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Interpreting Falling Weight Deflectometer (FWD) Data (for Asphalt and Concrete Pavements)

FINAL REPORT

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By
Kevin Alland
Nathan Bech and
Julie M. Vandenbossche (PI), P.E., Ph.D.

University of Pittsburgh

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Technical Advisor
William Dipner
PA Department of Transportation | District 11-0
45 Thoms Run Road | Bridgeville PA 15017
wdipner@pa.gov, Phone: 412.429.3814

16. Abstract
Falling weight deflectometer (FWD) testing is a valuable method for assessing the structural condition of existing pavement structures. For jointed plain concrete pavements (JPCPs), FWD testing is used to detect voids, monitor joints and crack performance, and backcalculate the modulus of elasticity of the existing Portland cement concrete (PCC) and the k-value of all supporting layers. For asphalt concrete (AC) pavements, FWD testing is used to backcalculate the stiffness of each layer and to estimate the amount of damage in the existing asphalt. This report summarizes the testing protocols and data analysis procedures recommended. The report consists of three primary sections. The first section describes the testing protocols recommended for FWD data collection. The second section defines the changes proposed to current PennDOT documents (including Publication 242, Publication 408, and the PennDOT Pavement ME Design Preliminary User Input Guide) based on the findings of this study. The third section is an appendix that is divided into four separate appendices: A-Scheduling and performing FWD testing; B-Data analysis guidelines; C-Research findings and D-Laboratory and field testing.

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INTRODUCTION

Falling weight deflectometer (FWD) testing is a valuable method for assessing the structural condition of existing pavement structures. For jointed plain concrete pavements (JPCPs), FWD testing is used to detect voids, monitor joints and crack performance, and backcalculate the modulus of elasticity of the existing Portland cement concrete (PCC) and the k-value of all supporting layers. For asphalt concrete (AC) pavements, FWD testing is used to backcalculate the stiffness of each layer and to estimate the amount of damage in the existing asphalt. This report summarizes the testing protocols and data analysis procedures recommended. The report consists of three primary sections. The first section describes the testing protocols recommended for FWD data collection. The second section defines the changes proposed to current PennDOT documents (including Publication 242, Publication 408, and the PennDOT Pavement ME Design Preliminary User Input Guide) based on the findings of this study. The third section is an appendix that is divided into four separate topics, as defined below:

Appendices

a. Scheduling and performing FWD testing
b. Data analysis guidelines
c. Research findings
d. Laboratory and field testing

FWD DATA COLLECTION PROCEDURES

The FWD data collection procedure recommended for JPCPs is described first and this is followed by a description of the data collection procedure recommended for AC pavements.

JPCP Pavements

Testing Protocol

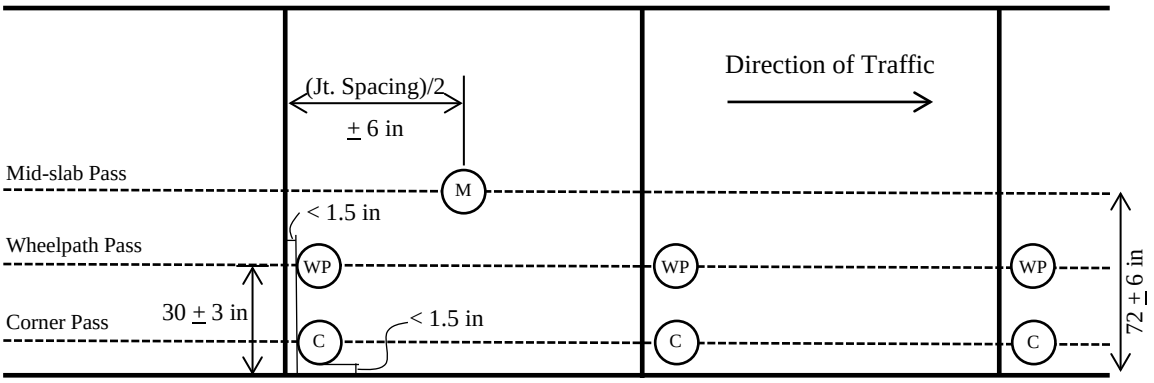
FWD testing performed on JPCPs is intended to assess the pavement condition prior to concrete pavement restoration or prior to placing a concrete overlay. The guidelines presented here were developed for JPCP and may not be applicable for jointed reinforced concrete pavements (JRCP). FWD testing should only be performed when the subgrade is not frozen, and the ambient temperature is less than 70 ^{o}F.

All FWD testing performed on JPCP should include the use of eight deflection sensors. The recommended distance of each of the eight sensors from the center of the load is provided in Table 1.

Table 1. Sensor offsets for performing FWD testing on JPCPs.

Sensor	Offset (in)
1	0
2	-12
3	8
4	12
5	18
6	24
7	36
8	60

FWD testing should be performed at three locations: 1. in the wheelpath on the leave side and adjacent to the transverse joint (WP); 2. in the corner on the leave side of the transverse joint (C); and 3. at mid-slab (M), as shown in Figure 1. The WP and C testing should be performed on each slab, while the mid-slab testing can be performed on every sixth slab. If the sixth slab has developed a crack, then an uncracked adjacent slab should be tested. The testing will be performed in three separate passes, which is also shown in Figure 1. It should also be noted that the data measuring instrument (DMI) should be re-zeroed the beginning of each pass to ensure consistency between the three passes.)



Note: Conduct mid-slab testing every 6 slabs. Adjust mid-slab testing locations to avoid cracked slabs

Figure 1. FWD test locations for JPCPs.

All three passes should be completed within the same day for all slabs tested. The sequence in which each pass should be completed will be different depending on when the testing will be performed. The required testing sequence for ***daytime*** and nighttime testing is summarized in Table 2.

Table 2. JPCP FWD testing sequence

Pass	Daytime Order	Nighttime Order	Drops	Time Restrictions
Mid-slab	1	3	3@16,000 lb	See Appendix A-1
Wheelpath	2	2	3@9,000 lb 3@16,000 lb	
Corner	3	1	3@9,000 lb 3@12,000 lb 3@16,000 lb	

Note: The loading sequence should begin with three seating drops at the 12,000-lb load level at each location, without collecting data.

Production Rate

Performing mid-slab testing on every sixth slab will take approximately 5% of the time allocated for testing. Therefore, for an 8-hr test period, approximately 25 minutes would be allocated towards the completion of mid-slab testing. A production rate of 0.4 lane miles per day is estimated but it is assumed this will increase as more experience with this test protocol is gained.

Data Analysis

A web-based application has been developed by Pitt for the FWD Analysis of Concrete Slabs (Pitt-FACS) that can be used to assist in establishing overlay design inputs (elastic modulus of the concrete and k-value), assessing joint performance, and in identifying possible voids beneath the slab. A user's manual for Pitt-FACS is provided in Appendix B-1.

Supplemental Testing

Additional information can be collected to assist with the interpretation of the FWD data. This information can be obtained through coring, dynamic cone penetrometer (DCP) testing, and measuring the temperature profile in the slab at the time FWD testing is performed.

Coring

- The slab thickness is a key parameter for predicting, assessing, and designing rehabilitation alternatives and for backcalculating layer properties. Cores pulled from the pavement can be used to confirm slab thickness and, if desired, to measure the material properties of the concrete. Coring at mid-slab will help reduce the effect of the core hole on the future performance of the slab. The diameter of the core pulled should be as follows:
    - Slab thickness: 2-in diameter core
    - Split tensile strength (ASTM C 496) and elastic modulus (ASTM C 469) (Needed when designing bonded concrete overlays.): 6-in diameter cores
- DCP testing may be performed in the core holes to establish the stiffness of the granular layers beneath the slab. This information is useful for determining if the time of day that mid-slab testing is performed needs to be restricted, as described in Appendix C-2. Guidance on performing DCP testing is provided in Appendix A-2.

Slab Temperature

The temperature profile in the slab at the time FWD testing is performed can be estimated within Pitt-FACS. However, the effect of shading by geological features or other sun barriers on pavement temperature will not be captured. If a significant portion of the pavement is shaded, the pavement temperature in this region should be measured using temperature holes. Four temperature holes should be drilled to the depths shown in Figure 2. Temperatures should be measured at a minimum of every 30 minutes, either manually with a temperature probe, or through the use of a datalogger.

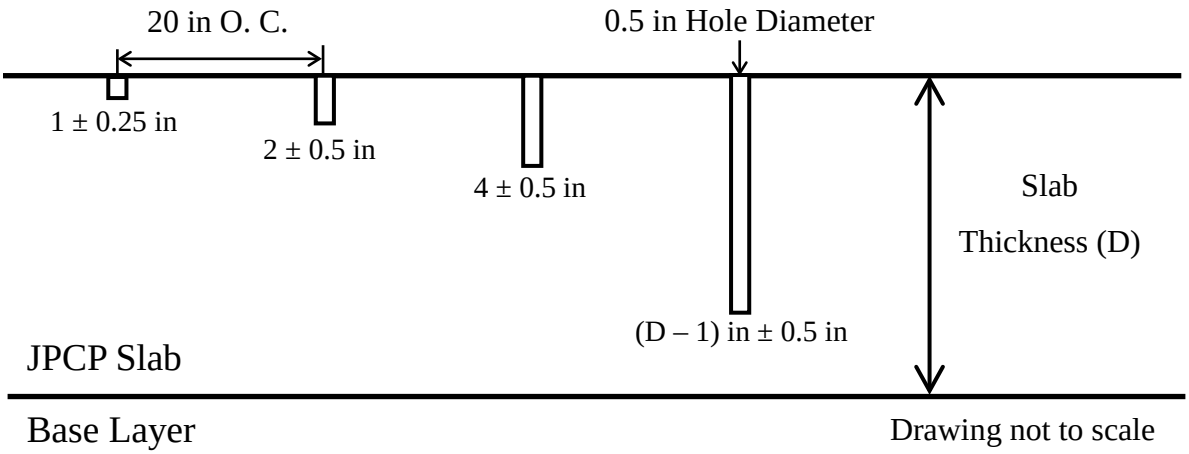


Figure 2. JPCP pavement temperature hole depths.

