Methodology for Temperature and Load Compensation in Full-Scale Traffic Tests on Flexible Airport Pavements

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ABSTRACT: A methodology is described that is used to compensate for changes in wheel load and asphalt material temperature during full-scale traffic tests on flexible airport pavements. The method is based on a known failure and structural model describing failure due to rutting in the subgrade of a flexible pavement and the assumption that the pavement structure deteriorates according to the cumulative damage factor (CDF) relationship (Miner's Rule). The test results are separated into portions during which the temperature is approximately constant and the wheel load is constant. The CDF for each portion of the test results is computed with the failure model fixed. But, by definition, CDF = 1.0 at failure and the failure model is allowed to move vertically until the CDF at failure = 1.0. The method has been applied to results from full-scale traffic tests run at the Federal Aviation Administration (FAA) National Airport Pavement Test Facility (NAPTF). Two examples are given. The first is a comparison of the performance of a conventional structure with the performance of an equivalent structure having an asphalt stabilized base and where the wheel loads are the same in both cases, but the temperature, and therefore the stiffnesses, of the asphalt layers changed during the tests. The number of passes to failure is also normalized to a standard temperature. The second example is normalization of the number of passes to failure to a standard load for a test in which the wheel loads varied over a range of 22.68 to 31.75 metric tons (MT) (50,000 to 70,000 lb). The test objective was to compare the performance of a six-wheel gear with that of a ten-wheel gear on the same structure. The compensation procedure allowed the results for both gears to be rationally compared, even though both experienced different loading histories. The first example assumes a linear failure model, allowing an analytic solution, and the second example assumes a non-linear failure model, requiring a numerical iterative solution procedure.

KEY WORDS: Airport, pavement, full-scale, testing, NAPTF.

1 INTRODUCTION

The response to loading and the structural life of flexible airport pavements are strongly dependent on the stiffness of the asphalt layers. Stiffness is, in turn, a strong function of the temperature of the asphalt layers. During structural testing of flexible airport pavements, it is usual for the temperature of the asphalt layers to change during testing if the structural life of the pavement is greater than a few hundred load applications (passes). Also, when comparing the results from different test series, the pavement temperature is rarely the same during each series, particularly when the tests are run at different test facilities. In particular, most of the test results referenced in this paper were obtained at the FAA National Airport Pavement Test Facility (NAPTF), a facility which is too large for affordable climate control. Therefore, a standardized method is required to compensate for the effects of varying temperature during a test and to reduce the combined results to a standard temperature so that the results from different tests can be compared on a rational basis.

A procedure was developed that is based on a known failure and structural model describing failure due to rutting in the subgrade of a flexible pavement and the assumption that the pavement structure deteriorates according to the cumulative damage factor (CDF) relationship (Miner's Rule). In addition, the method can be applied to any system variable that varies during a test series. In particular, full-scale tests can require changes in the magnitude of the applied loads if the structural life of the pavement is underestimated during the design of the pavement structure or as a result of assumed material properties not being properly achieved during construction.

After describing the general procedure, two examples are given. The first example is for varying temperatures during tests on two different structures and uses a linear vertical strain versus load coverages failure model. The linear model allows for an analytic solution that clearly illustrates the application of the method. The second example is for two different load configurations with mixed traffic loading and uses a nonlinear failure model. The nonlinear model is of a form expected to be used in future FAA flexible airport pavement thickness design procedures and requires an iterative numerical solution.

2 OUTLINE OF THE PROCEDURE

In this paper, the method is applied specifically to structural, traffic, and failure models as implemented in the FAA's airport pavement thickness design computer program FAARFIELD (FAA 2009a and 2009b). The details of the structural and traffic models are not relevant to this paper except to know that a layered elastic model is used to compute flexible pavement responses, and the traffic model converts the number of load applications to coverages through geometric transformations. Details of the failure model are provided to illustrate its use in compensation. (An outline of the development to date of the FAA's computer programs for airport pavement design and evaluation can be found in (FAA 2012).)

