Vermont Agency of Transportation

Pavement Design Guide

March 12, 2002

Vermont
Agency of Transportation

Flexible Pavement Design Procedures
for use with the

1993 AASHTO Guide for Design of Pavement Structures

The Vermont Agency of Transportation procedure for the design of new or reconstructed pavement structures is based on the 1993 AASHTO Guide for Design of Pavement Structures, referred to simply as the ‘93 Guide in this procedure. Pavement structures designed using this procedure must have a minimum 20-year Design Lane ESAL estimate of 100,000.

This procedure outlines the method to follow for flexible pavement design using sand borrow to achieve a frost resistant design without accounting for the structural benefit of this material. A three-stage construction strategy is described to allow the construction of a new pavement structure (first stage) and a planned rehabilitation of two future pavement overlay treatments (second and third stages) to achieve a 40-year design life.

1. **Several Design Inputs Are Required**

    a. Traffic
    b. Serviceability Loss
    c. Reliability Level
    d. Subgrade Resilient Modulus
    e. Pavement Material Parameters
    f. Frost Depth

2. **Traffic**

    a. 20-year and 40-year 18 kip ESALs for flexible pavement should be requested from the Traffic Research Unit, specifying the route number, project location, and the construction and 20- and 40-year design years.

    b. The designer must also review for the potential traffic effects caused by the proximity of seasonal traffic generators, i.e. recreational, school, logging, etc., and apprise Traffic Research of these findings in the ESAL request.

    c. Determine design lane 18 kip ESAL equivalents.

        1) Use distribution factors and procedure described below unless Traffic Research provides Design Lane ESALs.

2) Find the 18 kip ESAL distribution factor for the desired lane configuration from Table 1.

Table 1--18 Kip ESAL Distribution Factors	
Number of Lanes	**Distribution Factors**
2	0.50
3	0.50
4 (Rural)	0.60
4 (Urban)	0.60

3) Multiply the ESAL values by the Distribution factors to obtain Design Lane estimates for both 20- and 40-year 18 kip ESALs.

3. **<u>Serviceability Loss</u>**

a. Initial design serviceability index for flexible pavement is presently accepted as 4.0 for new pavement construction.

b. Subtract the terminal serviceability index of 2.5 for federal, state and Class 1 Town Highways (or 2.0 for local routes) from the initial serviceability index.

c. This equals the Design Serviceability Loss, ΔPSI.

4. **<u>Reliability Level</u>**

a. Overall design reliabilities shall be reflective of those listed in Table 2 and are dependent upon the functional classification of the project route. Urban/rural distinction is determined as defined by Federal-Aid Urban Areas and the VAOT "Functional Classification" map.

Table 2—Reliability Factors for Pavement Design		
Functional Classification	**Urban**	**Rural**
Interstate	99%	95%
Principal Arterial	95%	90%
Major and Minor Collectors and Minor Arterial	90%	85%
Local	75%	75%

These reliability factors may be used directly when implementing a DARWin solution reflecting a three-stage design. DARWin automatically accounts for the compounding effects of staged construction as described in Section 4.5 of Part I (pg. I-63 of '93 Guide).

b. If the designer opts for a nomographic solution (pg. II-32 '93 Guide), then the input reliability required must be the cube root (reflecting a three-stage strategy) of the overall reliability level desired in Table 2. The resulting reliability levels for nomograph use are listed in Table 3.

Table 3—Required Single-Stage Reliability Factors For a Three-Stage Pavement Design (when applying a nomographic solution)		
Functional Classification	**Urban**	**Rural**
Interstate	99.7%	98.3%
Principal Arterial	98.3%	96.5%
Major and Minor Collectors and Minor Arterial	96.5%	94.7%
Local	90.9%	90.9%

c. A standard deviation of 0.45 is required for both DARWin and nomograph solutions.

5. **Subgrade Resilient Modulus**

Since the Agency is promoting a frost resistant design, it is considered unlikely that the true subgrade soil will experience overstressing. Therefore, the "Subgrade Resilient Modulus" for design should be that provided by the sand borrow material. Unless the Pavement Design Committee or the Materials and Research Section provides more specific information, use 9,000 psi for the unfactored sand borrow modulus.

6. **Pavement Material Parameters**

The load carrying capacity of the pavement structure, referred to as the Structural Number (SN), must be sufficient to accommodate the estimated traffic. The layer and drainage coefficients must be identified for the materials composing the pavement that will provide the required SN.

$$SN = a_1\, d_1 + a_2\, d_2\, m_2 + a_3\, d_3\, m_3$$

DARWin's Layered Thickness Design procedure economizes the design by ensuring each material is not overstressed. The Layered Design Analysis is described in Section 3.1.5 of Part II of the '93 Guide (pp. II-35 - II-37).

Consult the "Material Properties" for recommended layer coefficient and resilient modulus values, unless the Pavement Design Committee or the Materials and Research Section provides more current information.

The drainage coefficients, m_2 & m_3, should be assumed to be 1.0 unless more specific information suggests otherwise.

7. **Frost Depth**

a. Determine the project maximum frost penetration on the Frost Penetration map. (Interpolate as necessary)

b. Determine the Design Frost Depth from Table 4 based on the appropriate functional classification for the project and the frost penetration determined above.

Table 4--Design Frost Depths

Functional Classification	**Factor**	**Maximum Frost Depth**						
		50"	55"	60"	65"	70"	75"	80"
Interstate	0.90	45"	50"	54"	55"	63"	68"	72"
Principal Arterial	0.80	40"	44"	48"	52"	56"	60"	64"
Minor Arterial	0.70	35"	39"	42"	46"	49"	53"	56"
Collectors	0.60	30"	33"	36"	39"	42"	45"	48"
Local Routes	0.40	20"	22"	24"	26"	28"	30"	32"

c. A more precise Design Frost Depth may be determined from multiplying the frost penetration resolved from the appropriate map by the factor listed in Table 4 for the corresponding functional classification of the road being designed.

8. <u>**Pavement Design Thickness Determination**</u>

Pavement structure design is composed of 1) the Asphalt Cement Concrete (ACC) requirements generated by a 20-year ESAL design, 2) the subbase requirements generated by a 40-Year ESAL design, and 3) sufficient frost resistant material to achieve frost depth calculated in "7."

a. Generate a full layered thickness design based on the appropriate material characteristics and the <u>20-Year</u> ESAL estimate.

b. Generate a full layered thickness design based on the appropriate material characteristics and the <u>40-Year</u> ESAL estimate.

c. Final design typical will consist of the ACC thickness determined in "a" conforming to the suggested layer thickness provided in Table 7. Use the subbase thickness determined in "b."

d. Determine the Sand Thickness

1) Subtract the final design typical depth determined in "8.c." from the Design Frost Depth.

2) Round to the nearest 3" increment.

e. The following minimum thicknesses shall be used for any new or reconstructed pavement structure.

1) The minimum ACC thickness shall be 4" for designs less than 1,000,000 ESALs and 5" for designs greater than 1,000,000 ESALs.

2) The minimum subbase material thickness shall be 18" for State Routes and 12" for Rural Town Highways.

3) The minimum sand thickness shall be 12" for State Routes unless subgrade soil is classified as A-1-a or A-3 (determination made from borings). Rural Town Highways can be 6".

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Metric Flexible Pavement Design Procedures
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<u>1993 AASHTO Guide for Design of Pavement Structures</u>

The Vermont Agency of Transportation procedure for the design of new or reconstructed pavement structures is based on the 1993 AASHTO Guide for Design of Pavement Structures, referred to simply as the Guide in this procedure. Pavement structures designed using this procedure must have a minimum 20-year Design Lane ESAL estimate of 100,000.

This procedure outlines the method to follow for flexible pavement design using sand borrow to achieve a frost resistant design without accounting for the structural benefit of this material. A three-stage construction strategy is described to allow the construction of a new pavement structure (first stage) and a planned rehabilitation of two future pavement overlay treatments (second and third stages) to achieve a 40-year design life.

1. **<u>Several Design Inputs Are Required</u>**

    a. Traffic
    b. Serviceability Loss
    c. Reliability Level
    d. Subgrade Resilient Modulus
    e. Pavement Material Parameters
    f. Frost Depth

2. **<u>Traffic</u>**

    a. 20-year and 40-year 80 kN ESALs for flexible pavement should be requested from the Traffic Research Unit, specifying the route number, project location and the construction and 20- and 40-year design years.

    b. The designer must also review for the potential traffic effects caused by the proximity of seasonal traffic generators, i.e., recreational, school, logging, etc., and apprise Traffic Research of these findings in the ESAL request.

    c. Determine design lane 80 kN ESAL equivalents.

        1) Use the following distribution factors and procedure unless Traffic Research provides Design Lane ESALs.

2) Find 80 kN ESAL distribution factor for desired lane configuration from Table 1.

Table 1--80 kN ESAL Distribution Factors	
Number of Lanes	**Distribution Factors**
2	0.50
3	0.50
4 (Rural)	0.60
4 (Urban)	0.60

3) Multiply ESAL values by Distribution factors to obtain Design Lane estimates for both 20- and 40-year 80 kN ESALs.

3. **<u>Serviceability Loss</u>**

a. Initial design serviceability index for new flexible pavement is presently accepted as 4.0, and for other rehabilitative treatments it is 4.2.

b. Subtract the terminal serviceability index of 2.5 for federal, state and Class 1 Town Highways (or 2.0 for local routes) from the initial serviceability index.

c. This equals the Design Serviceability Loss, ΔPSI.

4. **<u>Reliability Level</u>**

a. Overall design reliabilities shall be reflective of those listed in Table 2 and are dependent upon the functional classification of the project route. Urban/rural distinction is determined as defined by Federal-Aid Urban Areas and the VAOT "Functional Classification" map.

Table 2--Reliability Factors for Pavement Design		
Functional Classification	**Urban**	**Rural**
Interstate	99%	95%
Principal Arterial	95%	90%
Major and Minor Collectors and Minor Arterial	90%	85%
Local	75%	75%

These factors may be used directly when implementing a DARWin solution reflecting a three-stage design. DARWin automatically accounts for the compounding effects of staged construction as described in Section 4.5 of Part I (p. I-63 of the Guide).

b. If the designer opts for a nomographic solution (p. II-32 of the Guide), then the input reliability required must be the cube root (reflecting a three-stage strategy) of the overall reliability level desired in Table 2. The resulting reliability levels for nomograph use are listed in Table 3.

Table 3--Required Single-stage Reliability Factors for a Three-stage Pavement Design (when applying a nomographic solution)		
Functional Classification	**Urban**	**Rural**
Interstate	99.7%	98.3%
Principal Arterial	98.3%	96.5%
Major and Minor Collectors and Minor Arterial	96.5%	94.7%
Local	90.9%	90.9%

c. A standard deviation of 0.45 is required for both DARWin and nomograph solutions.

5. Subgrade Resilient Modulus

Since the Agency is promoting a frost resistant design, it is considered unlikely that the true subgrade soil will experience overstressing. Therefore, the "Subgrade Resilient Modulus" for design should be that provided by the sand borrow material. Unless the Pavement Design Committee or the Materials and Research Section provides more specific information, use 60 MPa for the unfactored sand borrow modulus.

6. Pavement Material Parameters

The load carrying capacity of the pavement structure, referred to as the Structural Number (SN), must be sufficient to accommodate the estimated traffic. The layer and drainage coefficients must be identified for the materials composing the pavement that will provide the required SN.

$$SN = a_1\, d_1 + a_2\, d_2\, m_2 + a_3\, d_3\, m_3$$

DARWin's Layered Thickness Design procedure economizes the design by ensuring each material is not overstressed. The Layered Design Analysis is described in Section 3.1.5 of Part II of the Guide (pp. II-35 - II-37).

Tables 4 and 5 list recommended resilient modulus values, unless the Materials and Research Division provides more current information.

a. Pavement

Table 4--Resilient Moduli for Pavement Materials

Material	Resilient Modulus
ACC Type I	4,500 MPa
ACC Type II	4,250 MPa
ACC Type III	3,900 MPa

b. <u>Subbase</u>

Table 5—Resilient Moduli for Subbase Materials	
Material	**Resilient Modulus**
Crushed Stone	225 MPa
Crushed Gravel	175 MPa
Gravel	150 MPa

c. Consult the "Material Properties" for recommended layer coefficient values, unless the Pavement Design Committee or the Materials and Research Section provides more current information.

d. Drainage coefficients, m_2 & m_3, should be assumed to be 1.0 unless more specific information suggests otherwise.

7. **<u>Frost Depth</u>**

a. Use the Metric Frost Penetration Map to determine the maximum frost penetration. (Interpolate as necessary)

b. Determine the percent of frost protection from Table 6 based on functional classification.

Table 6--Design Frost Depths								
Functional Classification	**Factor**	**Maximum Frost Depth (in cm)**						
		140	150	160	170	180	190	200
Interstate	0.90	126	135	144	153	162	171	180
Principal Arterial	0.80	112	120	128	136	144	152	160
Minor Arterial	0.70	98	105	112	119	126	133	140
Collectors	0.60	84	90	96	102	108	114	120
Local Routes	0.40	56	60	64	68	72	76	80

c. Multiply the maximum depth times the design percentage for the design frost depth.

8. <u>Pavement Design Thickness Determination</u>

Pavement structure design is composed of 1) the Asphalt Cement Concrete (ACC) requirements generated by a 20-year ESAL design, 2) the subbase requirements generated by a 40-Year ESAL design, and 3) sufficient frost resistant material to achieve the frost depth calculated in “7.”

a. Generate a full layered thickness design based on the appropriate material characteristics and the <u>20-Year</u> ESAL estimate. If the suggested pavement thickness criteria are used from Table 7, the thicknesses will have to be soft-converted to the nearest 0.1 inch for the layered thickness design input.

b. Generate a full layered thickness design based on the appropriate material characteristics and the <u>40-Year</u> ESAL estimate. Do <u>not</u> adjust the ACC derived thicknesses since these have an effect on the subbase design.

c. Final design typical will consist of the ACC thickness determined in “a” conforming to the suggested layer thicknesses provided in Table 7. Use the subbase thickness determined in “b,” and round to the nearest 25 mm.

d. Determine the Sand Thickness

   1) Subtract the total design typical depth determined in “8.c.” from the designed frost depth provided in Table 6. This difference represents the sand thickness required.