The following should also be considered when measuring temperature profiles in the field

- The deepest hole should be drilled first and the shallowest hole last to allow time for heat dissipation.
- A small amount of mineral spirits should be placed in the bottom of each hole to conduct heat between the concrete and the temperature sensor and the whole should be covered by a piece of duct tape to prevent debris from falling into the whole.
- Temperature measurements should not be recorded until at least 20 min after the last hole is finished.
- The temperature at the bottom of each hole should be recorded every 30 minutes during FWD testing. Temperature measurements can be performed manually using a temperature probe, or automatically using a datalogger. Automated temperature measurements should occur every 5 min.

Solar Radiation Measurements

If the pavement temperature profile is not measured, the temperature profile of the pavement can be estimated within Pitt-FACS. The accuracy of this estimate can be improved by providing measurements of the incoming solar radiation or estimates of the sky condition during FWD testing. The solar radiation can be established using three separate methods similar to the three-level hierarchy adopted in the Pavement ME Design Procedure. Level 1 provides the most accurate information, and Level 3 provides the least accurate information.

- *Level 1:* Measure the incoming shortwave solar radiation using a pyranometer installed on the roof of the FWD van. A cost estimate for the equipment required for these measurements is provided in Appendix A-3.
- *Level 2:* Estimate the sky condition once an hour. The sky condition is estimated based on the number of octas (1/8 of the celestial dome) which are filled with clouds. The number of octas corresponding to each sky condition can be seen in Table 3.

Table 3. Sky condition estimates

Sky Condition	Octas Filled
Clear	<1/2
Few	1/2 to 2
Partly Cloudy	2 to 4
Mostly Cloudy	4 to 7
Overcast	>7

- *Level 3:* Use the measurements of the sky condition provided by local weather stations in the National Oceanic and Atmospheric Administration (NOAA) Automated Surface Observation System (ASOS) (https://www.ncdc.noaa.gov/data-access), or at nearby grid points in the National Aviation and Space Administration (NASA) Modern Era Reanalysis for Research and Applications Version 2 (MERRA2) dataset (https://gmao.gsfc.nasa.gov/reanalysis/MERRA-2).

The advantages of using Level 1 or Level 2 values for the sky condition are explained in Appendix C-1.

AC Pavements

Testing Protocol

FWD testing is performed on AC pavements to develop inputs for the following designs:

- AASHTO Design Guide (AASHTO 1993)
    - AC overlay of AC pavement, non-destructive testing (NDT) method
- Pavement ME (ARA Inc. 2004)
    - AC over AC overlay design, Level 1
    - Unbonded JPCP over AC overlay design, Level 1

The same FWD data collection procedures can be used to establish the design inputs for all three of these overlay designs. FWD testing should performed when the mid-depth asphalt temperature is between 40 ºF to 100 ºF but not during spring thaw or when the subgrade is frozen.

FWD testing should be performed in the outer wheelpath of the driving lane along the entire length of the test section, as shown in Figure 3. The distance between the lane-shoulder joint and the outer wheelpath may be altered if the visible wheelpath is outside the recommended range, such as on a curve or in a lane narrower than 12 ft. It is recommended that FWD testing be performed every 50 ft for pavements in maintenance functional category (MFC) A, B, and C and every 100 ft for pavements in MFC D or E. The distance between test locations can be adjusted according to the demands of specific projects.

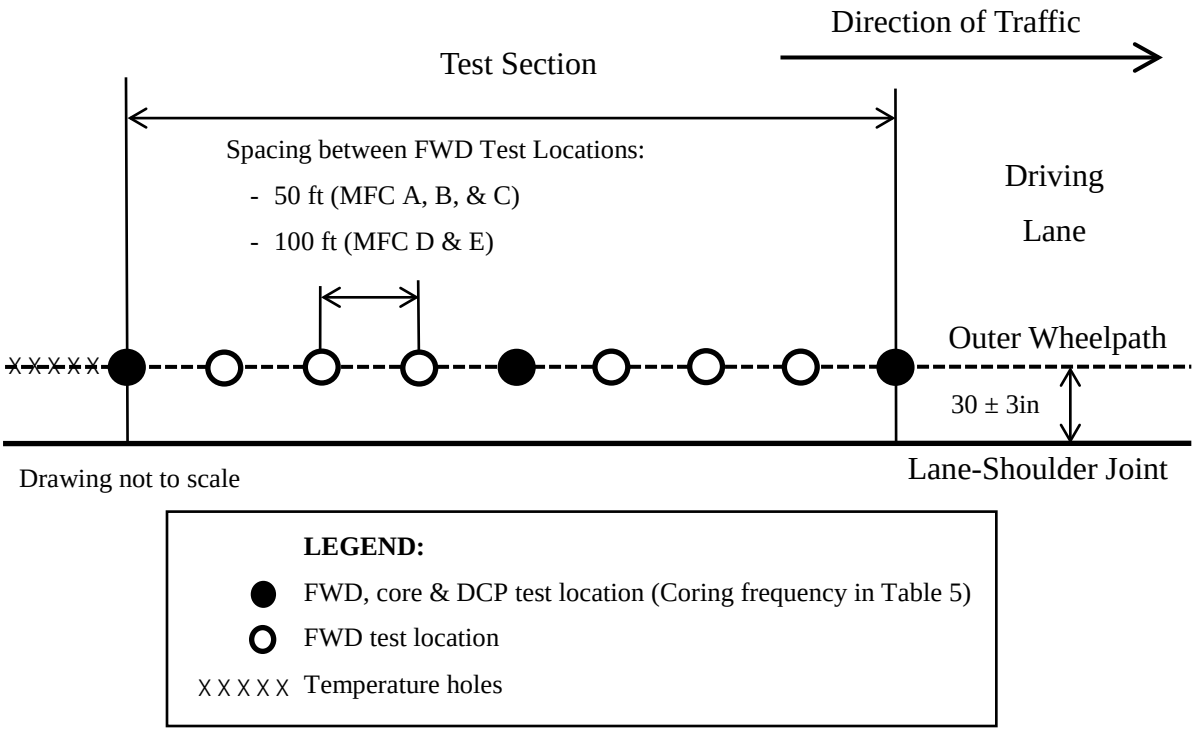


Figure 3. FWD test plan for AC pavements.

Eight deflection sensors should be used at the locations shown in Table 4. A total of six FWD drops should be performed at each test location: three seating drops at the 12,000-lb load level, and three measurement drops at the 9,000-lb load level. No deflections should be recorded for the seating drops, and the maximum sensor deflections should be recorded for each drop.

Table 4. Sensor offsets for performing FWD testing on AC pavements

Sensor	Offset (in)
1	0
2	8
3	12
4	18
5	24
6	36
7	48
8	60

Production Rate

FWD testing can be performed at a rate of 0.5 miles per hour when testing every 50 ft and 1.0 mile per hour when testing every 100 ft.

Data Analysis

The stiffness of each layer can be backcalculated using the guidelines described in Appendix B-3.

Supplemental Testing

Additional data collection can be performed to assist with the interpretation of the FWD data. This includes pulling cores, performing soil borings and DCP testing and measuring the temperature profile in the pavement at the time FWD testing is performed.

Coring

After FWD testing is completed, cores should be taken at regular intervals in the outer wheelpath of the driving lane (see Figure 3). Table 5 shows the recommended core spacings based on the MFC of the pavement. Cores should be sufficiently thick that the average thickness of all stabilized layers can be established. If the core is damaged, the thickness of the stabilized layers should be measured from the core hole.

Table 5. Core spacing for flexible pavements

Maintenance Functional Classification (MFC)	Cores Per Mile
A (Interstate highways)	5
B (Major arterial highways)	3
C, D, and E	2

Laboratory testing must be performed on the cores if the Level 1 inputs are being used to design an AC/AC overlay in Pavement ME. Guidance on the preparation and laboratory testing of the cores is provided in Appendix B-5.

If laboratory testing is performed, each core must contain at least 22 lb of the existing asphalt (see Appendix B-5 for a list of required laboratory tests). At least 33 lb of existing asphalt is required for a nominal maximum aggregate size of 37.5 mm. The existing asphalt is defined as all wearing, binder, and base courses and any open-graded asphalt permeable base (if present). Tables 6 and 7 show the minimum number of 6-in diameter cores required to satisfy the asphalt weight requirement as a function of existing asphalt thickness.

Table 6. Minimum number of 6-in diameter cores based on asphalt thickness (nominal max aggregate size less than or equal to 25 mm)

Asphalt Thickness (in)	Minimum Number of Cores
6 to 7.5	4
8 to 10.5	3
11 to 21.5	2
22 or more	1

Table 7. Minimum number of 6-in diameter cores based on asphalt thickness (nominal max aggregate size less than or equal to 37.5 mm)

Asphalt Thickness (in)	Minimum Number of Cores
6 to 6.5	6
7 to 8.5	5
9 to 10.5	4
11 to 16.5	3
17 or more	2

Soil Borings

Soil borings may be performed in core holes to determine the thickness of the unbound layers and to extract samples of unbound layers for laboratory resilient modulus testing.

