STRATEGY FOR HANDLING THE STATISTICS OF TRUCK WEIGHT DATA IN ALAKSA[1]

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ABSTRACT

Based on the premise that truck weight data primarily serves to provide truck load/stress information for the highway design engineer, the present HPMS/TMG requirement for weights of individual classes of trucks is not necessary. Thus the number of Weigh-in-Motion (WIM) sites for the state is determined on a basis of other needs for WIM data. In Alaska the adopted goal for WIM sites is to provide reasonable coverage of the State's Highway System, while at the same time satisfying secondary truck weight enforcement plans and providing data useful in support of the Pavement Management System and highway design. Since it is axle crossings, not truck type, that is used, sample size constraints are easily met.

The statistical analysis was directed to determine the accuracy needed by the WIM to determine highway stress from truck equivalent-single-axis-loads (ESAL's). From an analysis of the error sources in the AASHTO design equation for layered flexible pavement, such as structural number, resilient modulus of the subgrade, and change in Present Serviceability Index, it was found that a coefficient of variation for ESAL's of 50 to 70% is adequate. This translates to a coefficient of variation of about 13% in axle weight at the individual WIM site. All the present fixed WIM technologies and the data analysis techniques meet this requirement providing a 90-10 accuracy. Regression analysis of single and tandem axle truck crossings versus ESAL's determined from VTRIS showed remarkably consistent behavior. Further it is possible to estimate the ESAL's from classification counts by determining the number of crossing of each axle type using the equation: *ESAL's = 0.072 x single axle crossings + 0.45 x tandem axles crossings*.

INTRODUCTION

Data reports following the guidelines of the Highway Performance Monitoring System (HPMS) Field Manual and its companion Traffic Monitoring Guide (TMG) call for sufficient samples to determine volumes of vehicles as well as classification of vehicles to different levels of accuracy depending on the road classification and whether or not an urban area is an non-attainment area. The implication in the Traffic Monitoring Guide is that there is a need to determine the weight distribution of each class of truck.

With the limited number of each class of trucks, the HPMS sample requirement is not a reasonable requirement, particularly for Alaska. Determining the weight distribution, for example, for all Class 9 trucks serves little purpose, since what is important for highway engineers is the cumulative highway stress defined by and measured in Equivalent Single Axle Loads[3] (ESAL's) from the truck traffic. With the focus on understanding the statistics of the stress imparted by the truck, rather than on the statistics of the gross vehicle weight (GVW) of a given class of truck, the following questions are addressed:

- ✓ How do the other sources of error in highway design reflect into ESAL determination?
- ✓ How do these errors translate into setting requirements on WIM? and
- ✓ How can these data be used for simple estimation of ESAL's?

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2. Ms. Walters is the Road Log Manager for Alaska Department of Transportation and Public Facilities. Dr. Whitford worked for AKDOT/PF while on Sabbatical leave from the School of Civil Engineering at Purdue University (1997-98).
3. One ESAL is equivalent to 18,000 Pounds Force per Square Inch (psi), 18 kips/square inch (ksi), or 124 million Pascals (MPa).

Classification counts give the number of each class of truck. Empty/loaded data can be obtained from use of the VTRIS[4] software provided by the Federal Highway Administration (FHWA).

Regression of the 1995 and 1996 WIM data from the Alaska sites is examined for potential use in the analysis. Using only axle crossing data segregated by all truck single axles and all truck tandem axles, the sample size should be fairly adequate depending on the truck traffic. On the other hand, with the stress dependent on the fourth power of the weight, more samples will be needed when converting directly to ESAL's.

