% Random/Thermal Crack Routing & Sealing (see Note) • 0.10% Partial-Depth Pavement Patching (Mill & Fill Surface)
ACTIVITY 8 — YEAR 40
• 100% Longitudinal Shoulder Joint Routing & Sealing • 100% Centerline Joint Routing & Sealing • 50% Random/Thermal Crack Routing & Sealing (see Note) • 0.50% Partial-Depth Pavement Patching (Mill & Fill Surface)

Note: For random/thermal crack routing and sealing, assume 110 ft/station/lane.

MAINTENANCE AND REHABILITATION ACTIVITY SCHEDULE
FULL-DEPTH HMA PAVEMENT
AND HMA OVERLAY OF RUBBLIZED PCC PAVEMENT

Figure 54-7.C
(Continued)

54-7.04 Cost Analysis

The costs of all major pay items will be based upon the anticipated quantities for the contract. Computations that are used to develop costs will be documented for each major pay item. Contact the BDE for the Department's computer analysis procedures for life-cycle cost estimation.

54-7.05 Selection Process

Pavement selection will be based on the annual costs of initial construction and life-cycle activities amortized over the 45-year service life of the pavement. A discount rate of 3% will be used to determine annual costs and no adjustment for inflation will be required. Use the following equation to determine annual costs of alternatives during the selection process:

$$A = D + M + CRF_n\,[C + R_1(PWF_{n1}) + R_2(PWF_{n2}) + \ldots + R_n(PWF_{nn})] \qquad \text{Equation 54-7.1}$$

where:

A = total annual cost per mile

D = annual administrative and overhead cost per mile (assumed to be equal for all pavement types; therefore, do not include in analysis)

M = total annual maintenance cost per mile (assumed to be equal for all pavement types; therefore, do not include in analysis)

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CRF_n = capital recovery factor for year n calculated as follows (see Figure 54-7.D):

$$= \frac{i(1+i)^n}{(1+i)^n - 1} \qquad \text{Equation 54-7.2}$$

i = discount rate (assumed to be 0.03 (i.e., 3%))

n = year within analysis period in number of years after initial construction

C = initial construction cost per mile

R_1 = first rehabilitation cost per mile

R_2 = second rehabilitation cost per mile

R_n = n^{th} rehabilitation cost per mile

PWF_{nn} = present worth factor for the nth number of years after initial construction that the nth rehabilitation activity is performed calculated as follows (see Figure 54-7.D):

$$= \frac{1}{(1+i)^{nn}} \qquad \text{Equation 54-7.3}$$

n1 = number of years after initial construction that the first rehabilitation activity is performed

n2 = number of years after initial construction that the second rehabilitation activity is performed

nn = number of years n after initial construction that the nth rehabilitation activity is performed

If the difference in life-cycle cost between one alternative and the others is greater than 10%, select the alternative with the lowest life-cycle cost. If the difference in life-cycle costs is 10% or less, the selection will be based upon the alternate pavement bidding process described in Section 54-1.04 with only those alternatives within the 10% being taken forward to bidding. However if the project does not fit the criteria for alternate pavement bidding, or one pavement type is preferrable, the project will be referred to the Pavement Selection Committee. If the Committee agrees alternate pavement bidding is not appropriate, the Committee will select the pavement type.

The Pavement Selection Committee consists of five Department personnel (i.e., three from the Central Office and two from the district). Regional Engineers, Deputy Directors, and other high-ranking personnel are excluded from the Committee. Committee meetings usually are held by conference calls. Factors that are considered by the Committee during the selection process are documented in the AASHTO *Guide to Design of Pavement Structures*. Committee deliberations are considered confidential and only the Committee's recommendation as to the final pavement selection is recorded.

n	CRF_n①②	PWF_n①③
5	--	0.8626
10	--	0.7441
15	--	0.6419
20	--	0.5537
25	--	0.4776
30	--	0.4120
35	--	0.3554
40	--	0.3066
45	0.04079	--

Notes:

① *Factors in Figure 54-7.D are applicable only for a discount rate of 3% (i.e., i = 0.03).*
② *See Equation 54-7.2.*
③ *See Equation 54-7.3.*

CAPITAL RECOVERY AND PRESENT WORTH FACTORS

Figure 54-7.D

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54-8 PAVEMENT DESIGN SUBMITTALS

54-8.01 Submittal Requirement

Pavement designs for projects involving new construction, reconstruction, or widening greater than or equal to 6 ft will be submitted to BDE for approval if the project involves:

- more than 4,750 sq yds of pavement;
- more than $500,000 in pavement costs (see 20 ILCS 2705/2705-590);
- high stress intersections, experimental pavements, or special designs;
- requests for design exceptions; or
- an expired pavement design (see Section 54-8.03).

Once approved, BDE will post pavement designs on the IDOT website for public information. Approved pavement designs shall be included in the Phase I project report or file.

54-8.02 Submittal Content

All pavement design submittals that are forwarded to the BDE will include the following items:

1. Transmittal Memorandum. Include a memorandum of transmittal with the submittal. The memorandum should include information such as the route, section, county, district, and recommendation.

2. Sketch Map. Include a sketch map showing the location of the subject improvement and the limits of the analysis.

3. Typical Sections. Include typical cross-sections for standard pavement types and for any special or unusual pavement treatment. Typical sections should include information detailing the number of lanes, shoulders, curbs and gutters, and other information as appropriate. Usually, only one typical section is required.

4. Subgrade Stability Chart. If the soil condition is found to be fair or granular, include the subgrade stability chart used in the analysis. Also, include documentation for any unusual soil conditions that affect pavement design (e.g., laboratory test results). Because a majority of Illinois soils exhibit poor subgrade stability, it is not necessary to include detailed information to substantiate this fact.

5. Design Calculations. Include all design calculations and assumptions (e.g., traffic factor calculations, thickness calculations, thickness nomographs and related charts, temperature location map). Clearly illustrate how the pavement thickness was determined.

6. Economic Analysis. Include all calculations and assumptions related to the economic analysis (e.g., first-cost analysis, life-cycle cost analysis, total annual cost per mile,

maintenance and rehabilitation activities, capital recovery and present worth factors, discount rate). Clearly illustrate how the economic analysis was performed.

7. Unit Cost Sheets. Provide unit cost computation sheets that document the unit costs used for each major pay item involved in each of the alternative designs. The unit costs must be based on the total anticipated quantities of the major pay items involved for the entire contract section.