Dynamic Cone Penetrometer Testing

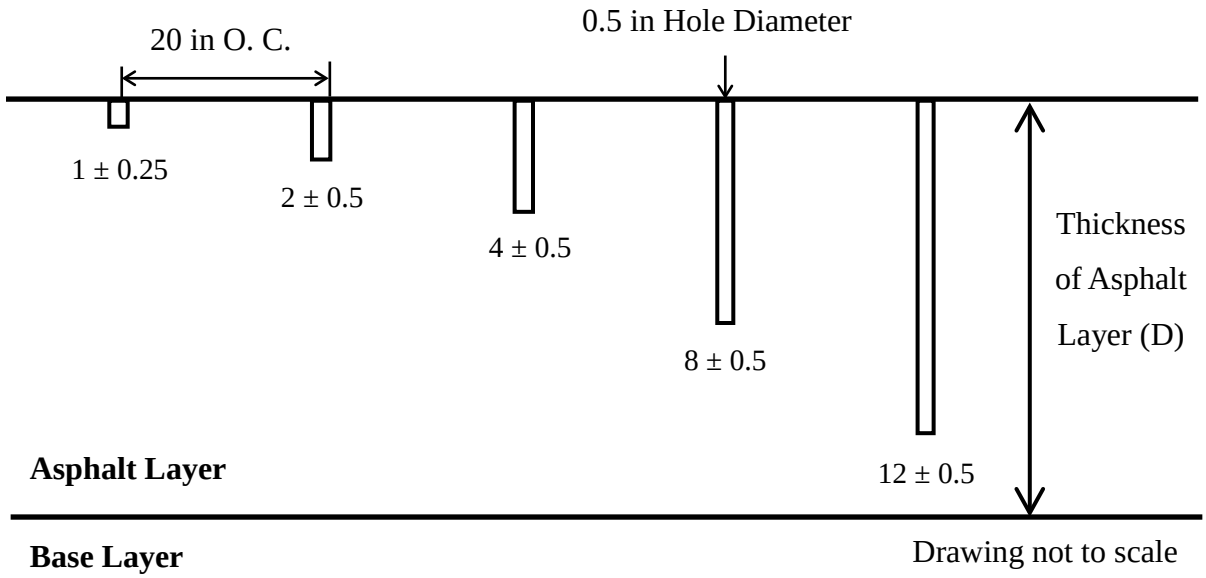
For projects where soil borings are not performed, DCP testing can be performed in each core hole to estimate the thickness and stiffness of the unbound layers. Guidelines on the use of DCP testing can be found in Appendix A-2. Procedures for determining layer thickness and estimating layer stiffness using the DCP can be found in Appendix B-2

Asphalt Temperature

The mid-depth temperature of the asphalt may be estimated using the BELLS3 equation. This equation and guidelines for its use can be found in Appendix B-4. If a significant portion of the test section is shaded, it is recommended that the asphalt temperature be measured directly using temperature holes.

The temperature holes should be drilled in the outer wheelpath at one end of the test section for sections one mile or less in length, and at both ends for test sections greater than one mile. Shading at the location of the temperature holes should be representative of pavement shading

along the entire test section. Temperature hole locations can be moved from the ends of the test section to ensure representative shading. The hole depths to be used for an asphalt pavement are shown in Figure 4.



Note: Holes deeper than the AC layer thickness (D) should not be drilled. In this case, a final hole with depth (D-1) inches should be drilled.

Figure 4. Depths and locations of temperature holes for flexible pavements.

The following should also be considered when measuring temperature profiles in the field:

- The deepest hole should be drilled first and the shallowest hole last to allow time for heat dissipation.
- A small amount of mineral spirits should be placed in the bottom of each hole to conduct heat between the asphalt and the temperature sensor and the hole should be covered by a piece of duct tape to prevent debris from falling into the hole.
- Temperature measurements should not be recorded until at least 20 minutes after the last hole is finished.
- The temperature at the bottom of each hole should be recorded every 30 minutes during FWD testing. Temperature measurements can be performed manually using a temperature probe, or in an automated manner using a datalogger. Automated temperature measurements should be recorded at 5-minute intervals.

PROPOSED CHANGES TO PENNDOT DOCUMENTS

The following section contains text from PennDOT Publication 242 (PennDOT 2016), PennDOT Publication 408 (PennDOT 2011), and the PennDOT Pavement ME Design Preliminary User Input Guide (Bhattacharya et al. 2017) that can be updated based on the results from this study. Sections of text recommended for removal are denoted by font with a strike-through. Text to be added is indicated by italics. Unchanged text is included in normal font to provide context for the edits. Notes commenting on the recommended edits are also provided at the end of each recommended edit

Publication 242

Section 4.3 – Joints

Recommended Edits

A. General Guidelines. Concrete joint partial-depth repair, joint rehabilitation and longitudinal joint repair should be considered in an effort to preserve or extend the life of an existing PCC pavement when it is not going to be overlaid. These items of work should be performed on pavements that are just beginning to show distress at the joints, even though the pavement's serviceability may still be satisfactory. In addition, if joint performance is a problem ~~then the joint should be replaced with full-depth concrete pavement patching regardless of the amount of spall repair required.~~ *load transfer should be restored. Poor joint performance is indicated by the presence of faulting greater than 1/8 inch, or through analysis of FWD testing in the wheelpath. If the concrete surrounding the joint is sound, this can be accomplished using dowel bar retrofits. A full-depth concrete repair should be performed if the joint is exhibiting both poor load transfer and there is also deterioration at the joint.*

5. Dowel Retrofit. Dowel retrofits are primarily used on roadways that receive heavily channeled loadings where transverse joints or cracks would benefit from improved load transfer. *Dowel bar retrofits are beneficial for joints that are in good condition, but are exhibiting poor joint performance, making them susceptible to the development of faulting. The presence of faulting is an indicator of poor joint performance. If diamond grinding is performed to remove the faulting without addressing the issues contributing to the faulting, the faulting will redevelop and at a more rapid rate. Therefore, joints exhibiting faulting are good candidates for dowel retrofits as long as sound concrete exists throughout the depth of the slab. Pitt-FACS can be used*

to identify poor performing joints using the corrected load transfer efficiency (LTE) and corrected differential deflection (DD). FHWA Guidelines suggest dowel bar retrofits be performed if the corrected LTE is less than 60% or if the DD normalized to 9,000 lb is greater than 0.01 in. Slab stabilization should be performed along with dowel bar retrofits if voids are present in the corners. Pitt-FACS can also be used to assist with identifying slab corners that have voids. Load transfer restoration and slab stabilization should be performed as part of CPR or prior to the construction of a bonded concrete or AC overlay. Working cracks in jointed reinforced concrete pavements (JRCP), which are not showing signs of deterioration, are also candidates for dowel bar retrofits.

Dowel retrofits involve the installation of epoxy-coated, smooth dowel bars into the wheel paths of existing concrete pavement across cracks or transverse joints without dowels (see Publication 72M, Roadway Construction Standards, RC-26M and Publication 408, Specifications, Section 527). A power-driven, self-propelled saw is used to make two parallel cuts per dowel bar slot for a minimum of four slots simultaneously, with saw cuts parallel to the roadway centerline. After the slots have been prepared and cleaned, the dowels are prepared and placed into the slots. The slots are then filled with concrete patching material and cured. Measurement and payment includes eight dowel bars per joint or crack.

Notes

Full-depth repair of joints with poor load transfer efficiency, but in otherwise good condition, are expensive and can increase the risk of future faulting by increasing the number of joints. Dowel bar retrofits between existing dowels can increase LTE, without the risk of patch settlement or poorly epoxied dowel bars.

Instructions for using Pitt-FACS to determine the corrected LTE and corrected DD and for detecting the presence of voids, is shown in Appendix B-1.

Section 4.5 – Slab Stabilization

Recommended Edits

Pumping action and subgrade consolidation and settlement may create small voids beneath the slab. Most of the voids develop near transverse joints and cracks - particularly at outside slab corners. The loss of slab support results in excessive slab deflections and stresses and causes joint faulting, corner breaks, diagonal cracking and, finally, the complete breakup of the slab. The following conditions indicate a loss of slab support:

1. Transverse joint faulting
2. Fines near joints or cracks on the traffic lane or shoulder
3. Small depressions (blow holes) in the shoulder at the transverse joint or crack
4. Corner breaks

Slab stabilization is a technique that attempts to stabilize the slab by filling voids at the slab/subbase interface with a cement/pozzolan grout. When voids are filled sufficiently, full support is restored. Slab stabilization shall be done according to Publication 72M, Roadway Construction Standards, RC-26M and Publication 408, Specifications, Section 679. Slab stabilization does not correct pavement surface depressions, increase the pavement's design structural capacity, or eliminate faulting. However, the pavement's structural integrity can be restored by filling voids to reduce deflections, which then reduces the potential for future pumping, faulting and slab cracking.

Performing slab stabilization on JPCP pavements without a void can cause uneven slab support, leading to premature failure. Therefore, slab stabilization should only be performed when a void is detected.

To reduce the amount of water that enters the pavement and contributes to pumping, joint and crack sealing must be performed in conjunction with slab stabilization. Also, subsurface drains should be kept in good condition.

For estimating purposes, at least 25 percent of the transverse joints and all patch joints should be stabilized if no preliminary testing has been performed. Estimate 1 cubic foot of grout per hole (0.25 bag of cement per hole). Refer to Publication 72M, *Roadway Construction Standards*, RC-26M for the number and pattern of holes to use at a joint or crack. To improve the effectiveness of full-depth patching, grout the patch with a two-hole pattern (the holes are drilled into the concrete adjacent to the patch). For the passing lane, grout the downslope side of the superelevation.

To economize the use of Maintenance and Protection of Traffic (MPT), drill the pavement just prior to stabilization; both crews (drilling and stabilizing) can be protected by the same traffic control devices. Refer to Publication 213, *Temporary Traffic Control Guidelines*, for MPT setup requirements. For the same reasons, confine drilling and stabilizing to a single lane at any one time.

