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Allowable Axle Loads on Pavements

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Research Project
Final Report #2011-02

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This report documents the development of a procedure to determine the structural adequacy and need of seasonal axle load restrictions for Minnesota low-volume roads. This procedure has been implemented into a new program, TONN2010. Since it is anticipated that the results of this study will be widely used by Mn/DOT, city, and county engineers, as well as consulting engineers involved in analysis of the falling weight deflectometer (FWD) data collected by the transportation agencies, an emphasis was made on development of a simple, easy to implement procedure. To simplify the procedure's implementation, the number of inputs was minimized. TONN2010 utilizes pavement layer thicknesses, FWD deflection basins, air temperature of the previous day, pavement surface temperature at the time of testing, pavement location, and anticipated traffic. All the inputs required by TONN2010 can be easily obtained by the user. Using these inputs, TONN2010 proceeds to 1) backcalculate layer moduli using the backcalculation procedure developed in this study, 2) adjust the backcalculated moduli using MnPAVE temperature and seasonal adjustment factors, and 3) estimate pavement axle load capacity by mechanistic-empirical analysis. In addition to detailing TONN2010, the report further describes selection of the damage models, development of the backcalculation design procedure, determination of the critical structural responses, development of new structural rating indexes, and finally the calibration and validation of the proposed procedure.

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EXECUTIVE SUMMARY

The current falling weight deflectometer (FWD) deflection analysis process used by Mn/DOT and implemented into the computer program TONN was developed from Investigation 603. It converts the measured maximum FWD deflection to an equivalent Benkelman Beam deflection and compares it to the allowable deflection of an asphalt surface thickness and anticipated traffic. Since the maximum FWD deflection is greatly affected by the subgrade stiffness, the TONN program may underestimate the allowable axle load for soft clay subgrades but overestimate it for stiff sand or granular subgrades. Hence, the TONN program does not fully account for the structural contribution of the constructed layers in the pavement, which may lead to either overdesign and unnecessary construction costs, or underdesign and consequent rehabilitation costs. Mn/DOT consequently approved the development of TONN2010, the update to Mn/DOT's existing TONN program.

The first step in the development of TONN2010 was to simplify the number of inputs required by the procedure. It is anticipated that TONN2010 will be used by state, city, and county engineers, hence while the procedure should be robust, it should be easy to implement for users with all levels of pavement engineering expertise. Minimizing inputs is a first step in this direction. TONN2010 utilizes pavement layer thicknesses, FWD deflection basins, previous day's air temperature, pavement surface temperature at the time of testing, pavement location, and anticipated traffic.

Furthermore, as the main objective of this study was to improve upon the original TONN and better estimate pavement axle load capacity through mechanistic-empirical analysis, the selection of appropriate damage models is an important part of the TONN2010 procedure development. Eighteen different failure criteria models (in fatigue cracking, subgrade deformation, and base failure) for asphalt pavements were identified and reviewed. Of these candidates, the MnPAVE models were selected for implementation into the TONN2010 procedure. In addition to being localized to Minnesota low-volume roads, the MnPAVE performance models are easy to implement and required few inputs, in line with the goals stated above. Furthermore, MnPAVE's climatic inputs being a part of TONN2010 ensures consistency with the MnPAVE design method, which pavement engineers have adopted throughout the state. In order to achieve consistency with MnPAVE, the TONN2010 pavement evaluation process adopts MnPAVE's seasonal durations (or "five seasons").

Once desired pavement inputs, including those depicting climate, and pavement response models had been selected, it was necessary to determine a FWD backcalculation method to accommodate TONN2010. The backcalculation process involves the selection of the pavement model capable of generating a deflection profile at the pavement surface (forward analysis) and the search for model parameters that result in the generated deflection profile that best resembles the measured deflection basis. A number of requirements were identified for the backcalculation procedure needed for TONN2010, and many publically available backcalculation procedures were reviewed in light of these requirements. None of these procedures satisfied the criteria. Therefore, a simple backcalculation procedure was developed as part of the work of developing TONN2010. The procedure utilizes a database of pre-calculated deflection basins from the layered elastic analysis for a 10,960-lb load uniformly distributed over a circular area with a

radius of 5.9 in. The vertical deflections were computed at the top surface, at lateral positions of 0, 8, 12, 18, 24, and 36 inches away from the center of the load.

The backcalculation procedure determines the layer moduli, which represent the pavement system only for the environmental conditions at the time of testing. However, the environmental conditions vary throughout the year. The asphalt modulus is temperature dependant whereas moisture conditions affect the base and subgrade moduli. To account for these effects in mechanistic-empirical analysis, the backcalculated moduli should be adjusted, and the TONN2010 accommodates such an adjustment. The following procedure was adopted for adjustment of the backcalculated AC modulus using the BELLS3 procedure. BELLS3 uses the pavement surface temperature measured by the FWD, the previous day's average air temperature, thickness of the asphalt, and time of the test to estimate the third-depth temperature. The mean air temperature can be easily obtained from meteorological sources.

After the backcalculated layer moduli are adjusted to account for the seasonal effects, the critical pavement responses (strains and deflections) are computed using the layered elastic program MnLAYER (Khazanovich and Wang 2010) embedded into TONN2010. The responses are computed for five seasons. After the critical responses are determined for each season, the damage analysis is performed. Damage analysis for TONN2010 involves: 1) AC fatigue cracking damage analysis; 2) subgrade rutting damage analysis, 3) base shear failure analysis, and 4) base deformation analysis. This procedure completes the TONN2010 program.

Finally, the TONN2010 procedure was calibrated using data from MnROAD Cells 83 and Cell 84 (on MnROAD's "Farm Loop"). This calibration specifically assigned values to three key calibration constants (identified in the models as C_{AC}, C_{RUT}, and C_{DW}). The project work also involved a comprehensive comparison of the completed TONN2010 with the original TONN and alternative procedures for the evaluation of pavement structural capacity. This comparison involved an analysis of almost 8400 deflection basins at various Minnesota counties, and it frames TONN2010 as an attractive alternative to the currently available procedures.

Though the TONN2010 procedure (delivered to Mn/DOT as a Fortran program that can be run in an MS-DOS environment) has been calibrated using MnROAD data and validated against a number of deflection basins, it is important for Mn/DOT to conduct a comprehensive verification of the TONN2010 predictions for a wide range of pavement structures and site conditions through a comparison of TONN2010 ratings with actual pavement performance. If necessary, the calibration coefficients of TONN2010 can be adjusted to improve performance predictions. Furthermore, to enable the wide adoption of the TONN2010 procedure, it is important to develop a user-friendly interface (such as a graphical user interface) for the TONN2010 program delivered to Mn/DOT. The development of an interface was outside of the scope of this study.

1. INTRODUCTION

Deflection testing and analysis is routinely used to evaluate the spring load capacity of pavements and to design structural overlays. The current falling weight deflectometer (FWD) deflection analysis process used by Mn/DOT and implemented into the computer program TONN is not very reliable. The process used by TONN to interpret FWD deflection measurements was developed from Investigation 603. It converts the measured maximum FWD deflection to an equivalent Benkelman Beam deflection and compares it to the allowable deflection for a given asphalt surface thickness and anticipated traffic. Since the maximum FWD deflection is greatly affected by the subgrade stiffness, the TONN program may underestimate the allowable axle load for soft clay subgrades but overestimate it for stiff sand or granular subgrades. The structural contribution of the pavement structure constructed layers is not fully accounted for. Therefore, there is a need to upgrade the TONN program.

This reports documents development of a procedure to determine the structural adequacy and need of seasonal axle load restrictions for Minnesota low-volume roads. This procedure has been implemented into a new program, TONN2010. Since it is anticipated that the results of this study will be widely used by Mn/DOT, cities and county, as well as consulting engineers involved in analysis of the FWD data collected by the transportation agencies, an emphasis was made on development of a simple, easy to implement procedure.

To simplify the procedure's implementation, the number of inputs was minimized. TONN2010 utilizes pavement layer thicknesses, FWD deflection basins, previous day's air temperature, pavement surface temperature at the time of testing, pavement location, and anticipated traffic. It backcalculates layer moduli using the backcalculation procedure developed in this study, adjusts the backcalculated moduli using MnPAVE temperature and seasonal adjustment factors, and estimates pavement axle load capacity by mechanistic-empirical analysis. All the inputs required by TONN2010 can be easily obtained by the user.

The subsequent sections of the report describe selection of the damage models, development of the backcalculation design procedure, determination of the critical structural responses, development of new structural rating indexes, as well as calibration and validation of the proposed procedure.

2. DAMAGE MODELS

As stated above, the main objective of this study was to improve the process of pavement axle load capacity estimation through mechanistic-empirical analysis. Therefore, selection of appropriate damage models is an important part of the procedure development. The following failure criteria models for asphalt pavements were identified:

- Fatigue Cracking
  - NCHRP 1-10 (Finn et al 1977)
  - SHRP A-003A (SHRP 1994)
  - Swedish Road Administration (2000)
  - MEPDG (AASHTO 2008)
  - Asphalt Institute (1983) MS-1
  - MnPAVE (Chadbourn et al 2002)
  - Shell Petroleum (Claussen et al 1977)
  - University of California, Berkeley (SHRP 1993)
- Subgrade Permanent Deformation
  - Ayres (1997)
  - MEPDG (AASHTO 2008)
  - Asphalt Institute (1983)
  - Swedish Road Administration (2000)
  - MnPAVE (Chadbourn et al 2002)
  - University of Dresden (Werkmeister 2003)
  - South African Mechanistic Design Method (Theyse et al 1996)
  - Nottingham University (Brown et al 1977)
  - U.S. Army Corps of Engineers (Chou 1976)
- Base Failure Criteria
  - South African Mechanistic Design Method (Theyse et al 1996)
  - MnPAVE (Chadbourn et al 2002)

The MnPAVE models were selected for implementation into the TONN2010 procedure due to the following reasons:

- MnPAVE is a mechanistic-empirical design procedure which is calibrated for Minnesota low-volume roads.
- MnPAVE is becoming a widely used tool for design of Minnesota low-volume roads and overlays. It is desirable that the pavement axle load capacity evaluation program is compatible with the pavement design program.
- MnPave performance models are simple, easy to implement, and require only a few inputs.
- Many MnPAVE climatic inputs are well established. Utilization of these inputs will simplify implementation of TONN2010 program and keep consistency with the MnPAVE design method.

The details of the MnPAVE damage models are described below.

Fatigue Cracking Models

Fatigue cracking is the failure of a pavement due to repeated vehicle loads and is an important design criterion for flexible pavements. Traditional asphalt concrete (AC) cracking models assume cracking begins at the bottom of the asphalt layer and propagates upward to the surface [bottom-up], where the cracking is initially due to tensile stresses and strains at the bottom of the asphalt layer under the load. The intensity of these strains and stresses depends on the magnitude and geometry of the axle loading and properties of the pavement system (i.e. layer thicknesses, moduli, etc.).

Pavement damage in fatigue cracking is typically defined as the ratio of the number of load applications to the allowable number of load applications. Asphalt fatigue transfer functions relate the number of load repetitions to reach certain levels of failure in cracking (i.e. crack initiation, 10-percent cracked area of the pavement surface, etc.) to the maximum strains at the bottom of the AC layer. The MnPAVE form of the Asphalt Institute model for the allowable number of load repetitions is as follows (Finn et al. 1977, Chadbourn et al 2002):

$$N_f = C \cdot K_{F1} \cdot 10^{-3} \cdot \varepsilon_h^{-3.291} \cdot E^{-0.854} \quad (1)$$

where C is a correction factor based on air voids and binder content and K_{F1} is a shift factor that accounts for calibration with existing R-value designs, ε_h is the maximum tensile horizontal strain at the bottom of the AC layer, and E is the AC modulus.

MnPAVE requires computing AC tensile strains for each of the five MnPAVE seasons. The AC fatigue damage is computed for each season and accumulated according to Miner's fatigue rule.

Subgrade Permanent Deformation Models

Permanent deformation, also known as rutting, is the failure of a pavement due to poor consolidation or lateral movement of layer materials due to repeated vehicle loads. Rutting of sub-surface pavement layers occurs when the strength or stiffness of a sub-surface layer is either lower than required or somehow compromised. The MnPAVE model of equation 2 utilizes a similar model to the Asphalt Institute model (Asphalt Institute 1983).

$$N_d = 0.0261 \cdot \varepsilon_c^{-2.35} \quad (2)$$

It should be noted that the MnPAVE rutting damage model assesses only rutting damage in the subgrade and does not consider damage in the granular base layer or bituminous.

Base Shear Failure Criteria

A maximum allowable stress criterion has been implemented in MnPAVE to protect against aggregate base failure (see figure 1). The failure criterion is based on the traditional Mohr-Coulomb criterion and has the following form:

$$\sigma_1 < \sigma_{1_{critical}} = \sigma_3 \cdot \tan^2(45 + \frac{\phi}{2}) + 2 \cdot C \cdot \tan(45 + \frac{\phi}{2}) \quad (3)$$

ϕ = internal friction angle (°)

C = cohesion

σ_1 = maximum allowable major principal stress

σ_3 = minor principal stress or confining pressure for the triaxial test

This criterion states that the base fails when the maximum shear stress σ_1 exceeds the critical value $\sigma_{1_{critical}}$. Therefore, the ratio of these two parameters, SR= $\sigma_1 / \sigma_{1_{critical}}$ is an indicator of how close the base is to shear failure when it is loaded by an axle load. The smaller this ratio is the less likely the base will fail.

It should be noted that MnPAVE assumes the same Mohr-Coulomb criterion parameters, C and ϕ, for the material regardless of the season at which the stresses in the base are computed. Although this assumption might be reasonable for the internal friction angle, ϕ, it is not realistic for cohesion, C. Indeed, in early spring, after base thawing, cohesion may be much lower than for the rest of the year, even for the same moisture conditions. When the base is frozen, the cohesion is very high.

To address this limitation, TONN2010 adopted the following seasonal cohesion values

$$C_i = sc_i \; C$$

where

C_i = seasonal cohesion for the base layer for season i

C = MnPAVE Late Spring default cohesion for Class 5 base (= 6 psi)

sc_i = seasonal cohesion adjustment factors; by default are equal to 10, 0.2, 1, 1.3, and 1 for the MnPave Winter, Early Spring, Late Spring, Summer, and Fall seasons, respectively.