Figure 1 shows a scatter chart of the flexible airport pavement full-scale test data available for which the initial mode of failure was determined to have been shear flow in the subgrade. The vertical axis is the maximum vertical strain at the top of the subgrade computed by FAARFIELD for the test structure under the test load configuration. The horizontal axis is the number of coverages to failure, where failure is defined as the point at which upheaval outside the wheel-track is greater than 25 mm (1 inch), signifying deep structural failure in one or more of the unbound layers, usually targeted to be in the subgrade. Both axes are plotted on logarithmic scales to base 10. The upper curve is a least squares linear regression line for all data points. The bi-linear line below the regression line (square blue markers) is the failure model for flexible pavement design as currently implemented in FAARFIELD and adopted from LEDFAA 1.3. The curved line (square black markers) is a prototype of a nonlinear model expected to be implemented in the next release of FAARFIELD. The shape of the curve is compatible with the shape of the four- and six-wheel alpha factor curves developed for use in the FAA's procedure for calculating Pavement Classification Number (PCN) by a standardized method (FAA 2011a).

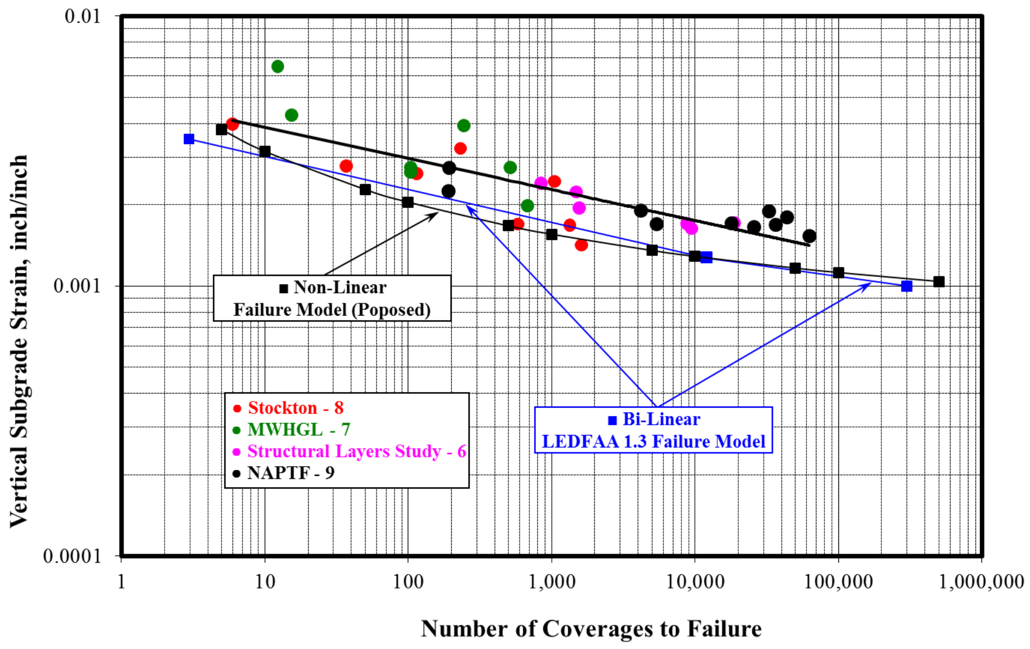

Figure 1: Full-scale subgrade failure test data with FAARFIELD failure models plotted on the chart. Data up to NAPTF CC3 is included.

The steps in the compensation procedure for an arbitrary failure model are listed below.