   2) Round the sand thickness <u>up</u> to the nearest 50 mm increment.

e. The following minimum thicknesses shall be used for any new or reconstructed pavement structure.

   1) The minimum ACC thickness shall be 100 mm for designs less than 1,000,000 ESALs and 125 mm for designs greater than 1,000,000 ESALs.

   2) The minimum subbase material thickness shall be 500 mm for State Routes and 300 mm for Rural Town Highways.

   3) The minimum sand thickness shall be 300 mm for State Routes unless subgrade soil is classified as A-1-a or A-3 (determination made from borings). Rural Town Highways can be 150 mm.

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Low Volume Pavement Design Procedures for Use with the

1993 AASHTO Guide for Design of Pavement Structures

The Vermont Agency of Transportation procedure for the design of low volume pavement structures is based on the 1993 AASHTO Guide for Design of Pavement Structures, referred to simply as the '93 Guide in this procedure. Pavement structures designed using this procedure are intended for those instances where the 20-year Design Lane ESAL estimate is less than 100,000.

This procedure outlines the pavement design method to follow for both paved and unpaved structures experiencing low traffic volumes. Owing to the typical functional classification associated with facilities experiencing these minimal loadings, reliability and frost depth requirements are relaxed from the more stringent "new pavement" procedures.

The following tables provide recommended typical sections for the paved or unpaved conditions presented, and whether design ESALs are available or just construction year ADTs:

Table 1 - Paved Roads

ADT	ESAL	Pavement		Gravel Subbase	Sand
		Type III	Type II		
≤ 110	≤ 75,000	1¼"	1¾"	12"	(not required)
≤ 150	≤ 100,000	1½"	2"	12"	12"

Table 2 - Unpaved Roads

ADT	ESAL	Aggregate Surface	Gravel Subbase	Sand
≤ 40	≤ 25,000	6"	12"	(not required)
≤ 150	≤ 100,000	6"	12"	12"

If the designer prefers, a detailed design may be undertaken using the following guidelines.

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Procedure for a Paved Low Volume Road

1. **Several Design Inputs Are Required**

   a. Traffic
   b. Serviceability Loss
   c. Reliability Level
   d. Subgrade Resilient Modulus
   e. Pavement Material Parameters
   f. Frost Depth

2. **Traffic**

   a. 20-year 18 kip ESALs for a flexible pavement should be requested from the Traffic Research Unit, specifying the route number, project location and the construction and design years.

   b. The designer must also review for the potential traffic effects caused by the proximity of seasonal traffic generators, i.e., recreational, school, logging, etc., and apprise Traffic Research of these findings in the ESAL request.

   c. Use the total 18 kip ESAL equivalents for the pavement design due to the high potential of meandering traffic and the typical lack of pavement markings.

3. **Serviceability Loss**

   a. Initial design serviceability index for flexible pavement is presently accepted as 4.0 for new pavement construction.

   b. Subtract the terminal serviceability index of 2.0 from the initial serviceability index.

   c. This equals the Design Serviceability Loss, ΔPSI.

4. **Reliability Level**

   a. Use 50% reliability for this procedure.

   b. A value of 0.45 may be used for both DARWin and nomograph design applications to provide for a solution.

5. **Subgrade Resilient Modulus**

Since we are not promoting a frost resistant design, an effective roadbed soil resilient modulus of 3000 psi is recommended. This value is available in Part II, Chapter 4, Table 4.3, of the AASHTO Guide.

6. **Pavement Material Parameters**

a. Consult the "Material Properties" for recommended layer coefficient and resilient modulus values, unless the Pavement Design Committee or the Materials and Research Section provides more current information.

b. Drainage coefficients, m_2 & m_3, should be assumed to be 1.0 unless more specific information suggests otherwise.

7. **Frost Depth**

a. Although it is not considered imperative to design for frost penetration under this procedure, the frost design component is provided for use at the designer's discretion.

b. Find the maximum frost penetration on the Frost Penetration map. (Interpolate)

c. Determine the amount of frost protection required from Table 3.

Table 3--Design Frost Depths

Functional Classification	Factor	Maximum Frost Depth						
		50"	55"	60"	65"	70"	75"	80"
Local Routes	0.40	20"	22"	24"	26"	28"	30"	32"

8. **Pavement Design Thickness Determination**

It is suggested to base the pavement structure design on a one-stage scenario. However, the designer may use two- or three-stage criteria if it can be justified.

a. Generate a full layered thickness design based on the appropriate material characteristics and the 20-Year ESAL estimate.

b. Determine the Sand Thickness

1) Subtract the final design typical depth determined in “8.a.” from the designed frost depth.

2) Round to the nearest 3" increment.

c. A minimum of 3" of ACC shall be used for paved structures developed under this procedure.

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Procedure for an Unpaved Low Volume Road

This procedure is patterned after the method described in Part II, Chapter 4 Low-Volume Road Design, in the AASHTO Guide. It is an iterative and tedious process requiring a graphical evaluation of a range of base thicknesses for a given set of traffic and performance conditions. A copy of Table 4.4 provides a useful format to organize the various design input parameters.

1. **Several Design Inputs Are Required**

   a. Serviceability Loss
   b. Rutting Criteria
   c. Roadbed Resilient Modulus
   d. Pavement Material Parameters
   e. Traffic
   f. Frost Depth

2. **Serviceability Loss**

   A recommended Design Serviceability Loss, ΔPSI, for this procedure is 3.0. However, the procedure may accommodate a range of 1.0 to 3.5.

3. **Rutting Criteria**

   A rut depth of 2.0 inches is recommended. While the typical range is 1 to 2 inches, the procedure may accommodate a rut depth of 0.5 to 3.0 inches.

4. **Roadbed Resilient Modulus**

   It is recommended to use the range of "poor" quality roadbed resilient moduli from Table 4.2, for inclusion in column 2 of Table 4.4.

5. **Material Parameters**

   a. "Gravel" is the recommended subbase material for structures designed under this procedure. However, if other sources provide a more economical design, and the designer prefers a more stable material, consult the "Material Properties" for recommended resilient moduli for other base materials.

   b. Use the resilient modulus for the selected material in column 3 of Table 4.4.

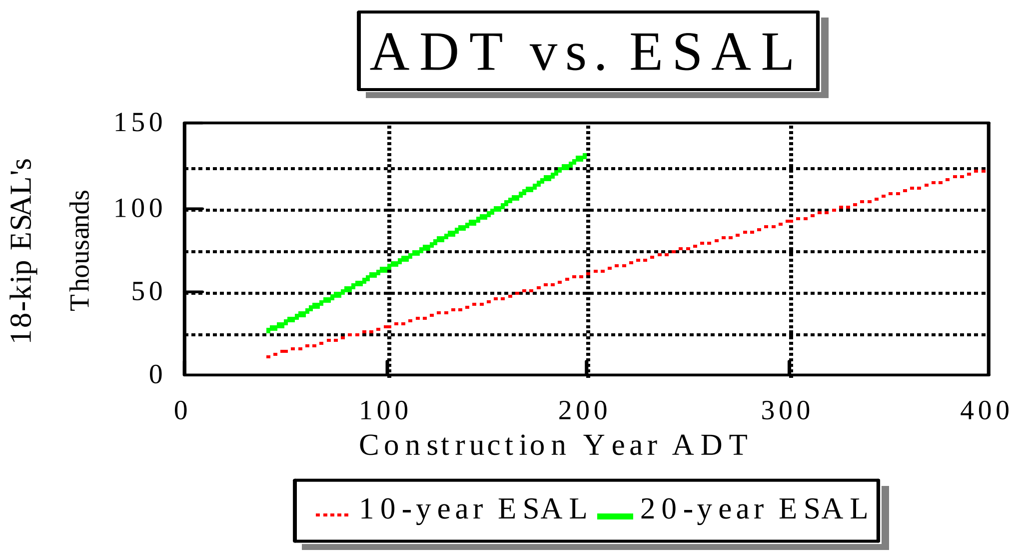
6. **Traffic**

a. 20-year 18 kip ESALs for a flexible pavement should be requested from Traffic Research, specifying the route number, project location, and the construction and design years. Figure 1 shows the relationship between the specified design-period ESALs and the construction year ADT. For typical low volume facilities, this may be used as a rough approximation in lieu of a detailed ESAL estimate.

b. The designer must also review for the potential traffic effects caused by the proximity of seasonal traffic generators, i.e. recreational, school, logging, etc., and apprise Traffic Research of these findings in the ESAL request.

c. Use the total 18 kip ESAL equivalents for the pavement design due to the high potential of meandering traffic and the typical lack of pavement markings.

d. Prorate the design ESALs according to the suggested season lengths for "U.S. Climatic Region III" presented in Table 4.1. Enter these values in column 4 of Table 4.4. Be sure the column totals the original design ESALs.



7. **<u>Total Damage as a Function of Base Thickness and Limited by Serviceability Loss</u>**

   a. Enter Figure 4.2 with the current "Base Layer Thickness." Determine the "Allowable 18-kip Equivalent Single Load Applications" based on the base material resilient modulus, the roadbed material resilient modulus, and the allowable serviceability loss. Enter this value in column 5 of Table 4.4.

   b. Column 6 of Table 4.4 shows the ratio of column 4 to column 5, and represents that season's contribution to the loss in serviceability. Depending on the base thickness being evaluated, the "Total Damage" may be higher or lower than 1.0. A total damage of 1.0 equates to the serviceability loss criteria established for this analysis.

   c. Repeat this procedure until several Total Damage/Base Thickness data points are established both higher and lower than 1.0. Plot these data points on a graph.

8. **<u>Total Damage as a Function of Base Thickness and Limited by Rutting Criteria</u>**

   a. Enter Figure 4.3 with the current "Base Layer Thickness." Determine the "Allowable 18-kip Equivalent Single Load Applications" based on the base material resilient modulus, the roadbed material resilient modulus, and the allowable rutting depth. Enter this value in column 7 of Table 4.4.

   b. Column 8 of Table 4.4 shows the ratio of column 4 to column 7, and represents that season's contribution to the rutting criteria. Depending on the base thickness being evaluated, the "Total Damage" may be higher or lower than 1.0. A total damage of 1.0 equates to the rutting depth criteria established for this analysis.

   c. Repeat this procedure until several Total Damage/Base Thickness data points are established both higher and lower than 1.0. Plot these data points on a graph.

9. **<u>Base Thickness Design</u>**

   The design base thickness is determined by interpolation of the graph produced by the previous two steps. Interpolate to a total damage equal to 1.0 for <u>both</u> Serviceability and Rutting criteria. The design base thickness is the <u>larger</u> thickness of the two.

10. **<u>Frost Design</u>**

   a. If the designer deems frost protection necessary, the design frost depth shall be determined from the Frost Penetration Map.

   b. If the surface course and base do not provide the depth needed for frost protection, the difference may be made up with sand borrow.

11. Design Limitations

a. All unpaved structures shall have 6" of “Aggregate Surface Course” material (see current Standard Specification book for item number) as the wearing surface.

b. Whichever base material is used, a 12" maximum thickness is recommended. When this design procedure results in a base thickness in excess of 12" the difference may be “converted” into a sand borrow “subbase.” Enter Figure 4.5 with a 12" Final Base Thickness and a 9000-psi Subbase Modulus. The “Decrease in Base Thickness” is the difference between the designed thickness resulting from this procedure and the 12" maximum. Decreases less than 4" may be ignored. The thickness of sand borrow subbase is determined by using the appropriate Base Modulus.

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Rigid Pavement Thickness Design Procedures
for use with the

1993 AASHTO Guide for Design of Pavement Structures

The Vermont Agency of Transportation procedure for the design of new or reconstructed pavement structures is based on the 1993 AASHTO Guide for Design of Pavement Structures, referred to simply as the '93 Guide in this procedure, and the 1998 Supplement to the AASHTO Guide for the Design of Pavement Structures, Part II, - Rigid Pavement Design & Rigid Pavement Joint Design. These design procedures may be applied to conditions that exceed a 20-year Design Lane Rigid ESAL estimate of 70,000.

This procedure outlines the method to design the thickness of a rigid pavement. A three-stage construction strategy is described to allow the construction of a new pavement structure (first stage) and two future planned rehabilitation treatments (second and third stages) to achieve a 40-year design life.

1. **Several Design Inputs Are Required**

a. Pavement Type
b. Traffic
c. Serviceability Indices
d. Pavement Material Parameters
e. Frost Depth
f. Effective Modulus of Subgrade Reaction
g. Reliability Level
h. Load Transfer
i. Overall Drainage

2. **Pavement Type**

The pavement type, jointed plain concrete pavement (JPCP), jointed reinforced concrete pavement (JRCP), or continuously reinforced concrete pavement (CRCP), has no effect on the thickness design generated by the AASHTO model. JPCPs are recommended for use in Vermont.

3. **Traffic**

a. 20-year and 40-year 18 kip ESALs for rigid pavement should be requested from the

Traffic Research Unit, specifying the route number, project location, and the construction, 20-, and 40-year design years.

b. The designer must also review for the potential traffic effects caused by the proximity of seasonal traffic generators, i.e., recreational, school, logging, etc., and apprise Traffic Research of these findings in the ESAL request.

c. Determine design lane 18 kip ESALs.

1) Use the distribution factors and procedure described below unless Traffic Research provides Design Lane ESALs.

2) Find the 18 kip ESAL distribution factor for the desired lane configuration from Table 1.

Table 1--18 kip ESAL Distribution Factors

Number of Lanes	Distribution Factors
2	0.50
3	0.50
4 (Rural)	0.60
4 (Urban)	0.60

3) Multiply the ESAL values by the appropriate Distribution factor(s) to obtain Design Lane estimates for both 20- and 40-year 18 kip ESALs.

3. **Serviceability Indices**

a. The initial design serviceability index for rigid pavement is presently accepted as 4.5.

b. The terminal serviceability index is 2.5 for all state and Class 1 Town Highways. For local routes, use 2.0.