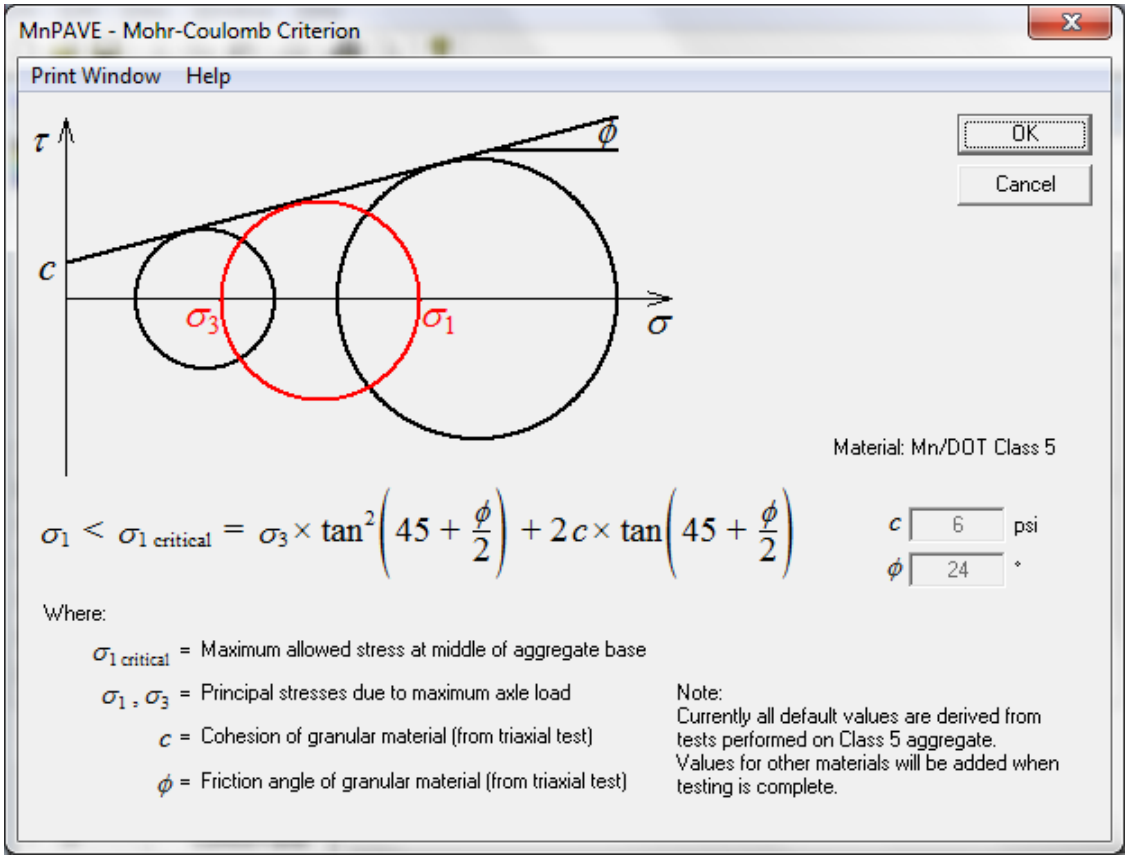


Figure 1. MnPAVE Mohr-Coulomb criterion input screen.

Base Deformations

The MnPAVE rutting model does not consider rutting in the base layer. The MEPDG uses the following equation to predict rutting in the unbound base:

$$\Delta_{p(soil)} = \beta_{s1} k_{s1} \varepsilon_v h_{soil} \left(\frac{\varepsilon_o}{\varepsilon_r} \right) e^{-\left(\frac{\rho}{n}\right)^{\beta}} \quad (4)$$

where:

$\Delta_{p(Soil)}$ = Permanent or plastic deformation in the layer/sublayer, in.
n = Number of axle load applications.
ε_o = Intercept determined from laboratory repeated load permanent deformation tests, in/in.
ε_r = Resilient strain imposed in laboratory test to obtain material properties ε_o, β, and ρ, in/in.
ε_v = Average vertical resilient or elastic strain in the layer/sublayer and calculated by the structural response model, in/in.
h_{Soil} = Thickness of the unbound layer/sublayer, in.
k_{s1} = Global calibration coefficients; k_{s1}=1.673 for granular materials and 1.35 for fine-grained materials.
β_{s1} = Local calibration constant for the rutting in the unbound layers; the local calibration constant was set to 1.0 for the global calibration effort.

$$Log\beta = -0.61119 - 0.017638(W_c) \quad (5)$$

$$\rho = 10^9 \left(\frac{C_o}{\left(1 - \left(10^9\right)^{\beta}\right)} \right)^{\frac{1}{\beta}} \quad (6)$$

$$C_o = Ln\left(\frac{a_1 M_r^{b_1}}{a_9 M_r^{b_9}} \right) = 0.0075 \quad (7)$$

W_c = Water content, percent.
M_r = Resilient modulus of the unbound layer or sublayer, psi.
$a_{1,9}$ = Regression constants; a_1=0.15 and a_9=20.0.
$b_{1,9}$ = Regression constants; b_1=0.0 and b_9=0.0.

The field-calibrated MEPDG procedure divides the base layer into thin sublayers and computes permanent deformations in the individual sublayers. Vertical strains should be determined at multiple locations to account for the effect of traffic wander. Although the MEPDG procedure is theoretically sound and robust, it is also too complex to be implemented in the proposed study. Therefore, an alternative simplified version of the procedure was proposed. It is based on the observation that if the properties of the base layer do not vary with depth, then rutting in the base layer according to the MEPDG can be expressed

$$Rut = \sum_{i=1}^{n} \chi \varepsilon_i = \chi \sum_{i=1}^{n} \varepsilon_i h_i \quad (8)$$

where Rut is the rutting in the base layer, ε_i is the vertical strain in the sublayer I, and χ is the coefficient. If the number of sublayers is increasing then equation 8 can be re-written as follows:

$$Rut = \lim_{n \to \infty} \chi \sum_{i=1}^{n} \varepsilon_i h_i = \chi \int_0^h \varepsilon \, dz = \chi (w_0 - w_h) \quad (9)$$

where w_0 is the vertical deflection at the top of the base layer, w_h is the vertical deflection at the bottom of the base.

Equation 9 suggests that limiting the difference between the vertical deflections at the top and bottom base surfaces would reduce a potential of the base rutting.

MnPAVE Climate Seasons

Environmental effects have a major influence on pavement performance. Mechanistic-empirical design procedures offer a rational approach for accounting of these effects by subdividing the pavement performance period into time increments and adjusting pavement system properties according to representative temperature and moisture conditions for the pavement. Different design procedures use different time intervals. For example, the MEPDG uses one month time increments. Although this permits refinement of the design process, it also creates an unnecessary complexity.

MnPAVE considers five seasons (Ovik, 2000):

- Early Spring. The season when the aggregate base is thawed and nearly saturated, but the subgrade remains frozen.
- Late Spring. The season when the aggregate base has drained and regained partial strength, but the subgrade is thawed, near saturated, and weak.
- Summer. The season when the aggregate base is almost fully recovered, but the subgrade has only regained partial strength.
- Fall. The season when both the aggregate base and subgrade have fully recovered.
- Winter. The season when all pavement layers are frozen.

The duration of each season is dependent on the geographic location of the pavement section and the climate it experiences. MnPAVE software provides information on the duration and average seasonal air temperatures for each season at the specific location being evaluated. A screen shot showing this information is given in figure 2.

In this study, the MnPAVE seasons were adopted for the TONN2010 pavement evaluation process. The seasonal durations were adopted to be equal to the MnPAVE durations. The mean air temperatures for all of the seasons except for Early Spring were adopted. Field testing at MnROAD indicated that although the mean air temperature may be low due to low nighttime temperatures, the high daytime temperatures induce a relatively high asphalt layer temperature causing a reduction in asphalt stiffness for several hours. Therefore, significant damage can be accumulated in the afternoon during this time. To address this issue, the mean air temperature for Early Spring was assumed to be equal to the Late Spring mean air temperature.

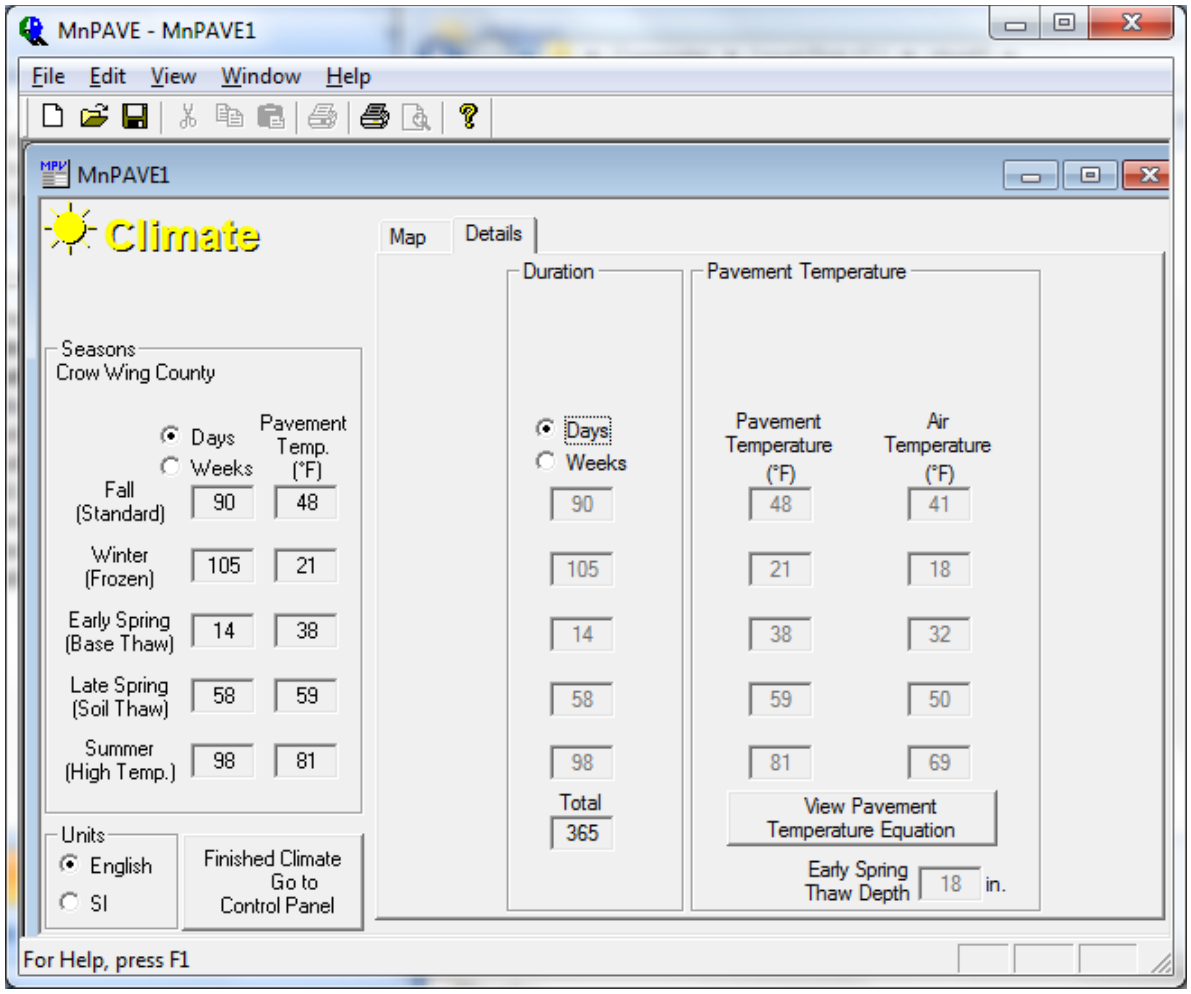


Figure 2. MnPAVE climate window.

3. BACKCALCULATION

FWD backcalculation is a method for analysis of deflection data which involves determination of the elastic properties of the pavement system. The backcalculation process involves selection of the pavement model capable of generating a deflection profile at the pavement surface (forward analysis) and search for the model parameters that results in the generated deflection profile closely matching the measured deflection basis.

For the purpose of this study, the following requirements were identified for the backcalculation procedure:

- Since the layered elastic model is the widely accepted model for structural analysis of flexible pavements in mechanistic-empirical design including MnPAVE, the backcalulation model should be based on layered elastic theory.
- The layered system should include four layers: AC surface, base, subgrade, and a very stiff layer. The first three layers can be of various thicknesses and the last layer is semi-infinite.
- The backcalculation process should not require user-defined seed values.
- The apparent depth to the stiff layer should be either an input parameter provided by the user or should be determined in the process of backcalculation.
- The procedure should permit seamless communication with the rest of the bearing capacity evaluation procedure.

Several publicly available backcalculation procedures were evaluated in this study. However, none of them was found to satisfy all of the above criteria. Therefore, a simple backcalculation procedure was developed in this study. The procedure utilizes a database of pre-calculated deflection basins from the layered elastic analysis for a 10,960-lb load uniformly distributed over a circular area with a radius of 5.9 in. The vertical deflections were computed at the top surface, at lateral positions of 0, 8, 12, 18, 24, and 36 inched away from the center of the load. The following structural systems were considered:

- Top layer (representing the AC layer)
  - Modulus of elasticity, E_1, of 100, 200, 300, 400, 600, 800, 1200, 1500, 2000, 3000, and 4000 ksi
  - Thickness, h_1, of 2, 3, 4, 5, 6, 8, 10, and 12 in
- Second layer (representing the granular base)
  - Modulus of elasticity, E_2, of 10, 20, 30, 40, 50, 60, 80, 100, and 999 ksi
  - Thickness, h_2: 3, 6, 9, 12, 18, 24, 36, and 48 in
- Third layer
  - Modulus of elasticity, E_3, of: 10 ksi
  - Thickness, h_3, of 12, 24, 36, 48, 60, 120, 180, and 240 in
- Fourth layer
  - Modulus of elasticity, E_4, of: 1,000 ksi
  - Thickness: semi-infinite

The developed procedure has the following limitation:

- AC layer thickness is greater than 2 in and less than 12 in

- Base thickness is greater than 3 in and less than 48 in
- Base modulus of elasticity is always not less than the subgrade modulus of elasticity, but is not greater than 100 times
- The AC modulus of elasticity is not less than one-tenth of the base modulus and not greater than 400 times of the base modulus.
- The subgrade is at least 12 in thick, but not thicker than 240 in.