8. Other Information. Include adequate documentation that describes any unusual factors affecting design or that influenced the pavement selection (e.g., construction staging, high-stress locations, Local Agency requests, unusual traffic volumes, traffic count summary sheets, memoranda from District Bureaus of Planning, references to current Interstate cost studies).

9. Recommendation. Clearly identify and describe the recommended design and the basis for pavement selection.

54-8.03 Shelf-Life of Approved Pavement Designs

To ensure the department is using relevant data in the design and selection of pavements as required by the Department of Transportation Law (20 ILCS 2705/2705-590), the approval of a pavement design will expire after 5 years.

To avoid having a pavement design expire in close proximity to a project's anticipated construction letting date, the submittal of pavement designs to BDE for approval should be coordinated with the letting date; or the design should be updated and re-submitted for approval sufficiently in advance of the letting date.

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54-9 DESIGN EXAMPLES

The following examples illustrate typical IDOT pavement designs:

* * * * * * * * * *

Example 54-9.1

Given: New rural 4-lane State highway in central Sangamon County
ADT (Design Year Traffic) PV = 17,225 SU = 360 MU = 360
Subgrade Support Rating (SSR) = Poor
Asphalt Binder Type = PG64-22

Because this involves new construction, it will be necessary to perform both a rigid and flexible mechanistic design. See Figure 54-1.A.

Solution:

Step 1: Determine the actual traffic factor using Equations 54-4.1 and 54-5.1.

TF_R (Actual)

$$TF_{R(A)} = 20\frac{(0.15 \bullet 0.32 \bullet 17{,}225) + (143.81 \bullet 0.45 \bullet 360) + (696.42 \bullet 0.45 \bullet 360)}{1x10^6}$$

= 2.74

TF_F (Actual)

$$TF_{F(A)} = 20\frac{(0.15 \bullet 0.32 \bullet 17{,}225) + (132.50 \bullet 0.45 \bullet 360) + (482.53 \bullet 0.45 \bullet 360)}{1x10^6}$$

= 2.01

Step 2: Check minimum traffic factor.

TF_R (Minimum)

$$TF_{R(M)} = 20\frac{(143.81 \bullet 0.45 \bullet 250) + (696.42 \bullet 0.45 \bullet 750)}{1x10^6}$$

= 5.02

TF_F (Minimum)

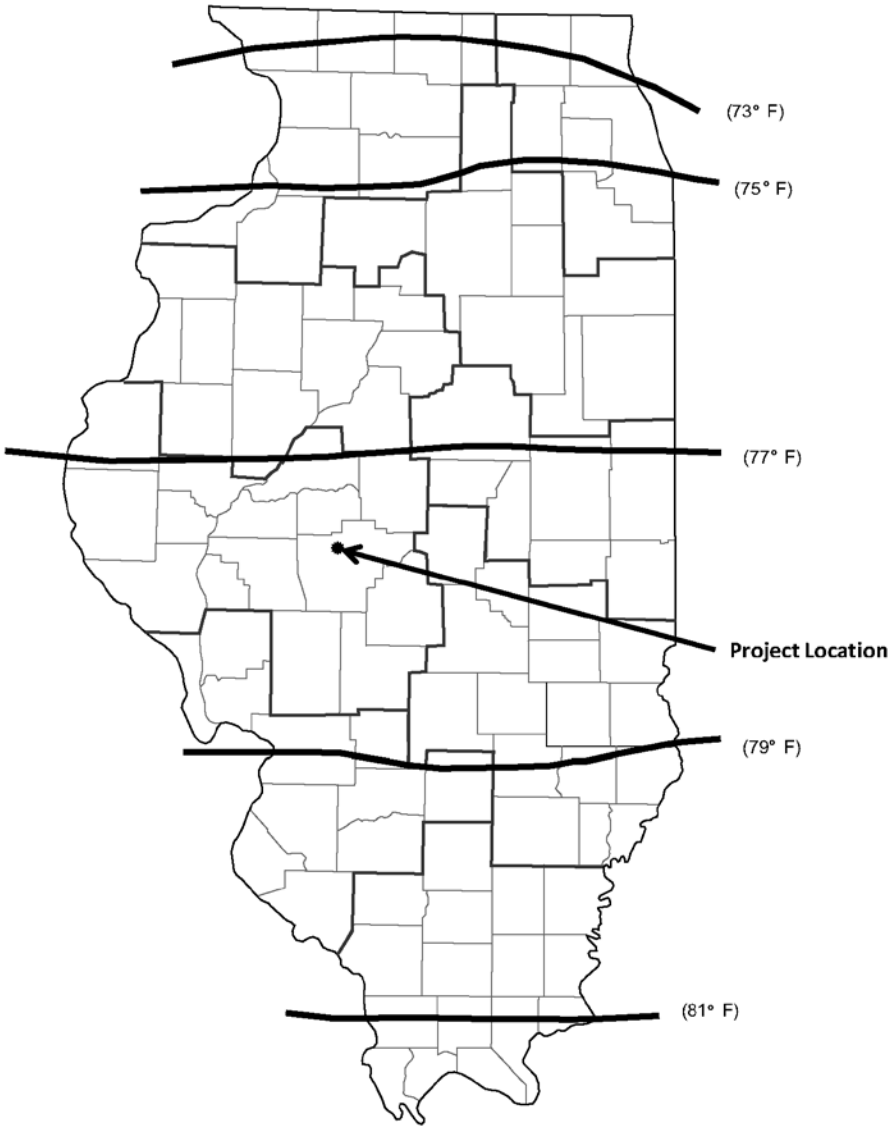
$$TF_{F(M)} = 20\frac{(132.50 \bullet 0.45 \bullet 250) + (482.53 \bullet 0.45 \bullet 750)}{1x10^6}$$

= 3.56

Because the minimum traffic results in a higher traffic factor, use the minimums.

Step 3: Determine the pavement HMA mixture temperature from Figure 54-5.C and round up to the nearest 0.5 degree. See Figure 54-9.A.

HMA Mix Temperature = 78.0°F



Note: The minimum design HMA mixture temperature will be 73°F.

HMA MIXTURE TEMPERATURE
(Mechanistic Design: Flexible Pavement)

Figure 54-9.A

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Step 4: Determine Design E_{HMA} from Figure 54-5.D and round to nearest 10. See Figure 54-9.B.