Notes

The Pitt-FACS user's manual can be found in Appendix B-1. The theory behind the development of the web application can be found in Appendix C-4. Pitt-FACS should be used in conjunction with engineering judgement, and other available information, such as distress surveys. For example, if the normalized deflections in the corner of the slab are well below the estimated cutoff level for all joints, except a few locations where the deflections are near the cutoff, voids are likely at those locations. This is especially true if these joints are exhibiting faulting or signs of pumping. In contrast, if all deflections along a section are near the cutoff, with a few sections slightly above the cutoff, voids may not occur at these locations. This is especially true if the joints appear to be in good condition, with no evidence of pumping, and the joints are well sealed.

Section 6.2.A – Subgrade Soils, Resilient Modulus, Falling Weight Deflectometer (FWD)

Notes

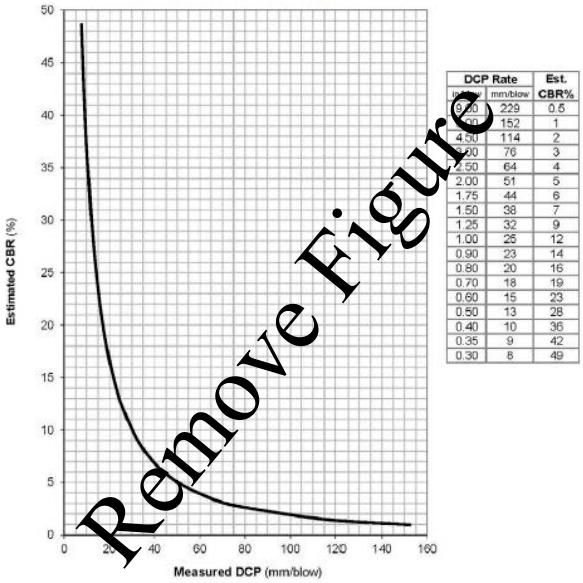
It is recommended that the procedures historically used in defining the inputs needed for overlay design using the AASHTO 1993 Design Guide continue to be used. It is important that these values be consistent with the values used in the design performance equation for the AASHTO 1993 Design Guide and therefore the means of establishing them should be consistent.

Section 6.2.A – Subgrade Soils, Resilient Modulus, Field Dynamic Cone Penetration (DCP)

Recommended Edits

A DCP test provides a measure of a material's in-situ resistance to penetration. The test is performed by driving a metal cone into the ground by repeatedly striking it with a 17.6 pound hammer dropped from a distance of 2.26 feet. The penetration of the cone is measured after each blow and is recorded to provide a continuous measure of shearing resistance up to 5 feet below the ground surface. DCP test results may be used and converted to Mr values via the CBR conversion. Use Figure 6.2 to facilitate the conversion. The use of DCP is limited to roadways with MFC = B, C, D & E. *The test is performed by driving a metal cone into the ground by repeatedly striking it with a 17.6-lb hammer, dropped from a distance of approximately two feet. The penetration of the cone is measured after each blow and is recorded to provide a continuous measure of shearing resistance below the ground surface. DCP test results may be converted to CBR using the equations shown below.*

FIGURE 6.2
DCP - CBR CORRELATION



DCP Rate		Est.
in/blow	mm/blow	CBR%
9.00	229	0.5
6.00	152	1
4.50	114	2
3.00	76	3
2.50	64	4
2.00	51	5
1.75	44	6
1.50	38	7
1.25	32	9
1.00	25	12
0.90	23	14
0.80	20	16
0.70	18	19
0.60	15	23
0.50	13	28
0.40	10	36
0.35	9	42
0.30	8	49

Log (CBR) = 2.20 - 0.71Log (DCP)^1.5 + 0.075
where: CBR = California Bearing Ratio (%)
DCP = Dynamic Cone Penetration Rate (mm/blow)

Transportation Research Institute
Technion Institute of Technology
Pub. No. 87-065, Moshe & Livneh (1987)

For high-plasticity clay soils (CH) (Webster et al. 1994):

$$CBR = \frac{1}{(0.07292 \times DPI)}$$

For low-plasticity clay soils (CL) (CBR < 10)) (Webster et al. 1994):

$$CBR = \frac{1}{(0.43228 \times DPI)^2}$$

For all other soils (Webster et al. 1992):

$$log(CBR) = 2.46 - 1.12 \times (\log(25.4 \times DPI))$$

Where:
DPI = Dynamic penetration index (in/blow)
CBR = California bearing ratio (%)

Note: The DPI to CBR equations presented are calibrated for the DCP configured with the standard-length drive rod. If a drive rod extension is used, these equations should be used with caution.

Notes

Test procedures for the dynamic cone penetrometer can be found in Appendix A-2. Recommendations for estimating CBR and M_r using DCP data can be found in Appendix B-2. It is recommended that Figure 6.2 be removed and replaced in order to standardize the DPI to CBR relationship across all PennDOT documents. Currently, Publication 242 recommends using the relationship in Figure 6.2 for all soil types and the PennDOT Pavement ME Design Preliminary User Input Guide recommends using the Webster, et al. 1992 relationship, shown above, for all soil types. An investigation of these equations and the Webster, et al. 1994 equations has shown that they are very similar (see Appendix C-9 for more details). Thus, it is recommended that the Webster, et al. 1992 and Webster, et al. 1994 equations be used in both Publication 242 and in the PennDOT Pavement ME Design Preliminary User Input Guide.

Section 11.7 – Falling Weight Deflectometer (FWD) Testing Programs

Recommended Edits

FWD data is required whenever a structural pavement overlay design is required, and California Bearing Ratio (CBR) or Resilient Modulus data are not available. Furthermore, FWD data are required for Concrete Pavement Restoration (CPR) *projects and overlays of existing concrete pavements* ~~, to determine the amount of required patching.~~ *to determine the location of full depth repairs, dowel bar retrofits, and sub-sealing.* The District Pavement Management Engineer/Pavement Manager obtains FWD data by submitting a testing request to the BOMO. It is desirable for the District project design staff to have this data prior to Final Design.

Testing of JPCP pavements should be performed when the ambient temperature is less than 70 ºF, and the subgrade is not frozen. Mid-slab testing should occur when the equivalent linear temperature gradient (ELTG) is less than 0.5 ºF/in. Guidance on how to ensure testing is not being performed outside of this window can be found in the PennDOT FWD Data Collection Procedures document. ~~Note that testing concrete pavement joints for CPR projects can only be performed when the air temperature does not exceed 70°F, and no FWD testing can be performed if the subgrade is frozen.~~ *No FWD testing of JPCP pavements should be performed if the subgrade is frozen or during spring thaw.*

FWD testing of AC pavements should only be performed when the mid-depth asphalt temperature is between 40ºF and 100ºF. This will typically exclude performing FWD testing on asphalt pavements less than 14 in thick during the daytime in June, July or August. FWD testing of AC pavements should not be performed if the subgrade is frozen or during spring thaw. Also, testing should not be done more than two years prior to construction, since conditions may worsen and design requirements may change over that period of time. *PennDOT FWD Data Collection Procedures contains more information on FWD testing of both JPCPs and AC pavements.*

Notes

Backcalculation of JPCP pavement layer properties is not reliable when the ELTG is greater than 0.5 ºF/in. Details on how this was determined are provided in Appendix C-2. Recommended testing times to ensure the ELTG is less than 0.5 ºF/in during testing, are shown in Appendix A-1. Details on the analysis used to develop these times can be seen in Appendix C-1. Details on the analysis used to establish seasonal restrictions for FWD testing of AC pavements are provided in Appendix C-7.

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Section 679.3(b) – Slab stabilization, construction, deflection testing

Recommended Edits

(a) General. Do not begin this work until it is satisfactorily shown that qualified personnel, with successful experience, are available at the job. Do not perform work if daytime temperatures are below 35 ºF or if the subgrade and/or base course material is frozen.

(b) Deflection Testing. If no preliminary testing was performed, test each joint and crack as directed, and as follows: Do not perform testing if air temperature exceeds 70 ºF. Do not test during

spring thaw conditions or if subgrade is frozen. Furnish and maintain four gauges capable of detecting slab movement to within 0.001 inch. Use approved gauge mounts. Furnish and maintain a vehicle having a dual-tire single axle with an 18,000-pound single axle load. Verify by measuring the force of gravity upon a certified scale. Position two gauges as shown on the Standard Drawings. Zero both gauges to the pavement surface with no force on the slab on both sides of the joint or crack. Slowly move the test vehicle into position and stop when the test axle is in the position shown on the Standard Drawings for the loaded approach slab condition. Read both gauges and record the results. Move the test vehicle slowly across the joint and stop it in the position shown on the Standard Drawings for the loaded leave slab condition. Read both gauges and record the results. Repeat this procedure at every transverse joint and crack. Stabilize all joints or cracks that have a loaded slab corner deflection of 0.020 inch or more, and a joint efficiency at 65%* or more. Patch and stabilize all joints or cracks that have a loaded slab corner deflection of 0.020 inch or more, and a joint efficiency of less than 65%. Joint efficiency (JE) is defined as follows: JE = Unloaded Slab Corner Deflection x 100 Loaded Slab Corner Deflection * Use the highest Loaded Slab Corner Deflection and the lowest joint efficiency at each joint or crack.

Work locations can also be identified using the FWD and marked by the PennDOT representative.