4. **Pavement Material Parameters**

The designer must establish the properties of the concrete slab. Recommended design values are provided in Table 2. These values, the modulus of rupture (S_c') and elastic modulus (E_c), may be estimated from the 28-day compressive strength of the PCC (f_c') using the following relationships (most PCC specifications call for a minimum of 5,000 psi concrete):

$$S_{c'} = 9.0 \times \sqrt{f_{c'}}$$

$$E_c = 57{,}000 \times \sqrt{f_{c'}}$$

and

Table 2--28-Day Mean Properties for PCC Pavement Slabs	
Property	**28-Day Mean Values**
Modulus of Rupture (S_c')	650 psi
Elastic Modulus (E_c)	4,000,000 psi

5. **Frost Depth**

Since a rigid pavement is stiffer than a flexible pavement, it is better able to withstand the effects of frost action. This quality makes the rigid slab more robust to the localized stress concentrations of frost susceptible materials. It is recommended, in concert with a properly designed underdrain system under both shoulders that a full pavement structure depth of 30 inches is considered for rigid designs on the NHS in Vermont. Rigid designs considered for all other roads may be designed at 24 inches. Only a permeable base is recommended for use directly under the slab. Once the slab and base thickness have been determined, the difference in the full structure depth may be made up using crushed stone, or some other well-draining unbound material.

6. **Effective Modulus of Subgrade Reaction**

The modulus of subgrade reaction, the k-value, is an elastic constant, which defines the material's stiffness or resistance to deformation. The k-value is used for slab design, instead of the more familiar resilient modulus, because tests have demonstrated a high correlation between its use for predicting the slab's performance as it rests on a dense liquid and actual stress/strain measurements. The k-value is fundamentally similar to the resilient modulus. The resilient modulus represents a material's elastic response (stress vs. strain) and the k-value represents the stiffness as a function of the elastic (resilient) modulus. Through extensive testing, AASHTO has determined the theoretical relationship to be: $k = \dfrac{M_R}{19.4}$

A mean effective k-value of 200 psi/in is appropriate for most designs being considered for Vermont. This value is consistent with the conditions prevalent throughout most of the state. However, the designer should consider a project-specific effective k-value.

The effective modulus of subgrade reaction is determined from several different factors: the effective (seasonally adjusted) resilient modulus of the roadbed soil, base type and thickness, depth to a rigid foundation (ledge), and the loss of support of the base material. This process is detailed in section 3.2.1 of the guide, but the DARWin software greatly simplifies this task to allow for quick analysis of various conditions. While the guide refers to the "subbase" as the material the slab is in direct contact with, both DARWin and this procedure refer to this material as "base."

a. Seasonal resilient moduli for the roadbed soil are required. If resilient moduli from backcalculated Falling Weight Deflectometer tests are available and used, they must be factored by one-third to be consistent with the development of the AASHTO model. The conditions used for the development of the recommended k-value of 200 psi/in are consistent with the soil conditions set forth in section 4.1 (Tables 4.1 and 4.2) of the guide: 2½ months at 20,000 psi, 1½ months at 1,500 psi, 4 months at 3,300 psi, and 4 months at 4,900 psi.

b. Base material properties are provided in Table 3. An eight-inch thick Portland cement-treated permeable base, with an elastic modulus of 1,500,000 psi, was used in the determination of the 200 psi/in recommended effective k-value. A 14-inch thick asphalt cement-treated base, with an elastic modulus of 250,000 psi, will also provide an effective k-value of 200 psi/in. While this is not recommended for a rigid design, for comparison, 40 inches of crushed stone can also provide an effective k-value of 200 psi/in.

Table 3--Elastic Moduli for Pavement Base Materials

Material	Elastic Modulus
Portland Cement Stabilized Permeable Base	1,500,000 psi
Asphalt Cement Stabilized Permeable Base	250,000 psi
Dense Graded Crushed Stone	30,000 psi

c. The depth to a rigid layer, or ledge, does not affect the effective k-value unless it is within ten feet of the surface. Within this limit, the modulus is affected exponentially as the ledge is closer to the slab. No ledge was considered for the determination of the 200 psi/in recommended effective k-value.

d. The loss of support (LS) factor attempts to account for voids created under the slab from erosion and/or differential vertical soil movements which compromise slab support. For stabilized bases, an LS of no more than 1.0 should be used. A value of 1.0 was used for the determination of the 200 psi/in recommended effective k-value. The LS varies between 1 and 2 for a dense-graded crushed stone with very little fines (material passing the #200 sieve). The Agency's dense-graded crushed stone gradation specification stipulates ≤6% fines.

7. **Reliability Level**

a. Overall design reliabilities shall be reflective of those listed in Table 4 and are dependent upon the functional classification of the project route. Urban/rural distinction is determined as defined by Federal-Aid Urban Areas and the VAOT "Functional Classification" map.

Table 4—Reliability Factors for Pavement Design

Functional Classification	Urban	Rural
Interstate	99%	95%
Principal Arterial	95%	90%
Major and Minor Collectors and Minor Arterial	90%	85%
Local	75%	75%

These factors may be used directly when implementing a DARWin solution reflecting a three-stage design. DARWin automatically accounts for the compounding effects of staged construction as described in Section 4.5 of Part I (p. I-63 of the '93 Guide).

b. If the designer opts for a nomographic solution (pp. II-45-46 of the '93 Guide), then the input reliability required must be the cube root (to reflect the recommended three-stage strategy) of the overall reliability level desired in Table 4. The resulting reliability levels for nomograph use are listed in Table 5.

Table 5--Required Single-stage Reliability Factors for a Three-stage Pavement Design (when applying a nomographic solution)		
Functional Classification	**Urban**	**Rural**
Interstate	99.7%	98.3%
Principal Arterial	98.3%	96.5%
Major and Minor Collectors and Minor Arterial	96.5%	94.7%
Local	90.9%	90.9%

c. A standard deviation of 0.35 is required for both DARWin and nomograph solutions.

8. **<u>Load Transfer</u>**

The Load Transfer Coefficient, J, is a factor used in the AASHTO rigid pavement design model to account for the ability of a concrete pavement to distribute loads across discontinuities, such as joints or cracks. The load transfer coefficient has a direct impact on slab thickness. The magnitude of this coefficient is dependent on three factors: pavement type, shoulder type, and the presence of load transfer devices, i.e., dowels.

Jointed plain concrete (JPCP) is recommended for rigid pavement design of Vermont roads. Tied PCC shoulders are recommended for the maintenance of a good joint. Load transfer devices are strongly recommended. A reasonable J-value consistent with the foregoing conditions is 3.0.

9. **<u>Overall Drainage</u>**

Since the base course being recommended for rigid designs is very permeable, this material should provide excellent drainage quality. The lateral underdrain system should promote a dry condition, since any water approaching the structure laterally from the ditches should be intercepted. Any subbase material, for instance dense-graded crushed stone, should provide good drainage quality.

If close attention is paid to the drainage quality of the foregoing structural features, the amount of time the structure experiences saturation should be minimized. A reasonable estimate for the C_d parameter is 1.2.

10. Design Details

The above procedure will allow the designer to determine the thickness(es) of the various materials included in a rigid pavement structure design. There are, however, numerous other design details that the designer must consider, develop, and include in the PS&E documents for a successful rigid pavement project.

Among, but certainly not all, of the details that will be needed are:

1. The design of the load transfer devices (size, coating, number, placement, etc.).
2. The slab joint spacing.
3. The joint design (depth of cut, width of cut, type of seal, size of seal reservoir, etc.).
4. The design of joints and minimum slab widths proximate to obstructions or other fixed objects such as manhole, drop Inlets, intersecting roads, curbs, etc.

Since these design details are constantly evolving this procedure does not attempt to define what is appropriate. The designer will need to research the current state of the practice with regard to these details. Many sources of information are available to the designer for this information including other state DOT's, FHWA, and the various trade associations, such as the Portland Cement Association.

March 13, 2002

Vermont
Agency of Transportation

Pavement Rehabilitation Design Procedures
for use with the

1993 AASHTO Guide for Design of Pavement Structures

Introduction

These procedures are intended as a guide for the designer who is considering a pavement design treatment less extensive than a full reconstruction of the pavement structure. These procedures are also intended for project-level development. Consistent with the Agency's Vermont State Design Standards' Level of Improvement (LOI) philosophy, major transportation projects involving extensive improvements will take place only in major corridors. Other parts of the State's roadway system will receive less extensive improvements, such as preservation or rehabilitation treatments, depending on how heavily they are used, and how important they are to statewide mobility. The LOI is a recognition that with limited resources, it is not possible to upgrade every road and bridge in Vermont to its ideal condition. The Agency's Pavement Management Section uses a similar philosophy for determining the treatments for the projects assigned to the Paving Program.

Since there is no formula or definitive equation available to determine a proper rehabilitation design, these procedures outline a process for selecting the most "preferred" rehabilitation method given particular conditions and limitations. Reference is made to the "1993 AASHTO Guide for Design of Pavement Structures," hereinafter referred to as "the Guide." These pavement rehabilitation procedures are based on collective Agency experience and are supplemented by information provided in the Guide.

Rehabilitation design is not as straight forward as new pavement design. In pavement rehabilitation design, an analysis must be performed to ensure treatment of the problem (that is, the cause of the distress), and not simply the relief of the symptom (the actual distress itself). However, at least three steps must be undertaken for a proper rehabilitation analysis.

1. Problem Definition
2. Development of Potential Problem Solutions
3. Selection of the Preferred Treatment

A glossary of selected terms

Alligator (fatigue) Cracking	A series of small, jagged, interconnected cracks caused by failure of the asphalt concrete surface under repeated traffic loading.
Longitudinal	Parallel to the centerline of the pavement.
Pavement Structure	Constitutes the constructed materials, ACC and unbound subbase, placed over the undisturbed existing soil, or subgrade.
Reflection Cracking	The fracture of the asphalt concrete above the cracks, or joints, in the underlying pavement layer(s).
Rutting	The occurrence of longitudinal surface depressions in the wheel paths.
Shoving	Permanent, usually longitudinal, displacement of a localized area of a pavement surface caused by traffic pushing against the pavement.
Subgrade	The natural, processed, or fill soil foundation upon which a pavement structure is placed.
Transverse	Perpendicular to the pavement centerline.

I. Problem Definition

A. Constraint Identification

There are several constraints the designer should establish prior to the collection of field data. The designer should also understand how these constraints may affect the suitability of candidate solution selections. The following list is representative, but certainly not exhaustive, and is an indication of the types of considerations affecting analyses.

1. Project Funding

The funding available for a project may have a profound effect on the treatment(s) to be considered. Funding constraints may force the designer to consider less extensive treatments of the problem, an increase of funding, or a staged treatment.

2. Minimum Desirable Life

Another important constraint is the expected performance life of the project. More extensive treatments provide a longer life, less total disruption to the traveling public, and should generally provide for a better investment. Consideration should be given to designating a longer performance life for projects involving higher functional classification facilities.

3. Geometric Design Problems

Geometrics are generally not addressed directly when considering primarily a pavement rehabilitation. However, extensive geometric deficiencies, and the treatments needed to correct them, may help to justify a higher degree of rehabilitation needed for the pavement structure.

4. Clearances/Permits

Clearances and permits may constrain the development of a project design by virtue of the coordination necessary to acquire them. In this sense, they may affect only the project delivery schedule. But, it is conceivable that certain State and Local regulations may also affect the extent of a recycling treatment or reconstruction activity.

5. Agency Policies

Depending on the existing conditions, certain Agency policies, e.g., shoulder paving for bicycle use or the extent of treatment allowed by the Agency's Level of Improvement, can eliminate specific treatments from consideration.

6. Urban/Rural

Some treatments may be discouraged from being used on projects within densely developed urban areas. Numerous storm drains and utility shut-offs might discourage the use of most recycle in-place treatments. A chip seal treatment is not a good candidate in an urban area due to the amount of loose stone that will tend to clog the closed drainage systems. The presence of curbs will often dictate pavement milling before subsequent treatments. A treatment, which requires the pulverization of the underlying PCC pavements, might cause damage to wells and aquifers in rural areas, and may damage foundations and utilities in urban areas. These limitations should be carefully evaluated as project conditions warrant.

7. Flexible /Rigid (ACC/PCC) pavements.

The presence of PCC pavements just below the surface of the roadway will eliminate some of the deeper treatments from consideration. The designer might only be left with surface type treatments.

In some high traffic urban areas, the severity of rutting and shoving of ACC pavements due to abnormally high ESALs and/or stopping conditions may indicate the need to consider an option other than that of an ACC pavement, e.g., reconstruction or a thin PCC overlay. Although Superpave™ may mitigate some of these distresses, there may still be conditions unsuitable for an ACC mix.

B. Data Collection

Appropriate data collection is required so an informed decision may be reached when developing the candidate list of solutions and ultimately when selecting the preferred rehabilitation method. The extent of treatments governed by the LOI and examples of data to be collected are: Traffic, Climate, Materials & Soil, Pavement Condition, Drainage, and Safety.

1. Traffic

a. The designer must review for the potential traffic effects caused by the proximity of seasonal traffic generators, i.e., recreational, school, logging, etc., and apprise Traffic Research of these findings in the ESAL request.

b. A range of traffic design years, for instance, 5, 10, 15, 20, and 40-year 18 kip design lane ESALs for flexible and rigid pavements, should be requested from the Traffic Research Unit, specifying the route number, project location, and the construction year. Other design years may need to be requested, depending on the rehabilitation strategy(ies) being evaluated. The range of traffic design years is necessary to adequately determine the preferred treatment.

2. Climate

Determine the frost penetration for the project location from the statewide map.

Any ACC treatment will require consideration of ambient temperatures in order to be able to select the appropriate Performance Grade (PG) asphalt regardless of the ACC pavement type used.

3. Materials & Soils

Route Logs and record plans may provide information relating to the materials composing the existing pavement structure. Pavement cores, test pits, and borings can be used to sample the pavement structure to supplement the existing documentation of the pavement materials. If any PCC is present within the pavement structure, this should also be noted, since the existence of PCC anywhere within the pavement structure requires an analysis consistent with PCC properties. When project constraints allow for treatments beyond simple corrective measures, the following data are necessary to fully evaluate the suitability of any rehabilitation treatments.

a. The thickness of all layers should be measured as accurately as possible. This will entail several measurements to ensure a representative assessment is made and to identify any anomalous deviations from the expected typical.

b. Samples of ACC layers and stabilized base should be examined to assess asphalt stripping, degradation, air voids, binder viscosity, and the presence of any hazardous materials, such as asbestos.

c. Samples of granular base and subbase should be visually examined and a gradation performed to assess degradation by contamination with fines.