Design E_{HMA} = 610 (i.e., 607 rounded to 610)

Step 5: Determine the Design HMA strain using the $TF_{F(M)}$ and Figure 54-5.E. See Figure 54-9.C.

Design HMA strain = 84

Step 6: Using the Design E_{HMA} and the Design HMA strain, determine the flexible thickness from Figure 54-5.F for a poor subgrade and round up to the nearest 0.25 in. See Figure 54-9.D.

Thickness = 10.75 in. (i.e., 10.74 in. rounded up to 10.75 in.)

Step 7: Determine the limiting strain criterion (LSC) design thickness from Figure 54-5.I and round up to the nearest 0.25 in. See Figure 54-9.E.

LSC Thickness = 16.00 in. (i.e., 15.78 in. rounded up to 16.00 in.) is greater than standard design thickness – use HMA thickness of 10.75 in.

Step 8: Using the $TF_{R(M)}$ and Figure 54-4.E for a poor subgrade, determine the rigid thickness. Round up to the nearest 0.25 in. See Figure 54-9.F.

Thickness = 9.00 in. for tied shoulder (i.e., 8.87 in. rounded up to 9.00 in.)

Results: For the design thickness, use a 10.75 in. full-depth HMA pavement or a 9.00 in. JPCP with tied PCC shoulders. A 45-year life-cycle cost analysis must be performed to determine the pavement type.

If a modified AASHTO rigid pavement design thickness was required to match an existing pavement using the above given factors, the following solution would apply:

Given: IBR = 3

Solution:

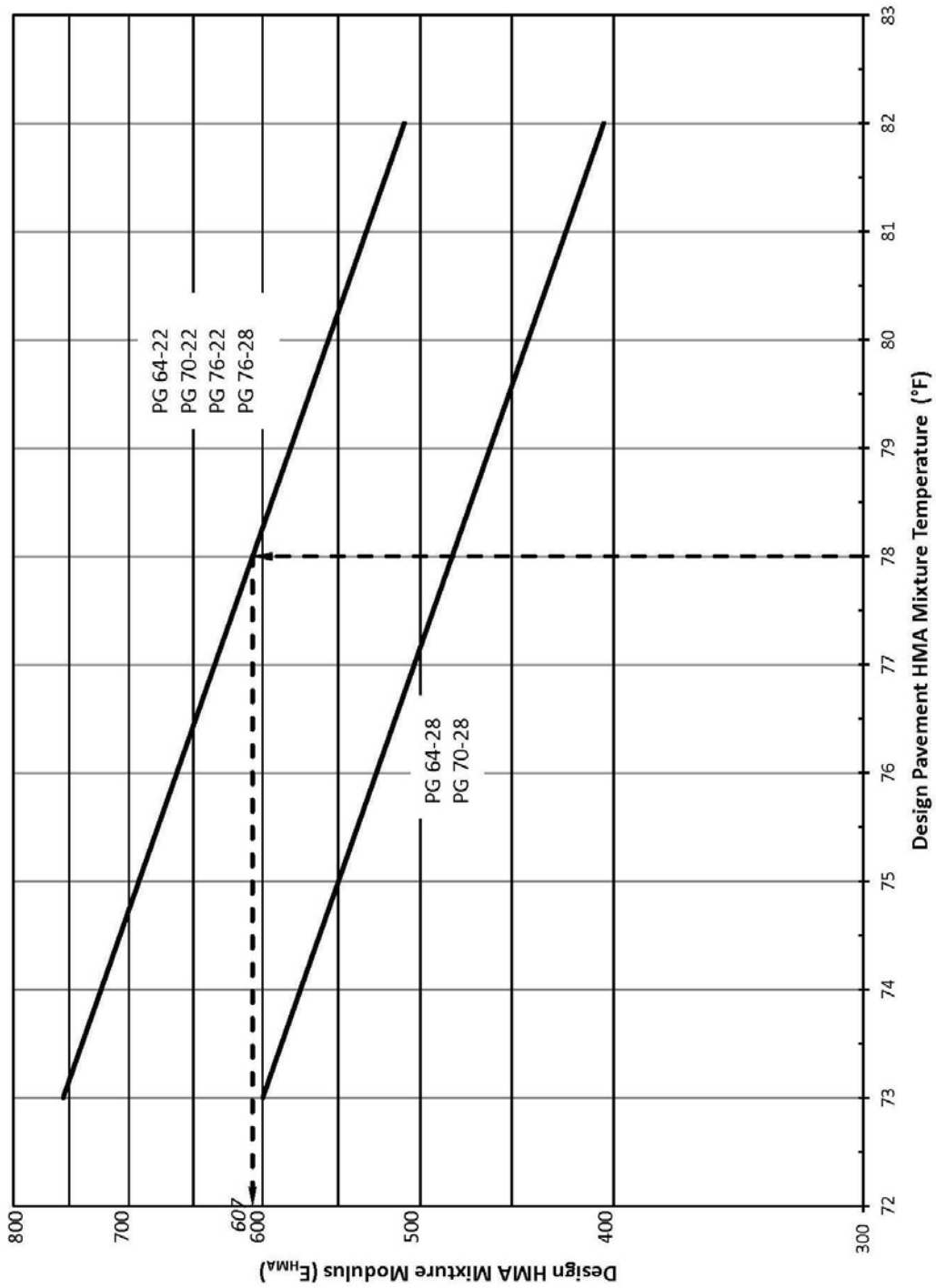
Step 1: The actual TF_R applies.

$TF_{R(A)}$ = 2.74

Step 2: Using the $TF_{R(A)}$ and Figure 54-4.M and an IBR of 3, determine the rigid pavement thickness. Round up to the nearest 0.25 in. See Figure 54-9.G.

Thickness = 9.25 in. (i.e., 9.15 in. rounded up to 9.25 in.)

Step 3: For a conventional flexible pavement, see Example 54-9.2.