1. Assume that the shape of the failure model is correct, but that it can be moved vertically on the scatter plot to match the failure point of a particular full-scale test.
2. Separate the coverages of the particular full-scale test into regions where the asphalt temperature and the applied load are simultaneously approximately constant.
3. For each region, compute a value of asphalt modulus using a suitable relationship between modulus and temperature, or establish a value of asphalt modulus from measurements made during the testing.
4. Compute the maximum vertical strain at the top of the subgrade for each region using the applied load and asphalt modulus established for each region.
5. From the failure model, compute the number of coverages to failure for each region corresponding to the maximum strains calculated in step 4.
6. Compute:

$$CDF_T = \frac{C_1}{C_{1F}} + \frac{C_2}{C_{2F}} + \ldots \frac{C_I}{C_{IF}} \ldots + \frac{C_N}{C_{NF}}$$

Where : C_I = the number of coverages applied in region I

C_{IF} = the number of coverages to failure computed for region I

$\frac{C_I}{C_{IF}}$ = the cumulative damage factor, CDF_I, for region I

CDF_T = the total CDF for the complete test to failure

7. In terms of cumulative damage factor, the definition of failure is that CDF_T = 1.0. Therefore, if the computed value of CDF_T is not equal to 1.0, adjust the vertical position of the failure model and repeat steps 5 and 6 to make CDF_T = 1.0.

To this point in the process, the *CDF* has been found for each region, with the failure model adjusted to satisfy the definition of failure in terms of Miner's Rule. But the individual *CDF*s for each region are expressed in terms of the temperature or asphalt stiffness and the applied load for each of the regions, and it is still not possible to find the number of coverages to failure corresponding to the complete test at some standard condition. This can be done by converting the number of coverages for each region to a standard temperature or asphalt stiffness and applied load by using the final failure curve established during the procedure given above and corresponding to CDF_T = 1.0. The asphalt stiffness used for design in FAARFIELD is 200,000 psi (690 MPa). This corresponds to an asphalt temperature of 92°F (33.3°C) according to the stiffness versus temperature relationship published in (FAA 2009a) and the FAARFIELD help file. This temperature is therefore taken to be the reference temperature for the NAPTF normalization procedure. A reference applied load is also selected if necessary. The following relationship, which states that the damage caused by two different sets of conditions is the same if the *CDF*s for each set of conditions are equal, then holds:

$$\frac{C_I}{C_{IF}} = \frac{C_{IR}}{C_{IFR}}$$

Where C_{IR} is the unknown normalized number of coverages, and C_{IFR} is the number of coverages to failure for region *I* corresponding to the reference conditions of asphalt stiffness and applied load. The number of coverages to failure for the reference condition, C_{IFR}, is found using the strain computed for the reference applied load in each region at the asphalt stiffness corresponding to 92°F (33.3°C). The computed strain is entered into the final failure curve corresponding to CDF_T = 1.0. The total number of coverages normalized to 92°F (33.3°C) and the reference applied load, C_R, is then found with the following relationships:

$$C_{IR} = C_I \frac{C_{IFR}}{C_{IF}}, \text{ and}$$

$$C_R = \sum_{I=1}^{N} C_{IR}$$

3 EXAMPLE WITH LINEAR FAILURE MODEL

By assuming a linear model in the log-log axes, the failure model is expressed as:

$$C_F = \left(\frac{A}{\varepsilon}\right)^B$$

Where:
C_F = coverages to failure
ε = vertical strain at the top of the subgrade
A = vertical offset of the log-log failure model
B = slope of the log-log failure model

Then, following the procedure described in the previous section:

$$
\begin{aligned}
CDF &= \frac{C_1}{C_{1F}} + \frac{C_2}{C_{2F}} + \frac{C_3}{C_{3F}} + \ldots \\
&= \frac{C_1}{A^B}\varepsilon_1^B + \frac{C_2}{A^B}\varepsilon_2^B + \frac{C_3}{A^B}\varepsilon_3^B + \ldots \\
&= \frac{C_1\varepsilon_1^B + C_2\varepsilon_2^B + C_3\varepsilon_3^B + \ldots}{A^B}
\end{aligned}
$$

Therefore, to give a CDF of 1.0

$$A = (\ C_1\varepsilon_1^B + C_2\varepsilon_2^B + C_3\varepsilon_3^B + \ldots)^{\frac{1}{B}}$$