AN ERROR ANALYSIS OF THE AASHTO EQUATION VARIABLES

In spite of the FHWA's Long Term Pavement Performance (LTPP) Program's assessment[5] of the AASHTO equation (see Equation 1) for layered flexible pavement, showing that the equation has a significant number of shortcomings, many states use some version of the equation for highway design. This paper utilizes the equation to frame the statistical requirements for ESAL's. The major variables are modulus of resiliency of the subgrade (M_R), the strength characteristics of the layered flexible pavement or Structural Number (SN), and the change in present serviceability index (ΔPSI) over the pavement's lifetime. One term included by many states to account for errors in the growth in trucking $Z\sigma$ is not included in this analysis.

$$\log_{10} W_{18} = Z\sigma + 9.36\log_{10}(SN+1) + \frac{\log_{10}\left(\frac{\Delta PSI}{2.7}\right)}{0.4+\log_{10}(SN+1)^{5.91}} + 2.32\log_{10}(M_R) - 8.27 \qquad \text{Equation 1}$$

Where W_{18}= Cumulative ESAL's in kips,
$Z\sigma$ = Statistical term showing Reliability (from Z tables) and Standard Deviation generally added for unknowns in projection of truck traffic over the design life of the pavement,
SN = Structural Number,
ΔPSI = The allowable change in ride quality before repair is indicated, and
M_R= The Modulus of resiliency of the subgrade.

Coefficient of Variation of Structural Number

The Structural Number is a function of the properties of the various materials that make up the Hot Mixed Asphalt Concrete (HMAC), the granular base and granular subbase. The basic equation that governs the present calculation of the Structural Number is given in Equation 2.

$$SN = a_{HMAC}\, d_{HMAC} + a_{base}\, d_{base}\, m_{base} + a_{subbase}\, d_{subbase}\, m_{subbase} \qquad \text{Equation 2}$$

Where a = structural coefficient of the material
d = depth of the course indicated in inches
m = the coefficient for drainage

In the typical pavement in Alaska the usual structural number is in the range of 3.5 to 5.5. In this paper we have used SN=4 as the nominal amount. For many of the Alaskan pavements, there will be two inches of HMAC,

4. VTRIS, Vehicle Travel Information System software is provided by FHWA for the data reduction of WIM data into ESAL's and other relevant information. (W tables).
5. National Research Council, *Evaluation of the AASHTO Design Equations and Recommended Improvements*, SHRP-P-394, April 1994.

which contributes about 22% of the structural strength[6], six inches of course granular base, which contributes about 21% of the strength and the rest (for a total of one meter below the base of the asphalt layer) will be about 30 to 33 inches of granular subbase, which provides the remainder (57%) of the structural strength.

Table 1 presents a possible set of data that could be considered in determining the non-nominal conditions that occur in the field construction of pavement. The structural coefficients depend on the quality control in the batch processing of the HMAC and in the grading and selection of material sources of the granular base and subbase courses. The adjustment for moisture or drainage depends on the level of precipitation, the site grading and the design of the pavement itself. That adjustment also seems to depend on temperature or at least the freeze-thaw cycle. For each layer, the error sources are combined as independent statistical quantities. The resultant coefficient of variation (COV) of 27% is determined. Nourledin et al. (1994)[7] indicates individual error quantities that combine for an estimate of 32% for Structural Number in design. The same authors also show a COV ranging from 5% to 18% with a typical value of 11% and a back calculation COV of 5%. For this analysis 15% is used.

Table 1. Estimated Coefficient of Variation COV for Structural Number.

Layer	Variable	Nominal Value	Estimated Standard Deviation	Contribution to COV Percent	COV for the layer Percent
HMAC	Structural Coefficient	0.44	0.02	4.5%	
HMAC	Thickness of Laid HMAC	5cm	0.5 cm	10%	
HMAC					11%
BASE	Structural Coefficient	0.14	0.01	7%	
BASE	Installed Thickness	15 cm	3 cm	20%	
BASE	Moisture/Drainage	1	0.05	5%	
BASE					22%
SUBBASE	Structural Coefficient	0.09	0.01	5.6%	
SUBBASE	Installed Thickness	80 cm	4 cm	5%	
SUBBASE	Moisture/Drainage	1	0.1	10%	
SUBBASE					12%
COEFFICIENT OF VARIATION FOR STRUCTURAL NUMBER = 27%					

Source: Discussion with Highway Engineers and Nourdelin et. al. (1994)

<u>Coefficient of Variation of Modulus of Resiliency of the Subgrade</u>

The Modulus of Resiliency M_R is the coefficient that best describes the performance of the subgrade or soil below the pavement. It is the soil's reaction to an applied force in pounds per square inch or in million Pascals (MPa). In Alaska, the subgrade in many locations is subjected to heavy freezing and annual thawing; a process, which means that any single number of MR used in the AASHTO equation, must account for annual climatic changes.