HMA MIXTURE MODULUS (E_{HMA})
(Mechanistic Design: Flexible Pavement)

Figure 54-9.B

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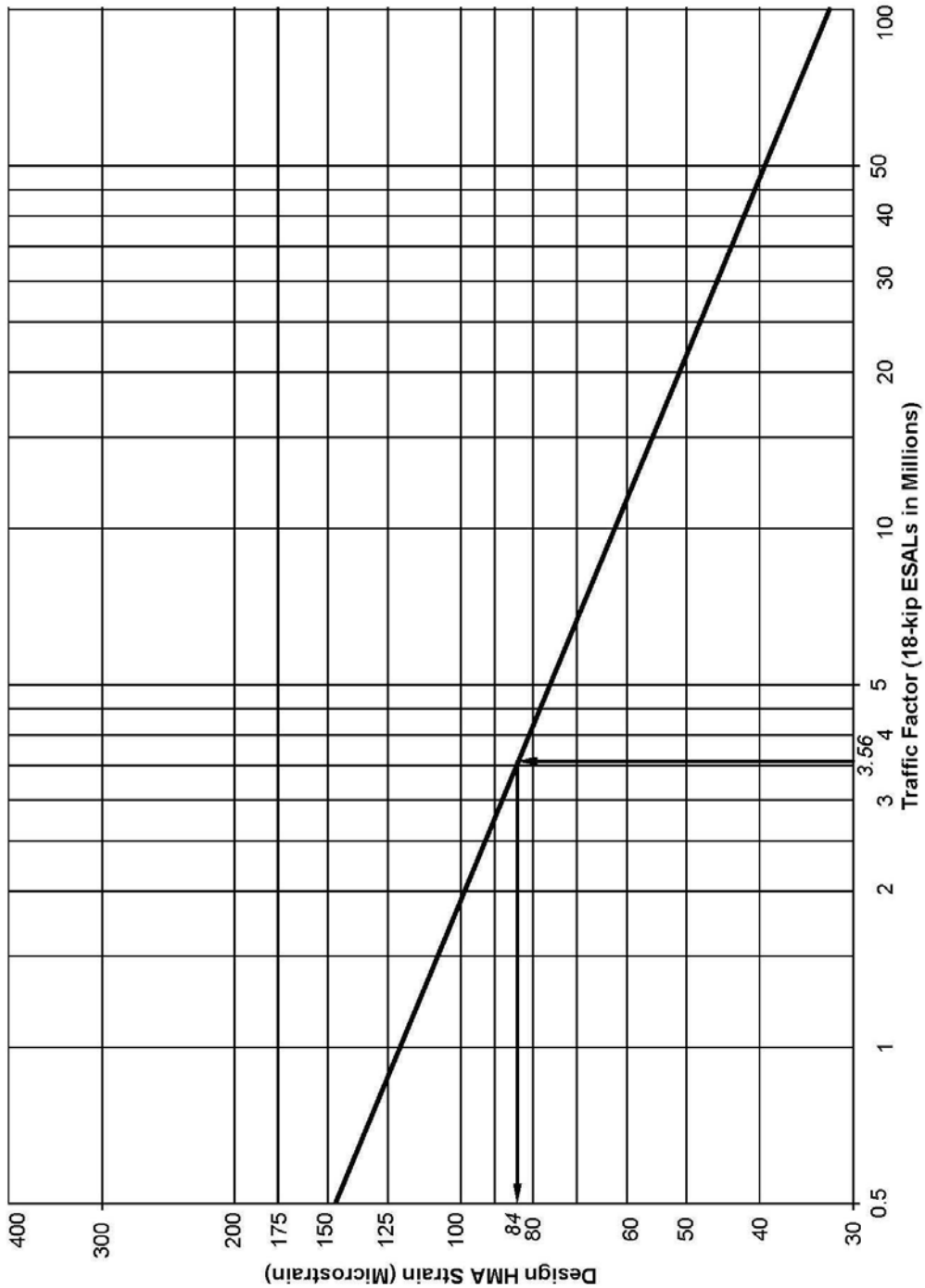
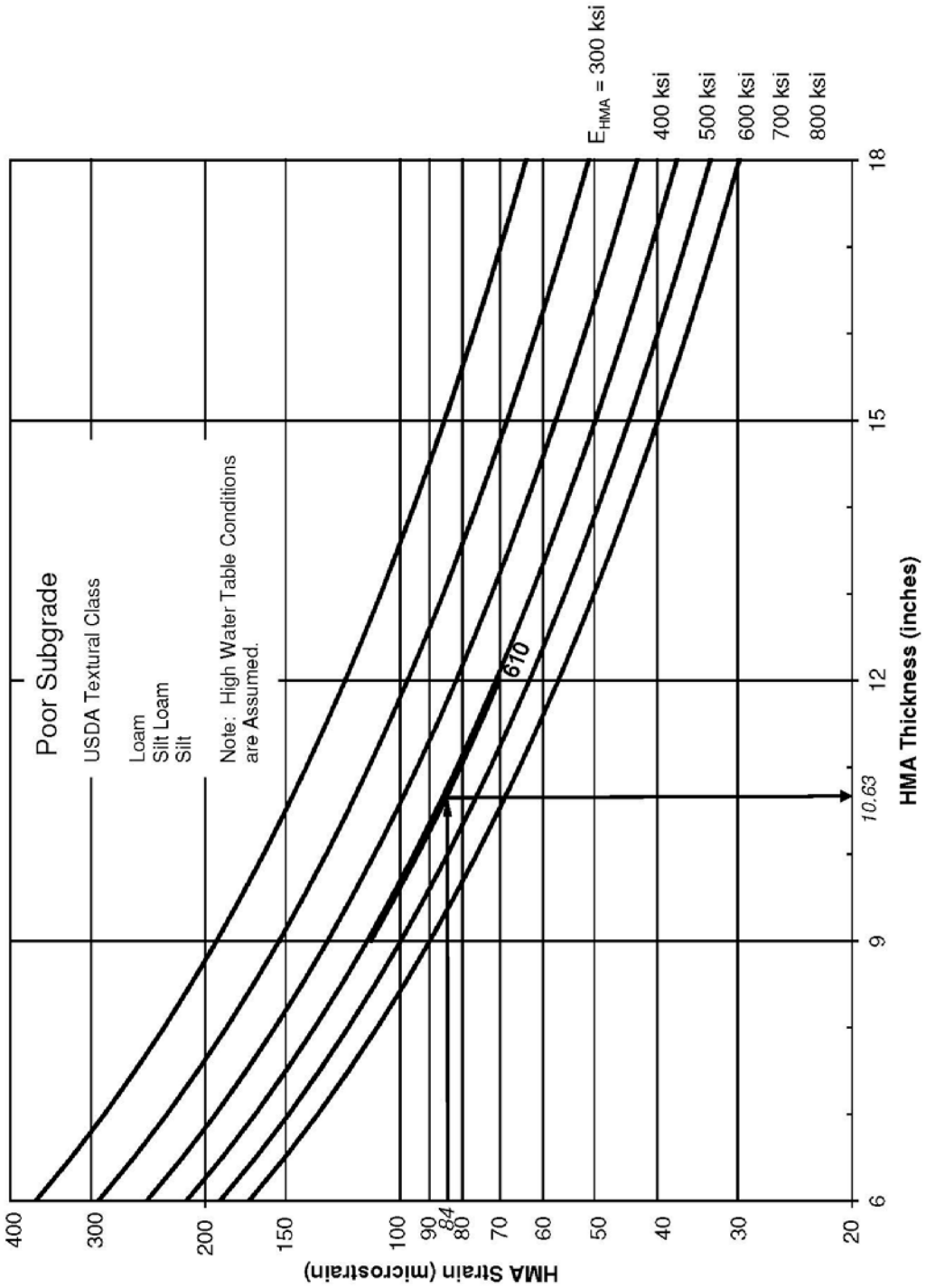