Construction cycle 1 (CC1) at the NAPTF included a conventional flexible pavement test item and a stabilized base test item, both on a medium strength, approximately 9 CBR, clay subgrade. The conventional test item consisted of 127 mm (5 inches) of P-401 asphalt surface course, 203 mm (8 inches) of P-209 crushed aggregate base course, and 305 mm (12 inches) of crushed screenings conforming to the P-154 specification. (See (FAA, 2011b) for complete specifications of the FAA standard construction materials items.) The stabilized base test item consisted of 127 mm (5 inches) of P-401 asphalt surface course, 127 mm (5 inches) of P-401 asphalt base course, and 203 mm (8 inches) of P-209 crushed aggregate subbase course. One half of each test item (designated "north") was trafficked with a six-wheel triple-dual-tandem (3D) Boeing 777 gear configuration and the other half (designated "south") was trafficked with a four-wheel dual-tandem Boeing 747 gear configuration (2D). Wheel loads were 20.4 MT (45,000 lb). The structures were designed to have the same structural lives using a slightly modified version of the computer program LEDFAA available at the time (LEDFAA was the precursor to FAARFIELD). Figure 2 shows the rut depths measured at two positions in each test item (north and south are considered separately) and the temperature of the surface asphalt measured during trafficking. See (Hayhoe, 2004) for more details.

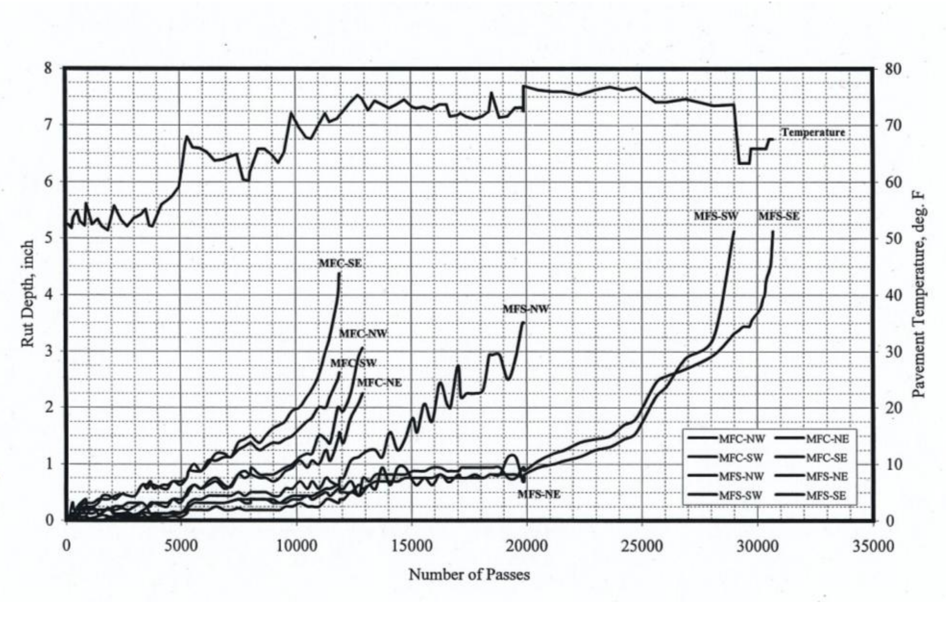

Figure 2: Rut depth and temperature measurements for conventional and stabilized base test items on medium strength subgrade trafficked during construction cycle 1.

The nomenclature in figure 2 for test item identification is: M = medium-strength subgrade, F = flexible, C = conventional, S = stabilized base, N = north traffic lane (triple-dual-tandem), S = south traffic lane (dual-tandem), W and E = rut depth measurement positions on a test item.

The primary objective of the test program was to compare the performance of conventional and stabilized base flexible pavements under four- and six-wheel loading. A secondary objective was to compare the performance of conventional and stabilized base pavements under the same traffic. Posttraffic testing showed that the subbase of test item MFS-North, with six-wheel traffic, failed prematurely. This left only the four-wheel trafficked test items available for comparing the different pavement structures. As can be seen in the figure, the stabilized base pavement sustained about two and one-half times the number of passes before failure as the conventional pavement, at approximately 30,000 passes versus approximately 12,500 passes. However, the temperature history of the two pavement types was very different during trafficking, and the conclusion that the stabilized base pavement performed significantly better than the conventional pavement could not be supported without consideration of the temperature effects.