6. For the more usual pavement of 4 inches the HMAC provides 35% of the strength.
7. Nourledin, et al, *Deviation of Predicted Performance of Flexible Pavements Using AASHTO Model*, (1994)

The swings in the M_R can range from a high under frozen conditions of 15,000 pounds of force per inch (psi) (103 MPa) to 2000 psi (13.8 MPa) shortly after thaw begins. The M_R then slowly increases as moisture drains from the soil achieving some stable value until freeze-up occurs when it returns to its frozen value. Using a simple model and varying the level of freeze/thaw and the timing of thaw, a COV of about 10% is calculated from these models.

The usual way of determining the M_R is by back calculating the values from Falling Weight Deflectometer (FWD) test data. While the FWD tests seem quite repeatable, analysis of the LTPP data (SHRP P-394)[8] indicates that the back-calculation and the laboratory test results differ, usually by an average factor of 4.48. Even with that bias, the spread in these data is still very high, showing a very weak correlation or high COV. However, correcting for the bias reduces the number of over-estimates considerably.

Others who have done studies seem to differ widely on the COV of the Subgrade Resilient Modulus. For example, Hall and Thompson (1994) back-calculated the Modulus of Resiliency for 493 highway sections. Those data indicate a COV of about 45%. On the other hand, Nourledin et.al (1994) give back-calculations that indicate a COV of 15.6%. Janoo and Berg (1992), using the Corps of Engineers Falling Weight Deflectometer (FWD) software program reported errors that were higher when pavement is treated as a whole rather than by its layers. They estimate COV of 35% for unfrozen clay and 18% for unbound sand subgrade. Siddharthan et. al (1992) present their results from processing FWD data which shows the COV for the asphalt layer as about 41%, the unbounded stone base course as 34% and the granular subgrade as 8.7%. Measurements taken for soils at six locations in Indiana for a Freeze-Thaw evaluation give an estimated COV of about 25%. This author interviewed four professors in Geotechnical Engineering for their best estimate of the COV. Their answers were 10%, 35%, 50% and "at least 50%". Thus, a COV of 35% is used in this analysis for the Modulus of Resiliency.

Coefficient of Variation of Present Serviceability Index

The Pavement Serviceability Performance Concept was developed in 1962[9]. It has been shown that a pavement begins its life in excellent condition, but as traffic loads are applied and the pavement is subject to temperature variation and cycles of freezing and thawing weather, there will be a deterioration of the pavement until eventually it reaches an unacceptable ride quality.

The Present Serviceability Rating (PSR) is obtained from ride quality judged by a panel of trained "raters" who drive/ride on the pavement and rate it on smoothness of ride. The accumulation of loads and the cumulative effects of weather cause the pavement to eventually reach a level called the terminal serviceability rating. At this point raters and state highway departments feel that the pavement can no longer perform as it was intended. PSR's of about 2 to 2.5 reflect this condition.

PSI was established to reflect the addition of some profile data to the rating process. In the AASHTO equation the lifetime is accounted for by the inclusion of the ΔPSI term. A substantial PSR rating effort using 24 trained raters was done in Kansas.[10] The data, taken on a large sample of Kansas flexible, layered highway segments

8. SHRP-P-394, op. cit.
9. The Present Serviceability Rating was initially set out as part of a national road test by AASHTO in the early 1960's.

of pavement, yielded a COV in PSI of 24%. Since there is a similar variation in rating at the beginning of the life of the pavement, a COV of ΔPSI would be 1.41 times as much or 35%.