DESIGN HMA STRAIN
(Mechanistic Design: Flexible Pavement)

Figure 54-9.C



HMA THICKNESS DESIGN CHART
(Mechanistic Design: Flexible Pavement: SSR = Poor)

Figure 54-9.D

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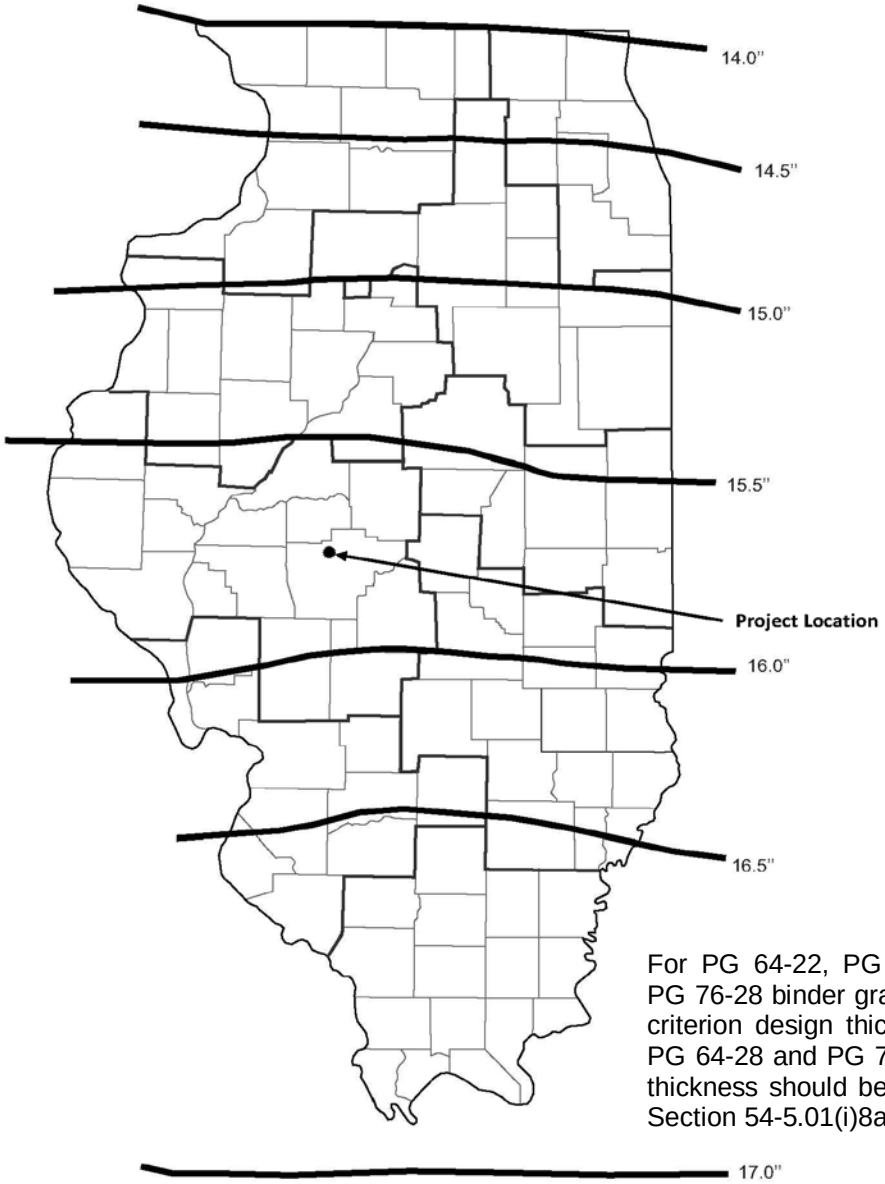
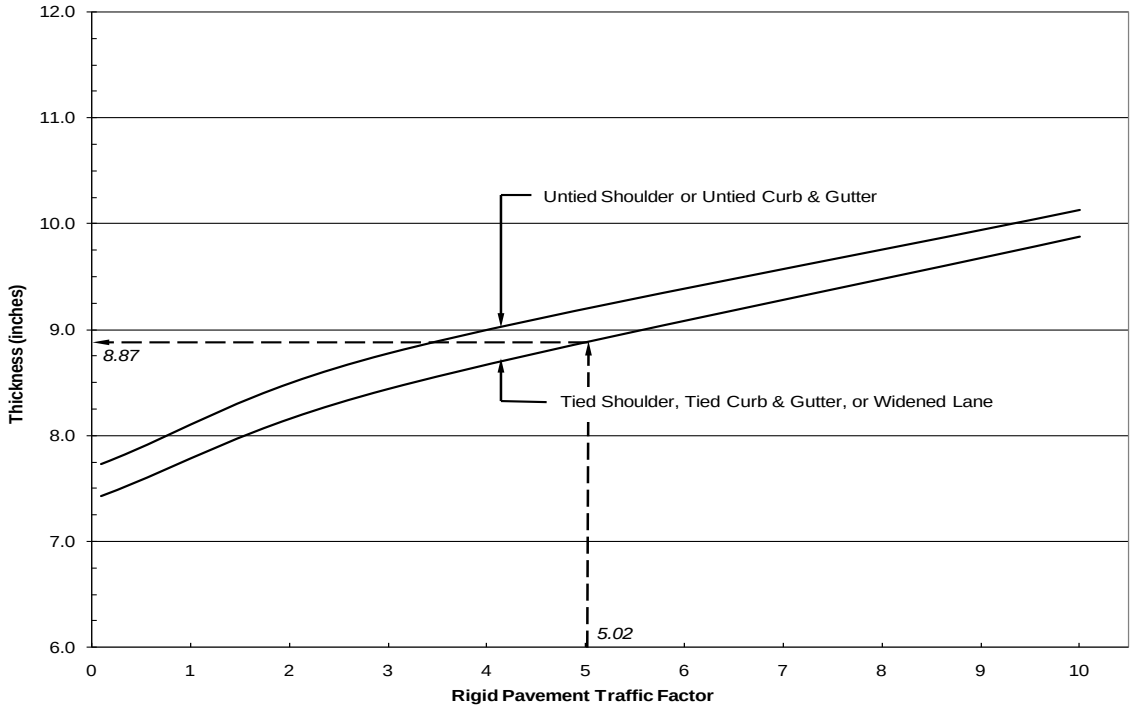