Pavement ME Design Preliminary User Input Guide

Section 8.1.5 – Rehabilitation: Condition of existing flexible pavement, Rehabilitation input level 1

Recommended Edits

Deflection basins measured on the project provide valuable information and are believed to result in more reliable rehabilitation designs. Measured deflection basins are used to estimate the in-place “damaged” elastic modulus values for each structural layer and subgrade of the existing pavement. Thus, FWD testing should be carried out in cracked (that will not be repaired) and non-cracked areas. ~~Backcalculation of the elastic layer modulus values are determined or calculated external to the Pavement ME software. There are several effective back-calculation programs including Modulus 6, EverCalc, Elmod, ModComp etc.~~ *It is recommended that the backcalculation program EVERCALC 5.0 or the Pavement ME Backcalculation Tool be used to perform backcalculation.* The average back-calculated values for a specific design section should

be entered for each pavement layer and subgrade soil. Often there are outliers that should be deleted from the analysis. *Refer to PennDOT FWD Data Collection Procedures for additional guidance on backcalculation of flexible pavements.* These elastic modulus values for each pavement layer and subgrade are discussed in the next chapter of the User Input Guide.

The average total amount of **thermal or transverse cracking** in terms of feet per mile should be entered for the project. In addition, the severity rating of transverse cracking should be selected from three options: low, medium, and high. This is a critical input that affects the rate at which reflection cracks come through the AC overlay. The severity level depends on actual severity of existing cracks. The designer can also use the distress data and information included in PennDOT's RMS database.

The other input required for rehabilitation input level 1 is the **average rut depth** within each pavement layer and subgrade. Since this is difficult to measure, Table 8.1 lists the percentages to be used in distributing the total rut depth measured at the surface to each pavement layer and subgrade.

Table 8.1—Ratios to Distribute Total Rut Depth to Individual Layers

Flexible Pavement Layer	Ratio of Total Rut Depth Distributed to Each Layer[1]
AC	0.75
Granular Aggregate Base	0.10
Subgrade	0.15

[1]A total rut depth of 0.5 in. would have 0.75*0.5 = 0.375 in. rut depth in the AC layer, 0.05 in. in the granular aggregate base, and 0.075 in. in subgrade.

These percentages were determined through the global calibration process under NCHRP projects 1-37A and 1-40D and are based on a limited number of studies at the global and local levels (Colorado, Montana, etc.). The values will be verified based on the local calibration study for PennDOT using the LTPP and non-LTPP roadway segments by determining the values that result in the lowest standard error of the rut depth transfer function.

An evaluation of the AC/AC Overlay Design procedure using rehabilitation input Level 1 has shown that this procedure overestimates fatigue cracking in most cases. Additionally, the overlay design thickness is very sensitive to the backcalculated stiffness of the existing asphalt, which can be highly variable. ***The AC/AC Overlay Design procedure using rehabilitation input Level 1 should not be used for overlay type selection or thickness design.***

Notes

An evaluation of the AC/AC Overlay Design procedure using Level 1 inputs, detailed in Appendix C-6, has shown that the overlay design thickness is very sensitive to the backcalculated stiffness of the existing asphalt concrete. Additional investigation, detailed in Appendix C-7, has shown that the backcalculated stiffness of the existing asphalt concrete is highly variable. Thus, it is recommended that the AC/AC Overlay Design procedure using Level 1 inputs **not be used for design purposes.** If the procedure is used with Level 1 inputs, adjustment factors have been developed for the backcalculated stiffness of the asphalt concrete, which is the critical input. The development and use of these adjustment factors is detailed in Appendix C-8.

Section 8.1.5 – Rehabilitation: Condition of existing flexible pavement, Rehabilitation input level 2

Recommended Edits

If deflection testing and data are unavailable to estimate the in-place condition of the AC layers, the use of input level 2 is reasonable without significantly increasing the cost of the pavement evaluation. For input level 2, three inputs are required to determine the condition of the existing pavement layers. These inputs are listed and defined below.

1. The average total amount of **fatigue or alligator cracking** within the wheel path area in terms of percent of total lane area should be entered for the project. In addition, the severity rating of alligator cracking should be selected from three options: low, medium, and high. The designer can also use the distress data and information included in PennDOT's RMS database.
2. The average total amount of **thermal or transverse cracking** and severity, which is the same as for rehabilitation input level 1, as defined above.
3. The **average rut depth** within each pavement layer and subgrade, which is the same as for rehabilitation input level 1, as defined above.

An evaluation of the AC/AC Overlay Design procedure using rehabilitation input Level 2 for existing asphalt has shown that this procedure underestimates fatigue cracking in most cases. As a result, fatigue cracking predicted by the AC/AC Overlay Design using Rehabilitation Input

*Level 2 should be **used with caution** for overlay type selection or in designing the overlay thickness.*

Notes

See Appendix C-6 for details on the evaluation of the AC/AC Overlay Design procedure using Level 2 inputs.

Section 8.1.5 – Rehabilitation: Condition of existing flexible pavement, Rehabilitation input level 3

Recommended Edits

Five subjective **pavement ratings (structural and environmental)** are used to describe the condition of the pavement surface. They are defined in the MEPDG Manual of Practice (ARA Inc. 2015) and considered appropriate for PennDOT. Table 8.2 relates the subjective structural condition survey ratings included in the Pavement ME software to the percent of fatigue or alligator cracking (all levels) of total lane area. Table 8.3 relates the subjective environmental condition survey ratings included in the Pavement ME software to feet per mile thermal or transverse cracking (all levels). Select appropriate condition rating in the software.

Table 8.2—Pavement ME Level 3 Condition Ratings Related to Existing Alligator Cracking in Percent Lane Area

Structural Rating	Existing Alligator Cracking in Percent Lane Area (All levels of severity)
Excellent	< 5
Good	5-15
Fair	15-35
Poor	35-50
Very Poor	>50

Table 8.3— Pavement ME Level 3 Condition Ratings Related to Existing Transverse Cracking in Feet per Mile

Environmental Rating	Existing Transverse Cracking in Feet per Mile (All levels of severity)
Excellent	< 50
Good	50-150
Fair	150-400
Poor	400-800
Very Poor	>800

The other input required for input level 3 is the **average total rut depth** measured at the surface of the AC layer. The Pavement ME Design software distributes that total rut depth

measured at the surface to the different layers using the layer percentages determined under the NCHRP Project 1-37A.

An evaluation of the AC/AC Overlay Design procedure using rehabilitation input Level 3 for existing asphalt has shown that this procedure underestimates fatigue cracking in most cases, but to a lesser extent than when using Level 2 inputs. As a result, fatigue cracking predicted by the AC/AC Overlay Design using rehabilitation Level 3 inputs should be ***used with caution*** *for overlay type selection or in designing the overlay thickness.*

Notes

See Appendix C-6 for details on the evaluation of the AC/AC Overlay Design procedure using Level 3 inputs.

Section 8.2.10 – Foundation support for rehabilitation of rigid pavements

Recommended Edits

The foundation support resilient modulus at optimum moisture content and maximum dry unit weight conditions can be estimated based on soil class (Level 3) or California Bearing Ratio (CBR) (Level 2) and entered for rehabilitation design similar to the design of new jointed and continuous concrete pavements. Recommendations for estimating the subgrade resilient modulus is provided in Section 9.6.2.

For rehabilitation design, it is far more accurate to measure the subgrade dynamic k-value along the project, and enter it directly into the Pavement ME Design software for the month tested. The subgrade dynamic k-value can be measured through deflection testing on top of existing slab. This process is by far the most accurate approach that gives subgrade support along the project.

- The project dynamic k-value is determined by back-calculation from deflections from FWD deflection testing.
- Note that the “dynamic” k-value is approximately twice the “static” k-value, which is the input used in the AASHTO 1993 (AASHTO 1993) design procedure . Thus, a static k-value of, for example, 100 psi/in used in the old AASHTO procedure for the subgrade would represent a dynamic k-value of 200 psi/in.
- ~~Procedures to calculate the project subgrade dynamic k-value are found in Section 9.6.2.~~
- *The dynamic k-value should be calculated using Pitt-FACS.*

Notes

Instructions for using Pitt-FACS for backcalculating the modulus of subgrade reaction can be found in Appendix B-1. Pitt-FACS will perform backcalculation using the AREA method for tests in the center of the slab, which were performed with an equivalent linear temperature gradient less than 0.5 ºF/in. No backcalculation results will be reported if ELTG>0.5 ºF/in during testing. Suggested testing times to ensure ELTG<0.5 ºF/in can be seen in Appendix A-1. The analysis relating to the development of the backcalculation procedure within Pitt-FACS and to establishing the ELTG restriction can be found in Appendix C-2.

Section 8.2.11 – Condition of existing PCC surface for JPCP rehabilitation design

Recommended Edits

These inputs describe the amount of cracking and slab repairs of the existing PCC slabs and any repairs previously made to the JPCP and the transverse joint load transfer efficiency. Two inputs are required for the existing PCC layer when designing an AC overlay of an existing JPCP or for restoration (e.g., diamond grinding, slab replacement, etc.):

1. Percentage of slabs that are transversely cracked or have been replaced before rehabilitation. This input could range from 0 to over 20 percent.
2. Percentage of slabs that will be replaced as part of the rehabilitation project. This could range from 0 up to the percent cracked before rehabilitation. (Note: The Pavement ME software input text for this input has an error. This input is defined as the percentage of cracked slabs or replaced slabs replaced during rehabilitation).

The following examples explain the inputs and results achieved:

- If 0 percent slabs cracked prior to rehab and 0 percent slabs are replaced as a part of rehabilitation, then the future prediction will start at 0 percent slabs cracked. The inputs for this example are thus 0 percent (before) and 0 percent (during) restoration.
- If 10 percent slabs are cracked prior to rehab and 0 percent slabs are replaced as a part of rehabilitation, then the future prediction will start at 10 percent. The inputs for this example are thus 10 percent (before) and 0 percent (during) restoration.
- If 10 percent slabs are cracked prior to rehab and 3 percent slabs are replaced as a part of rehabilitation, then the future prediction will start at 7 percent slabs cracked. The inputs for this example are thus 10 percent (before) and 3 percent (during) restoration.