Understandably this method is subjective and highly dependent on the training of the raters. The addition of physical measurements like roughness and crack spacing has led to a more tractable numbers. Instrumented vehicles measuring the highway's profile in detail have put the rating[11] on an even more standardized basis called the International Roughness Index (IRI), which uses a measurement of the actual highway profile. The specification for the Dynatest road profiler indicates a COV of 5% after six runs. IRI Calibration Tests run by Mr. Scott Gartin, Director of Alaska's Pavement Management System, indicate COV results ranging from 0 to 23% with about 9% as the average. However, the determination of IRI and the corresponding pavement conditions have been the subject of some effort by LTPP.[12] Scatter diagrams of the data taken to date show poor correlation indicating the lack of understanding. The data for the wet-freeze climate indicates an IRI with a mean of 51 and standard deviation of 35;[13] This would imply a COV in IRI 68%. Using the relation developed in Figure 1 from efforts to compare the PSI and IRI[14], a COV of 68% in IRI is equivalent to a COV in PSI of about 35% or ΔPSI of 49%. A COV of 35% has been assumed for ΔPSI.

Translating to Errors in ESAL and Combining

In order to combine these errors the equivalent impact each has on ESAL's is needed. Figures 2, 3, and 4 show the relationship obtained from Equation 1 for each of the variables. These are summarized in the last column of Table 2.

Table 2 COV of Error Sources and ESAL

Error Source	Range of COV of Error Source	COV Used in Error Analysis	Related COV in ESAL's
Structural Number	5% to 32%	15%	63%
Resilient Modulus of Subgrade	10% to 70%	35%	62%
Change in Present Serviceability Index	9% to 68%	35%	50%

With the concept that the error source added by the expected errors in measuring ESAL's should be in the range of the errors contributed by the other error sources, the COV that can be attributed to ESAL determination from a given WIM site should be between 50% and 70%. The combination of the error sources amounts to a combined COV of 101% in units of ESAL's. A 50% COV in ESAL determination yields an overall COV of 113% or only an 11% increase in error. Likewise a 70% COV in ESAL determination yields an overall COV of 123% or an 18% increase in error.

10. Moore, R.K., Clark, G.N. and Plumb, G.N., *Present Serviceability-Roughness Correlations Using Rating Panel Data,* 1987. TRB.
11. FHWA Roughness Equipment, Calibration and Data Collection, HPMS Field Manual (FHWA Order M 5600.1B) Appendix J, August 1993.
12 . National Research Council, Sensitivity *Analyses for Selected Pavement Distress,* SHRP-P-393, April 1994.
13. Ibid., page 69.
14. Paterson, W.D.O., *International Roughness Index: Relationship to Other Measures of Roughness and Ride Quality* (1986). TRB.