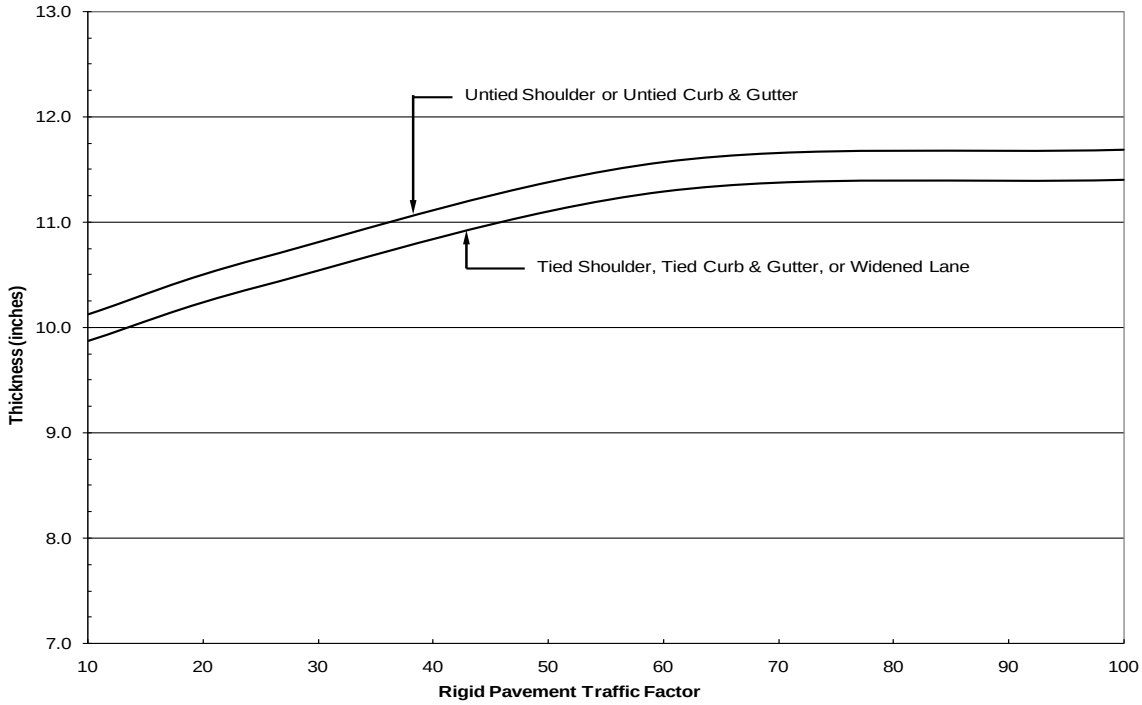
For PG 64-22, PG 70-22, PG 76-22, and PG 76-28 binder grades, read limiting strain criterion design thickness off of map. For PG 64-28 and PG 70-28 binder grades, the thickness should be corrected according to Section 54-5.01(i)8a.

Note: Thickness values based upon Mean Monthly Pavement Termperature at 4 in. depth correlated to July Mean Monthly Air Temperature, axle load of 20,000 lb, strain of 70 με and E_{Ri} of 2 ksi.

MAXIMUM PAVEMENT THICKNESS
(Limiting Strain Criterion Design: Flexible Pavement)

Figure 54-9.E





Note: Use of untied shoulder design requires BDE approval.

RIGID PAVEMENT DESIGN CHART
(Mechanistic Design: SSR = Poor)

Figure 54-9.F

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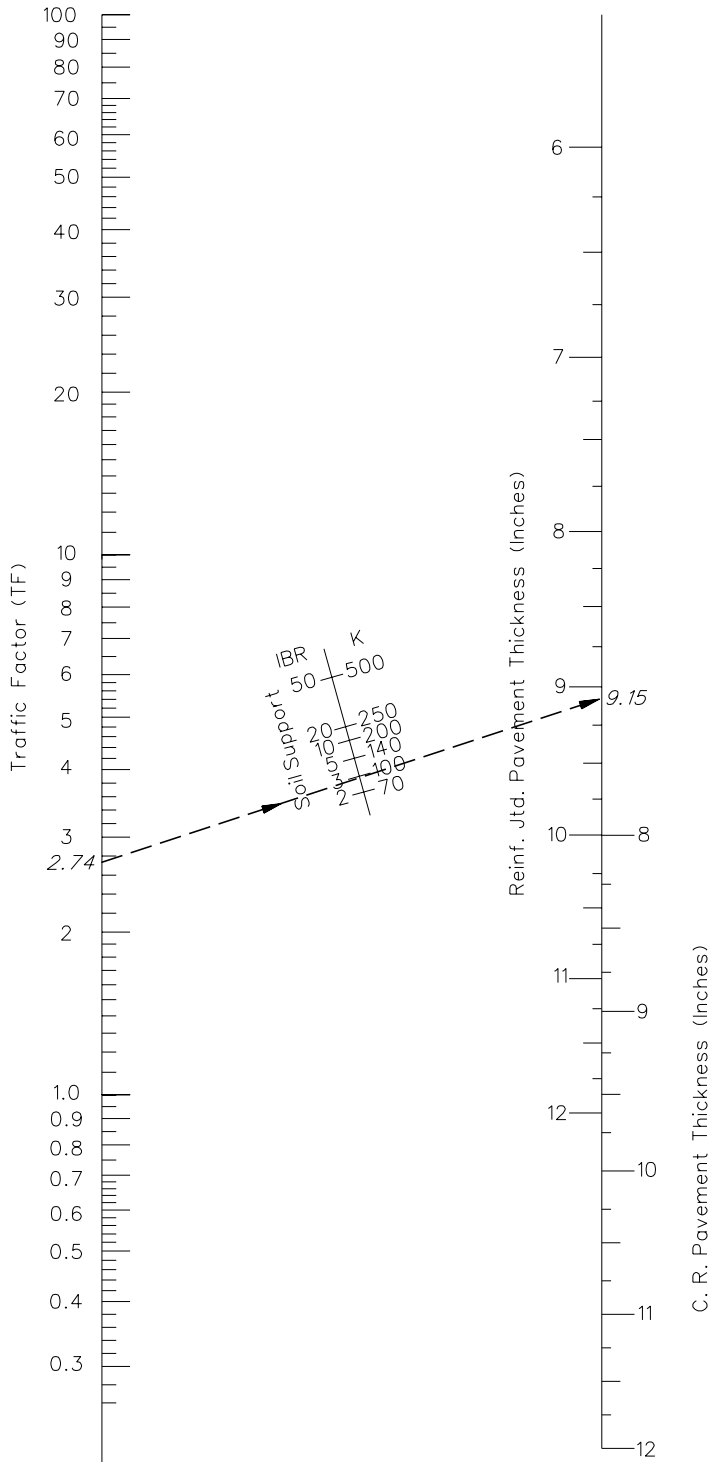