These two inputs are important because they define the in-place fatigue damage of the JPCP which is used to predict future damage and cracking of the PCC slabs.

The transverse joint load transfer efficiency (LTE) input is used in the AC overlay reflection cracking prediction. The joint LTE can range from less than 25 percent (very poor) to above 80 percent (very good). ~~The LTE can be measured at a representative number of joints using the FWD in the outer wheel path of the slab in cooler weather when the air temperature is 80°F or less. If FWD testing is not possible, then the following guidelines are provided:~~ *Prior to the design of an AC overlay, FWD testing should be performed at all the joints, and the corrected LTE should be determined using Pitt-FACS. If poor LTE (less than 65%) occurs at a doweled joint, the LTE should be restored at these joints by performing dowel bar retrofits prior to the construction of the overlay.*

The following values for LTE (Level 3) should be used when designing AC overlays of PCC pavements.

- Doweled joint: 70 percent
- Non-doweled joint with stabilized base course: 50 percent
- Non-doweled joint with granular base course: 30 percent

Notes

An analysis of the effect of curling and warping on the measured joint performance, along with a procedure for adjusting the results can be seen in Appendix C-3. A sensitivity analysis on the effect of the joint LTE on the predicted transverse cracking of the overlay can also be found in Appendix C-3. The AC/JPCP module in Pavement ME is extremely sensitive to the measured LTE. However, neither the documentation for developing the reflective cracking model (Lytton et al. 2010), or implementing the model into Pavement ME (Titus-Glover et al. 2016), mention using measured LTE in the calibration. Therefore, it is recommended that Level 3 values be used to avoid predictions of unrealistically long pavement lives.

Section 9.1 – Pavement layers for flexible pavement design, AC and asphalt stabilized base layers, For rehabilitation

Recommended Edits

For rehabilitation, the existing AC and overlay layers are restricted to four layers~~. When two layers are entered to represent the existing AC, only two overlay layers can be used. Conversely, if three~~

~~overlay layers are entered, only one layer can be used to represent the existing AC layers. Results from deflection basin testing and the backcalculation of elastic layer modulus values should be used to determine whether the existing AC layers are confined to one or two layers.~~ *If the stiffness of the existing layers is determined using FWD testing, all existing bituminous layers, including wearing, binder, and base courses, open-graded friction courses, asphalt-treated permeable base, microsurfacing, and chip seals should be combined as one existing AC layer in design. Layers that will be removed through milling prior to the placement of the overlay should not be included in the existing asphalt layer. If backcalculation is not used to determine the stiffness of the asphalt layers, the layers should be combined logically such that the total number of asphalt layers, including the overlay, is less than or equal to 4.*

~~**NOTE 12** For rehabilitation, it is recommended that the existing AC layers be combined as one layer, unless there is a specific reason why two layers should be simulated.~~

Notes

Backcalculation of the stiffness of multiple asphalt stabilized layers is generally not accurate.

Section 9.3.1 – Mixture volumetric properties

Recommended Edits

The volumetric properties should represent the mixture after compaction at the completion of construction. Obviously, the project-specific values will be unavailable to the designer because the project has yet to be built. These parameters should be available from previous construction records and can be analyzed to determine typical values for inputs. The following summarizes the recommended input parameters for AC mixtures.

NOTE 13 Pavement ME uses Effective Asphalt Content by Volume while PennDOT collects Effective Asphalt Content by Weight.

Air voids, effective asphalt content by volume, and unit weight:

- ***New AC Mixtures****: Use the average values from historical construction records for a particular type of AC mixture. Table 9.3 includes the volumetric properties based on the target values for common AC mixtures used in Pennsylvania.*

Table 9.3—Volumetric Properties for Pennsylvania's Dense-Graded Mixtures

Volumetric Property	Superpave Mixtures					
	Surface Mixtures		Binder		Base	
	9.5 mm	12.5 mm	19 mm	25 mm	25 mm	37.5 mm
Average Air Voids, %	7.0	7.1	6.97	6.3	7.07	7.4
Effective Asphalt Content by Volume, %	11.98	10.45	9.87	9.2	9.47	8.6
Density, pcf	148.5	156.5	150.5	150.0	150.0	150.0

The following volumetric equations can be used to estimate the input parameters.

Air Voids, V_a:

$$V_a = \left(1 - \frac{G_{mb}}{G_{mm}}\right) \times 100$$

~~**Voids in Mineral Aggregate, VMA**~~:

~~$$VMA = 100 - \left(\frac{G_{mb}(P_s)}{G_{se}}\right)$$~~

Voids in Mineral Aggregate, VMA:

$$VMA = 100 - \left(\frac{G_{mb}(P_s)}{G_{sb}}\right)$$

Effective Asphalt Content by Volume, V_{be}:

$$V_{be} = VMA - V_a$$

Where:
V_a = Air voids (%)
VMA = Voids in mineral aggregate (%)
V_{be} = Effective asphalt content by volume (%)
G_{mb} = Bulk specific gravity of the AC mixture
G_{mm} = Maximum theoretical specific gravity of the AC mixture
~~G_{se} = Effective specific gravity of the combined aggregate blend~~
G_{sb} = Bulk specific gravity of the combined aggregate blend
P_s = Percentage of aggregate in mix by weight (%) (P_s=100-P_b)

- ***Existing AC Mixtures****: Laboratory testing should be performed on cores to determine the volumetric properties of the existing asphalt when Level 1 inputs are used. All existing bituminous layers are combined for rehabilitation design, so each volumetric property input should be a weighted average of all existing bituminous layers. See PennDOT FWD Data Collection Procedures for additional guidance on coring and determining the volumetric properties of existing asphalt. For input Level 2 or 3, mix volumetric inputs of*

the existing asphalt can be estimated using information from previously performed laboratory testing on cores that are representative of the existing asphalt. Alternatively, mix volumetric inputs for input level 2 or 3 can be estimated using values from Table 9.3.

Notes

As noted in Section 8.1.5, it is recommended that the AC/AC Overlay Design procedure using Level 1 inputs **should not be used for design purposes.**

An error was fixed in the VMA equation. G_{se} was changed to G_{sb}. Guidelines for determining the volumetrics of existing asphalt from cores can be found in Appendix B-5.

Section 9.3.2 – Mechanical properties

Recommended Edits

Existing AC Mixtures: For rehabilitation design of flexible pavements, the dynamic modulus of the existing AC layers is needed. ~~For rehabilitation input levels 2 and 3, the dynamic modulus inputs are the same as for new AC mixtures discussed above. For rehabilitation input level 1, the dynamic modulus values represent the backcalculated elastic modulus values.~~

~~Deflection basins should be measured over a range of temperatures, even if the deflection testing is completed within the same day so that the backcalculated elastic layer modulus values can be determined for at least two temperatures: one representing the morning hours and one representing the late afternoon hours. If there is no significant difference between the back-calculated elastic modulus values, one average value can be used.~~

~~Two other inputs are needed: (1) the frequency of deflection testing – a default value of 20 Hz is recommended to represent the FWD and (2) the temperature representative of the average backcalculated elastic modulus value – the mid-depth temperature of the layer used in the backcalculation process measured during deflection testing.~~

For rehabilitation input Level 1, the dynamic modulus of the existing AC layers is defined using the volumetric properties and aggregate gradation of the existing AC layers, the backcalculated stiffness of the existing AC layers, the load frequency of the FWD, and the temperature of the existing AC layers at the time of FWD testing. The volumetric properties and aggregate gradation of the existing AC layers can be determined by coring and performing laboratory testing, as detailed in Section 9.3.1. Note that the same cores used to determine the

volumetric properties can be used to determine gradation inputs. The backcalculated stiffness of the existing AC layers should be determined using the guidelines for FWD testing of flexible pavements found in PennDOT FWD Data Collection Procedures.

An evaluation of the AC/AC Overlay Design procedure using rehabilitation input Level 1 has shown that the overlay design thickness is very sensitive to the backcalculated stiffness of the existing asphalt, which can be highly variable. ***The AC/AC Overlay Design procedure using rehabilitation input Level 1 should not be used for overlay type selection or thickness design.***

For rehabilitation input levels 2 and 3, only the aggregate gradation is needed to establish the dynamic modulus. The aggregate gradation can be estimated using information from previously performed laboratory testing on cores that are representative of the existing asphalt. Typical values from Table 9.6 can be used is this information is not available.

Notes

As noted in Section 8.1.5, it is recommended that the AC/AC Overlay Design procedure using Level 1 inputs **should not be used for design purposes.**

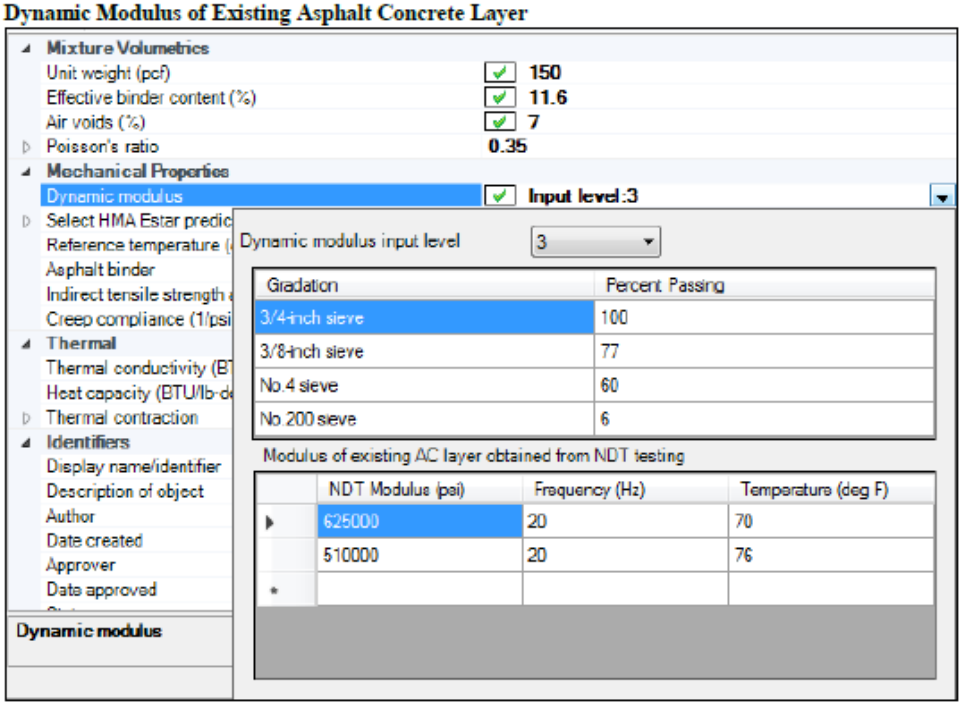
Additionally, analysis of LTPP data showed that performing multiple tests in the same location over the course of the day does not improve the accuracy of distress predictions. Therefore, testing multiple times in the same day is not required. Details of this analysis can be found in Appendix C-6.