Figure 1. Relationship of COV IRI to COV PSI

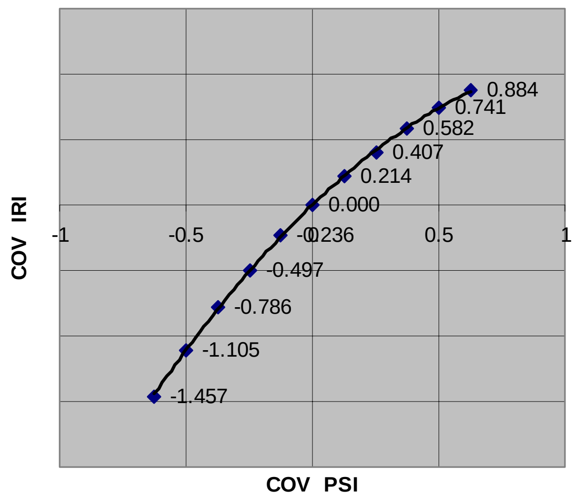


Figure 2. COV in ESAL's as a function of COV in Structural Number

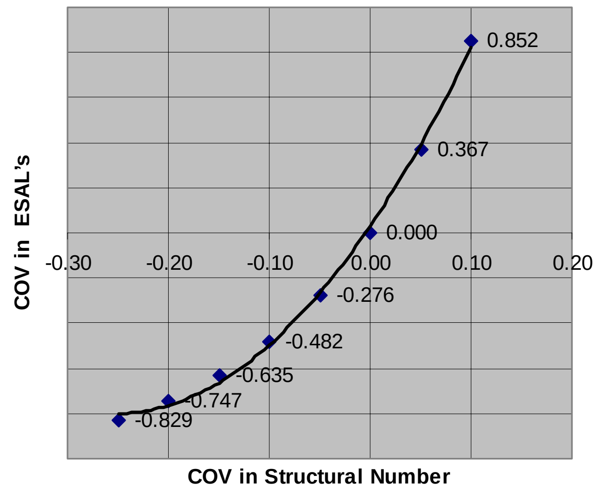


Figure 3. Relationship of COV in ESAL's with COV of Subgrade Modulus (M_R) [SN =4;ΔPSI=1.7]

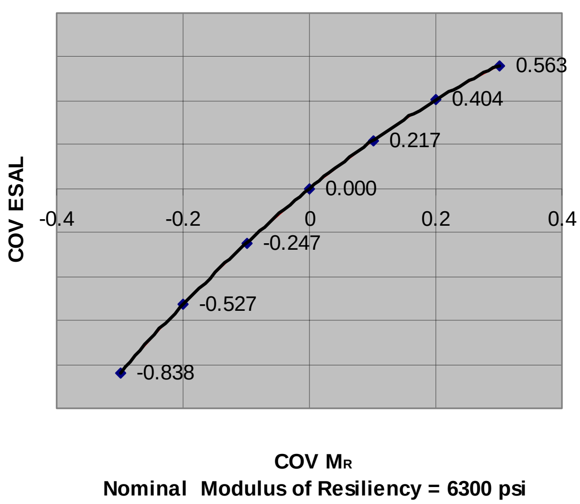
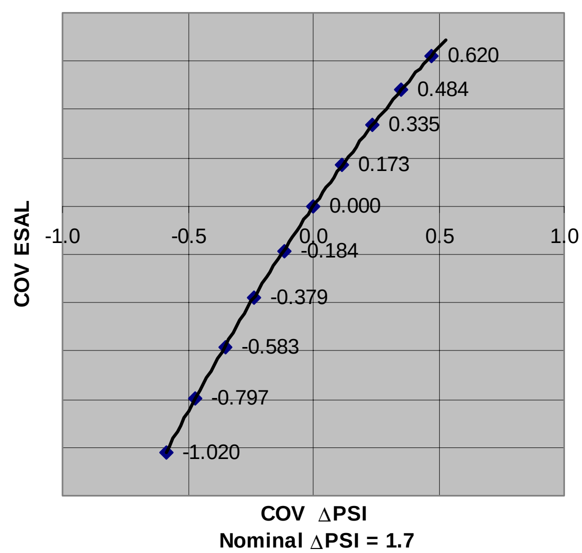


Figure 4. Relationship of COV ESAL's with COV ΔPSI [MR=6300psi; SN=4]



WHAT DO THESE ERRORS TRANSLATE INTO IN TERMS OF NUMBER OF SAMPLES TAKEN?

At this point the discussion turns to the nature of the WIM data and how many crossing of each type of axle are required to obtain a reasonable prediction of the load. Since the WIM data directly determines weight and the data reduction software places each weight into a bin, the accuracy of the WIM equipment reflects itself into a potential COV caused by placing the data into the wrong weight bin. Each bin is given the ESAL according to the midpoint of the bin. Three error sources exist: (1) WIM equipment accuracy, (2) effects of calibration and climate on the equipment and (3) the errors from the established bins for each axle weighing. Equation 3 expresses the fourth power formula for converting from single axle weight to ESAL.

$$ESAL_{mean} + \Delta ESAL = \left(\frac{18{,}000 + \sigma}{18{,}000}\right)^4 = \left(1 + \frac{\sigma}{18000}\right)^4 = (1 + COV_{weight})^4 \qquad \text{Equation 3}$$

If the mean ESAL at 18,000 pounds is 1 and ΔESAL set at the standard deviation then Equation 4 results.

$$COV_{weight} = (1 + COV_{ESAL})^{0.25} - 1 \qquad \text{Equation 4}$$

This equation is shown in Table 3. To meet our COV at 50 to 70% the COV required of the weight measurement is between 10.7 and 14.3%.