These limitation do not prevent analysis of the majority of practical problems to be addressed using the procedure. The backcalculation process involves the following steps:

Step 1. Select layer thicknesses

The user should provide thicknesses of the AC layer, base, and subgrade. If the pavement structure has more than 3 layers, some layers should be combined. The following procedure should be followed:

- All asphalt bound layers should be considered to be a single AC layer. It is important to note that if the input AC layer thickness is less than 2 in then the program will use an AC thickness of 2 inches in the analysis. If the input thickness is greater than 12 in then the program will use 12 inches in the analysis. In this case, a lower than actual AC stiffness will be backcalculated; however, it should be noted that backcalculation for such thick pavements may require a special procedure.
- Unbond layers should be classified as part of either the base or subgrade layer. A combined thickness of all layers classified as part of the base should be provided. The total base thickness should be less than 48 inches. The minimum base thickness is 3 in. If the base thickness is unknown, it is recommended to use a base thickness of 12 in.
- The user should provide the subgrade depth which is defined as the distance from the bottom of the base layer to the top of the apparent stiff layer. If the user provides a subgrade thickness greater than 12 in then the analysis will be conducted using a subgrade thickness which is the closest to the input thickness among the following values: 12, 24, 36, 48, 60, 120, 180, and 240 in. If the user provides a subgrade thickness less than 12 inches, this indicates that the backcalculation process should be performed for each of these thicknesses.

Step 2. Interpolate deflection database for the AC and base thickness.

Using bi-cubic spline interpolation, interpolate the deflection database for the input AC and base thicknesses. After that, determine the bi-cubic spline coefficients for the interpolation coefficients for determination of the deflections for the intermediate values of E_1 and E_2.

Step 3. Minimize error function

Using on-grid optimization, find elastic moduli E_1 and E_2 that minimize the following error function:

$$ERR = \sum_{1}^{6} \left(\frac{D_{meas}(r_i)}{D0_{meas}} - \frac{D_{calc}(r_i)}{D0_{calc}} \right)^2 \quad (10)$$

where Dr_{meas} and Dr_{calc} are FWD measured and FWD calculated deflections at lateral distances (0, 8, 12, 18, 24, or 36 in) away from the center of the load, respectively; $D0_{meas}$ and $D0_{calc}$ are measured and calculated deflections under the center of the load, respectively.

It should be noted that due to the limitation of the deflection profile database, the optimization is performed only for the range of E_1 values from 100 to 4,000 ksi and for E_2 values from 10 to 999 ksi. In some cases the values of the elastic moduli E_1 and E_2 that minimize the error function are the lower or upper bounds of the deflection profile database. In such cases, there is no guarantee that a good match between the FWD measured and simulated deflections is achieved. There are many reasons that can cause this problem. Local fluctuation of the layer thickness or material quality, measurement error, and presence of distresses can be mentioned among others.

To warn user about a potential backcalculation problem, the program reports the minimization completion codes provided in Table 1. The code is reported for each deflection basin.

Table 1. Backcalculation completion codes.

Code	Legend
0	successful completion
1	E_2 has reached the lower bound
2	E_2 has reached the upper bound
10	E_1 has reached the lower bound
11	E_1 and E_2 have reached the lower bounds
12	E_1 has reached the lower bound and E2 has reached the upper bound
20	E_1 has reached the upper bound
21	E_1 has reached the upper bound and E2 has reached the lower bound
22	E_1 has reached the upper bound and E2 has reached the lower bound

Step 4. Determine pavement system moduli

Equation 11 ensures minimization of the discrepancy between the measured and calculated FWD deflection basins normalized to their respective maximum deflections. It is important to note that the calculated FWD deflections are determined for the FWD load, P_0, of 10,960 lb and the subgrade modulus, E_3, of 10,000 psi. However, since layered elastic analysis is used for deflection calculation, the following relationship is valid for deflections computed for an elastic system with the same layer thickness, but different elastic moduli and FWD load magnitude:

$$D\left(E_{ac},E_{base},E_{subgr},P,r\right)=\frac{P}{P_0}\frac{E_3}{E_{subgr}}D\left(E_1,E_2,E_3,P_0,r\right)\ \ if\ \ \frac{E_{ac}}{E_{base}}=\frac{E_1}{E_2}\ and\ \frac{E_{base}}{E_{subgr}}=\frac{E_2}{E_3} \qquad (11)$$

where E_{ac}, E_{base}, and E_{subgr} are elastic moduli of the top (AC), second (base), and third (subgrade) moduli of elasticity, P is the FWD load.

Analysis of equation (11) leads to the conclusion that any combination of the modili E_{ac}, E_{base}, and E_{subgr} that satisfy the conditions

$$\frac{E_{ac}}{E_{base}}=\frac{E_1}{E_2}\ and\ \frac{E_{base}}{E_{subgr}}=\frac{E_2}{E_3} \qquad (12)$$

have the same value of the error function defined by equation (10) which minimized the discrepancy between the load normalized measured and calculated deflections. Therefore, the total difference between the actual measured and load normalized deflections can be minimized by adjustment of the subgrade modulus and keeping the ratios between the moduli constant.

The discrepancy between measured and calculated deflections can be defined as

$$ERR2=\sum_1^6\left(D_{meas}(r_i)-\frac{P}{P_0}\frac{E_3}{E_{subgr}}D\left(E_1,E_2,E_3,P_0,r_i\right)\right)^2 \qquad (13)$$

To minimize function ERR2, the subgrade modulus should satisfy the following condition:

$$\frac{\partial\left(\sum_1^6\left(D_{meas}(r_i)-\frac{P}{P_0}\frac{E_3}{E_{subgr}}D\left(E_1,E_2,E_3,P_0,r_i\right)\right)^2\right)}{\partial E_{subgr}}=0 \qquad (14)$$

This leads to the following expression for E_{subgr}

$$E_{subgr}=\frac{P}{P_0}E_3\frac{\sum_1^6\left(D\left(E_1,E_2,E_3,P_0,r_i\right)\right)^2}{\sum_1^6\left(D\left(E_1,E_2,E_3,P_0,r\right)D_{meas}(r_i)\right)} \qquad (15)$$

Since the conditions in equation 11 should be satisfied, the asphalt and base modulus can be found from the following equations:

$$E_{ac} = \frac{P}{P_0} E_1 \frac{\sum_1^6 \left(D\left(E_1, E_2, E_3, P_0, r_i\right)\right)^2}{\sum_1^6 \left(D\left(E_1, E_2, E_3, P_0, r\right) D_{meas}(r_i)\right)} \quad (16)$$

$$E_{sbase} = \frac{P}{P_0} E_2 \frac{\sum_1^6 \left(D\left(E_1, E_2, E_3, P_0, r_i\right)\right)^2}{\sum_1^6 \left(D\left(E_1, E_2, E_3, P_0, r\right) D_{meas}(r_i)\right)} \quad (17)$$

Step 5. Conduct final error check and report the completion code

The backcalculation process described above involves the use of interpolated deflection basins. To ensure a more realistic evaluation of the degree of discrepancy between the measured and generated deflection basins, the MnLAYER layered elastic program is used to simulate the deflection basin with the backcalculated moduli. The discrepancy parameter, defined by equation 13, is computed and reported.

Example

Consider the deflection basin collected on Meeker County State Aid Highway CSAH 18 summarized in Table 2. The pavement has an 8-in thick AC layer. The base was assumed to be 12-in thick. The backcalculation procedure was performed for various depths of the apparent stiff layer.

Table 2. An example of a FWD profile collected on Meeker CSAH 18.

FWD	FWD Sensor Deflection, in					
Load, lb	0 in	8 in	12 in	18 in	24 in	36 in
6185	0.00872	0.00756	0.00674	0.00657	0.00449	0.00288

Table 3 presents backcalculated layer moduli, a completion code, and a discrepancy parameter for each subgrade thickness used in the backcalculation. Figure 3 presents a comparison of the measured FWD deflections and MnLAYER deflections calculated using the subgrade thicknesses of 12, 36, 60, and 120 in with the corresponding backcalculated layer moduli.

Analysis of Table 3 shows that backcalculation for subgrade thicknesses of 12, 24, 36, and 48 in resulted in the elastic moduli E_1 or E_2 having the values of either the lower or upper bounds of the elastic moduli in the deflection profile database. This resulted in a significant discrepancy between the measured and computed deflection profiles as indicated by the last column of Table 3. Backcalculation for thicknesses of 60, 120, 180, and 240 inches resulted in low discrepancies between the measured and computed deflection basins. The subgrade thickness of 120 in led to the best match between the measured and computed deflections. In Table 3, note the fairly large moduli shifts between the 60 and 120 inch, both results with similar errors.

Table 3. Summary of backcalculated parameters for the example of a FWD profile collected on Meeker CSAH 18.

Subgrade Thickness, in	E_{AC}, psi	E_{base}, psi	$E_{subgrade}$, psi	Completion Code	ERR2 (Eq 13)
12	1752452	4381.13	4381.13	21	0.354838
24	2232153	5580.38	5580.38	21	0.143143
36	1621991	40549.78	4054.98	22	0.024848
48	1682740	48409.71	4840.97	2	0.001774
60	1457632	55629.68	5786.6	0	0.000069
120	1729130	33428.22	10411.1	0	0.000064
180	1762011	27190.23	12464.81	0	0.000196
240	1897168	21631.24	13871.01	0	0.000342

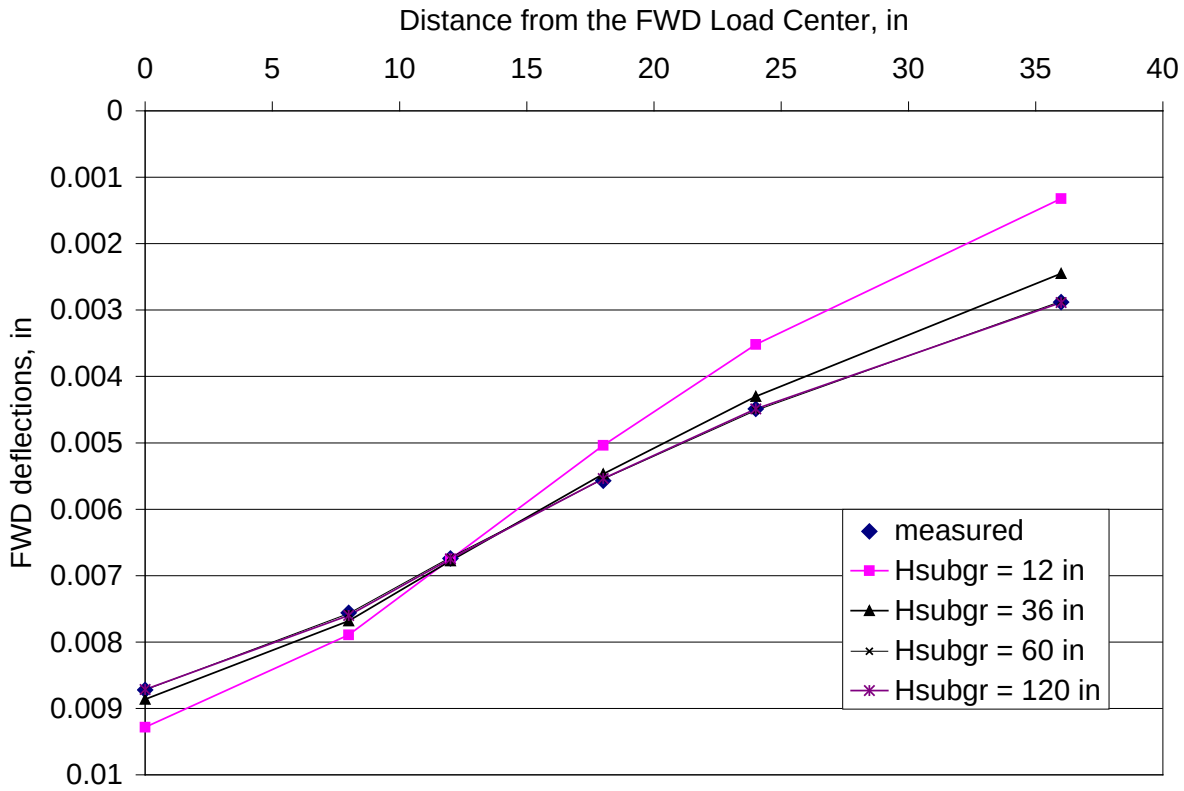


Figure 3. Comparison of measured and calculated deflections.

Figure 3 also shows that the deflection basins computed for the subgrade depth of 12 and 36 in significantly deviate from the measured deflection basin. At the same time, the deflection basins computed for the subgrade depth of 60 and 120 in are almost identical and match the measured deflection basin. It is important to note that computation of the deflection basin in this error checking step is independent from the process of forward calculation in the backcalculation step. Although both routines are based on layered elastic analysis and utilize the same structural model, the backcalculation process utilized interpolation of the pre-computed deflection basins

whereas the deflection basins presented in figure 3 are generated by MnLAYER. This confirms the robustness of the backcalculation routine.

4. TEMPERATURE AND SEASONAL ADJUSTMENT OF BACKCALCULATED MODULI

The backcalculation procedure described above determines the layer moduli which represent the pavement system only for the environmental conditions at the time of testing. However, the environmental conditions vary throughout the year. The asphalt modulus is temperature dependant whereas moisture conditions affect the base and subgrade moduli. To account for these effects in mechanistic-empirical analysis, the backcalculated moduli should be adjusted.