4. Falling Weight Deflectometer (FWD) Testing

FWD non-destructive testing (NDT) is performed by the Agency's Pavement Management Section and affords a means for determining the effective structural number (SN_{EFF}) of a flexible pavement structure, or an effective slab depth (D_{EFF}) of an equivalent rigid pavement, when testing a rigid pavement or a composite (ACC/PCC) pavement. It is important to identify the existence of bedrock if it is located within 20 vertical feet of the pavement surface. Shallow bedrock will affect any non-destructive deflection testing.

FWD testing may be used to measure deflections in the pavement structure at an interval sufficient to adequately assess conditions. The testing frequency should be consistent with the variability in the pavement structure. FWD tests are routinely conducted at half-mile increments in the right wheel path. The opposing lane should be tested at alternating locations so that information is obtained at quarter-mile increments. Multiple lane highways should be tested across the section to obtain representative information. Areas that are deteriorating, and have been identified for repair, should be tested. Ultimately, the testing frequency is the designer's decision and should address project-specific variability.

5. Pavement Condition

Information on pavement condition may be available from several areas. The designer is encouraged to check with Pavement Management and/or the appropriate Maintenance district(s).

Pavement Management collects pavement condition data on a statewide basis. This information may provide the designer some indication of the distress(es) prevalent in the project area. Pavement Management also conducts project-level structural and roughness surveys. They may also be able to define the rate of deterioration that the pavement is experiencing as well as the friction value of the pavement.

The Maintenance districts are a valuable resource for determining on-going pavement problems, e. g., frost heaves, ice-jacking, and settlement conditions. The districts are better equipped to identify problem locations and maintenance history.

Once the resources available in Pavement Management and the Maintenance districts have been drawn upon, the designer is better informed to make a field investigation to verify and define the extent and severity of the pavement conditions. Sampling along the project in the lanes, which are experiencing the heaviest volumes of traffic, should be used to quantify the distresses. In general, pavement distress surveys are taken within single traffic lane test sections, 500 feet in length. The SHRP Distress Manual, SHRP-P-338 (1993), provides additional guidance for conducting distress surveys. Any modifications to the distress severity criteria documented in this procedure, or used by the Pavement Management Section, take precedence over that listed in SHRP-P-338.

Two primary forms of distress generally encountered on Vermont highways include cracking and surface deformation. The remaining miscellaneous distresses occur due to project specific conditions.

a. Cracking: Strictly speaking, any cracks evident at the surface of the pavement constitute a “failure” of the bound material. The designer should be particularly concerned about these crack failures when they allow moisture infiltration into the unbound portion of the pavement structure or cause stripping of the bound material. Fatigue, longitudinal, reflective, and transverse cracks constitute the majority of cracking distress on Vermont highways. If these distresses are not addressed in a timely manner, they can often develop into potholes.

b. Surface deformations: Deformations manifest themselves in several ways at different times of the year. Examples of surface deformations include rutting, shoving, roughness, settlement conditions, frost heaves, and ice-jacking.

c. Miscellaneous: This type of distress includes shoulder drop-off, lane-to-shoulder separation, water bleeding and pumping, and utility cuts.

6. Drainage

The districts can also provide condition information, especially during the winter season, such as frost heave locations or ice jacking locations. These locations may indicate drainage problems requiring attention.

Whereas the moisture infiltration is identified with the pavement condition assessment, the drainage evaluation is concerned with how well the entire structure drains excess moisture away and how well the structure remains relatively dry.

Distress in both rigid and flexible pavements is often either caused or accelerated by the presence of moisture in the pavement structure. When designing a pavement rehabilitation, the designer must investigate the role of drainage in pavement performance. Distress types in flexible pavement may be caused or accelerated by the presence of moisture in the pavement structure and include stripping, rutting, depressions, fatigue cracking, and potholes. Differential frost heave and spring breakup (evidence of loss of support) both indicate the pavement structure retains excess moisture during the winter months. Drainage evaluation also requires investigation of the problem site, preferably during a wet weather period. The designer must consider the following issues and conditions during the site visit:

a. Where and how does water move across the pavement surface?

b. Where does the water collect on or near the surface?

c. How high is the water level in the ditches and/or do the ditches need cleaning?

d. Do the joints and cracks contain any water?

e. Does water pond on the shoulder?

f. Are the joints and cracks sealed well?

g. Inspect the condition of the culverts and underdrain.

h. When extremely bad drainage conditions exist, consideration should be given to the use of groundwater monitoring piezometers to assess the severity of the drainage problem. Other circumstances, which might warrant the consideration of this type of monitoring, include very high profile projects, or improvements to a prominent facility.

7. Safety

The safety afforded by our roadway pavement surfaces is of paramount importance. The designer must recognize and document the following conditions, which may jeopardize the safety of the traveling public:

a. Minor rutting, as little as ¼ inch, in combination with a significant rainfall event, can contribute to hydroplaning. Rutting and shoving pavements may cause loss of control, affecting the ability of the driver to properly control the vehicle. Severe rutting can cause the vehicle to “bottom out,” that is, exceed the capacity of the vehicle’s suspension, which can cause damage to the vehicle and the pavement.

b. Severe potholes can cause motorists to swerve out of their respective travel lanes, and into the path of an oncoming vehicle, while attempting to avoid the pothole. Potholes can also cause the driver to lose control of the vehicle or can damage the vehicle’s suspension. Potholes may be partially filled with loose debris, which may be dislodged by passing traffic, and thrown onto pedestrians or other vehicles’ windshields.

c. The presence of pavement debris or subbase materials is always a safety concern for motorcyclists.

d. Aggregate polishing and asphalt flushing compromise the skid resistance of the pavement surface. This can lead to premature loss of friction when braking or turning.

C. Data Evaluation

Evaluation of the data begins with classifying the extent (how much of the total project is subject to the given distress) and severity (how bad the failure is) of each distress. Determination of possible failure mechanisms is based on the extent and severity of the distresses identified. Once the failure mechanism(s) has been identified, an appropriate treatment can be considered.

1. Initial Evaluation of Existing Condition Data.

a. Cracks - fatigue, longitudinal, reflective, and transverse.

Fatigue cracks are indicative of either an insufficient structure or accelerated oxidation of the ACC caused by excessively high air voids. When observing cracks, during the initial portion of the evaluation, it cannot yet be determined which pavement structural component material, ACC or unbound subbase, is insufficient or inadequate. Further information is required to make this determination. FWD testing may help to quantify the degree of insufficiency of the structure.

Longitudinal cracks are an indication that insufficient or differential densities were developed at the time of construction. This lack of density most often occurs at the construction joints of the pavement materials. The movement of underlying PCC slabs may also cause this distress.

Reflective cracks follow any pre-existing crack pattern or joints from underlying pavement layers. Reflective crack patterns that exhibit a regular geometric pattern, i.e., regularly spaced transverse cracks or a consistent longitudinal crack close to the edge of the travel lane, are indicative of existing PCC slabs. Reflective cracks associated with PCC slabs indicate differential movement of the PCC slabs. This differential movement is worsened by traffic or by thermal contractions.

Thermal contraction of the ACC can cause transverse cracks and is worsened when combined with hardening, or low elasticity, of the binder.

b. Rutting is indicative of plastic flow in the ACC, i.e., soft binder, or excessively low air voids in the binder, lateral displacement of the unbound subbase materials, or overstressing of the subgrade.

c. Roughness may be a composite manifestation of various cracking distress and surface imperfections. Roughness may also indicate surface imperfections or undulations exacerbated by traffic or shoving of the ACC materials.

d. Any indications of inadequate drainage should be further investigated.

When the unbound base and subbase materials become saturated, the structure loses its stability. When a structure loses its stability in this way, it resembles a structure that is insufficiently stiff or too thin. The results of this cyclic phenomenon lead to fatigue cracking.

e. Non-Destructive Testing (NDT) from a Falling Weight Deflectometer (FWD) should provide data to establish whether the existing structure is sufficiently stiff to protect the subgrade from overstressing resulting from the design traffic loadings. An insufficient structure is a problem when the existing structure does not provide the support required (as designated by the structural number (SN)) for the loadings forecast during the performance period. This condition requires attention regardless how minor other distresses may be.

2. Determine Effective SN (SN_{EFF}).

The designer may utilize the expertise in the Pavement Management Section for assistance in this analysis. Pavement Management can provide not only an analysis of the effective SN for a project section, or sections, based on FWD data, but can also provide guidance for effective treatments.

However, designers are encouraged to become familiar with this component of pavement design. Pages L-23 through L-31 of the Guide provide guidance for this analysis. DARWin software has a module that automates this procedure as well. Several input data are required for a DARWin analysis. These data include: FWD deflection basins, pavement temperature at the time of the FWD test, thickness data for the pavement structure, and subgrade type information. Some of this information is part of the FWD data file. The rest must be determined.

a. The NDT method of SN_{EFF} determination follows an assumption that the structural capacity of the pavement is a function of its total thickness and overall stiffness. This is one of the single most useful measurements of the existing conditions for a pavement rehabilitation design. The relationship between SN_{EFF}, thickness, and stiffness is:

$$SN_{EFF} = 0.0045D\sqrt[3]{E_p}$$

where:

D = total thickness of all pavement layers above the subgrade, in inches, and

E_P = effective modulus of pavement layers above the subgrade, in psi.

E_P must satisfy the relationship using the deflection data as described above under FWD testing. If the subgrade resilient modulus and the total thickness of all layers above the subgrade are known, the effective modulus of the entire pavement structure (all pavement material above the subgrade) may be determined from the deflection measured at the center of the load plate using Equation 5.15 on page L-26 of the Guide. DARWin provides an estimate of SN_{EFF} directly, once the designer identifies the total pavement thickness, D, the ACC thickness, and the resilient modulus correction factor, C. All other parametric information is provided by DARWin's backcalculation of the deflection basin.

b. Design Subgrade Resilient Modulus.

Research has demonstrated that the backcalculated resilient modulus (M_R) values for the subgrade should not be used in the AASHTO pavement design model, i.e., the procedure by which the SN and the layered analysis are determined. A static M_R, a property derived by a laboratory triaxial test, must be used for this operation. However, this may not always be available.

Research has established a "C value" that represents the ratio of the static M_R to the backcalculated M_R. C can vary from 0.15 to 0.33, and applies to cohesive clay and silty soils. Until the Agency can develop a database of C values, the recommended range for Vermont is 0.25 to 0.33. C tends to increase with a decreasing amount of fines, i.e., material passing the 200 sieve.

The phenomenon resulting in the C value is primarily an effect of the limitations of laboratory procedures to duplicate *in situ* conditions. First, the repeated vertical stresses produced by triaxial testing do not exactly reproduce the actual loading pulse imparted by a moving wheel load. Second, the uniform confining pressures used by triaxial testing do not truly simulate the actual confining stress variation, nor does the confinement pressure truly simulate the passive resistance experienced *in situ*. Third, *in situ* anomalies and discontinuities in the materials themselves are impossible to replicate in the laboratory.

There does not appear to be sufficient justification, based on seasonal FWD data, to warrant further modification of the backcalculated M_R values, if they are taken during the summer months. FWD readings taken in Vermont during the summer months of June through September typically exhibit very little variation. Further, any pavement modulus backcalculated from data taken during this time period is representative of the "Effective Roadbed Soil Resilient Modulus," as described in section 2.3.1 of the Guide. The effective modulus is representative of the averaging of the damage which occurs throughout the season. Spring thaw conditions, which can occur during the months of April and May, should be avoided altogether since these readings would skew a design and their effects are already accounted for in any effective modulus characterization. Spring thaw conditions are short-lived, but site-specific. If a well draining structure is being tested, this is usually not a concern since the rapid evacuation of excess moisture serves to provide for a stable foundation. However, if drainage is determined to be a problem, FWD testing should be postponed until the pavement structure is unsaturated. This may take until the end of June.

If the effective (or adjusted) roadbed soil resilient modulus is less than 3500 psi, then 3500 psi should be used.

c. The temperature of the ACC during deflection testing is routinely measured for each FWD test. The ACC temperature should be measured directly at mid-depth of the pavement layer, or may be estimated from surface or air temperatures. This parameter is part of the FWD file and is translated by the DARWin software.

3. Design Inputs for Rehabilitation Pavement Design.

A higher level of reliability is recommended for the analysis of pavement rehabilitation options than is commonly used for a "new" pavement structure design. Since only one treatment is generally considered for "rehabilitation," that is if no staged construction is being considered, it is even more important for that one treatment to perform well for the entire analysis period. For this reason, the reliability levels for this procedure have been increased for better long-term performance over those endorsed for new pavement structures.

a. Design Parameters.

(1) Design Reliability Level

(a) Use a reliability of 99% for all NHS projects.

(b) Use a reliability of 95% for all other state routes, and Class 1 Town Highways.

(c) Use a reliability of 90% for all other Town Highways.

(d) Use a standard deviation of 0.45 for flexible pavements and 0.35 for composite or rigid pavements.

(2) Serviceability Indices

(a) The initial serviceability index is 4.0 for flexible pavement and is 4.5 for rigid pavement.

(b) The terminal serviceability index is 2.5 for NHS, State routes, and Class 1 Town Highways. Use 2.0 for all other local routes.

4. Identification of Failure Mechanism(s).

At this point in the evaluation of the data, a determination of the possible failure mechanism(s) is appropriate. If the designer is comfortable making this determination, Problem Definition is complete and the Development of Potential Solutions is the next step.

If the failure mechanism is not clear, or the evaluation of the distresses provides ambiguous conclusions, further testing may be warranted.

For instance, if the initial evaluation of the distresses indicates possible failure of either the ACC or the unbound subbase, perhaps a particular laboratory test will provide some further stability or permeability characteristics. Laboratory tests may be conducted to determine "static M_R," moisture, density, permeability, etc., to better evaluate the more involved rehabilitation treatments, e.g., reclaim, recycle, and reconstruction.