Table 3 Needed COV in Weight to meet Allowable ESAL COV

Allowable COV_{ESAL}	Translation COV_{WEIGHT}		
200%	31.6%		
150%	25.7%		
100%	18.9%		
70%	14.3%	⇐ **Range required**	
50%	10.7%	⇐ **for System**	
40%	8.8%		
21%	5%	⇐ Piezoelectric	**WIM**
10%	2.5%	⇐ Bending Plate	**Technol-**
6%	1.5%	⇐ Load Cell	**ogies**

The COV for weight for the number of stations proposed should be less than 12.5%. All three WIM technologies provide weights in that range. An analysis of the effects of separating the data into bins indicates that the errors related to being placed in the bin give a COV in weight between 10% and 12% over the range of single axle weights. The other effects of temperature and calibration errors appear to have a standard deviation in the range of 5 to 7% depending on the maintenance procedures and calibration routine. The combined error gives a COV of about 14%. Each WIM station will meet determine ESAL's to an accuracy of 90-10 with at least 80 crossings. This is a reasonable level of daily traffic for most Alaska highways. Analysis of the Tandem Axles gives similar results.

HOW CAN THESE DATA BE USED FOR SIMPLE ESTIMATION OF ESAL'S?

What does the pattern of truck axle weight suggest for determining ESAL data by using an equation and classification data? After reviewing the data presented, it would appear that the WIM site allocations are needed primarily for secondary enforcement of overweight trucks and adequate coverage geographically. This suggests that many of the sites will be located in conjunction with the weigh station they serve. The remainder have been placed in places to provide a representative sample of the truck traffic and on highways where there seem to be special reasons such as pavement evaluation data for LTPP or frost behavior. This then suggests that the number of WIM sites is more a function of those requirements than ones that reflect the accuracy of data. The 13 sites called for in the Alaska plan are ample to meet all these criteria.

Behavior of WIM Data

The statewide WIM data collected during 1995 and 1996 was entered into the Vehicle Travel Information System (VTRIS) software provided by FHWA. The W-4 Equivalency Factor tables from VTRIS were used to calculate the ESAL's and frequency data used in this analysis. The number of tridem axles in the data was found to be insignificant to this study and was combined with the tandem axle group.

The development of a model from which analysts would use the truck classification data by class of truck was examined. One can readily see from Table 4 that the errors in ESAL's are about 40 to 50% on a truck class by truck class basis. However, in looking at all the data, the values of a metric like ESAL's per truck, on the average, were reasonable. The urban sites exhibited a mean of 0.31 ESAL's per truck with a standard deviation of 0.08. The rural sites were poorer in that they showed a mean of 0.37 with a standard deviation of 0.16. The Coefficients of Variation were about 26% and 43%, respectively. Combined all trucks gave a mean of 0.34 with a standard deviation of 0.13. Thirteen sites thus give 90%-15% for one year and better than 90%-10% over two years.

When the results from VTRIS were analyzed logarithmic regression equations of ESAL's for each of the single axle and tandem axle loads[15] yielded adequate coefficients of determination. Axles at very, very low ESAL were omitted. Single axle and tandem axle groups combined the individual station equations to form a single equation for all stations during the year. This equation was tested for fit by entering the station data into the equation and comparing the actual values to the calculated values. The Coefficients of Determination (R^2) ranged from 0.79 to 0.96 for single axles and 0.65 to 0.97 for tandem axles. Table 5 presents the regression equations obtained, and Figure 5 shows the resultant curves for the overall equation. This analysis found that the WIM data collected tended to be consistent over all stations. The equations produced, though useful, are too specific to result in a total ESAL estimate from total crossings. Thus a more general type of equation was sought in order to test the best way to use classification data.