The following procedure was adopted for adjustment of the backcalculated AC modulus. First, the AC temperature at a depth of a 1/3 of the AC layer thickness at the time of FWD testing is determined using BELLS3 procedure (Lukanen et al 1998). BELLS3 uses the pavement surface temperature measured by the FWD, the previous day's average air temperature, thickness of the asphalt, and time of the test to estimate the third-depth temperature. The mean air temperature can be easily obtained from meteorological sources. For example, historical air temperature data for various Minnesota locations can be found on the Minnesota Climatology Working Group website (http://climate.umn.edu/). The BELLS3 model has the following form:

$$T(z) = 2.78 + 0.912 * T_s + (\log(z/3 - 1.25)\{-0.448 * T_s + 0.553 * T_{air,1\text{-}day} + 2.63 * \sin(hr_{18} - 15.5)\} + 0.027 * T_s * \sin(hr_{18} - 13.5) \quad (18)$$

where

$T(z)$	=	pavement temperature at depth z, ^oC
z	=	depth at which material temperature is to be predicted, mm
T_s	=	infrared surface temperature, ^oC
$T_{air,1\text{-}day}$	=	average air temperature the day before testing, ^oC
hr_{18}	=	Time of day, in a 24-hr clock system, but calculated using an 18-hr asphalt concrete (AC) temperature rise-and-fall time cycle
log	=	Base 10 logarithm

After the AC temperature at 1/3 of the AC layer thickness depth is determined, the modulus for the reference temperature of 22^{o}C (~72^{o}F) is computed using the equation developed by Lukanen et al, (1998) from the analysis of the Long Term Pavement Performance (LTPP) Seasonal Monitoring Program (SMP) data:

$$E_{ref} = E_{test} \times 10^{slope(T_{ref} - T_{test})} \quad (19)$$

The magnitude of the slope in the equation above depends on the individual characteristics of the mix such as the binder properties and aggregate characteristics. The range encountered in the LTPP SMP study for the slope was roughly bounded by -0.015 to -0.030. In this study, a slope value of -0.020 was adopted, though further study of this parameter using existing data for Minnesota is recommended.

After the AC modulus for the reference temperature is determined, representative seasonal AC moduli values are determined. The MnPAVE procedure for determining the average seasonal pavement temperature was adopted. The following equation is used:

$$Tp_i = Ta_i\left(1+\frac{1}{z+4}\right)-\frac{34}{z+4}+6 \quad (20)$$

where

T_{pi} = average seasonal pavement temperature at depth z for season i (^oF)
T_{ai} = average seasonal air temperature for season i (^oF)
z = depth at which material temperature is to be predicted, in.

The average seasonal air temperature for any Minnesota location can be found from the MnPAVE design software "climate" screen.

After the seasonal pavement temperatures are determined, the corresponding AC moduli are determined using equation 19.

The elastic properties of unbound materials are moisture dependent. Since moisture conditions vary from season to season, the backcalculated base and subgrade moduli are adjusted using the following equations:

$$E_{base,i} = E_{base} * \frac{bs_i}{b_{day}} \quad (21)$$

$$E_{subgr,i} = E_{subgr} * \frac{ss_i}{s_{day}} \quad (22)$$

where

E_{base}	=	backcalculated base modulus
$E_{base,i}$	=	average base modulus for season i
bs_i	=	base modulus season adjustment factor for season i
b_{day}	=	base modulus adjustment factor accounting for a difference in the moisture conditions for the test day; by default it is equal to 1
E_{subgr}	=	backcalculated subgrade modulus
$E_{subgr,i}$	=	average subgrade modulus for season i
ss_i	=	subgrade modulus season adjustment factor for season i
s_{day}	=	subgrade modulus adjustment factor accounting for a difference in the moisture conditions for the test day; by default it is equal to 1

If no other information is available, the MnPAVE seasonal adjustment factors should be used.

5. DETERMINATION OF STRUCTURAL RESPONSES

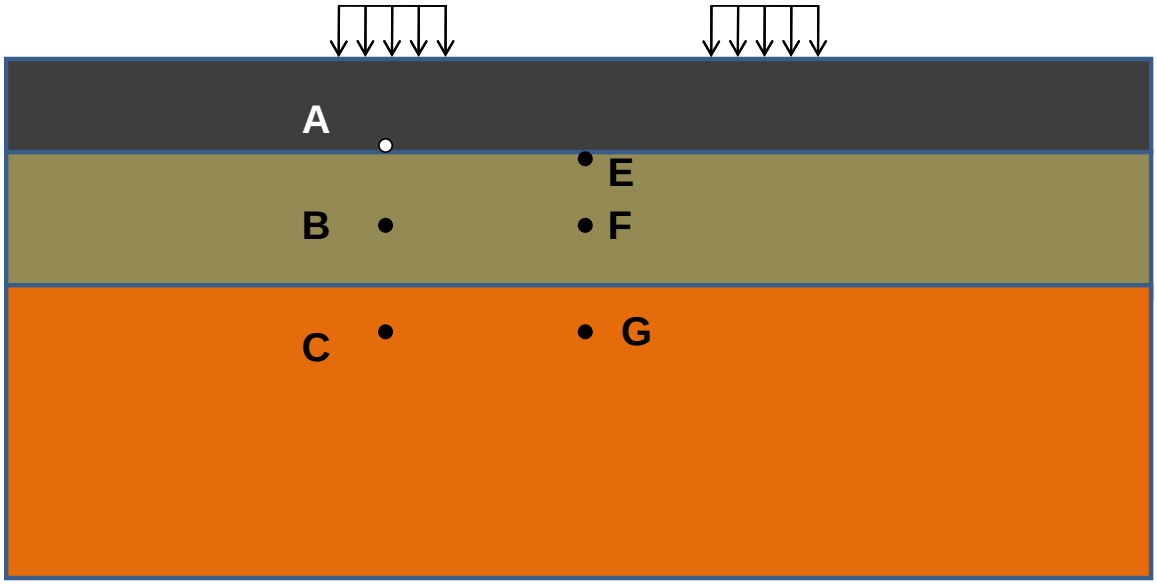
After the backcalculated layer moduli are adjusted to account for the seasonal effects, the critical pavement responses (strains and deflections) are computed using the layered elastic program MnLAYER (Khazanovich and Wang 2010) embedded into TONN2010. The responses are computed for five seasons. The pavement is assumed to be loaded by an 18,000 lbs single axle load. Only half of an axle (two wheels) is considered. Each tire footprint is assumed to have a radius of 3.8 in and the tire pressure is assumed to be equal to 100 psi. The wheels are assumed to be placed 13.5 in apart. The following evaluation points are used depending on the thickness of the base layer (see Figure 4):

If base layer thickness is less than or equal to 12 inches (Figure 4a), six evaluation points are considered

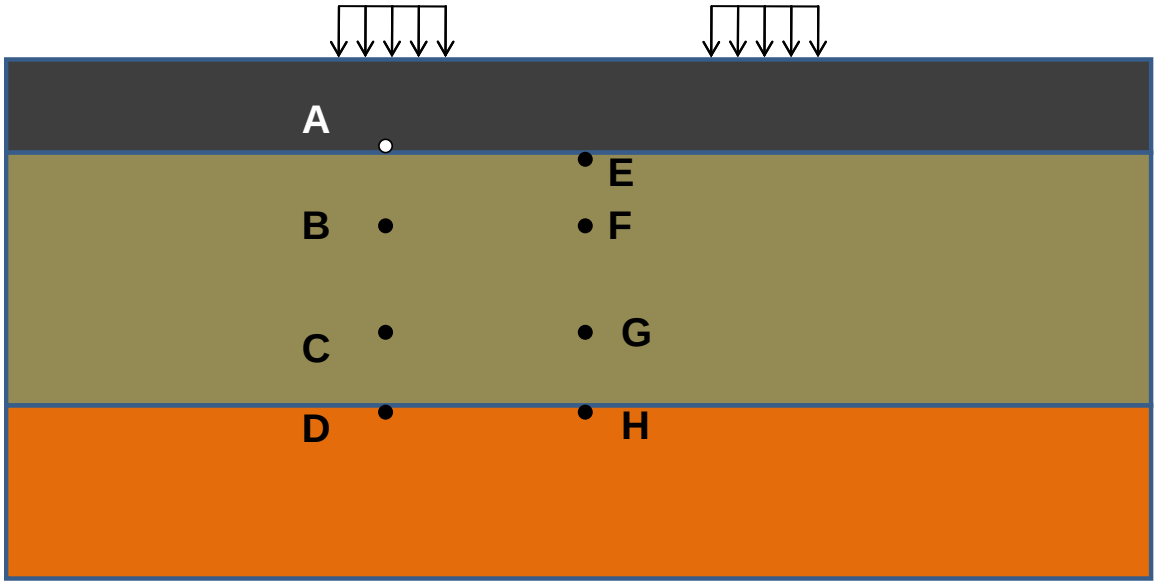
- Point A. Bottom of the AC layer, under the center of the wheel.
- Point B. 6 inches below the top of the base layer, under the center of the wheel
- Point C. 12 inches below the top of the base layer, under the center of the wheel
- Point E. Top of the base layer, mid-distance between the wheels
- Point F. 6 inches below the top of the base layer, mid-distance between the wheels
- Point G. 12 inches below the top of the base layer, mid-distance between the wheels

If base layer thickness is greater than 12 inches (Figure 4a), six evaluation points are considered

- Point A. Bottom of the AC layer, under the center of the wheel.
- Point B. Mid-depth of the base layer, under the center of the wheel
- Point C. 12 inches below the top of the base layer, under the center of the wheel
- Point D. Top of the subgrade, under the center of the wheel
- Point E. Top of the base layer, mid-distance between the wheels
- Point F. Mid-depth of the base layer, mid-distance between the wheels
- Point G. 12 inches below the top of the base layer, mid-distance between the wheels
- Point H. Top of the subgrade, mid-distance between the wheels



(a) base layer thickness less than or equal to 12 inches



(b) base layer thickness greater than 12 inches

Figure 4. Location of evaluation points in the structural model.

The maximum principal horizontal strain computed at point A is used in the subsequent AC damage calculation. The maximum vertical strains computed at points C and G (or D and H if the base layer exceeds 12 inches in thickness) are needed for subgrade rutting damage analysis. Stresses computed at points B and F are used to compute the principle and critical stresses as defined by equation 3. These stresses are used to compute the strength to stress ratios. The highest strength to stress ratio, SR_c, is used in the subsequent analysis as defined in equation 23.

$$SR_c = \max\left(\frac{\sigma_{B,1}}{\sigma_{B,1_{critical}}}, \frac{\sigma_{F,1}}{\sigma_{F,1_{critical}}} \right) \tag{23}$$

where

$\sigma_{B,1}$ and $\sigma_{F,1}$ are first principal stresses at points B and F, respectively.

$\sigma_{B,1_{critical}}$ and $\sigma_{F,1_{critical}}$ are critical stresses computed for points B and B, respectively.

Finally, the vertical displacement at point C is subtracted from the vertical displacement at point A and the vertical displacement at point G is subtracted from the vertical displacement at point E. The maximum of these two differences, DW, is be used in the subsequent analysis.

$$DW = \max\left(w_A - w_C, w_E - w_G\right) \tag{24}$$

where w_A, w_C, w_E, and w_G are vertical displacements at points A, C, E, and G, respectively.

6. DAMAGE ANALYSIS

After the critical responses are determined for each season, the damage analysis is performed. It involves:

- AC fatigue cracking damage analysis
- Subgrade rutting damage analysis
- Base shear failure analysis
- Base deformation analysis

The anticipated AC fatigue damage, DAM_{AC}, over the design life is determined using the following equation:

$$DAM_{AC} = \frac{CESAL}{365}\sum_{i=1}^{5}\frac{DAYS_i}{0.314\varepsilon_{A,i}^{-3.291}E_{ACi}^{-0.854}} \quad (25)$$

where

CESAL = anticipated cumulative number of ESALs over the pavement design life
$DAYS_i$ = duration of season i, days.
$\varepsilon_{A,i}$ = principal horizontal strain at point A for season i combinations of elastic properties
E_{ACi} = AC elastic modulus for season i.

The anticipated subgrade rutting damage, DAM_{rut}, over the design life is determined using the following equation:

$$DAM_{RUT} = \frac{CESAL}{365}\sum_{i=1}^{5}\frac{DAYS_i}{0.0261\varepsilon_{B,i}^{-2.35}} \quad (26)$$

where

CESAL = anticipated cumulative number of ESALs over the pavement design life
$DAYS_i$ = duration of season i, days.
$\varepsilon_{B,i}$ = vertical strain at point B for season i combinations of elastic properties

Analysis of equations 25 and 26 shows that for a given structure an increase in anticipated traffic leads to an increase in the anticipate damage. Also, if the anticipated damage exceeds that of the critical fatigue and cracking damage values then the pavement structure should be considered inadequate in fatigue or rutting, respectively. In MnPAVE analysis these critical values are equal to 1. However, since the backcalculated FWD values are not necessarily equal to elastic properties used in the MnPAVE calibration, these critical damage values can be different and should be determined through calibration, which will be presented below.

If the anticipated damage is equal to or less than the critical damage at the end of the design life then the pavement should be rated as a 10-tonn pavement. If the anticipated damage is greater than this critical damage this means that either the pavement would require a rehabilitation before the end of the design period or axle weights on the pavements should be restricted, especially during the seasons when the damage is the greatest. The following relationships are proposed to translate the anticipated damage of pavements into TONN indices:

$$TONN_{AC} = C_{AC}\, DAM_{AC}^{-0.25} \quad (27)$$

$$TONN_{rut} = C_{RTU}\, DAM_{rut}^{-0.25} \quad (28)$$

where

$TONN_{AC}$ = TONN index based on AC damage

$TONN_{RUT}$ = TONN index based on subgrade rutting

C_{AC} and C_{RTU} = calibration coefficients

If the stresses computed from an 18 kip single axle load result in a strength to stress ratio equal to 1 then the road should be classified as 9 TONN, with respect to base shear failure. Similarly for other strength to stress ratios, the TONN index can be defined as

$$TONN_{SF} = 9\left(\max_i\left(SR_i\right)\right) \quad (29)$$

where

$TONN_{SF}$ = shear failure TONN index based on the strength to stress ratio

SR_i = strength to stress ratio computed for point i

i = point (C or D) in the structural system

Finally, a higher difference in deflections at the base indicates a higher probability that the pavement will fail prematurely in compression. The following expression for the TONN base compression index was developed for the deflection differences:

$$TONN_{BD} = \frac{C_{BD}}{\max_i\left(DW_i\right)} \quad (30)$$

where

$TONN_{BD}$ = TONN index based on differential deflections in the base layer

C_{BD} = calibration parameter relating allowable axle load to difference in vertical base deflections

DW_i = difference in the vertical deflections computed for a pair of points i, in

i = a pair of points in the structural model (either A and E or F and G)

The overall $TONN_{2010}$ rating for the road is determined as the minimum of the four individual TONN ratings

$$TONN_{2010} = \min\left(TONN_{AC}, TONN_{RUT}, TONN_{SF}, TONN_{BD}\right) \quad (31)$$

The TONN2010 procedure has been encoded into a FORTRAN program that is executed with a user created input file containing information about the pavement system, climatic data, etc. as well as the FWD deflection data.