Tables 1 and 2 show summaries of the measured test results and the calculated normalized number of passes to failure for the two pavement structures. The trafficking was separated into three regions, which were designated to have had constant temperatures of 55°F (13°C), 65°F (18°C), and 75°F (24°C), as indicated in figure 2 and table 1. The transition from 65°F (18°C) to 75°F (24°C) is approximately linear from 9,500 to 12,500 passes. The region from The region from 10,000 to 12,500 passes was considered to be at a constant temperature of 75F to reduce the detail in the presentation of the results. The effect of this approximation on the results is small because, for both test items, the percentage of passes in this region is small compared to the total passes. The value of the coefficient *B* in the linear failure model was set to 6.22 based on the failure model in the then-current version of LEDFAA, $C_F = (A/\varepsilon)^B$ (see page 4 above). The pass-to-coverage ratio is different for the two structures because the LEDFAA pass-to-coverage model is dependent on the total depth of the structure to the top of the subgrade, and the two structures have different total depths. As can be seen, after normalization the total number of passes to failure for the two structures are almost identical (4910 compared to 4541 passes), considering the level of test error to be expected in full-scale testing and the unknown inaccuracies in the assumed shape of the failure model and in the normalization process.

Table 1: Summary of the test results and calculations for the conventional test item rut depth lines MFC-SW and MFC-SE shown on figure 2.

Temperature, °F	Passes	Coverages	Subgrade Strain	Coverages to Failure	CDF
55	5,000	7,500	0.001251	22,800	0.329
65	5,000	7,500	0.001308	17,277	0.434
75	2,000	3,000	0.001375	12,660	0.237
Totals	12,000	18,000			1.000
Normalized at 92	4,910	7,366	0.001500	7,366	1.000

Table 2: Summary of the test results and calculations for the stabilized base test item rut depth lines MFS-SW and MFS-SE shown on figure 2. W and E passes are averaged.

Temperature, ˚F	Passes	Coverages	Subgrade Strain	Coverages to Failure	CDF
55	5,000	6,200	0.001091	156,363	0.0397
65	5,000	6,200	0.001236	71,914	0.0862
75	21,000	26,040	0.001424	29,790	0.8741
Totals	31,000	38,440			1.0000
Normalized at 92	4,541	5,631	0.001861	5,631	1.0000

4 EXAMPLE WITH NON-LINEAR FAILURE MODEL

The second example is taken from construction cycle 5 (CC5) at the NAPTF, for which the primary objective was to measure the performance of six- and ten-wheel gear configurations for comparison with predictions from the layered elastic-based model in FAARFIELD and the alpha factor-based model in the CBR method of design. The CC5 tests were run as a preliminary step in recalibrating the FAARFIELD flexible pavement models. This was done because the layered elastic and alpha factor models give significantly different results as the number of wheels in a gear, or a closely spaced group of gears, increases beyond six. Also, it was not known which model of pavement failure best represents results from full-scale tests, and hence, by extrapolation, expected field performance. The ten-wheel gear configuration was modeled on the Airbus A380 wing and body group of gears but with closer spacing between the gears to accentuate whatever effect the number of wheels might have on the performance of the pavement.

Figure 3 shows the trafficking layout for the six- versus ten-wheel tests. The left side of the figure is the north side of the test pavement, and the right side is the south side. The pavement structure on both sides was a conventional flexible pavement consisting of 127 mm (5 inches) of P-401 asphalt surface course, 203 mm (8 inches) of P-209 crushed aggregate base course, and 834 mm (34 inches) of crushed screenings conforming to the P-154 specification. Wheel group one was trafficked first, with the inner four wheels lifted clear of the pavement when passing over the east end of the test item. Wheel loads were set at 22.7 MT (50,000 lb) at the start of the test, but pavement distress was very slow to accumulate. Therefore, wheel loads were increased in steps to 26.3 MT (58,000 lb), 29.5 MT (65,000 lb), and 31.8 MT (70,000 lb). Failure occurred under the 31.8 MT (70,000 lb) wheel loads. Acceptance testing of the subgrade indicated a strength of 3.0 to 3.5 CBR, which was used for design, whereas posttraffic trenching showed a subgrade strength of 5.0 to 5.5 CBR. Therefore, the pavement was overdesigned for the subgrade strength existing during trafficking.