Section 9.3.4 – Screenshots for the AC properties: New and existing layers

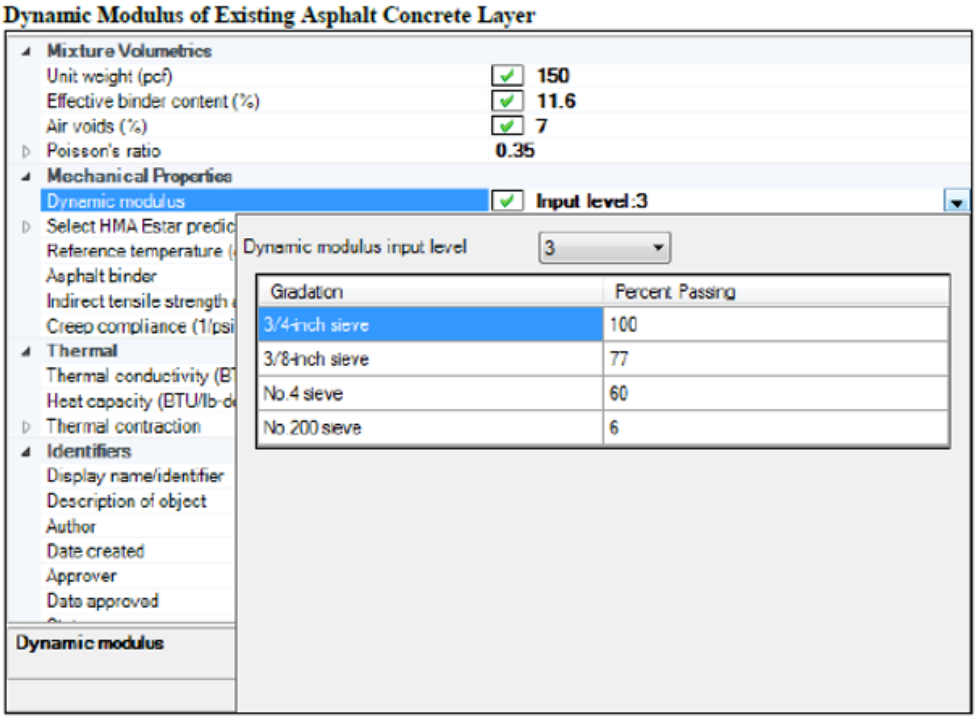
Recommended Edits

Image to change:

Dynamic Modulus of Existing Asphalt Concrete Layer



Revised image:



Notes

As noted in Section 8.1.5, it is recommended that the AC/AC Overlay Design procedure not be used with Level 1 inputs. The image has been revised to remove Level 1 input fields.

Section 9.5.1 – AC or PCC overlay of existing intact PCC slabs

Recommended Edits

Existing intact PCC properties are required for AC overlay, restoration, and for unbonded PCC overlay. Example screen shots showing the PCC material property inputs are included at the end of this section. The PCC properties are the same as for new PCC mixes with the following exceptions.

The modulus of elasticity of the existing PCC slab is determined through an assessment of the amount of slab cracking that will not be repaired (include all types: longitudinal, transverse, corner, diagonal). An effective (or damaged) modulus of elasticity value is estimated as follows:

- If the percent of cracked slabs is less than 10 percent, the effective modulus of elasticity is the same as that of the intact slab. There is no modulus reduction. See note below if the PCC slab elastic modulus is back-calculated from FWD deflections for reduction factor.
- If the percent of cracked slabs is 10 percent or greater, the effective modulus is selected from Table 9.12.

Table 9.12—Recommended Effective Modulus Values for Existing Intact PCC Slabs

Qualitative Description of Pavement Condition	Typical Modulus Ranges, psi	Default Modulus, psi
Good/Adequate (10 to 20 percent cracked slabs)	2 to 4 x 10^6	3.0 x 10^6
Marginal (20 to 50 percent cracked slabs)	1 to 2 x 10^6	1.6 x 10^6
Poor/inadequate (>50 percent cracked slabs)	0.2 to 1 x 10^6	0.65 x 10^6

Note that for FWD backcalculation of PCC slab elastic modulus for uncracked slabs, the resulting modulus value is essentially a dynamic value that must be reduced by multiplying by 0.8 to obtain a static uncracked value to input into the Pavement ME.

Pavement ME is now able to calculate the amount of AC overlay reflection cracking over time that emanates from transverse joints and transverse cracks.

- AC total transverse cracking: thermal plus reflective (feet/mile).
- The thermal cracking is from low temperature stresses (not joint or crack).
- The reflective cracking is from transverse joints plus transverse cracking.
- Thus, a JPCP with 15-foot joint spacing has a total of 4,224-feet of transverse joint length. If the input joint LTE is low, reflection cracking will occur through all of the transverse joints very rapidly. If aggressive maintenance is accomplished, these cracks may survive for several years before deteriorating into potholes and roughness.

The program requires the transverse joint LTE. LTE can be measured using an FWD when the air temperature is less than ~~80°F~~ *75ºF*. The ~~level~~ *LTE* depends heavily on the presence of dowel bars at the joint. *Pitt-FACS can be used to assist in interpreting the FWD data to establish the LTE for doweled pavement*s. If FWD testing cannot be performed, the following can be used as the default LTE.

- No dowel bars, granular base: 30 percent LTE.
- No dowel bars, stabilized base: 50 percent LTE.
- Dowel bars exist: 70 percent LTE.

Notes

An analysis of FWD data from the LTPP database shows that testing when pavement temperatures are greater than 75ºF leads to inaccuracies in the estimated joint performance. Details of this analysis are provided in Appendix C-3. Instructions on the use of Pitt-FACS can be found in Appendix B-1.

Section 9.5.3 – Restoration of JPCP

Recommended Edits

The restoration of a JPCP may include any of the following treatments, depending on the condition of the existing pavement.

- Diamond grinding for joint faulting and other unevenness that may exist is always required. Restoration cannot be run without grinding.
- Slab replacement and partial slab replacement for slab cracking or joint deterioration.
- Spall repair for joint spalling and deterioration.
- Tied PCC shoulder to increase structural capacity of the outer lane.
- Dowel bar retrofit for undoweled faulted JPCP.

Most of these projects are not "designed" using any structural procedure, but are based on applying a repair treatment to an existing JPCP that has various distresses and roughness. Pavement ME provides the ability to check the structural capacity of the restored pavement to handle future traffic loads. Pavement ME also provides the ability to predict the future service life of the restored pavement after various treatments. For example, if after slab replacement and diamond grinding a restored JPCP develops significant fatigue transverse cracking within 10 years, then this may not be a good candidate for restoration. Or, if a JPCP develops significant faulting within a few years, then retrofit dowels may be required.

The design of a restored JPCP requires the same inputs as a new JPCP design with the following exceptions.

- The percent slabs cracked prior to restoration and percent cracked slabs replaced during restoration as described in Section 8.2.11 are required inputs. This affects the future amount of fatigue transverse cracking predicted initially and into the future.
- The modulus of elasticity of the PCC slab at the time of restoration must be determined. This can be done through coring and running a compressive strength that can be used to

estimate the modulus of elasticity. The modulus can also be estimated through FWD testing and back-calculation to obtain a "dynamic" E value. This is then multiplied by a factor of 0.8 to adjust to a static E value. Note: the modulus of elasticity of an old, intact PCC slab should always be greater than 4 million psi. This should be the minimum input value. *The backcalculated elastic modulus of the PCC slab can also be calculated using the Pitt-FACS web application. When the elastic modulus of the slab is critical to the design thickness, such as bonded concrete overlays of concrete pavements, it is recommended that the elastic modulus and compressive strength of the existing concrete be determined from laboratory testing of cores using ASTM C469 and ASTM C39, respectively, due to the variability inherent with backcalculating the elastic modulus of the existing concrete.*

- If future transverse fatigue crack prediction is significant, a tied PCC shoulder can be included to reduce future cracking.
- If the JPCP has no dowels, then this must be entered into the Pavement ME. If future faulting is severe, then retrofit dowels of proper size can be entered into the program and the future faulting observed.
- The expected initial IRI must be input after diamond grinding. This value may be higher than traditionally achieved on new construction due to subgrade movement over the years which diamond grinding cannot totally remove. A typical IRI after diamond grinding is 50 percent of the existing IRI. If the existing IRI = 140 in/mile, after grinding the IRI may be about 0.5*140 = 70 in/mile.
- *The dynamic modulus of subgrade reaction can be calculated using the Pitt-FACS web application.*

Notes

The concrete pavement restoration design is based on the calibrated transfer functions for new concrete pavements and has not been calibrated based on the inputs specific to CPR design. This module may be useful as a guide but is not a substitute for sound engineering judgement in determining whether an overlay is required as part of a rehabilitation. In addition, there is no straightforward way of accounting for the condition of joints in this module.