7. CALIBRATION OF THE TONN2010 PROCEDURE

To finalize the TONN2010 procedure it is necessary to assign values to the calibration constants C_{AC}, C_{RUT}, and C_{DW}. In this study, these parameters were determined by an analysis of two MnROAD Farm Loop pavement sections: Cell 83 and Cell 84.

Cell 84 was designed to represent a 10-tonn road. It has a 5.5-in AC layer placed on a 9-in thick granular (Class 5) base. The pavement has an AC shoulder. Cell 83 was design to represent a 7-tonn road. It has a 3.5-in thick AC layer, an 8-in thick granular base, and a gravel shoulder. Both pavement sections have two lanes.

The cells were constructed in October of 2007. Since then they were subjected to a heavy traffic two weeks per year: one week in March and another week in August. The traffic consisted of two MnROAD, 80-kip and 102-kip, trucks and heavy farm equipment. After three years of testing, Cell 84 did not show any appreciable signs of distresses. The westbound lane of Cell 83 failed in the spring of 2009 and eastbound lane of Cell 83 failed in the summer of 2010. The distresses in both lanes included base and subgrade permanent deformation and AC cracking.

The performance data for Cells 83 and 84 indicate that they can be adequately classified as 7-tonn and 10-tonn pavements, respectively. However, since they were subjected to a non-typical traffic mix, which included a relatively small number of heavily overloaded vehicles, the design ESAL traffic for these sections was determined through simulation of performance of these sections using MnPAVE. MnPAVE analysis indicated that Cell 83 could sustain 160,000 ESALs over 20 years and Cell 84 could sustain 600,000 ESALs over 20 years. Rutting damage was found to be critical for both Cells. The expected performance for both pavements in AC fatigue damage is 48 years.

FWD deflection data were collected for both cells on August 22, 2008. The testing was conducted for four load levels: 6, 7.5, 9.5, and 12.5 kips. 172 deflection basins were collected for Cell 83 and 168 deflection basins were collected for Cell 84. In addition to the deflection data, the AC surface temperature measured by the FWD infrared sensor was recorded in the database.

Backcalculation of the FWD deflection basins was performed using the procedure described in section 3. The depth to the apparent stiff layer was assumed to be equal to 240 in. Table 4 summarizes the results of backcalculation.

Table 4. Summary of the results of backcalculation for MnROAD Farm Loop Cells 83 and 84.

	Cell 83			Cell 84		
	$E_{AC,}$ ksi	$E_{base,}$ ksi	$E_{subgr,}$ ksi	$E_{AC,}$ ksi	$E_{base,}$ ksi	$E_{subgr,}$ ksi
Mean	188	12.2	11.7	184	17.3	15.6
Minimum	91	7.5	7.5	139	13.0	12.4
Maximum	520	25.9	15.3	302	32.9	19.9

Analysis of Table 4 shows that the backcalculation for Cells 83 and 84 resulted in a remarkably close mean AC moduli. At the same time, the backcalculated AC moduli for Cell 84 exhibited

lower variability than the backcalculated moduli for Cell 83. A likely explanation of this phenomenon is that the effect of the AC-layer thickness deviation from the as-designed thickness. The as designed thickness was assumed in the backcalculation. Since Cell 83 is thinner, thickness variation has a greater relative effect on the results than the thicker section.

Table 4 also shows that the backcalculated base and subgrade moduli for Cell 83 are lower than the corresponding backcalcuate moduli for Cell 84. This can be explained either by poorer compaction of the base and subgrade in Cell 83 or the effect of damage inflicted in the base and subgrade on Cell 83 during Spring 2008 testing.

Using the results of backcalculation for each deflection basin, damage analysis was performed using the procedure described in Section 6. The seasonal parameters used in analysis are provided in Table 5.

Table 5. Seasonal parameters for the damage analysis.

	Winter	Early Spring	Late Spring	Summer	Fall
Duration, days	100	15	55	105	90
Mean air temperature, ^{o}F	18	50	50	70	41
Base stiffness adjustment factor, bs	10	0.35	0.85	1	1
Subgrade stiffness adjustment factor, ss	10	10	0.7	0.85	1

Figures 5 through 8 present the results of the damage analysis in terms of frequency distributions. Table 6 summarizes the mean values for the computed damage parameters for Cells 83 and 84. As expected, Cell 83 exhibited greater fatigue and rutting damage as well as greater base deflection differences, but a lower stress ratio, SR.

Table 6. Mean damage values for Cells 83 and 64.

	CELL 83	CELL 84
Rutting Damage	0.214	0.102
Fatigue Damage	0.133	0.053
Stress Ratio	1.017	1.243
Base Deformation (in mils)	18.14	11.47

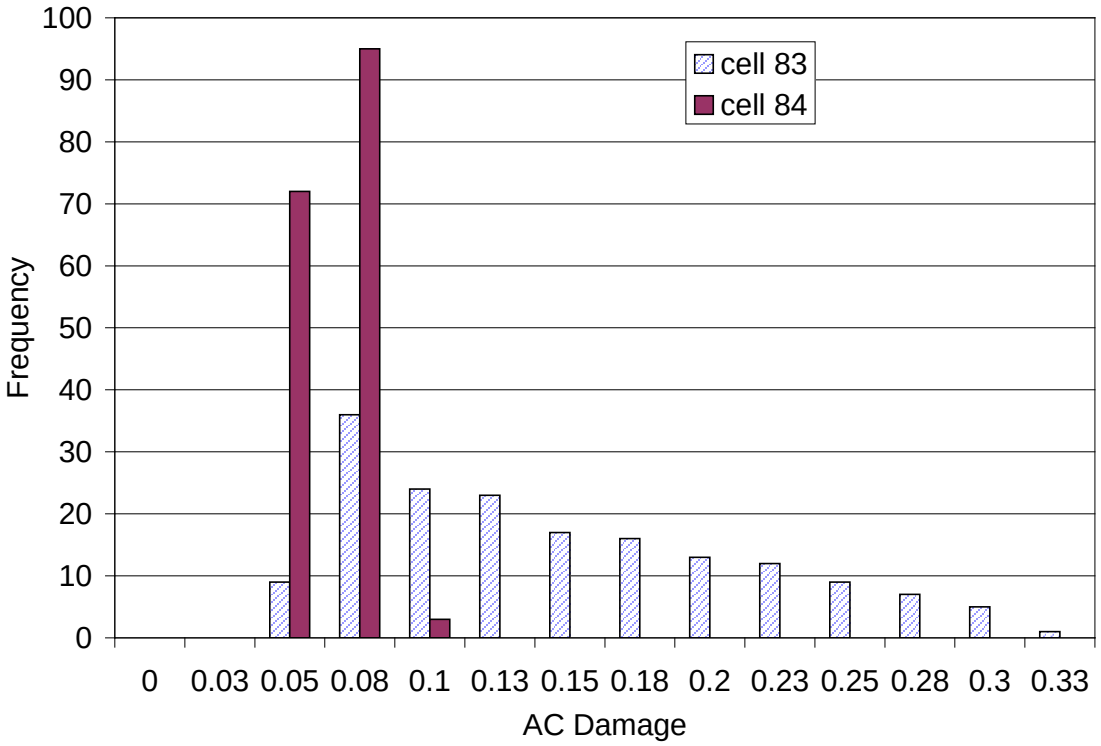


Figure 5. Predicted AC damage for MnROAD farm loop cells.

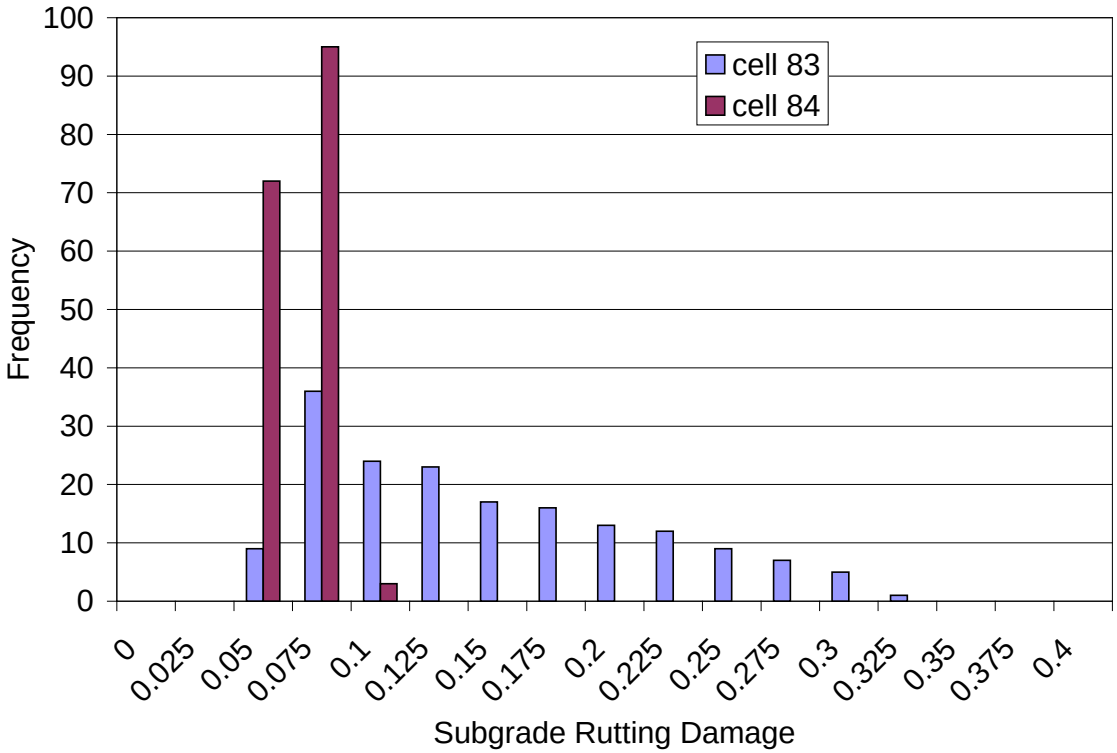


Figure 6. Predicted subgrade rutting damage for MnROAD farm loop cells.

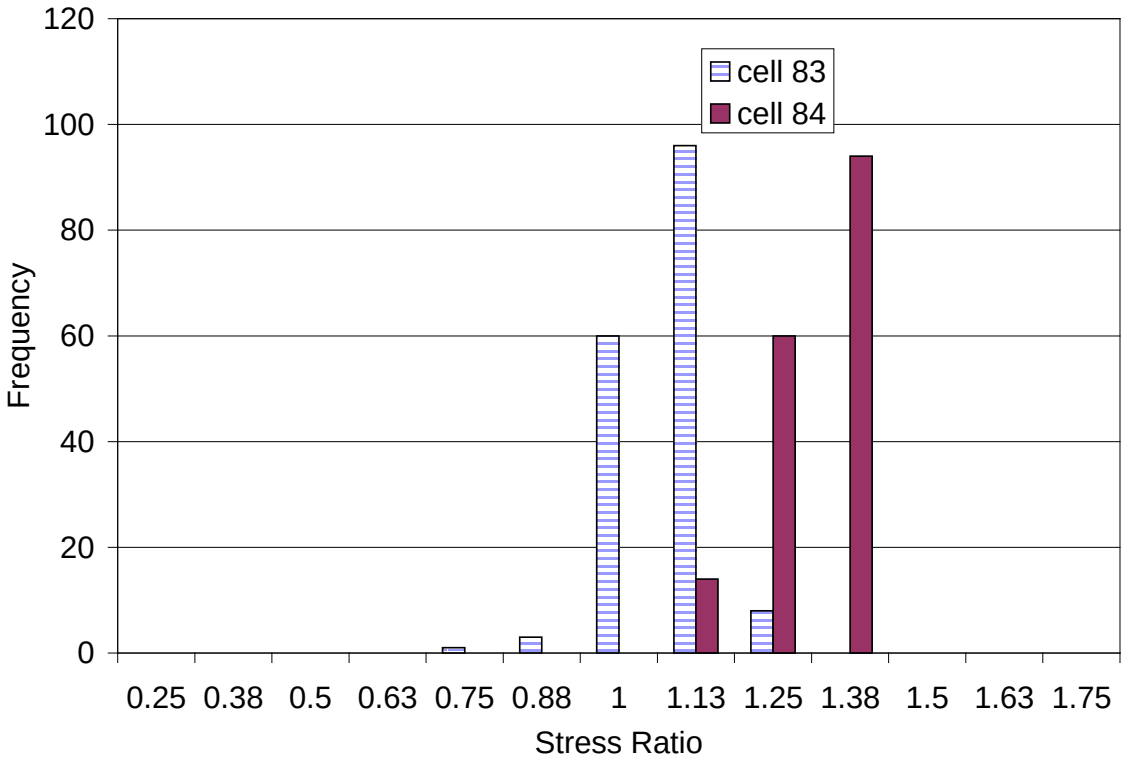


Figure 7. Predicted stress ratios for MnROAD farm loop cells.