5. Supplemental Laboratory Testing (if warranted).

M_R, moisture, density, and permeability may help determine the *in situ* M_R and other characteristics since the designer needs to know how the existing pavement structure responds to traffic loading and the environment.

a. A laboratory triaxial test (as determined by AASHTO T 292) is useful for determining the stiffness property referred to as "static M_R." This is a property consistent with the development of the AASHTO model for pavement design, and is often very different from the backcalculated value.

b. The moisture content (as determined by AASHTO T 238, T 255, or T 265) should be established for both *in situ* conditions prevalent during the FWD testing, and the springtime saturation conditions.

c. The density of the pavement materials (as determined by AASHTO T 191 or T 238) is an important property for both laboratory-testing purposes, and for in-place performance.

d. The permeability can affect the support properties of the pavement materials. The presence of excess moisture decreases a material's stiffness and its ability to withstand loading stresses.

e. The amount of air voids in the ACC can have a significant effect on the performance of this material. Too few air voids makes the ACC susceptible to rutting because the binder is prone to plastic flow. Too much air void content makes the ACC prone to fatigue cracking because the porous nature fosters oxidation of the mix, resulting in a brittle material.

II. Development of Potential Problem Solutions

A. Select Candidate Solutions

The treatments to be considered should address the distresses identified within the project area and correct the failure mechanism(s). Generally, the level of the treatment should be consistent with the severity and extent of the distresses present. For instance, distresses of a minor severity, which indicate a good pavement to begin with, should not warrant very extensive treatments. To treat minor distresses, appropriate treatments like single-course overlays or level and overlays should be considered. "Spot treatments" may also be considered, when appropriate, either by themselves, or to augment a more extensive rehabilitation treatment. A likely spot treatment considered for a severe frost heave might be the only extensive work done in a project that only needs a structural overlay. Indeed, a candidate solution need not be limited to a single treatment type, but may make use of a host of complementary treatments to maximize the rehabilitation performance.

Tables 1 through 6 below outline suggested treatments for each of the various distress types. When faced with a variety of distresses that call for a range of treatments, the designer should determine the "dominant" distress. The dominant distress represents the dominant cause of failure and therefore the treatment selected should serve to address this failure mechanism. Less extensive treatments typically have lower initial cost, but may not adequately address the cause of failure. As a result, the treatment life is compromised.

Completely addressing the failure mechanism may be cost prohibitive, and the optimum solution may lie somewhere in between completely eliminating it and ignoring it. This is a judgement call and cannot be quantified to most designers' satisfaction. The designer must recognize the constraint on treatment type required by the LOI Policy. The Policy serves to eliminate more extensive treatments on the lower functional classification facilities. However, if the treatment recommended by following this procedure is in excess of that allowed by the LOI, the LOI provides for more extensive treatments to address "structural deterioration," if it is cost effective. A life cycle cost analysis (LCCA) should provide the designer the justification for selecting a treatment not consistent with LOI.

Table 1 - Transverse Cracks				
		Extent (spacing)		
		>50 feet	<50 & >25 feet	<25 feet
Severity	Low: <¼" wide	N.A.	N.A.	Crack Sealing & Chip Seal
	Moderate: >¼" wide & <¼" deep	Crack Sealing	Crack Sealing	Mill and Fill, Hot In-place Recycling, Cold Recycle and Overlay, & Reclaim and Overlay
	High: >¼" wide & >¼" deep	Crack Sealing & Thin Overlay	Mill and Fill, Hot In-place Recycling, Cold Recycle and Overlay, & Reclaim and Overlay	Reconstruction

Table 2 - Fatigue Cracks					
(longitudinal)		Extent of Total Project (measured as length of lane per total project length)			
		<10%	10-24%	25-49%	>50%
Severity	Low: fine hairline	N.A.	Crack Sealing	Thin Overlay & Structural Overlay	Structural Overlay
	Moderate: alligator pattern clearly developed	Spot Leveling	Thin Overlay & Structural Overlay	Thin Overlay & Structural Overlay	Hot In-place Recycling, Cold Recycle and Overlay, & Reclaim and Overlay
	High: moderate severity with spalling and distortion	Spot Leveling	Spot Leveling, Thin Overlay, Structural Overlay, & Reclaim and Overlay	Hot In-place Recycling, Cold Recycle and Overlay, & Reclaim and Overlay	Reconstruction

Table 3 - Rutting		
		Project Segment Average*
Severity	Low: <⅛"	N.A.
	Moderate: ⅛-¼"	Thin Overlay
	High: ¼-½"	Thin Overlay & Mill and Fill
	Extreme: >½"	Mill and Fill, Hot In-place Recycling, & Reconstruction

Table 4 - Roughness		
(measured in IRI)		Project Segment Average*
Severity	Low: <150	N.A.
	Moderate: 150-250	Thin Overlay
	High: >250	Thin Overlay and Structural Overlay

*If the "project" is divided into two or more "segments," then this distress may be averaged for each segment.

Table 5 - Drainage		
		Failure specific treatment
Severity	Inadequate ditches	Correct ditch breakdown
	Underdrain malfunctioning	Flush or replace
	Frost heaves	Reclaim and Overlay & Reconstruction
	Excess fines in subbase	Reconstruction

Table 6 - Structure deficiency

(measured in SN)		Extent	
		<50%	>50%
Severity	<1	Thin Overlay	Thin Overlay, Mill and Fill, & Structural Overlay
	1-2	Thin Overlay, Mill and Fill, & Structural Overlay	Structural Overlay, Hot In-place Recycling, Cold Recycle and Overlay, & Reclaim and Overlay
	>2	Reclaim and Overlay & Reconstruction	Reconstruction

Table 7 lists the various preservation and rehabilitation treatment options. This table is intended to provide the designer with a reasonable life expectancy for a treatment considering time and ESALs. The actual performance depends on several variables; however, the treatments are listed in ascending order of treatment extent, estimated life, and cost. The site-specific factors that impact the performance include the pavement type, drainage, soils, frost depth, and the condition of pavement at the time of treatment. The typical Vermont pavement type can be classified into one of four general types:

Thick-on-Strong (>6 inches ACC on >18 inches subbase),
Thin-on-Strong (<5 inches ACC on >18 inches subbase),
Thin-on-Weak (<5 inches ACC on little or no subbase), and
Composite (ACC on PCC slabs).

When evaluating the expected performance of any given treatment, due consideration must be given to project specific conditions. Performance is based on environmental deterioration as well as traffic loading. Performance life based on traffic loading may be estimated by evaluation of the pavement structure before and after treatment. Appropriate layer coefficients, consistent with FWD findings and the effective SN, are necessary. The performance life based on environmental conditions is less of an analysis, but is based more on observation. One “rule of thumb” that may be used is directly applicable to reflective cracking: even with very little traffic, cracks reflect through new ACC at the rate of about one inch per year.

Furthermore, the condition of the pavement at the time of treatment will impact the treatment life. It stands to reason that a thin overlay applied to a road with low severity and low extent cracking will outperform the same overlay applied to a road that is fatigue cracked to the point where it is beginning to pothole. These are concepts that are difficult to quantify, yet the designer must understand and consider all the information when selecting a rehabilitation or preservation treatment.

Select at least three candidate solutions from the following rehabilitation treatments.

Table 7 - Rehabilitation Treatments

Treatment	New ACC Added	Estimated Life	Estimated ESAL Life* w/SN=3 & 4
Crack Sealing	-	3-4 years	-
Spot Leveling	-	-	-
Chip Seal	-	3-6 years	-
Thin Overlay	≤1½"	5-8 years	200,000 & 550,000
Mill and Fill	2"±	8-12 years	300,000 & 1,200,000
Structural Overlay	2-5"	6-12 years	300,000-2,000,000 & 1,200,000-7,000,000
Hot In-place Recycling	1-3"	6-10 years	200,000-700,000 & 800,000-3,000,000
Cold Recycle and Overlay	2½"	12 years	400,000 & 1,750,000
Reclaim and Overlay	≥4½"	8-12 years	1,400,000 & 5,500,000
Reconstruction	Varies	20-40 years	Design dependent

* ESAL life is governed by the support provided by the underlying materials. Therefore, greater support, i.e., a higher existing SN, provides higher ESAL life estimates. For instance, a 2-5" Structural Overlay should provide 300,000-2,000,000 estimated ESALs if the existing SN is 3, but the same 2-5" treatment on an existing SN of 4 should provide 1,200,000-7,000,000 ESALs of life.

1. Preservation Treatments.

The following treatments are appropriate when the majority of the cracking is considered minor in severity, rutting is minor to moderate in severity, and the structural condition required, i.e., SN, is within 0.5 of the existing. Additionally, the overlay thicknesses are typically no greater than 1½".

Ideally, the preservation treatments should be used as part of an overall rehabilitation strategy. For instance, crack sealing could be used to preserve the investment of a rehabilitation treatment. Crack sealing may be applied within three to four years, and again in six to eight years, if warranted, following the original rehabilitation.

This strategy serves to maximize the performance of the original rehabilitation and to postpone the next major rehabilitation treatment. At the very least, less extensive treatments tend to slow the rate of the deterioration. The exception to this strategy is the spot leveling used for rut filling or other corrective measures.

<u>Crack Sealing</u> - Cracks identified for sealing may be prepared by routing, sawing, or other means. The width of the crack determines the type of preparation. Cracks are then flush filled, or slightly “overbanded.” Refer to FHWA-SA-96-027 Handbook on Preventive Maintenance Treatments or NCHRP Synthesis 223 for more details.

<u>Spot Leveling</u> - This treatment can fill dips, improve ride, or fill wheel ruts. Its life span is highly dependent on the distress it is masking.

<u>Chip Seal</u> - This treatment usually involves spot leveling followed by asphalt emulsion and stone chips. Chip seals can improve the ride, seal cracks, and improve traction. A chip seal can last between 3 and 6 years.

<u>Thin Overlay</u> - This treatment is useful to improve the ride and correct minor rutting. A tack coat must always be used. Thin overlays vary in thickness and as such are not given structural credit. Depending on the pavement type and level of distress, they can be expected to last 5 to 8 years.

2. Minor Rehabilitation Treatments.

The following treatments are appropriate when cracking is moderate and rutting is severe, or if the structural deficiency is less than 2.0. The overlay thicknesses are typically in the 2-5" range.

<u>Structural Overlay</u> - This treatment involves pavement overlays designed to accommodate structural needs and traditionally requires a “leveling” course. A tack coat should be applied prior to the placement of any new pavement material or

leveling material. The leveling course should be sufficient to fill the existing void created by imperfections in the existing pavement surface. Once leveled, the surface provided should be true and uniform to accommodate any subsequent pavement lift(s). An overlay design, as described in Section III, Chapter 5 of the Guide, should be used to determine overlay thicknesses. The DARWin Software is a useful tool for this determination. A minimum design life based on ESALs should be 6 to 12 years.

<u>Mill and Fill</u> - This technique can be used to correct heavy rutting, retain "curb reveal," or remove pavements which are cracked or undesirable. Once milled by a cold planer, a tack coat should be applied. The new pavement layer(s) can be placed at varying thickness(es) to accommodate the curb reveal or increase structural needs. A leveling course is usually necessary for minor mill and fill operations. An overlay design, as described in Section III, Chapter 5 of the Guide, should be used to determine overlay thicknesses. DARWin, again, may be used to determine the structural need. This treatment may last from 8 to 12 years.

3. Major Rehabilitation Treatments.

The following treatments are appropriate when the majority of the cracking and rutting distresses are severe or when the structural deficiency is greater than an SN of 2.0. Overlay thicknesses are typically greater than 4".

<u>Mill and Fill</u> - This treatment is considered major when milling and replacing more than 2" of ACC and is typically used to remedy more extensive and severe distresses. Leveling is usually not necessary for major mill and fill operations. A minimum design life based on ESALs should be 15 years.

<u>Reclaim and Overlay</u> - This process involves pulverizing the existing pavement and some portion of the subbase. The pulverized material is then stabilized, if necessary, compacted, and then overlaid. Reclaiming will have greater success if the subbase is of good quality. This treatment is not capable of improving the stability of the subbase material; however, subbase material may be added to improve the gradation. It is usually used to eliminate existing crack patterns. Reclaim depths depend on pavement thicknesses as determined by pavement cores. Test pits are recommended to assess the quality of the unbound base and subbase materials. Water, calcium chloride, or asphalt emulsion can be used to stabilize the subbase-like material. Crushed aggregate can be added to achieve required gradations. Overlay thicknesses increase from the 4½" minimum thickness based on structural need. Structural need is determined by assessing the existing structural strength, reducing it for the reclaiming, and comparing that to the required structural number for the design period. It can be expected to last 8 to 12 years.

Cold Recycle and Overlay - The recycling is usually done on thicker cracked pavements. A 2 to 4 inch portion of the cracked pavement is milled, screened, mixed with asphalt emulsion, and relayed with the paver. It is a pavement-like product which helps delay reflective cracking. For in-place recycling, the equipment train requires a minimum of 2½" of pavement for support. Overlay thicknesses vary, but generally can start around 2½"± based on structural need. Structural need is determined through a process similar to that used for the reclaim and overlay treatment. It is expected to last 10 to 12 years.

Hot In-place Recycling - This treatment is performed by specialty contractors. It is usually used to eliminate existing crack patterns or to correct friction or rutting problems. The treatment includes heating and scarifying the existing pavement surface, adding additional stone or bituminous mix, compacting, and placing an additional bituminous overlay. Depending on the overlay thickness, this treatment can be expected to last from 6 to 10 years.

Reconstruction - This treatment is recommended when it is no longer cost effective to rehabilitate the existing pavement and when budgets allow. This treatment is intended for the complete replacement of the entire pavement structure, whether the roadway facility is relocated, or not, and consistent with the Agency's "Flexible Pavement Design Procedures" for new pavement structures.

B. Develop Preliminary Designs

1. Determine the primary treatment, construction duration, and construction costs.

2. Determine the subsequent maintenance treatments required for each candidate primary treatment to provide an appropriate analysis period. The construction duration and costs for each maintenance treatment will be needed for the LCCA.

III. Selection of the Preferred Treatment

The analysis associated with an LCCA may help the designer determine the preferred treatment. Generally, life-cycle costs refer to all costs, and in the complete sense, all benefits, that are involved with the construction and performance of a pavement during the analysis period. These costs include construction, maintenance, rehabilitation, salvage value, and user costs. Since it is desirable to minimize costs, user costs in particular, the effect of this minimization can be considered a "benefit." However, the lowest cost treatment is not necessarily the preferred treatment. LCCA, particularly probabilistic LCCA, allows the designer to evaluate risk and uncertainty associated with the treatments being considered and also to evaluate the pros and cons of a variety of treatment scenarios. Probabilistic LCCA is the method of choice; however, a simple discrete LCCA module is available in DARWin and may be used for this procedure.