Table 4. Applying the Truck Type Model to Classification Data for a Six WIM Site Sample

FHWA truck Class.	. Whtd Avg. SAL/tr	2. Avg. ESAL b Class	Sta. 123 -Rural Int			Sta. 126-Urban PA			Sta. 107-Rural Int.			Sta. 127-Rural PA			Sta 114-Urban MA			Sta. 143-Urban Int.		
			Trks	ESAL1	ESAL2	Trks	ESAL1	ESAL2	Trks	ESAL1	ESAL2	Trks	ESAL1	ESAL2	Trks	ESAL1	ESAL2	Trks	ESAL1	ESAL2
4	0.533	0.502	1	0.53	0.50	15	8.00	7.53	2	1.07	1.00	0	0.00	0.00	1	0.53	0.50	20	10.66	10.04
5	0.079	0.066	77	6.05	5.10	558	43.81	36.95	189	14.84	12.52	0	0.00	0.00	116	9.11	7.68	511	40.12	33.84
6	0.675	0.667	6	4.05	4.00	60	40.49	40.03	21	14.17	14.01	9	6.07	6.00	19	12.82	12.68	75	50.61	50.04
7	0.800	0.200	1	0.80	0.20	1	0.80	0.20	0	0.00	0.00	0	0.00	0.00	0	0.00	0.00	0	0.00	0.00
8	1.242	0.995	2	2.48	1.99	11	13.66	10.95	2	2.48	1.99	1	1.24	1.00	0	0.00	0.00	9	11.18	8.96
9	0.916	0.898	12	10.99	10.78	53	48.53	47.61	27	24.72	24.25	14	12.82	12.58	7	6.41	6.29	71	65.02	63.78
10	1.203	1.167	2	2.41	2.33	6	7.22	7.00	5	6.01	5.84	3	3.61	3.50	1	1.20	1.17	9	10.82	10.51
11	1.939	0.176	0	0.00	0.00	0	0.00	0.00	0	0.00	0.00	0	0.00	0.00	0	0.00	0.00	0	0.00	0.00
12	0.000	0.027	0	0.00	0.00	0	0.00	0.00	0	0.00	0.00	1	0.00	0.03	0	0.00	0.00	0	0.00	0.00
13	1.314	1.577	2	2.63	3.15	1	1.31	1.58	12	15.77	18.92	4	10.51	6.31	1	1.31	1.58	11	14.46	17.34
			103	30	28	705	164	152	258	79	79	32	34	29	145	31	30	706	203	194.51
	Actual ESAL		44			178			56			54			24			234		
	Esal/Truck		0.43			0.25			0.22			0.92			0.16			0.33		

Note The weighted Average by truck is calculated across all trucks of a given type and the Average ESAL is calculated across WIM sites.

Table 5. Trendline Analysis for 1996 Single and Tandem Axles

Site Number	Single Axle Equation	R^2	Tandem Axle Equation	R^2
101	y = -5.7283Ln(x) + 6.462	0.8015	y = -5.9438Ln(x) + 8.6477	0.9756
103	y = -31.525Ln(x) + 8.7587	0.9017	y = -7.3262Ln(x) + 17.561	0.7806
104	y = -18.989Ln(x) + 0.6162	0.9645	y = -6.2214Ln(x) + 9.1793	0.9739
106	y = -35.981Ln(x) - 9.7021	0.9126	y = -3.8446Ln(x) + 5.2151	0.9431
107	y = -22.341Ln(x) - 14.263	0.8233	y = -3.9827Ln(x) + 5.9486	0.9241
111	y = -10.404Ln(x) + 5.8385	0.8902	y = -2.5805Ln(x) + 6.3141	0.6537
114	y = -8.862Ln(x) - 1.8692	0.8471	y = -0.8897Ln(x) + 1.3391	0.9302
126	y = -41.34Ln(x) - 8.6899	0.8582	y = -6.5277Ln(x) + 9.6567	0.9213
127	y = -11.622Ln(x) + 0.0224	0.9911	y = -3.0058Ln(x) + 5.1591	0.9776
179	y = -20.696Ln(x) - 5.955	0.7927	y = -3.4542Ln(x) + 5.3688	0.9627
535	y = -9.6516Ln(x) + 2.183	0.8565	y = -2.2141Ln(x) + 7.9972	0.7423
All Data	**y = -204.82Ln(x) + 29.23**	**0.9054**	**y = -43.460Ln(x) + 81.763**	**0.9303**