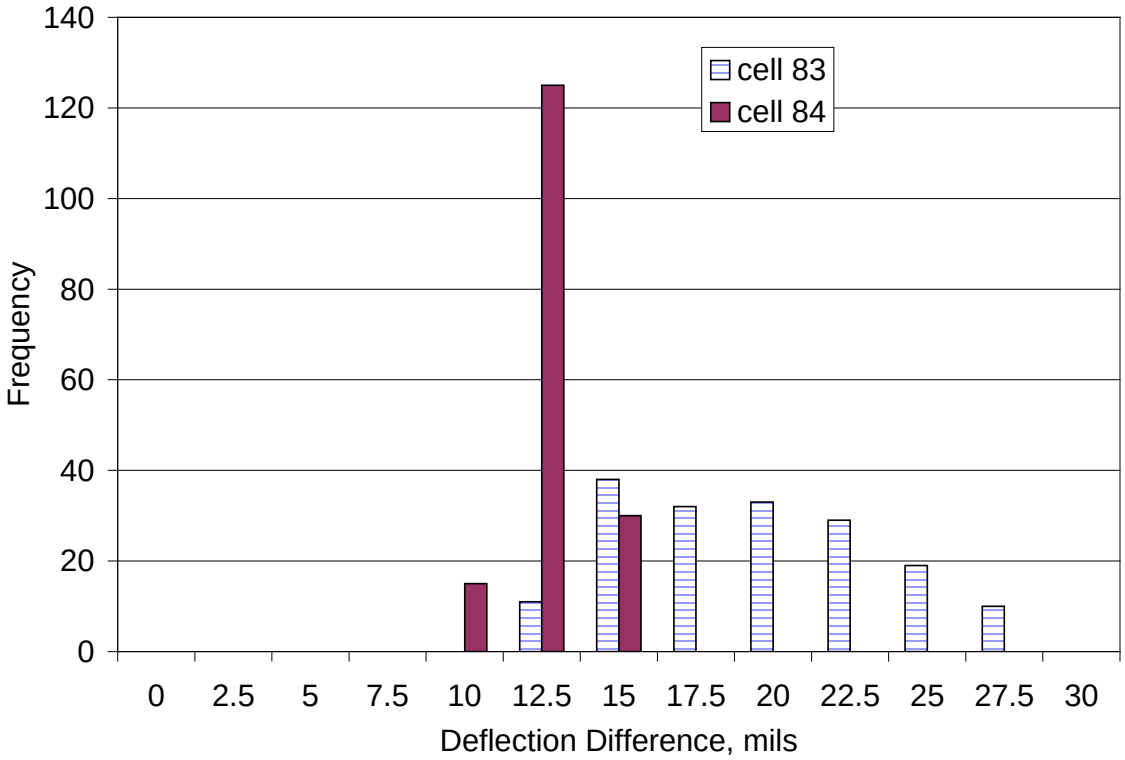


Figure 8. Predicted base deflection differences for MnROAD farm loop cells.

Using the data obtained for Cells 83 and 84, the following calibrated equations were proposed for TONN2010 indices:

$$\begin{aligned} TONN_{AC} &= 5.6\,DAM_{AC}^{-0.25} \\ TONN_{rut} &= 5.6\,DAM_{rut}^{-0.25} \\ TONN_{SF} &= 9\left(\max_i\left(SR_i\right)\right) \\ TONN_{BD} &= \frac{0.115}{DW} \end{aligned} \tag{32}$$

Table 7 present the mean values of the TONN2010 analysis for Cells 83 and 84. One can see that TONN2010 evaluates Cell 83 as a 6.65-tonn road and Cell 84 as a 10-tonn road.

Table 7. Mean damage values for Cells 83 and 84.

	CELL 83	CELL 84
$TONN_{RUT}$	8.5	10.0
$TONN_{AC}$	9.6	11.3
$TONN_{SF}$	9.2	11.2
$TONN_{DW}$	6.65	10.1
$TONN_{2010}$	**6.65**	**10.0**

Table 8 compares the $TONN_{2010}$ with other currently available TONN indexes computed using an MS EXCEL spreadsheet developed by Erland Lukanen from Mn/DOT. The spreadsheet TONN uses the current Investigation 603 TONN method and alternative procedures such as Investigation 183 (INV183), Soil Factor (SF), and AASHTO-93 (AASHTO) based methods. One can observe that TONN2010 agrees with the most conservative, INV183-based TONN index for Cell 83. Considering that Cell 83 failed during both Spring and Fall testing, these indexes appears to be reasonable while other procedures do not lead to sufficiently conservative estimates. On the other hand, for Cell 84, the INV183-based TONN index appears to be overly conservative since the pavement exhibited no damage under heavy axle loading. The AASHTO-93 –based procedure leads to a more reasonable assessment of the section as a 10-ton road. $TONN_{2010}$ matches this evaluation.

Table 8. Mean damage values for Cells 83 and 64.

	CELL 83	CELL 84
$TONN_{2010}$	**6.65**	**10.0**
TONN	7.7	11.4
$TONN_{inv183}$	6.8	8.1
$TONN_{SF}$	9.1	10.5
$TONN_{AASHTO}$	8.2	10.0

The analysis of the MnROAD Farm Loop sections leads to the conclusion that the proposed calibration coefficients are reasonable for assessment of pavement bearing capacity.

8. FURTHER EVALUATION OF THE TONN2010 PROCEDURE

The TONN2010 procedure was utilized for analysis of almost 8400 deflection basins collected in nine (Benton, Clay, Dakota, Houston, Lake, Meeker, Nicollet, Nobles, and Polk) Minnesota counties. The pavement sections have various structures with an AC layer thickness varied from 2.6 to 13.4 inches. The expected traffic for these sections ranged from 30,000 to 5,700,000 ESALs, Table 9 summarizes the test sections evaluated in this study.

Table 9. Test sections used for validation of the procedure.

	County	Section	AC Thickness, in	Design Traffic, 1000 ESALs
1	Clay	CSAH 10 (25-121)	7.6	2101
2	Clay	CSAH 26 (95-100)	13	2103
3	Clay	CSAH 31 (28TH-12)	10.3	835
4	Clay	CSAH 31 (12-80TH)	5.5	771
5	Benton	CSAH 2 (1-W.Lake)	4.1	280
6	Benton	CSAH 2 (W.Lake-21)	4.1	280
7	Benton	CSAH 4 (E.Co.Line-6)	3.6	214
8	Benton	CSAH 4 (23-3)	4.6	438
9	Benton	CSAH 29 (1-10)	4.4	914
10	Dakota	CSAH 23 (Lakeville So.Lmts-Dodd	8	2128
11	Dakota	CSAH 23 (Dodd-Lakeville So.Lmts	7.5	2128
12	Dakota	CSAH 31 (50-64)	6.3	851
13	Dakota	CSAH 31 (64-46)	6.1	2147
14	Dakota	CSAH 31 (46-64)	6.3	2147
15	Dakota	CSAH 46 (Pilot Knob-TH3)	6.2	5711
16	Dakota	CSAH 46 (TH3-Pilot Knob)	6.2	5711
17	Dakota	CSAH 46 (TH3-160th)	4.3	2460
18	Houston	CSAH 4	6.1	130
19	Houston	CSAH 25	2.6	31
20	Houston	CSAH 27	4.4	127
21	Lake	CSAH 2	9.1	517
22	Lake	CSAH 11 (W.Co.Line-12)	5.3	135
23	Lake	CSAH 11 (12-61)	5.3	292
24	Lake	CSAH 12 (11-121)	5.4	187
25	Lake	CSAH 12 (121-2)	5.4	560
26	Meeker	CSAH 1 (28-W.of 180th)	6.4	196
27	Meeker	CSAH 1 (w.of 180th-160th)	6.3	196
28	Meeker	CSAH 1 (160th-TH7)	7.1	131
29	Meeker	CSAH 18 (14-9)	7.5	190
30	Meeker	CSAH 18 (9-22)	4.8	190
31	Nicollet	CSAH 13	13.4	181
32	Nicollet	CSAH 16	8.7	195
33	Nicollet	CSAH 20	9.9	204
34	Nobles	CSAH 17	5	97
35	Nobles	CSAH 25	7.9	631
36	Polk	CSAH 12	6	56
37	Polk	CSAH 14	4.5	65
38	Polk	CSAH 22	9.6	93

Figure 9 shows the results of the TONN2010 analysis for Benton County State Aid Highway (CSAH) 2 from CSAH 1 to W. Lake. One can observe that the individual TONN ratings show a wide spread for individual locations. Subgrade rutting and shear strength ratings range from 8.8 to 16.5, whereas deflection difference and asphalt fatigue ratings were as high as 62 and 33, respectively. It should be noted that for some locations, the deflection difference rating was as

low as 11 and controlled the overall rating, but for other locations either subgrade rutting or stress ration criteria will be the limiting factor.

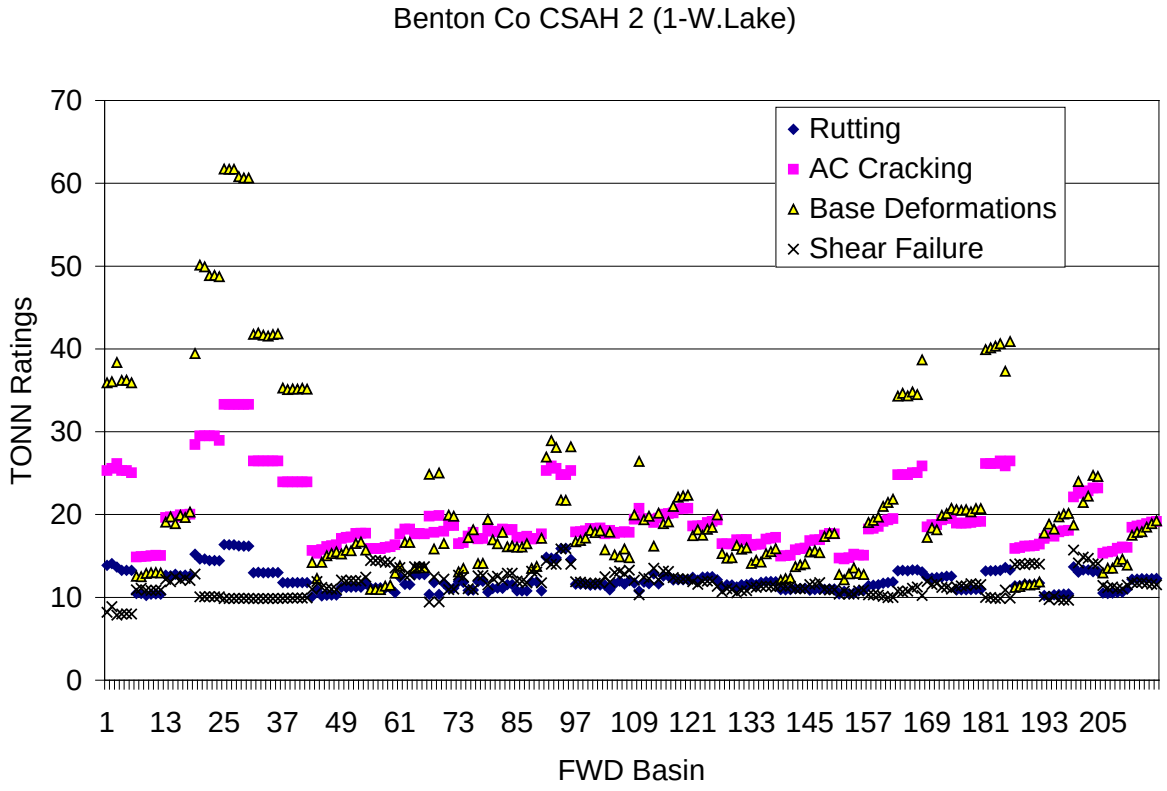


Figure 9. Results of TONN2010 analysis for Benton CSAH 2.

Figure 10 presents a comparison of the $TONN_{2010}$ results with other TONN rating procedures. One can observe that $TONN_{2010}$ agrees reasonable well with the other ratings. $TONN_{2010}$ is not as conservative as the INV 183-based rating, but it is more conservative than the current TONN which overestimates the pavement bearing capacity compared to other rating systems.

Figures 11 through 13 present comparison of $TONN_{2010}$ with other ratings for all the FWD basins evaluated in this study. One can observe from Figure 11 that $TONN_{2010}$ is more conservative than the current TONN for the majority of FWD basins. Nevertheless, for some locations $TONN_{2010}$ results in higher ratings. Similarly, $TONN_{2010}$ is less conservative than AASHTO-93-based TONN rating, but for same basins $TONN_{2010}$ rating is higher. A much closer agreement is observed between $TONN_{2010}$ and the INV183-based rating.

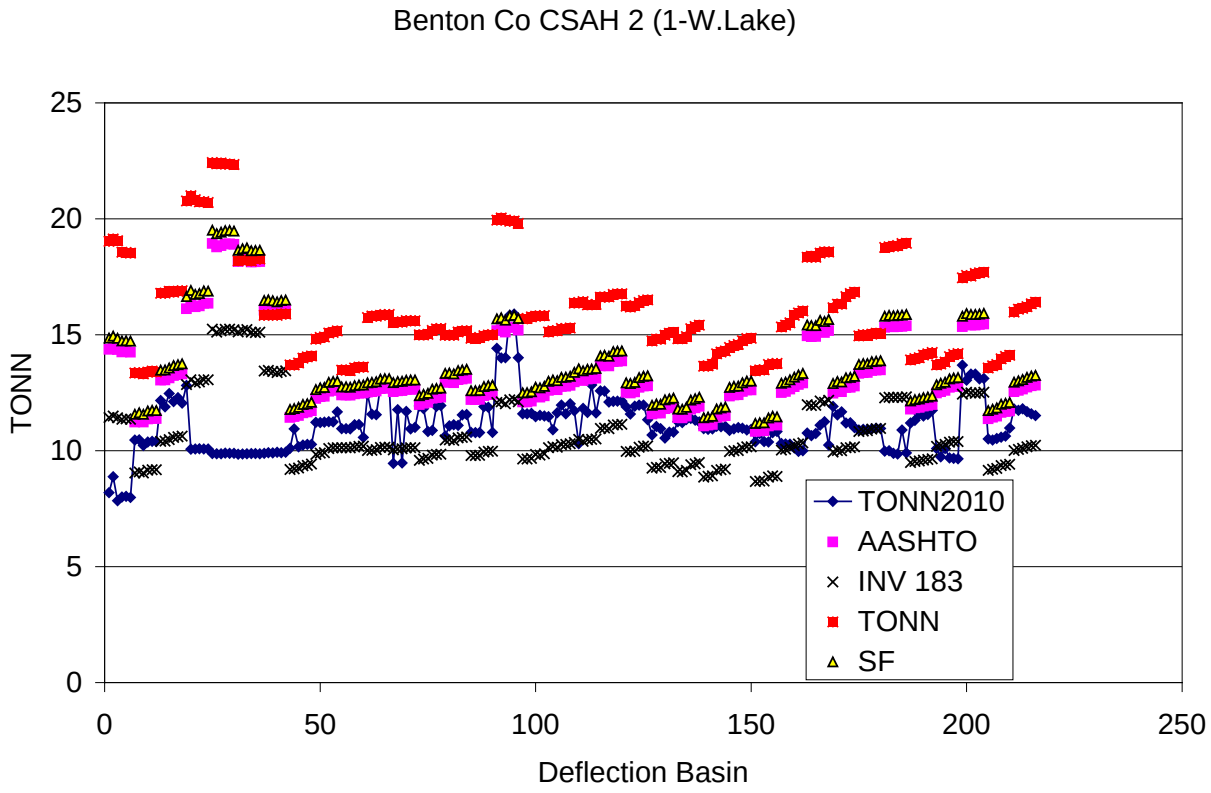


Figure 10. Comparison of various TONN ratings for Benton CSAH 2 section.