A. Cost Analysis

1. For LCCA, all candidate solutions should be reduced to similar service lives (the analysis period) to facilitate this analysis. If it is impractical to have all candidate solutions reduced exactly to the same service life, a few years' difference will not adversely affect the overall analysis. Use a real discount rate in the range of 3-5% and use constant dollars, i.e., ignore inflation since interest rates are generally 3-5% greater than inflation.

2. Agency Costs.

   a. If design work has already been done for the project, ignore these costs. This investment cannot be recovered and should not influence the designer's selection of the preferred treatment.

   b. The construction costs associated with each candidate solution, and any subsequent maintenance or rehabilitation work, must be determined. The construction durations for the various treatments must be estimated for user cost calculations.

   c. Salvage value should not be significant for most rehabilitation analyses, and may be ignored. If the salvage value of a treatment is considered, its calculation should be determined in a manner similar to straight-line depreciation. That is, the designer may apply the percentage of remaining service life to the initial cost to determine the "unused value" of the treatment in question.

3. User Costs.

Calculate user costs for each treatment alternative being considered. This includes the initial construction and all subsequent operations intended to achieve the full analysis period for each rehabilitation treatment alternative. Use the procedure summarized in "Life Cycle Cost Analysis in Pavement Design, Demonstration Project No. 115," (Publication No. FHWA-SA-98-040), DP-115 for short, to determine the user costs associated with any lane closure, or capacity reduction, required by construction activity. This procedure for calculating user costs is fairly straightforward, but intricate.

a. For most two-lane rural highways, and short urban projects, the total daily user costs from Table 8 may be used instead of performing a detailed analysis.

Table 8 - Total Daily User Costs

Length of workzone	≤ 1000 ADT	1000-5000 ADT	5000-10000 ADT	10000-20000 ADT
≤ 0.1 mile	$25	$200	$700	$3000
0.1-0.25 mile	$50	$300	$750	*
0.25-0.5 mile	$80	$450	$1000	*
≥ 0.5 mile	*	*	*	*

* Requires a detailed analysis.

b. If a detailed two-lane analysis is required, or a four-lane analysis is required, follow the procedure described in DP-115. Begin with a request for the traffic mix in passenger cars (PC), medium trucks (M), and heavy trucks (H), directional distribution, and hourly distribution from Traffic Research for the construction years under consideration covering the initial construction year and subsequent rehabilitation years.

c. When calculating user costs, the designer may assume a free flow capacity of 2100 vphpl (vehicles/hour/lane) and a work zone capacity of 1400 vphpl for four-lane divided highways. Determination of the values for two-lane highways is more involved, and must be consistent with the procedure described in the Highway Capacity Manual.

d. Determine the vehicle operating costs (VOC) for the passenger cars, medium trucks, and heavy trucks. The following typical values may be considered:

PC = \$10/hour ±\$2/hour
M = \$20/hour ±\$3/hour
H = \$25/hour ±\$5/hour

The user cost analysis described in DP-115 involves seven different conditions. The first three must be considered for any work zone where a speed limit reduction is used:

1) the added vehicle running cost to decelerate to the work zone speed,
2) the added VOC for traversing the work zone at the reduced speed, and
3) the cost of the time delay to traverse the work zone.

Additional costs are incurred if vehicles are forced to stop:

4) the additional vehicle running cost to come to a complete stop,
5) the additional VOC incurred for coming to a complete stop,
6) the VOC associated with traversing the queue delay, and
7) the VOC associated with sitting idle in the queue.

e. For simplicity, ignore detouring effects. That is, if vehicles avoid the construction area, there may be very little difference in their overall VOC.

4. Determine the net present value (NPV) of each candidate solution.

B. Preferred Rehabilitation Alternatives

The selection of the "Preferred Rehabilitation Treatment" may be based on the least total cost. However, consideration should be given to the user cost component, and how it may be minimized. If a probabilistic approach is used to determine the LCCA, the designer may consider the likelihood of a particular alternative's performance, and how it compares to the other alternatives considered.

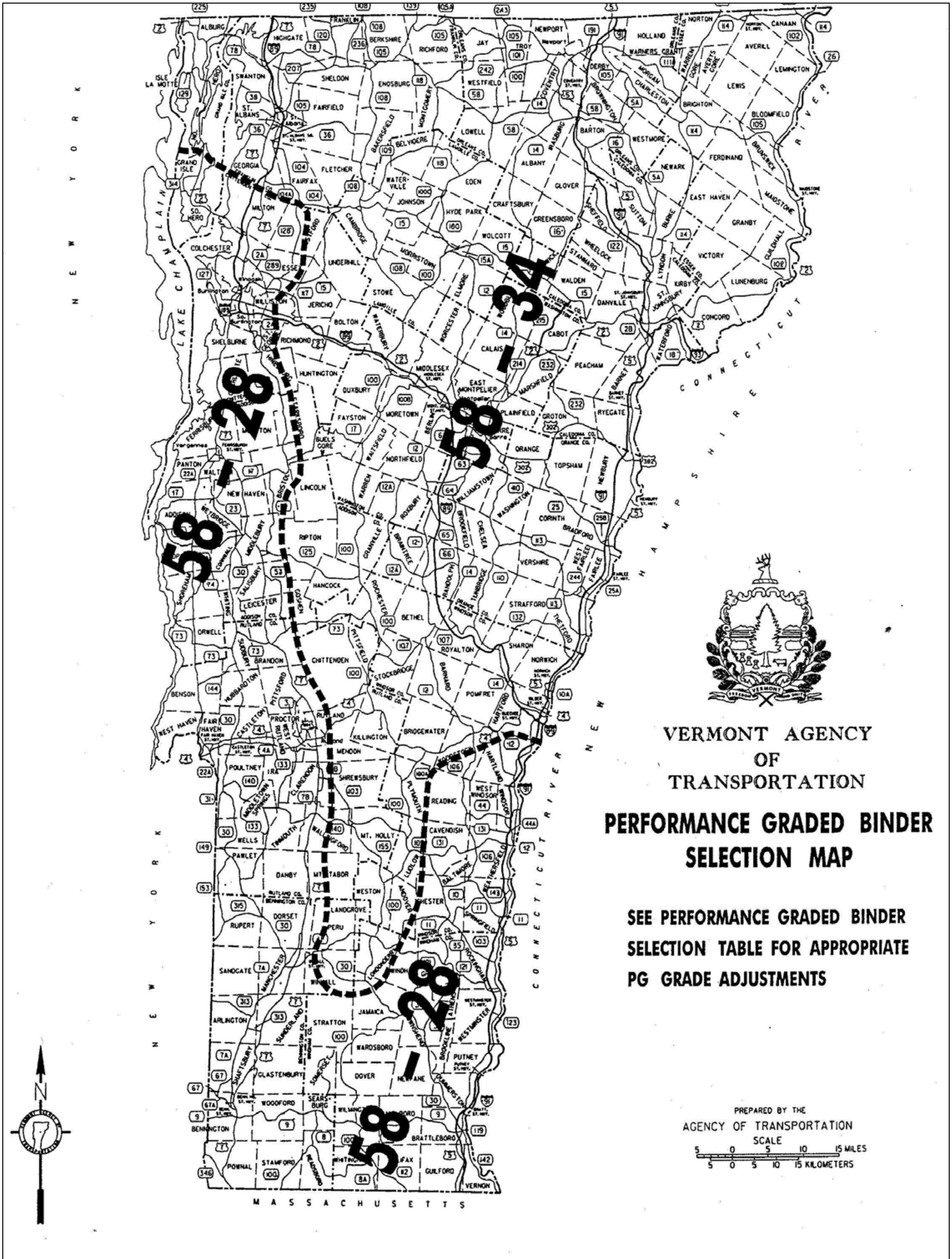
Vermont Agency of Transportation Pavement Materials Design Properties					
	Stabilities		**Layer Coefficient**		
Pavement Component	**Marshall (lbs)**	**Resilient Modulus (psi)**	**a_1**	**a_2**	**a_3**
Surface Courses					
Type II ACC	2,850	350,000	0.35		
Type III ACC	2,650	300,000	0.33		
Type IV ACC			0.32		
Type IIS ACC	1,350	140,000	0.32		
Type IIIS ACC	1,350	320,000	0.36		
Type IVS ACC		100,000	0.35		
Base Courses					
Asphalt Treated Permeable Base		100,000		0.33	
Type I ACC	3,050	360,000		0.30	
Type II ACC	2,850	350,000		0.32	
Type IS ACC	2,250	200,000		0.28	
Type IIS ACC	1,350	150,000		0.32	
Reclaimed stabilized base				0.18	
Subbase Courses					
Dense Graded Crushed Stone		30,000			0.14
Crushed Gravel		30,000			0.13
Pit Run Gravel		25,000			0.12
Granular Borrow		20,000			0.10
Sand Cushion		20,000			0.08

Vermont Agency of Transportation Metric Pavement Materials Design Properties					
	Stabilities		**Layer Coefficient**		
Pavement Component	**Marshall (N)**	**Resilient Modulus (MPa)**	**a_1**	**a_2**	**a_3**
Surface Courses					
Type II ACC	12,500	2,500	0.35		
Type III ACC	11,500	2,000	0.33		
Type IV ACC			0.32		
Type IIS ACC	6,000	1,000	0.32		
Type IIIS ACC	6,000	2,250	0.36		
Type IVS ACC		750	0.35		
Base Courses					
Asphalt Treated Permeable Base		750		0.33	
Type I ACC	13,500	2,500		0.30	
Type II ACC	12,500	2,400		0.32	
Type IS ACC	10,000	1,400		0.28	
Type IIS ACC	6,000	1,000		0.32	
Reclaimed stabilized base				0.18	
Subbase Courses					
Dense Graded Crushed Stone		200			0.14
Crushed Gravel		200			0.13
Pit Run Gravel		175			0.12
Granular Borrow		150			0.10
Sand Cushion		150			0.08

Vermont Agency of Transportation
Materials & Research Division

1989 Pavement Design Study
For Information Only and Should Not be Used for New Construction

Pavement Component	Strength		Coefficient		
	Marshall	CBR	a_1	a_2	a_3
Surface Courses					
Type II Bit. Conc. w/AC 20	1650	212	0.39		
Type III Bit. Conc. w/AC 20	1500	205	0.37		
Type IV Bit. Conc. w/AC 20	1500	137	0.37		
Type V Bit. Conc. w/AC 20	850		0.28		
All Types 1977-1988 w/AC 10	1300		0.34		
All Types 1972-1976 w/AC 5	1000		0.30		
All Types Prior to 1972	1400		0.36		
Base Courses					
Type I Bit. Conc. w/AC 20	1800	251		0.32	
Type II Bit. Conc. w/AC 20	1650	212		0.30	
Item 303 Plant Mix		95		0.22	
Bomag Recycle 50/50 Mix		45		0.13	
Subbase Courses					
Dense Graded Crushed Rock		70			0.13
Crushed Gravel		40			0.12
Pit Run Gravel		30			0.11
Sand Cushion		12			0.08
Granular Borrow		10			0.07


VERMONT AGENCY
OF
TRANSPORTATION
PERFORMANCE GRADED BINDER
SELECTION MAP
SEE PERFORMANCE GRADED BINDER
SELECTION TABLE FOR APPROPRIATE
PG GRADE ADJUSTMENTS
58 - 34
58 - 28
58 - 28
58 - 28
PREPARED BY THE
AGENCY OF TRANSPORTATION
SCALE
5 0 5 10 15 MILES
5 0 5 10 15 KILOMETERS
N
NEW YORK
LAKE CHAMPLAIN
NEW HAMPSHIRE
CONNECTICUT RIVER
MASSACHUSETTS
FREEDOM AND UNITY
VERMONT

Performance Graded Binder Selection Table

Adjusted PG Binder
on the Basis of Traffic Speed and Traffic Level

Design ESALs[1] (million)	Adjusted PG Binder Grade		
	Average Traffic Speed		
	< 20 km/h (12 mph)	20 to 70 km/h (12 to 44 mph)	> 70 km/h (44 mph)
< 0.3	PG 58-XX[2]	PG 58-XX	PG 58-XX
0.3 to < 3	PG 64-XX	PG 58-XX	PG 58-XX
3 to < 10	PG 70-28 [3]	PG 64-XX	PG 58-XX
10 to < 30	PG 70-28 [3]	PG 64-XX	PG 64-XX
> 30	PG 70-28 [3]	PG 64-XX	PG 64-XX

[1] Design ESALs are the anticipated project traffic level expected on the design lane over a 20-year period, regardless of the actual design life of the roadway.

[2] XX indicates the low temperature of the selected PG Binder determined from the Performance Graded Binder Selection Map, either -28 or -34.

[3] When the high-end temperature is adjusted two grades to a 70, the low-end temperature needs to be changed to a -28 if the selected PG binder is a PG 58-34. If selected PG binder is a PG 58-28, then no change to the low-end temperature is needed when changing the high-end temperature two grades to 70.

Examples:

(A) Selected PG Binder from Map = PG 58-28
Design ESALs (20 years) = 4,500,000
Average Traffic Speed = 85 km/h
Final PG Binder for project = PG 58-28 (no adjustment)

(B) Selected PG Binder from Map = PG 58-28
Design ESALs (20 years) = 4,500,000
Average Traffic Speed = 55 km/h
Final PG Binder for project = PG 64-28 (High End Temperature adjusted by one grade)

(C) Selected PG Binder from Map = PG 58-34
Design ESALs (20 years) = 4,500,000
Average Traffic Speed = 55 km/h
Final PG Binder for project = PG 64-34 (High End Temperature adjusted by one grade)

(D) Selected PG Binder from Map = PG 58-34
Design ESALs (20 years) = 4,500,000
Average Traffic Speed = 15 km/h
Final PG Binder for project = PG 70-28 (Adjustment made according to Footnote 3)

Design Thickness Guidelines for Marshall Mix Types of Asphalt Cement Concrete (ACC)

Pavement (ACC) layer thickness is a function of the aggregate gradation used for the mix type and is based on generally accepted norms used in the industry. While designers, based on actual project experience, may find it advantageous to “push” the recommended thickness limits, this practice is not encouraged by the Materials and Research Section and the Pavement Design Committee. As with all design guidance, the designer must take into consideration project-specific convenience and program optimization.