Section 9.5.4 – PCC overlay of existing flexible AC pavement

Recommended Edits

This section addressed the overlay design of a JPCP overlay over an existing flexible pavement (JPCP over existing AC). The key aspects of this design are as follows:

- The material inputs and design inputs are similar to that of new JPCP design. The Pavement ME has some limitations including longitudinal joint spacing of 12-foot minimum (6-foot by 6-foot slabs cannot be designed currently), transverse joint spacing of 10 to 20 feet, and slab thickness of a minimum of 6 inches. ~~The condition and damaged modulus of the existing AC layers is critical and must be assessed using either Levels 1, 2, or 3 as described in Section 8.2.11.~~
- *The condition and damaged modulus of the existing AC layers must be assessed using either level 1, 2, or 3 as described in Sections 8.1.5, 9.1, and 9.3. A sensitivity analysis of the JPCP over AC overlay design procedure has found that distress predicted by the JPCP over AC overlay design procedure is insensitive to the condition and damaged modulus of the existing AC layers, regardless of the input level. It is recommended that Level 2 or 3 inputs be used to define the condition of the existing AC layers.*
- The friction between the new JPCP overlay and the existing AC layer is critical to the success of the overlay.
    - Milling of the existing surface is recommended to level up the existing surface so that the PCC slab can be placed with uniform thickness to provide a smooth surface *as long as this does not result in milling across asphalt layers. Otherwise the cross-slope corrections should be addressed in the concrete layer*.
    - Milling of the existing surface is recommended to achieve a strong bond and friction between the existing AC layer and the PCC overlay. This bond/friction is essential for joint formation and for good structural performance of a composite slab/AC layers. Enter the "PCC-Base Contact Friction, Months Before Friction Loss" as the full design life of the JPCP overlay.
- The subgrade is modeled using a resilient modulus. The resilient modulus can be best estimated from back-calculation (level 1), but also from estimation from subgrade soil testing (level 2) or soil classification (level 3) as described in Section 9.6.2.

- *For bonded JPCP overlays of asphalt pavements, overlays less than 6.5 in thick, the overlay should be designed using the BCOA-ME design procedure, which can be found at http://www.engineering.pitt.edu/Vandenbossche/BCOA-ME/. In this procedure, the condition of the existing AC pavement is determined based on the condition of the asphalt roadway using information from a field condition survey. FWD testing is not required. This design procedure allows overlays to be designed with panels less than 1 lane width (e.g. 4 feet x 4 feet, 6 feet x 6 feet) and with overlays less than 6 inches thick.*

Notes

The sensitivity analysis of the JPCP/AC overlay design procedure can be found in Appendix C-10.

Section 9.6.2 – Resilient modulus, Level 2 FWD testing, backcalculation, and adjustment for flexible pavement

Recommended Edits

FWD testing can be conducted along the rehabilitation project and the resulting elastic modulus at each point determined through backcalculation. The mean resilient modulus for each layer is then computed by deleting any major outliers, following which the mean layer values are adjusted to lab conditions at optimum moisture and density for each unbound base and subgrade layer. Table 9.16 lists the adjustment ratios that should be applied to the unbound layers for use in design. More importantly, the in-place water content and dry density need to be entered in the Pavement ME Design software when the in-place resilient modulus values are used.

Table 9.16—Summary of the Adjustment Factors Recommended for Use in Pennsylvania to Convert Backcalculated Layer Modulus Values to Laboratory Equivalent Modulus Values

Layer & Material Type	Layer Description	Adjustment Factor, (M_R/E) FHWA Pamphlet
Aggregate Base Layers	Granular base under a PCC surface	1.32
	Granular base above a stabilized material (a Sandwich Section)	1.43
	Granular base under an AC surface or base	0.62
Subgrade Soil/Foundation	Soil under a stabilized subgrade	0.75
	Soil under a full-depth AC pavement	0.52
	Soil under flexible pavement with a granular base/subbase	0.35
~~AC Mixtures~~	~~AC surface and base layers, 41 °F~~	~~1.00~~
	~~AC surface and base layers, 77 °F~~	~~0.36~~
	~~AC surface and base layers, 104 °F~~	~~0.25~~

Notes

As noted in Section 8.1.5, it is recommended that the AC/AC Overlay Design procedure using Level 1 inputs **not be used for design purposes.** If Level 1 inputs are used, it is recommended that the backcalculated asphalt stiffness adjustment factors in Table 9.16 be replaced by the adjustment factors described in Appendix C-8.

The adjustment factors presented in Table 9.16 do not affect the accuracy of predicted fatigue cracking when used with the AC/AC overlay design procedure with Level 1 inputs. The adjustment factors do, however, increase the amount of predicted total rutting for sections having an unadjusted backcalculated granular base layer stiffness less than 15 ksi and/or an unadjusted backcalculated subgrade layer stiffness less than 25 ksi. The adjustment factors should be used with caution in these cases. More details on the evaluation of the adjustment factors in Table 9.16 can be found in Appendix C-6.

Section 9.6.2 – Resilient modulus, Level 2 Dynamic cone penetrometer (DCP) or CBR testing

Recommended Edits

Level 2 DCP testing. PennDOT uses DCP for pavement evaluations and in estimating the resilient modulus of the unbound materials and soils. *The following equations can be used to estimate the resilient modulus using the dynamic cone penetration rate (DPI).* ~~Equation 6 can used to calculate the resilient modulus from the penetration rate measured with the DCP. It is suggested that the DCP be considered for future use for rehabilitation design for the unbound pavement layers and subgrade, especially when FWD deflection basin data are unavailable.~~

~~$$M_R = 17.6\left(\frac{292}{(DPI)^{1.12}}\right)^{0.64}(C_{DCP}) \quad (6)$$~~

~~Where:~~
~~M_R = Resilient modulus of unbound material, MP$_a$.~~
~~*DPI* = Penetration rate or index, mm/blow.~~
~~C_{DCP} = Adjustment factor for converting the elastic modulus to a laboratory resilient modulus value.~~

Convert Dynamic Penetration Index (DPI) to CBR

For high-plasticity clay soils (CH) (Webster et al. 1994):

$$CBR = \frac{1}{(0.07292 \times DPI)}$$

For low-plasticity clay soils (CL) (CBR < 10)) (Webster et al. 1994):

$$CBR = \frac{1}{(0.43228 \times DPI)^2}$$

For all other soils (Webster et al. 1992):

$$log(CBR) = 2.46 - 1.12 \times (\log(25.4 \times DPI))$$

Where:
DPI = Dynamic penetration index (in/blow)
CBR = California bearing ratio (%)

Note: The DPI to CBR equations presented are calibrated for the DCP configured with the standard-length drive rod. If a drive rod extension is used, these equations should be used with caution.

Convert CBR to M_r

$$M_r = 2555\ (CBR)^{0.64}$$

Where:
CBR = California bearing ratio (%)
M_r = Resilient modulus (psi)

The resilient modulus estimated using the equations above must be adjusted to laboratory conditions using an adjustment factor. ~~The subgrade resilient modulus can be estimated (level 2) from the DCP tests using equation 6, but those values need to be adjusted to laboratory conditions.~~ Table 9.17 provides the adjustment factors recommended for use in estimating resilient modulus from the DCP penetration rate. (It should be noted and understood that the Pavement ME Design does not adjust the resilient modulus values calculated from the DCP, and the values in Table 9.17 have not been field-verified for PennDOT*). The adjustment factor should be applied using the equation below.*

$$M_{r,adjusted} = M_r \times C_{DCP}$$

Where:
M_r = Resilient modulus (psi)
C_{DCP} = Resilient modulus adjustment factor (Table 9.17)

Table 9.17—Resilient Modulus Values Derived for Subgrade Soil from DCP Tests for Use in Pennsylvania

Material/Soil Type		Condition	Adjustment Factor, C_{DCP}
Fine-Grained; Low Plasticity Soil	Clay-Silt	Above Optimum Water Content	1.90
	Soil-Sand Mix	At or Below Optimum Water Content	1.05
	Soil-Aggregate Mix with Large Aggregate	At or Below Optimum Water Content	0.60
Coarse-Grained Material	Soil-Aggregate Mix	At or Below Optimum Water Content	0.60
	Crushed Aggregate	At or Below Optimum Water Content	1.04

Level 2 CBR testing. The subgrade resilient modulus can also be estimated approximately from the CBR test, which can be entered into the software (level 2). Note that this equation is in the Pavement ME software.

$$\sout{M_r = 2225 \times CBR^{0.64}}$$

$$M_r = 2555 \times CBR^{0.64}$$

Where:
M_r = Resilient modulus (at CBR test specimen moisture content) (psi)
~~CBR = Soaked CBR value, % (AASHTO T193) (Valid for 2–12% water) (Note: Water content on CBR specimen must be entered into the Pavement ME under "Optimum gravimetric water content" input.~~
CBR = Soaked CBR value (calculated using AASHTO T193 (AASHTO 2013) and valid for 2–12% water) (%)

Note: Water content on CBR specimen must be entered into the Pavement ME under "Optimum gravimetric water content" input.

Notes

See Appendix B-2 for additional information on the analysis of DCP data. See Appendix C-9 for more information on the DPI to CBR and CBR to M_r correlations presented in this section and for examples of their use.

An error in the Level 2 CBR testing section was fixed. The equation was changed from $M_r = 2225*CBR^{0.64}$ to $M_r = 2555*CBR^{0.64}$.

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