RIGID PAVEMENT DESIGN NOMOGRAPH
(Modified AASHTO Design: Class I Facilities: JRCP and CRCP)

Figure 54-9.G

Example 54-9.2

Given: Existing 2-lane rural State highway in central Sangamon County being widened to 4 lanes – 12 ft widening each side (with resurfacing)
ADT (Design Year Traffic) PV = 17,225 SU = 360 MU = 360
Subgrade Support Rating = Poor IBR = 4
Asphalt Binder Type = PG64-22

Because the design requires widening with resurfacing, the solution will involve preparing a mechanistic flexible, a modified AASHTO flexible, and a composite design. See Figure 54-1.A.

Solution:

Step 1: The mechanistic flexible design will be the same as in Example 54-9.1.

Step 2: Determine the modified AASHTO flexible design. The actual flexible traffic factor will be used.

$TF_{F(A)}$ = 2.01 (from Example 54-9.1)

From Figure 54-5.M, determine the flexible pavement structural number (SN_F).

SN_F = 4.4 (See Figure 54-9.H)

From Note 1 in Figure 54-5.S, the minimum SN_F for a multilane State highway is 5.0.

Using Equation 54-5.4, determine the layer thickness of the surface course, base course, and subbase course.

$$SN_F = a_1 D_1 + a_2 D_2 + a_3 D_3$$

From Figure 54-5.O:

a_1 = 0.40 (HMA)
a_2 = 0.30 (Assumes HMAs Stabilized Granular Material 1,500 MS)
a_3 = 0.11 (Assumes Granular Material, Type B)
D_1 = 2.5 in. (Policy resurfacing with widening)
D_3 = 4 in. (Assumed)

$$5.0 = (2.5 \bullet 0.4) + (0.3\ D_2) + (4 \bullet 0.11)$$

$$D_2 = \frac{5.0 - (1.0) - (0.44)}{0.30} = 11.86 \text{ in. use 12 in.}$$

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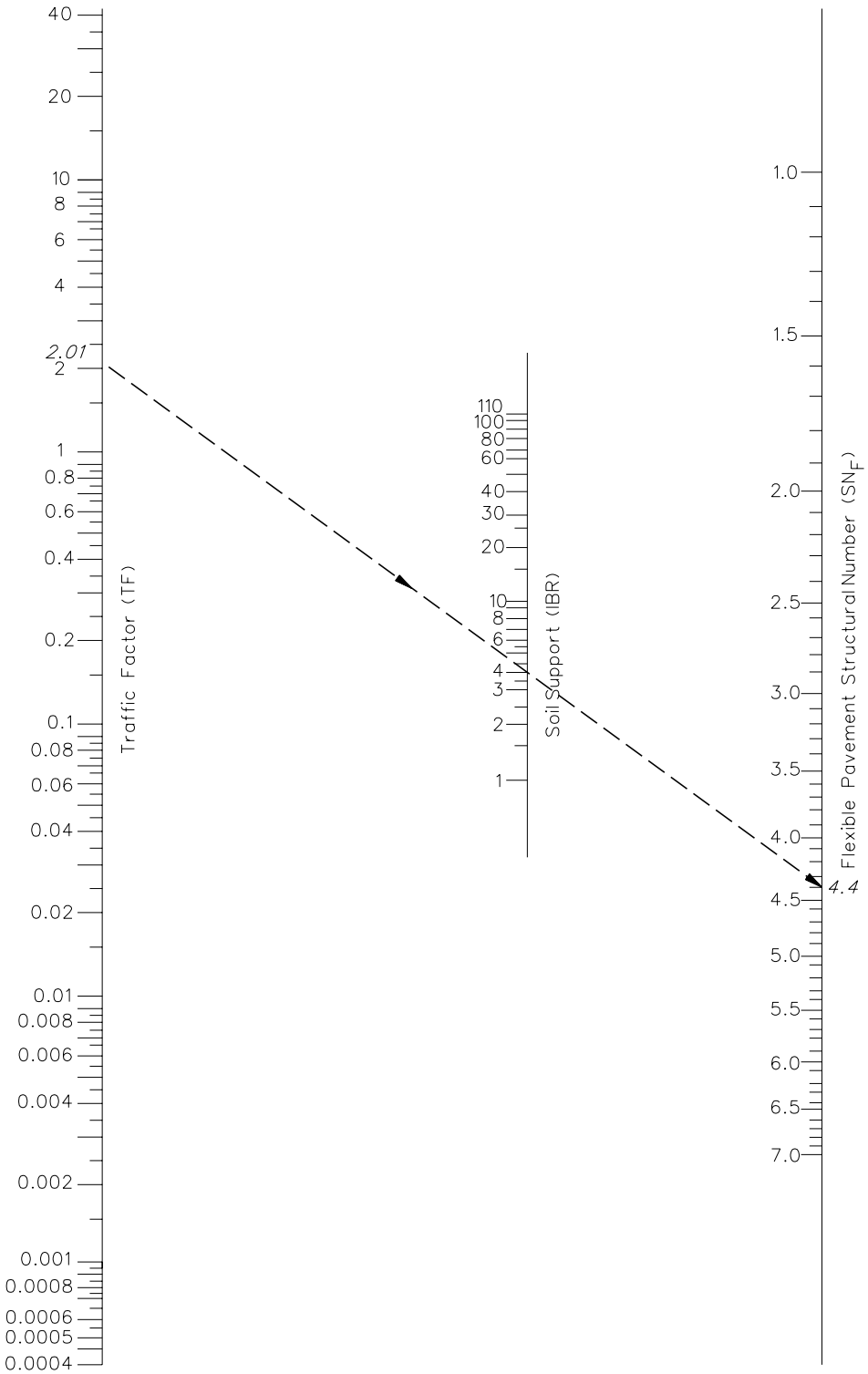