Figure 4 shows a photograph of the test vehicle loading modules in the ten-wheel group one configuration. The six-wheel group is toward the right of the photograph, and the extent of the wander pattern is shown.

Figure 5 shows the gear configuration used in FAARFIELD to compute the maximum vertical strain at the top of the subgrade for both design and for performing the normalization process. Strain was computed for the left group of ten wheels only. The black dots show the positions of the strain evaluation points. The maximum computed strain over all evaluation points is the strain used for design and normalization.

The trafficking results were normalized using the nonlinear failure model shown in figure 1 and the regions of constant load shown in table 3. An analytic solution could not be found for the non-linear failure model and an iterative procedure had to be used to find the vertical position of the failure curve that gave a total CDF of 1.0. This was implemented using Microsoft® Excel®. Also shown in table 3 are the total passes to failure after normalization to a reference load of 31.8 MT (70,000 lb) and a reference temperature of 92°F (33.3°C). The ratio of six-wheel passes to failure to ten-wheel passes to failure was 1.2 before normalization and 1.8 after normalization. This compares with a ratio of 1.7 predicted by FAARFIELD, and a ratio of 5.1 predicted by the CBR method with an alpha factor traffic model. It should also be noted that the number of passes to failure after normalization is one-fourth to one-third the number of passes to failure before normalization because most damage is done at the high loads, and the lower loads contribute very little to the number of passes to failure for normalized traffic.

Trafficking has now been completed on the south side of CC5 with the configuration shown in figure 3 for wheel group two, except that the transverse spacing of the six- and four-wheel gears was the same as for wheel group one. Additionally, the wheel load was set to 31.8 MT (70,000 lb) from the start of trafficking until failure. The ratio of six-wheel passes to ten-wheel passes was, in this case, 1.2. The results from the south side tests are still being analyzed and reconciled with posttraffic materials tests. Nevertheless, the full-scale tests on CC5 strongly indicate that the FAARFIELD layered elastic-based model provides a better prediction of pavement structural performance in full-scale tests than the CBR method with the current alpha factors for wheel groups with more than six wheels.