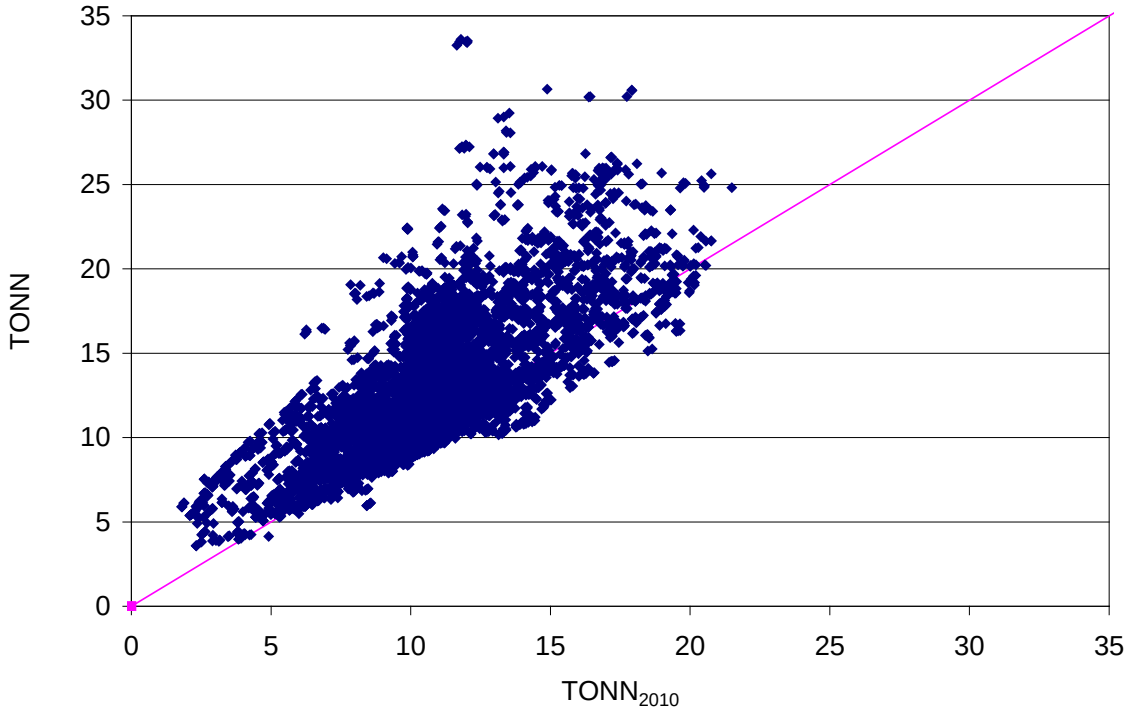


Figure 11. Comparison of $TONN_{2010}$ and current TONN (Inv 631) ratings.

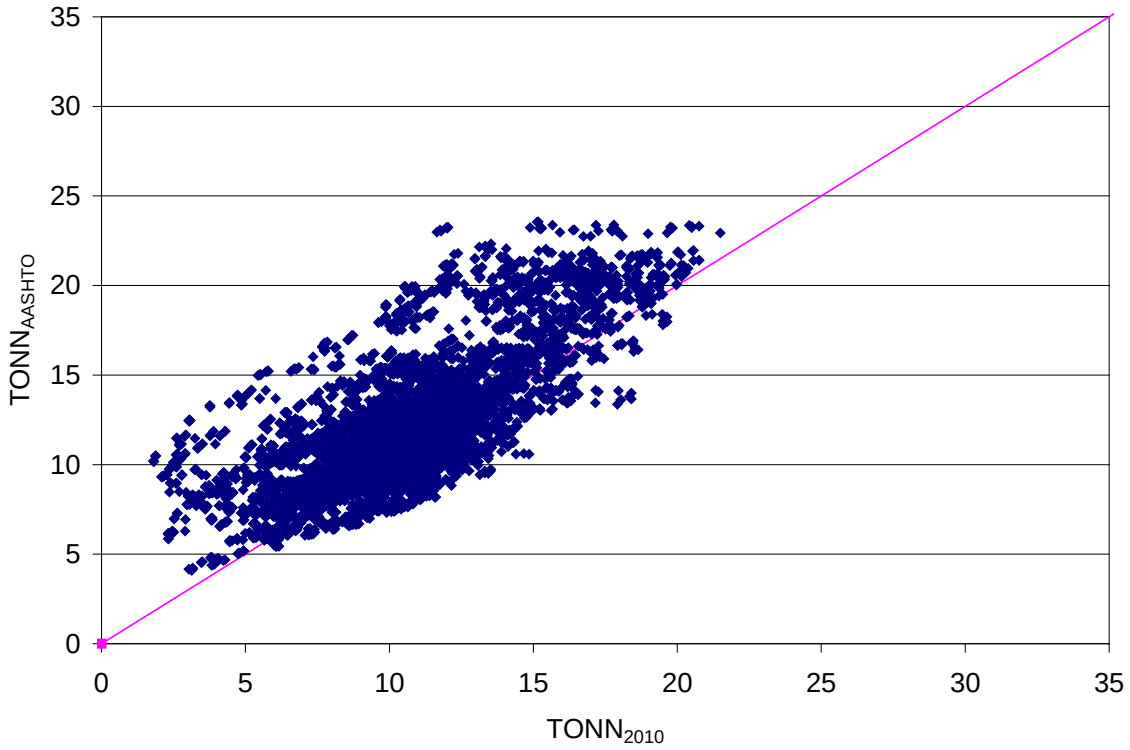


Figure 12. Comparison of TONN$_{2010}$ and AASHTO-93-based TONN ratings.

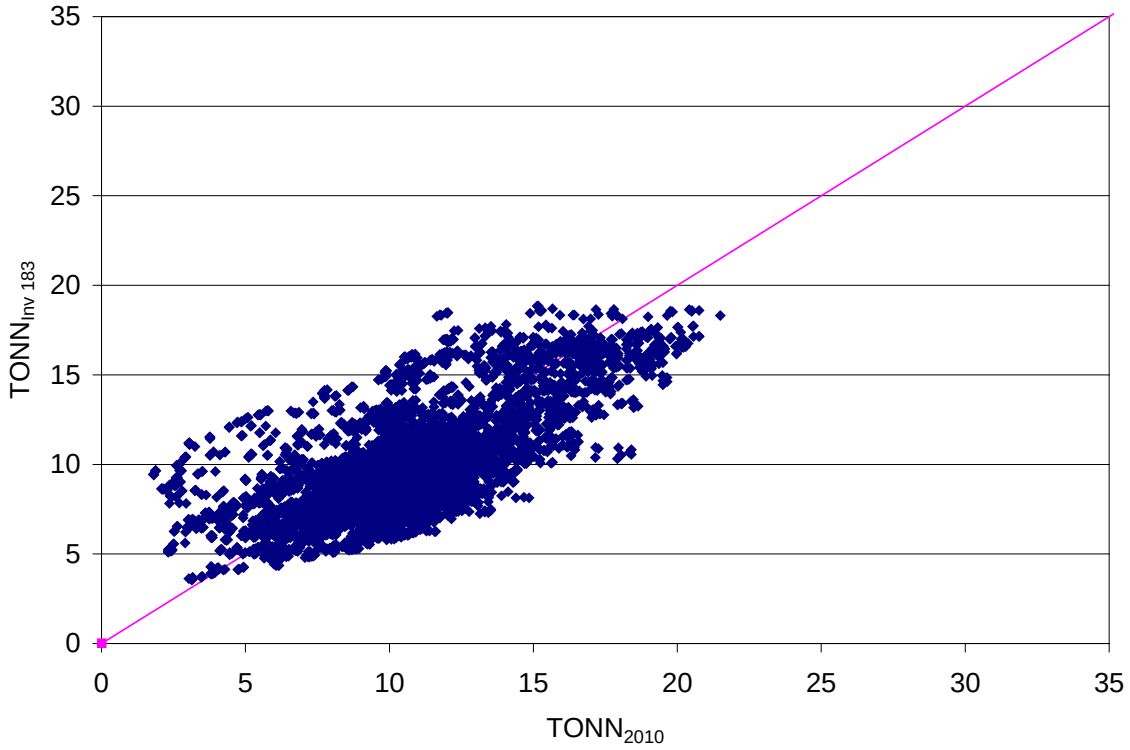


Figure 13. Comparison of TONN$_{2010}$ and AASHTO-93-based TONN ratings.

It can be concluded that reasonable agreement between the $TONN_{2010}$ and other ratings is observed. It should be noted, however, that a perfect correspondence between $TONN_{2010}$ and other ratings was not expected. Each of the other ratings uses only one criterion for pavement evaluation. $TONN_{2010}$ evaluates four different criteria prior to making an assessment. This makes $TONN_{2010}$ an attractive alternative to the current rating procedures.

9. CONCLUSIONS

This report documents research that resulted in development and calibration of a procedure for determination of the structural adequacy of low-volume flexible pavements from FWD deflection measurements. This procedure has been encoded into a computer program, TONN2010. Unlike the current TONN procedure, which utilizes only one criterion, the new procedure is based on four criteria. Three of these criteria, AC fatigue cracking, subgrade rutting, and base shear failure, are adopted from the Mn/DOT mechanistic-empirical design procedure for flexible pavements, MnPAVE. The forth criterion, which limits deflection difference in the base layer, was adopted based on a review of the MEPDG base rutting model.

The procedure has been calibrated using the deflection information from MnROAD pavement sections, and compared to the observed pavement performance. A comprehensive comparison with the current TONN and alternative procedures for the structural capacity evaluation has also been conducted. This evaluation involved an analysis of almost 8400 deflection basins at various Minnesota counties. It has been shown that TONN2010 is an attractive alternative to the currently available procedures.

It is important to conduct a comprehensive verification of the TONN2010 predictions for a wide range of pavement structures and site conditions through a comparison of TONN2010 ratings with actual pavement performance. If necessary, the calibration coefficients of TONN2010 can be adjusted to improve the performance predictions.

As described in Appendix A, TONN2010 is a Fortran program that can be run in the MSDOS environment. However, to enable wide use of this procedure it is important to develop a user friendly interface. Development of such an interface was outside of the scope of this study.

REFERENCES

AASHTO. (1993). *Guide for Design of Pavement Structures*. American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.

AASHTO. (2008). *Mechanistic-Empirical Pavement Design Guide, Interim Edition: A Manual of Practice*. American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.

Ayres, M. (1997). *Development of a Rational Probabilistic Approach for Flexible Pavement Analysis*. Ph.D. dissertation. University of Maryland, College Park, MD.

Brown, S.E, Pell, P.S. and Stock, A.E. (1977). "The Application of Simplified, Fundamental Design Procedures for Flexible Pavement." *Proceedings, 4th International Conference on the Structural Design of Asphalt Pavements,* Vol. 1, pp. 321-341. University of Michigan, Ann Arbor, MI.

Chadbourn, B. "MnPAVE Homepage." Minnesota Department of Transportation, Office of Materials and Road Research, Maplewood, MN. Accessed 28 March 2008. <http://www.mrr.dot.state.mn.us/research/mnpave/mnpave.asp>

Chadbourn, B., Dai, S., Davich, P. Siekmeier, J., and Van Deusen, D. (2002). *Mn/DOT Flexible Pavement Design - MnPAVE Beta Version 5.1*. Commercially unpublished report for the Office of Materials and Road Research, Minnesota Department of Transportation, St. Paul, MN.

Chou, T.Y. (1976). "Analysis of Subgrade Rutting in Flexible Airfield Pavements." *Transportation Research Record*, No. 616, pp. 44-48.

Claussen, A.I.M., Edwards, J.M., Sommer, P., and Uge, P. (1977). "Asphalt Pavement Design-Shell Method." *Proceedings, 4th International Conference on the Structural Design of Asphalt Pavements,* Vol. 1, pp. 39-74. University of Michigan, Ann Arbor, MI.

El-Basyouny, M.M. (2004). *Calibration and Validation of Asphalt Concrete Pavement Distress Models for 2002 Design Guide*. Ph.D. dissertation. Arizona State University, Tempe, AZ.

El-Basyouny, M.M., Witczak, M.W., El-Badawy, S., Timm, D., Dongre, R., Heitzman, M., and Dunning, R. (2005). "Verification for the Calibrated Permanent Deformation Models for the 2002 Design Guide." *Journal of the Association of Asphalt Paving Technologists,* Vol. 74, pp. 601–652.

Finn, F.N., Saraf, C.L., Kulkarni, R., Nair, K., Smith, W., and Abdullah, A. (1977). *Development of Pavement Structural Subsystems*. Final Report NCHRP 1-10. Transportation Research Board, National Research Council, Washington, D.C.

Garg, N., and Thompson, M.R. (1998). *Mechanistic-Empirical Evaluation of the MnROAD Low-Volume Road Test Sections*. Report FHWA-IL-UI-535. Illinois Department of Transportation, Springfield, IL.

Kenis, W.J. (1977). "Predictive Design Procedures—A Design Method for Flexible Pavements Using the VESYS Structural Subsystem." *Proceedings, 4th International Conference on Structural Design of Asphalt Pavements*, Vol. 1, pp. 101-130. University of Michigan, Ann Arbor, MI.

Khazanovich, L., and Wang, X. (2007). "MnLAYER: High-Performance Layered Elastic Analysis Program." *Transportation Research Record*, No. 2037, pp. 63-75.

Ovik, J.M., Birgisson, B., and Newcomb, D.E. (2000). *Characterizing Seasonal Variations in Pavement Material Properties for Use in a Mechanistic-Empirical Design Procedure*. Report MN-RC-2000-35. Minnesota Department of Transportation, St. Paul, MN.

Shackelton, M.C. (1995). *Modeling of the Permanent Deformation of Untreated Pavement Layers*. M. Eng. thesis. University of Pretoria, Republic of South Africa.