FLEXIBLE PAVEMENT DESIGN NOMOGRAPH
(Modified AASHTO Design: Class I Facilities)

Figure 54-9.H

Check:

$2.5 \bullet 0.4 = 1.00$

$12 \bullet 0.3 = 3.60$

$4 \bullet 0.11 = \underline{0.44}$

$5.04 > 5.0$ — OK

From Figure 54-5.S, minimum D_2 = 10 in. — OK

Step 3: Determine the composite design. The actual traffic factor will be used.

$TR_{R(A)}$ = 2.74 (from Example 54-9.1)

From Figure 54-6.A, determine the composite pavement structural number (SN_C).

SN_C = 3.55 (See Figure 54-9.I)

Using Equation 54-6.3, determine the layer thickness:

$$SN_C = 0.40\ D_S + 0.33\ D_B$$

$$D_B = \frac{3.55 - (0.4 \bullet 2.5)}{0.33}$$

D_B = 7.72 in. From Figure 54-6.D, minimum thickness is 8 in.

Results:

Mechanistic Flexible:	10.75 in. Full-Depth Flexible using PG64-22 12 in. Lime-Modified Subgrade
Modified AASHTO Flexible:	2.5 in. HMA Surface Course 12 in. HMA Binder Course 4 in. Aggregate Subbase
Composite Design:	2.5 in. HMA Surface Course 8 in. PCC Base Course

A first-cost economic analysis must be performed to determine the pavement type.

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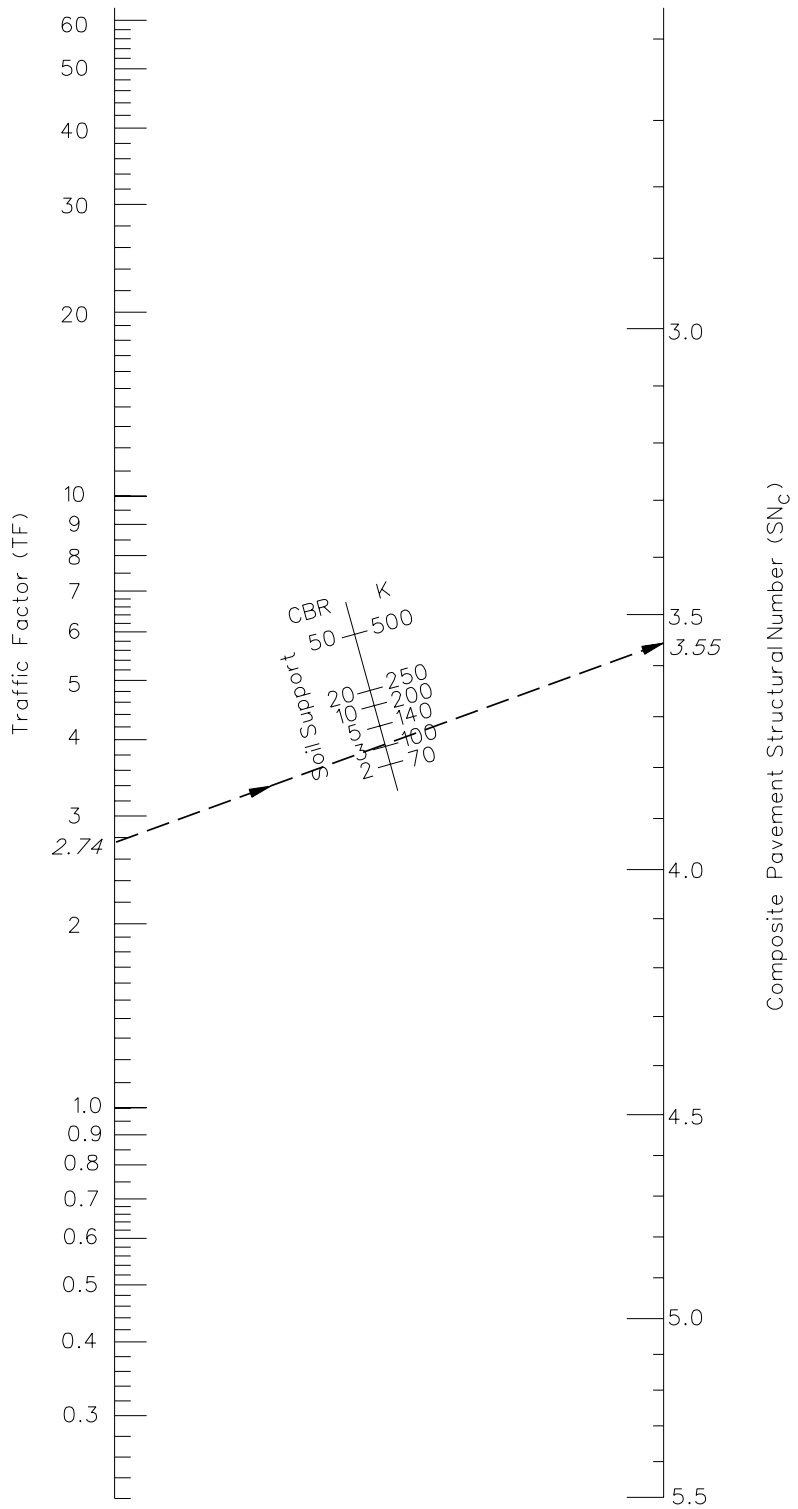

COMPOSITE PAVEMENT DESIGN NOMOGRAPH
(Class I Facilities)

Figure 54-9.I

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