The “normal” thickness ranges for ACC material should fall within an approximate range of two to three times the dimension of the maximum aggregate size, with a “recommended” thickness conveniently situated between these two extremes.

Marshall Thickness Recommendation (in.)			
Mix Type	Minimum	Recommended	Maximum
I	2½	3½	4
II	2	2½	3
III	1½	2	2¼
IV	1	1¼	1½
V	¾	1	1¼

Metric thickness ranges are provided using criteria similar to that used to develop the English recommendations.

Marshall Thickness Recommendation (mm)			
Mix Type	Minimum	Recommended	Maximum
I	65	80	95
II	50	65	75
III	40	50	55
IV	25	30	35
V	20	25	30

Consistent with the approximate nature of the industry criteria, both the English and Metric recommendations are not exact, but rather convenient values for their respective unit of measure.

Superpave Design Selection Tables

Selection of Design Gyrations

Design ESALs[1] (million)	Design Gyrations
< 0.3	50
0.3 to < 3	75
3 to < 10	100[2]
10 to < 30	100
> 30	125

(1) Design ESALs are the anticipated project traffic level expected on the design lane over a 20-year period. Regardless of the actual design life of the roadway, determine ESALs for 20 years and choose the appropriate N_{design} level.
(2) When the estimated design traffic level is between 3 to 10 million ESALs, the agency may specify 75 design gyrations.

Superpave Thickness Recommendations

Mix Type	Nominal Maximum Aggregate Size mm (in)	Suggested Nominal Depth, mm (in)		
		Minimum	Recommended	Maximum
MS	37.5 (1 1/2)	115 (4 1/2)	135 (5 3/8)	140 (5 1/2)
IS	25.0 (1)	75 (3)	90 (3 1/2)	100 (4)
IIS	19.0 (3/4)	60 (2 3/8)	70 (2 3/4)	80 (3 1/4)
IIIS	12.5 (1/2)	40 (1 1/2)	45 (1 3/4)	50 (2)
IVS	9.5 (3/8)	30 (1 1/4)	35 (1 3/8)	40 (1 1/2)

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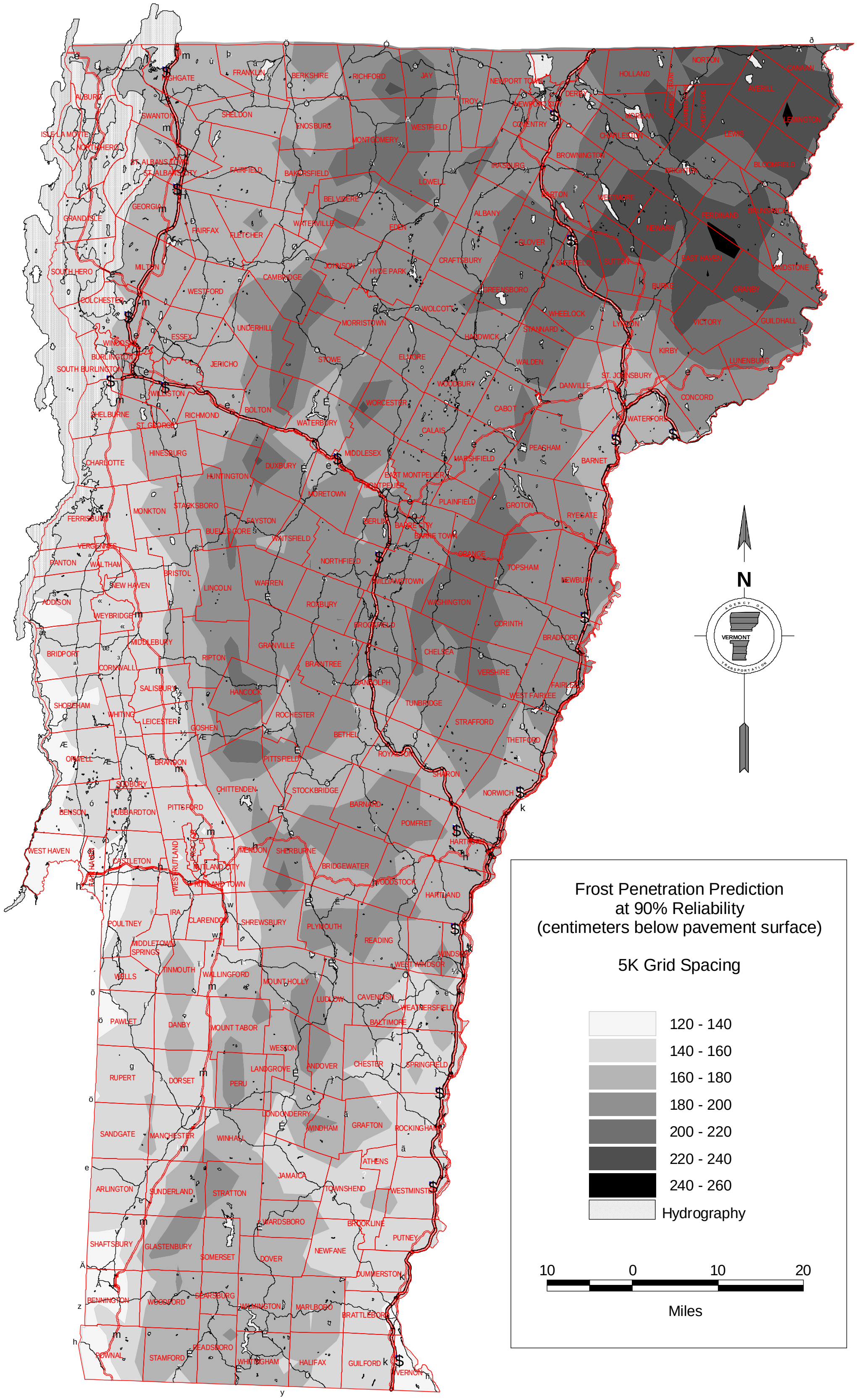
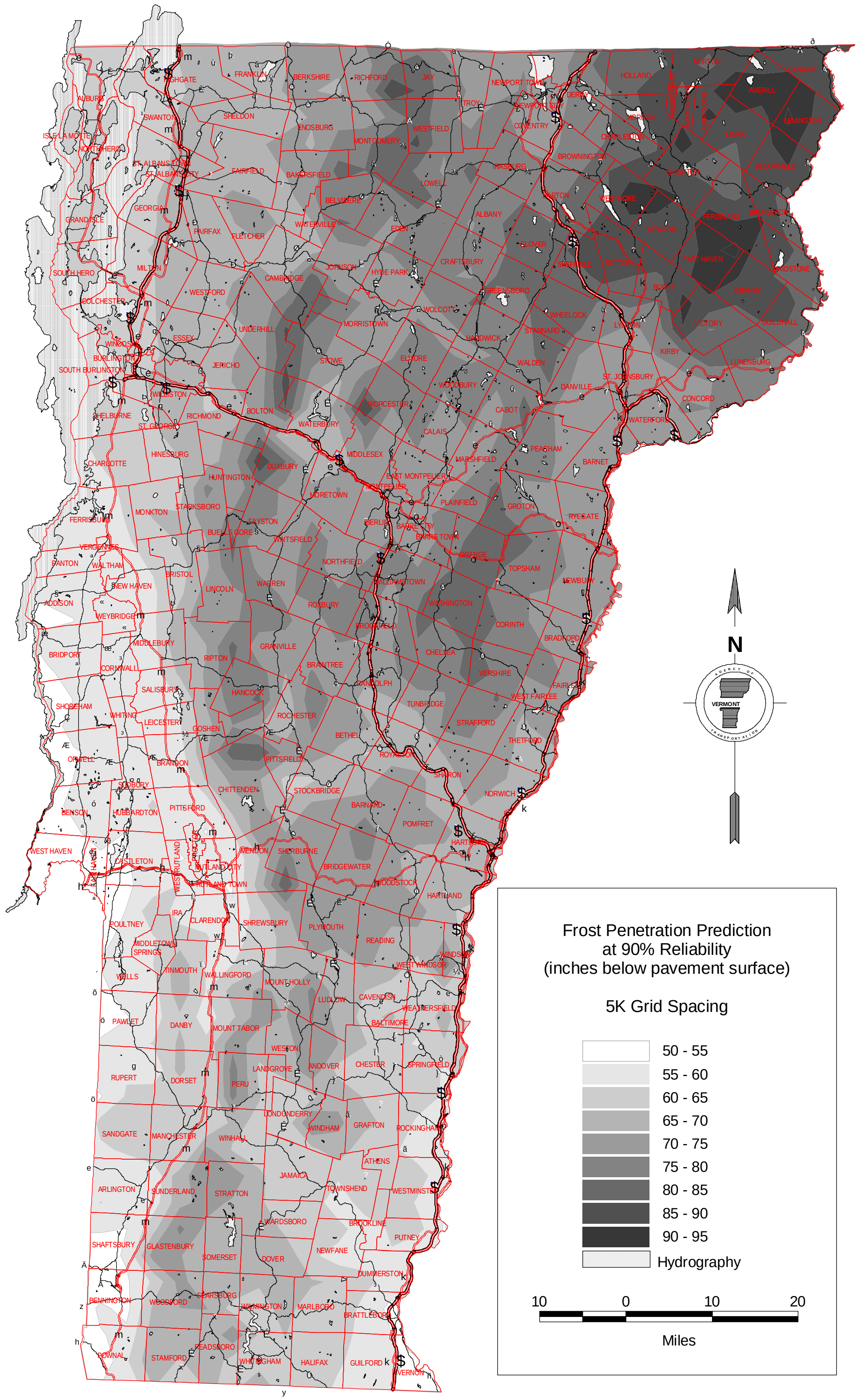

Frost Penetration Prediction
at 90% Reliability
(centimeters below pavement surface)
5K Grid Spacing
120 - 140
140 - 160
160 - 180
180 - 200
200 - 220
220 - 240
240 - 260
Hydrography
10
0
10
20
Miles
N
VERMONT


Frost Penetration Prediction
at 90% Reliability
(inches below pavement surface)
5K Grid Spacing
50 - 55
55 - 60
60 - 65
65 - 70
70 - 75
75 - 80
80 - 85
85 - 90
90 - 95
Hydrography
10
0
10
20
Miles
N
VERMONT

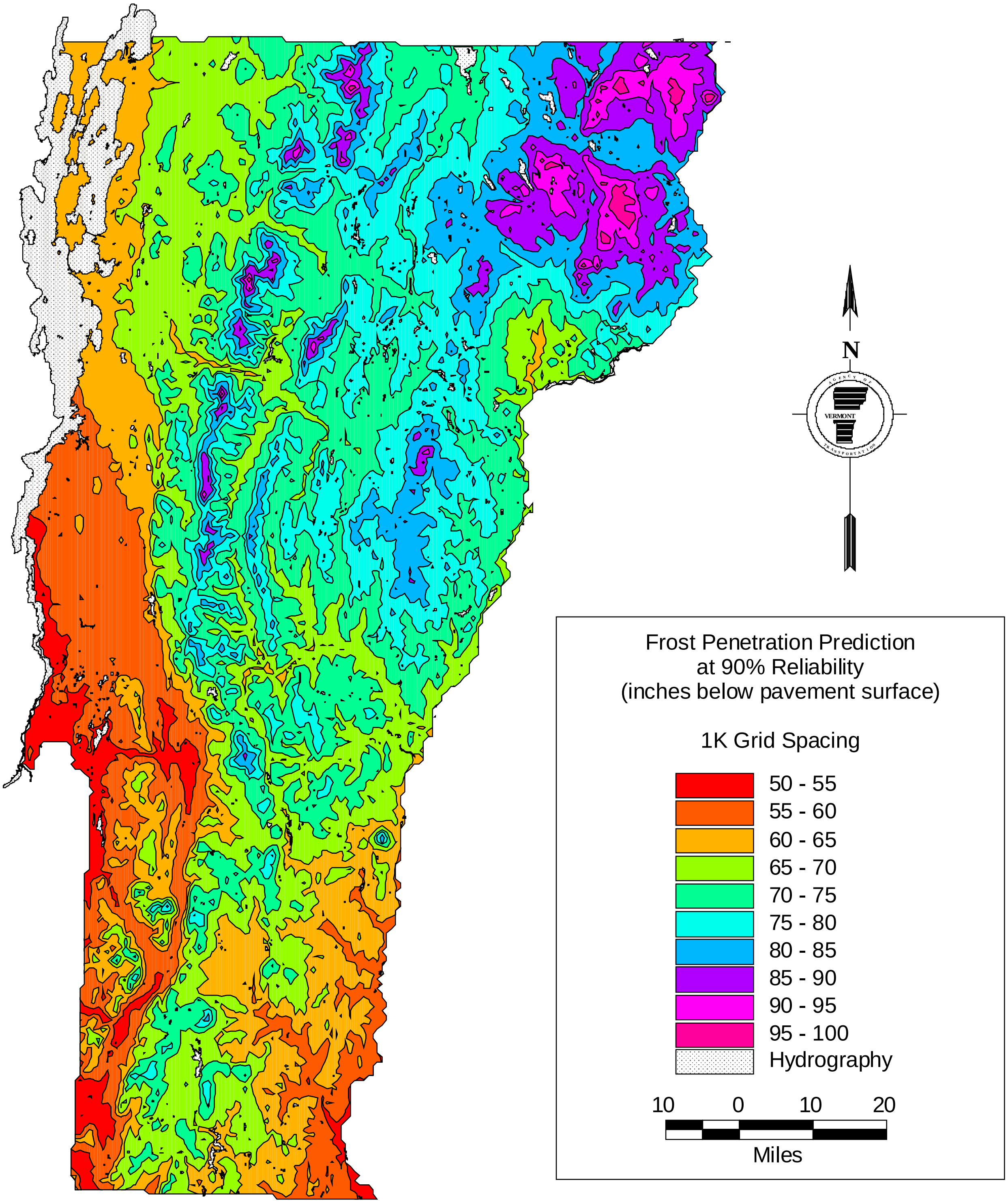
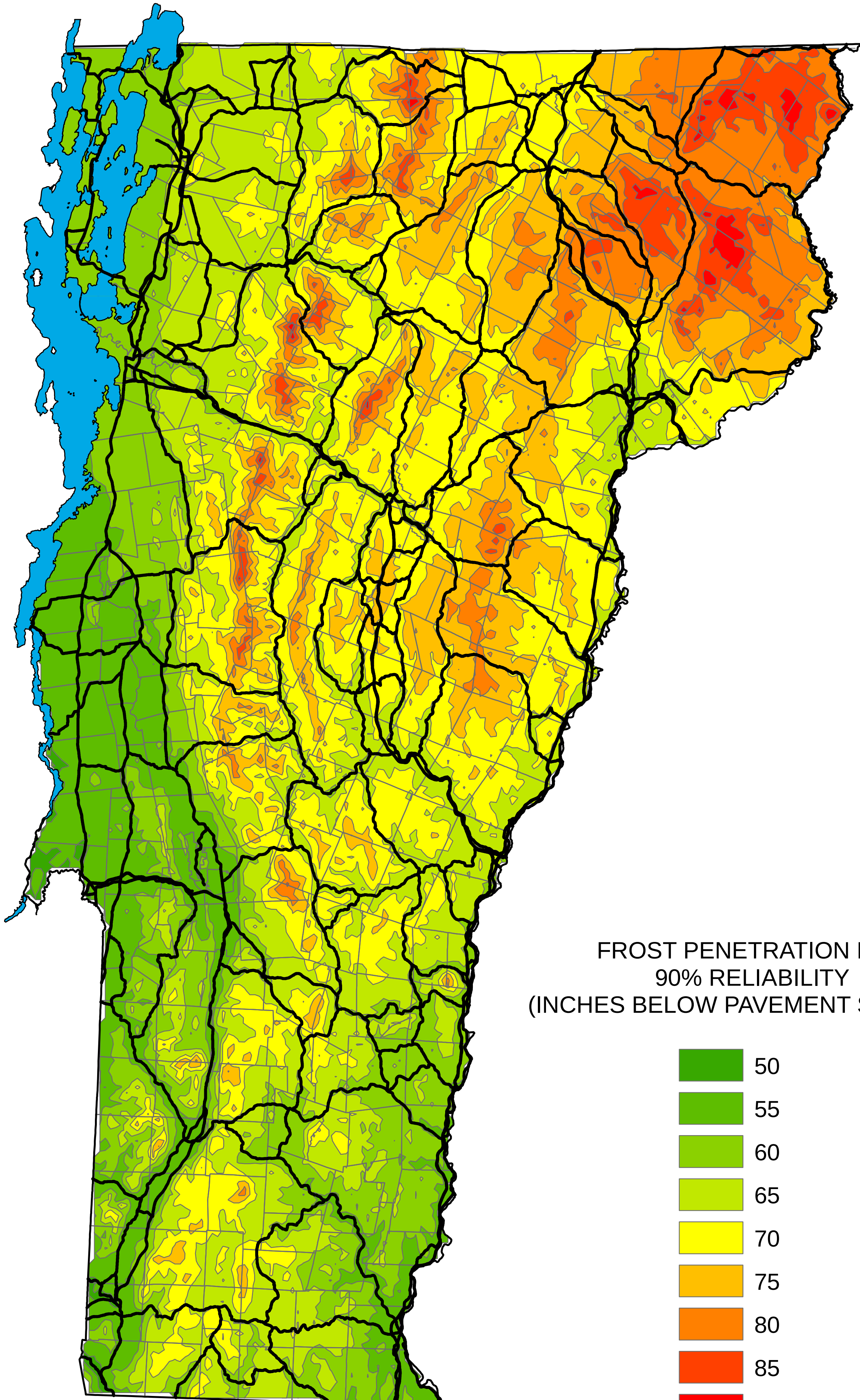