Shook, J.F., Finn, F.N., Witczak, M.W., and Monismith, C.L. (1982). "Thickness design of asphalt pavements—The Asphalt Institute method." *Proceedings, 5th International Conference on the Structural Design of Asphalt Pavements*, Vol. 1, pp. 17–44. Delft University of Technology, the Netherlands.

Strategic Highway Research Program. (1993). *Development and Validation of Performance Prediction Models and Specifications for Asphalt Binders and Paving Mixes*. SHRP Report A-357. National Research Council, Washington, D.C.

Strategic Highway Research Program. (1994). *Fatigue Response of Asphalt-Aggregate Mixes*. SHRP Report A-404. National Research Council, Washington, D.C.

Swedish Road Administration. (2000). *Bearbetning av deflktionsmatdata, erhallna vid provbelastning av vag med FWD-apparat*. Publication 2000:29. Vegverket, Borlange, Sweden.

The Asphalt Institute. (1993). *Thick design-Asphalt Pavements for Highways and Streets*. Manual Series, No. 1, MS-1, Lexington, KY.

The Asphalt Institute. (1983). *Research and Development of the Asphalt Institute's Thickness Design Manual (MS-1)*. Ninth Edition, RR-82-2, Bethesda, MD.

Theyse, H.L., De Beer, M., and Rust, F.C. (1996). "Overview of South African Mechanistic Pavement Design Method." *Transportation Research Record*, No. 1539.

Theyse, H.L. (2002). *Stiffness, Strength, and Performance of Unbound Aggregate Material: Application of South African HVS and Laboratory Results to California Flexible Pavements*. University of California Pavement Research Center, Davis, CA.

Werkmeister, S. (2003). *Permanent Deformation Behavior of Unbound Granular Materials in Pavement Constructions*. Ph.D. dissertation. University of Dresden, Germany.

APPENDIX A. USER GUIDE

The TONN2010 procedure has been implemented into a FORTRAN program. To execute the program, the user has to create an input file containing information about the pavement system, climatic data, etc. as well as the FWD deflection data. The file should be saved in the same directory as the program TONN2010.exe and two data files: DB_inp1.txt and backdefl.txt. To execute the program, the following command should be typed in the DOS prompt in the same directory where TONN2010 is located:

TONN2010 Input_file_name.txt DB_inp1.txt backdefl.txt

Figure A1 shows a screenshot with the command line:

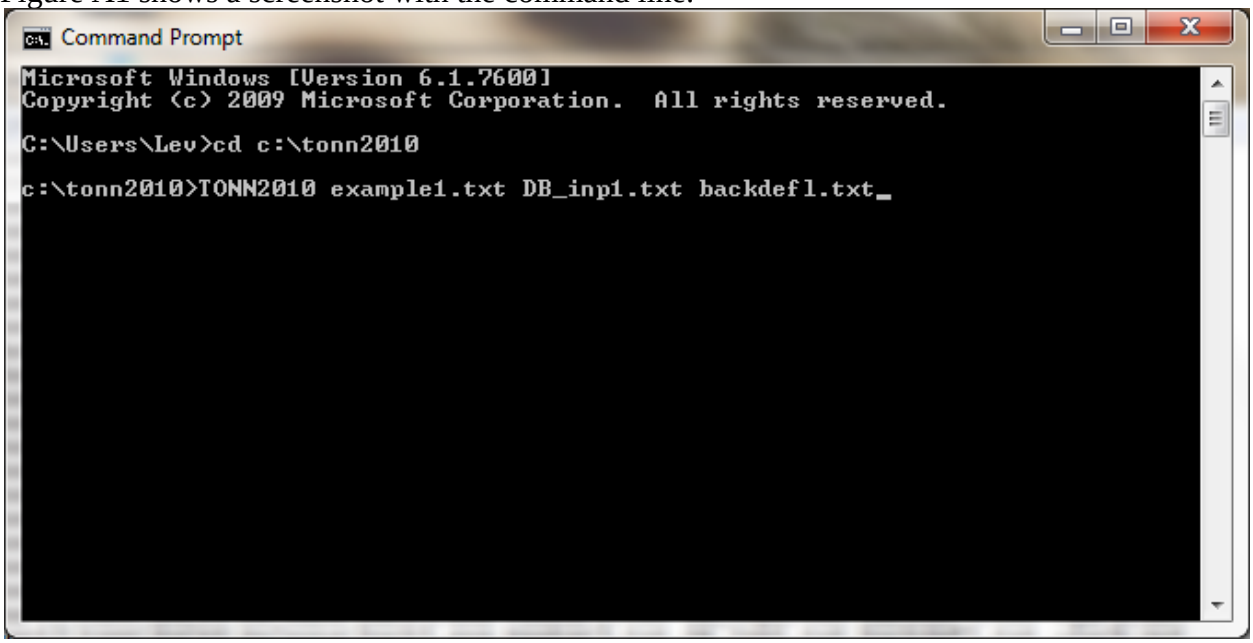


Figure A1. Example of TONN2010 execution

After execution, the program will create the following output files:

- **Input_file_name_back.out** – a file containing the measured and calculated FWD deflections
- **Input_file_name_Tdam.out** – a file containing the details of damage calculation for each deflection basin and each subgrade thickness used in the calculation
- **Input_file_name_TONN.out –** a file containing the backcalculated moduli and TONN indexes for each deflection basin and each subgrade thickness used in the calculation

If the input subgrade thickness is less than 12 inches then backcalculation is performed for multiple subgrade thicknesses. In this case, the program will also create files **Input_file_name_Tdam2.out and Input_file_name_TONN2.out** which contain the details of the damage calculation and TONN indexes, respectively, for each FWD deflection basin and the subgrade thickness which leads to the least discrepancy between the measured and calculated deflection basins.

The output files can be opened using Windows Notepad.

The user-created input file should have the following format, where any number of spaces can be used to separate values:

Line 1. Pavement structure
Hac Hbase Hsubgr

Hac – thickness of the asphalt layer. Hac should be not less than 2 in and not greater than 12 in
Hbase – thickness of the base layer Hbase should be not less than 3 in and not greater than 48 in
Hsubgr – thickness of the subgrade layer (i.e. the layer between the base and the rigid bedrock). Hsubgr should be no less than 12 in and no greater than 240 in. If the specified Hsubgr is less than12 in, the subgrade thickness is considered unknown and the analysis will be performed for several subgrade thicknesses

Example:
5.5 12 0

In this example the asphalt layer thickness is 5.5 in, the base layer thickness is 12 in and the subgrade thickness is unknown.

Line 2. Design traffic
Traf

Traf is the design traffic in ESALs

Example:
270000

Line 3. Seasonal duration information
Days(1) Days(2) Days(3) Days(4) Days(5)
where
Days(1) = number of days in the Winter season
Days(2) = number of days in the Early Season
Days(3) = number of days in the Late Spring season
Days(4) = number of days in the Summer season
Days(5) = number of days in the Fall season

Example:
100 15 55 105 90
In this example there are 100 days in the Winter season, 15 days in the Early Spring, 55 days in the later Spring, 105 days in the Summer, and 90 days in the Fall season,

Line 4. Mean air temperature in each season
Airtemp(1) Airtemp(2) Airtemp(3) Airtemp(4) Airtemp(5)

where

Airtemp(1) = mean air temperature for the Winter season, ^{o}F
Airtemp(2) = mean air temperature for the Early Spring season, ^{o}F
Airtemp(3) = mean air temperature for the Late Spring season, ^{o}F
Airtemp(4) = mean air temperature for the Summer season, ^{o}F
Airtemp(5) = mean air temperature for the Fall season, ^{o}F

Example:
18 50 50 70 41

In this example the mean air temperature in the Winter is 18^{o}F, the mean air temperature in the Early Spring is 50 ^{o}F, the mean air temperature in the Late Spring is 50 ^{o}F, and the mean air temperature in the Summer is 70 ^{o}F, and the mean air temperature in the Fall is 41^{o}F.

Line 5. Seasonal base modulus adjustment factors
adjB(1) adjB(2) adjB(3) adjB(4) adjB(5)

where

adjB(1) = base modulus adjustment factor for the Winter season, ^{o}F
adjB(2) = base modulus adjustment factor for the Early Spring season, ^{o}F
adjB(3) = base modulus adjustment factor for the Late Spring season, ^{o}F
adjB(4) = base modulus adjustment factor for the Summer season, ^{o}F
adjB(5) = base modulus adjustment factor for the Fall season, ^{o}F

Example:
10 0.35 0.65 0.95 1

In this example the base modulus adjustment factor for the Winter is 10, the base modulus adjustment factor for the Early Spring is 0.35, the base modulus adjustment factor for Late Spring is 0.65, and the base modulus adjustment factor for the Summer is 0.95, the base modulus adjustment factor for the Fall is 1.

Line 6. Seasonal subgrade modulus adjustment factors
adjB(1) adjB(2) adjB(3) adjB(4) adjB(5)

where

adjS(1) = Subgrade modulus adjustment factor for the Winter season, ^{o}F
adjS(2) = Subgrade modulus adjustment factor for the Early Spring season, ^{o}F
adjS(3) = Subgrade modulus adjustment factor for the Late Spring season, ^{o}F
adjS(4) = Subgrade modulus adjustment factor for the Summer season, ^{o}F
adjS(5) = Subgrade modulus adjustment factor for the Fall season, ^{o}F

Example:
10 10 0.65 1 1

In this example the subgrade modulus adjustment factor for the Winter is 10, the subgrade modulus adjustment factor for the Early Spring is 0.35, the subgrade modulus adjustment factor for Late Spring is 0.65, and the subgrade modulus adjustment factor for the Summer is 1, the subgrade modulus adjustment factor for the Fall is 0.8.

Line 7. FWD testing conditions

Tair_1F DayB DaySub

where

Tair_1F = previous day mean air temperature

DayB = adjustment factor for base layer modulus if believed to be substantially different from typical modulus for the day of testing (1 if typical, greater than one for unusually dry base, less than one for unusually wet base layer)

DaySub = adjustment factor for subgrade modulus if believed to be substantially different from typical modulus for the day of testing (1 if typical, greater than one for unusually dry subgrade, less than one for unusually wet subgrade)

Example:

67 1 1

Line 8. FWD sensors that should be used in backcalculation

weight(1) weight(2) weight(3) weight(4) weight(5) weight(6)

where:

weight(1) = index indicating if the deflections of the sensor located under the center of the FWD load plate should be used in backcalculation

weight(2) = index indicating if the deflections of the sensor located 8 in away from the center of the FWD load plate should be used in backcalculation

weight(3) = index indicating if the deflections of the sensor located 12 in away from the center of the FWD load plate should be used in backcalculation

weight(4) = index indicating if the deflections of the sensor located 18 in away from the center of the FWD load plate should be used in backcalculation

weight(5) = index indicating if the deflections of the sensor located 24 in away from the center of the FWD load plate should be used in backcalculation

weight(6) = index indicating if the deflections of the sensor located 36 in away from the center of the FWD load plate should be used in backcalculation

if weight(i) = 1 then the corresponding sensor should be used in backcalculation;
if weight(i) = 0 then the corresponding sensor should not be used in backcalculation and the corresponding deflection data will be ignored by the program;

Example:

1 1 1 1 1 1

Line 9. FWD measurement data (repeat as many times as necessary)

Tir Hour Pload defl(1) defl(2) defl(3) defl(4) defl(5) defl(6)

where

Tir – pavement surface temperature, ^{o}F
Hour – time of testing (decimal hours on a 0 to 24 hour basis)
Pload – total FWD load, lbs

defl(1) = FWD deflection (in mils) of the sensor located under the center of the FWD load plate should be used in backcalculation

defl(2) = FWD deflection (in mils) of the sensor located 8 in away from the center of the FWD load plate should be used in backcalculation

defl(3) = FWD deflection (in mils) of the sensor located 12 in away from the center of the FWD load plate should be used in backcalculation

defl(4) = FWD deflection (in mils) of the sensor located 18 in away from the center of the FWD load plate should be used in backcalculation

defl(5) = FWD deflection (in mils) of the sensor located 24 in away from the center of the FWD load plate should be used in backcalculation

defl(6) = FWD deflection (in mils) of the sensor located 36 in away from the center of the FWD load plate should be used in backcalculation

Example:

75.599996	7.15	5188.6125	11.51	8.33	6.53	4.51	3.12	1.8
75.599996	7.15	7581.7395	17.58	12.51	9.8	6.83	4.74	2.76
75.599996	7.15	9768.68325	22.29	16.65	13	9.05	6.35	3.6
75.599996	7.15	12935.4975	30.35	23.26	18.19	12.7	8.88	5.01
75.599996	7.16	5188.6125	10.81	8.13	6.42	4.54	3.29	1.96
75.599996	7.16	7589.74725	16.89	12.35	9.76	6.94	4.94	2.94
75.599996	7.16	9704.62575	22.17	16.47	12.92	9.17	6.59	3.88
75.599996	7.16	12893.46075	30.03	23.12	18.25	12.95	9.24	5.33

Input file example1.txt

```
5.5     12      0
270000
100     15      55      105     90
18      50      50      70      41
```

```
10      0.35    0.65    0.95    1
10      10      0.65    1       1
67      1       1
1       1       1       1       1       1
75.599996       7.15    5188.6125       11.51   8.33    6.53    4.51    3.12    1.8
75.599996       7.15    7581.7395       17.58   12.51   9.8     6.83    4.74    2.76
75.599996       7.15    9768.68325      22.29   16.65   13      9.05    6.35    3.6
75.599996       7.15    12935.4975      30.35   23.26   18.19   12.7    8.88    5.01
75.599996       7.16    5188.6125       10.81   8.13    6.42    4.54    3.29    1.96
75.599996       7.16    7589.74725      16.89   12.35   9.76    6.94    4.94    2.94
75.599996       7.16    9704.62575      22.17   16.47   12.92   9.17    6.59    3.88
75.599996       7.16    12893.46075     30.03   23.12   18.25   12.95   9.24    5.33
```

The program creates the following output files:
example1_BACK.out
example1_Tdam.out
example1_Tdam2.out
example1_TONN.out
example1_TONN2.out

Note that files *example1_Tdam2.out* and *example1_TONN2.out* are created only because in this example Hsbgr = 0.