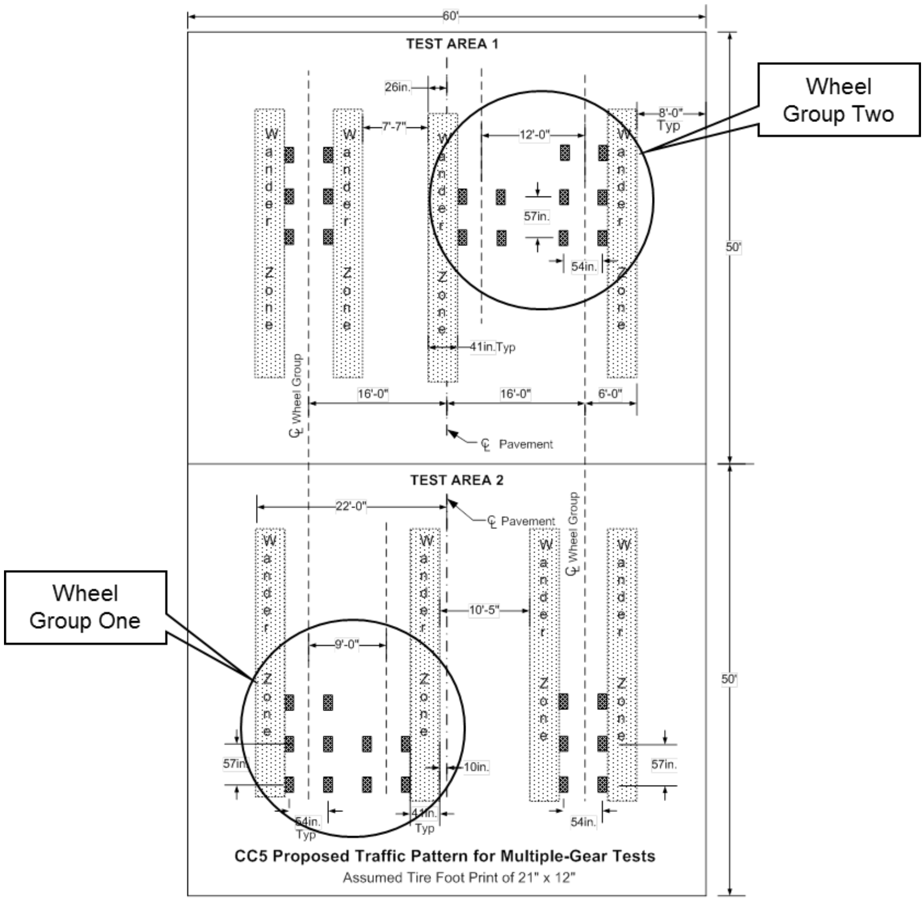

Figure 3: Trafficking layout for the six- versus ten-wheel test on CC5. Wheel groups one and two were tested at different times.

Figure 4: Trafficking the ten-wheel group of wheels on CC5 at 31.8 MT (70,000 lb).

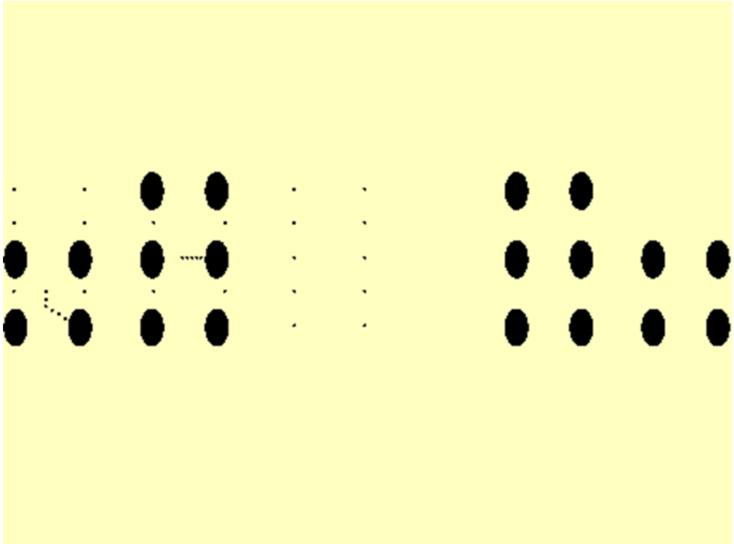

Figure 5: Gear configuration used to compute vertical strain at the top of the subgrade in FAARFIELD.

Table 3: Summary of the applied traffic with mixed loading and the normalized traffic results.

Wheel Load, MT (lb)	Passes Applied with the Ten-Wheel Gear	Passes Applied with the Six-Wheel Gear
22.7 (50,000)	7,917	7,917
26.3 (58,000)	4,954	4,954
29.5 (65,000)	5,621	5,621
31.8 (70,000)	3,778	9,031
Sum, not normalized	22,270	27,523
Sum, normalized to 31.8 MT and 92°F (33.3°C)	6,185	11,432
Ratio six-wheel to ten-wheel, normalized		1.8

5 CONCLUSIONS

A methodology has been described that can be used to compensate for changes in asphalt surface temperature and mixed load application during full-scale structural life testing of flexible airport pavements when failure occurs in the subgrade. The same method can also be used to normalize test results to a set of standard test conditions so that full-scale tests run under different conditions can be compared in a rational manner. The methodology has been used to provide a rational comparison between test results obtained at the NAPTF, for which changing conditions during the testing confounded and confused interpretation of the results without normalization. Application of the methodology has increased the level of confidence when applying the results of the tests to the development of airport pavement thickness design models.

6 ACKNOWLEDGEMENTS

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