Figure 22. Statewide maximum frost penetration depth in inches at 90% reliability derived from correlation of 90th percentile deterministic model results to 90th percentile AFDI.



FROST PENETRATION MAP
90% RELIABILITY
(INCHES BELOW PAVEMENT SURFACE)

Legend
50
55
60
65
70
75
80
85
91.312

The Pavement Design Committee has developed the following policy regarding pavement type selections:

> "Based on economic evaluations, the Vermont Agency of Transportation endorses the use of flexible pavements, i.e., asphalt cement stabilized concrete, for Vermont State highways. If conditions warrant, the designer may recommend a rigid design, i.e., Portland cement concrete. Examples of the possible warranting conditions that could prompt consideration of a rigid pavement include intersections with high volumes of turning and/or stop-and-go-traffic, steep stop controlled intersection approach grades, and/or community preference for routes under local control. The designer must provide justification for the proposed use of a rigid pavement. This justification will be in the form of a life-cycle cost analysis and will be consistent with the 'AASHTO Guide for the Design of Pavement Structures.' This analysis should demonstrate the economic superiority of the proposed rigid pavement over a forty year analysis period."

The AASHTO guide provides guidelines for the development of a life-cycle cost analysis. The designer may consider the following information useful.

1. Confirm with FHWA the inflation rate and discount rates that are required (or are reasonable) to be used for the life-cycle cost analysis.

2. Both flexible and rigid ESALs are required for the prospective design typicals for the analysis period.

3. The following maintenance scenarios may be considered for the respective pavement type:
- ACC: a mill-and-fill operation at 10-, 20-, and 30-year intervals
- PCC: at the 20 year interval repair of the PCC joints and overlay using a saw-and-seal treatment

  at the 30-year interval, cold-plane the 20-year overlay, repair the PCC as needed, repair the PCC joints, and proceed with another overlay using a saw-and-seal treatment.

4. Appropriate unit-price estimates shall be used for all materials and/or work elements. Then current PCC Concrete prices should be verified by consulting with FHWA and/or adjacent states. Traffic Control costs will be included in the analysis.

Simplified Pavement Design for Small Projects

The following procedure was developed in order to simplify the pavement design for projects that use a relatively low volume of bituminous concrete pavement. The goal was to allow the contractors some flexibility in choosing the type of pavement and grade of binder for small projects. In developing the table, we used the DARWin pavement design software and a range of design-lane esals, assuming a rural minor arterial highway with the following inputs:

Initial Serviceability:	4
Terminal Serviceability:	2.5
Reliability Level:	85%
Overall Standard Deviation:	0.45
Roadbed Soil Resilient Modulus:	200,000 kPA or 30,000 psi
Construction Stage:	3

Because this table was based on inputs for a rural minor arterial, you will find that the recommended pavement thickness for local roads with very low esals is a bit more than necessary. The designer is free to develop his or her own pavement design, but should keep in mind that the goal is to minimize number of different pavement types called for on small projects.

Draft Proposal: 26 Oct 2001
Revision #6: 26 Feb 2003

Simplified Pavement Design for Small [1] Projects

20-year ESALs [2]		**Wearing Surface** Type III or IV		**Base Course** [3] Type I or Type II		**Marshall**
1,000,000 < ESALs ≤ 3,000,000	2nd Lift	1½ in	40 mm	3 in	75 mm	75 blow
	1st Lift	1½ in	40 mm	3 in	75 mm	
250,000 < ESALs ≤ 1,000,000	2nd Lift	1½ in	40 mm	2½ in	65 mm	50 blow
	1st Lift	1½ in	40 mm	2½ in	65 mm	
ESALs ≤ 250,000	2nd Lift	1½ in	40 mm	N/A	N/A	50 blow
	1st Lift	1½ in	40 mm	3 in	75 mm	

	Wearing Surface in (mm)	**Base Course** in (mm)	**Marshall**
Parking Facilities [8]	1½ (40) Type III or IV	2 (50) Type II	75 blow
Multi-use Facilities (Bike paths, sidewalks, etc.)	2 (50) Type III	N/A	50 blow

[1] Small projects are defined as having less than 500 tons (U.S. Customary) per mix type.

[2] Design lane ESALs.

[3] Top of base course will be flush with top of concrete bridge deck (i.e., final two lifts of wearing surface shall be the same for the both the roadway approaches and the bridge deck).

[4] Subbase for all projects with ESALs (i.e., roadway/bridge) shall use 18 in (450 mm) subbase, either crushed gravel or dense graded crushed stone. Others (e.g., Park & Rides and Bike Paths) shall be as specified in the plans.

[5] Bridge project (metric) example: Total ESALs = 1,500,000 for two-lane rural route. Therefore, design lane ESALs = 750,000. Roadway approaches will consist of 40 mm over 40 mm (Type III or IV) over 65 mm over 65 mm (Type I or II). Total pavement depth will be 210 mm bituminous concrete pavement (BCP) over 450 mm subbase. On bridge there will be 40 mm over 40 mm (Type III or IV).

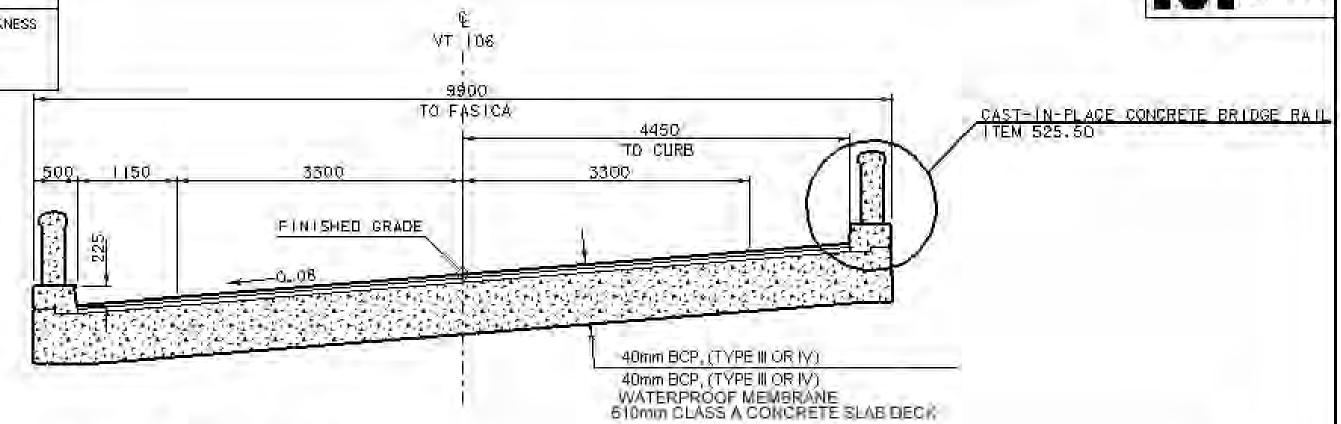
[6] On new construction projects, the designer should still take the effects of frost into consideration by providing a sand layer in accordance with the VTrans Flexible Pavement Design Procedures.

[7] The Contractor may choose to use either Type III or IV BCP for the wearing course(s) and either Type I or II BCP for the base course(s) or as directed by the Engineer. The following PG grades will be allowed per project special provisions or as directed by the Engineer:

PG 64-28	PG 64-34
PG 58-34	PG 58-28

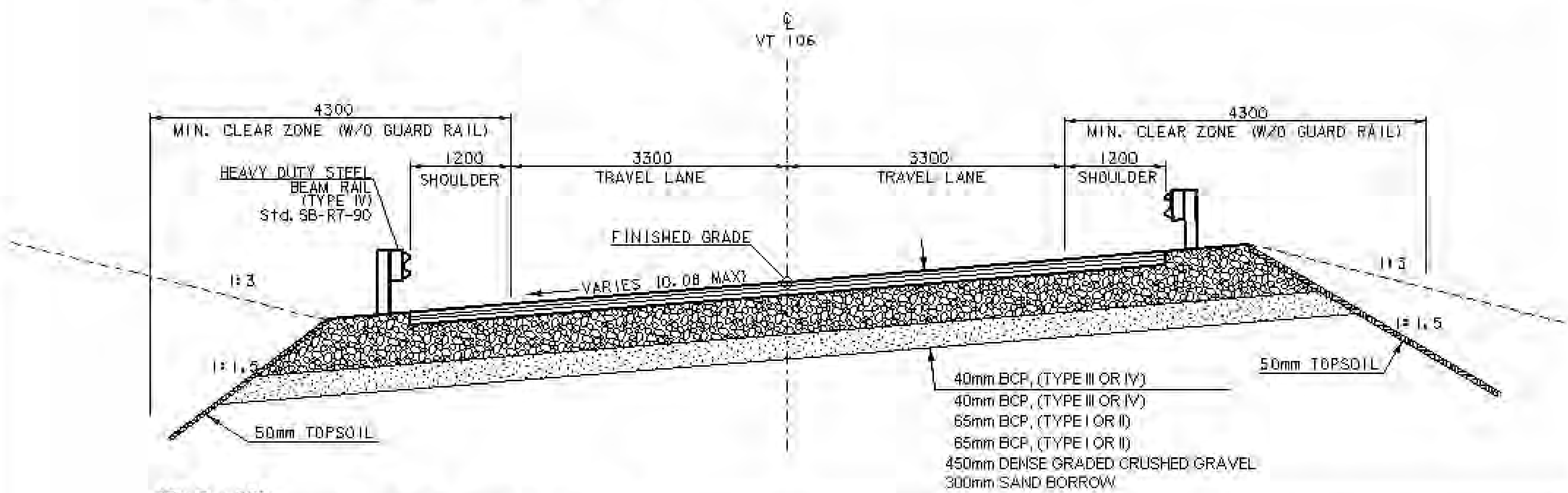
[8] Parking facilities include park & rides, parking lots, etc. Include 300 mm (12 inches) of unbound subbase when frost penetration is less than 750 mm (30 inches), 450 mm (18 inches) where frost penetration is greater than 750 mm (30 inches), and increase these subbase thicknesses by 150 mm (6 inches) when silt or clay subgrades are encountered.

MATERIAL TOLERANCES	
MATERIAL ITEM	TOLERANCE
PAVEMENT	+/-¼" TOTAL THICKNESS
SUBBASE	+/-1"
SAND BORROW	+/-1"



SCALE 1:250

TYPICAL BRIDGE SECTION



SCALE 1:250

TYPICAL ROAD SECTION

PROJECT NAME: WOODSTOCK
BRS 0151(8)
FILE NAME:
PROJECT LEADER:
DESIGNED BY:
PLOT DATE:
DRAWN BY:
CHECKED BY:
SHEET OF