Note: y = the frequency and x = ESAL's

A further regression analysis was performed on the WIM data from 1995 and 1996 by reversing the axes; that is plotting ESAL's as a function of the frequency of axles crossings for each single axle and tandem axle. The resulting linear equations have Coefficients of Determination of 82% for single axles, and 85% for tandem axles. These results are shown in Figure 5. The mean average ESAL's per truck calculation resulted in 0.06 for single axles and 0.34 for tandem axles with ESAL COV of 38% and 35%, respectively.

15. The analysis did not include tridem axles, as there are only a few class ten trucks in Alaska and treating the tridem axle configuration as a tandem is conservative.

Figure 5. Combined Axle Data for All Sites

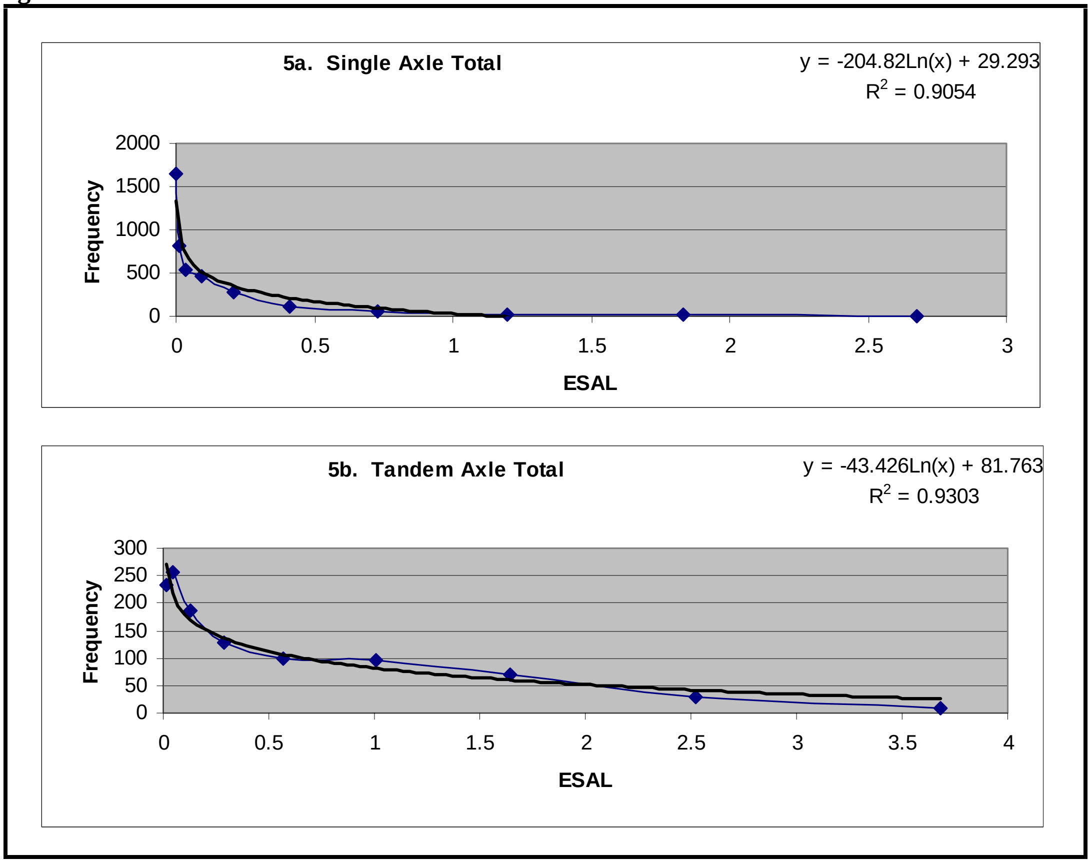
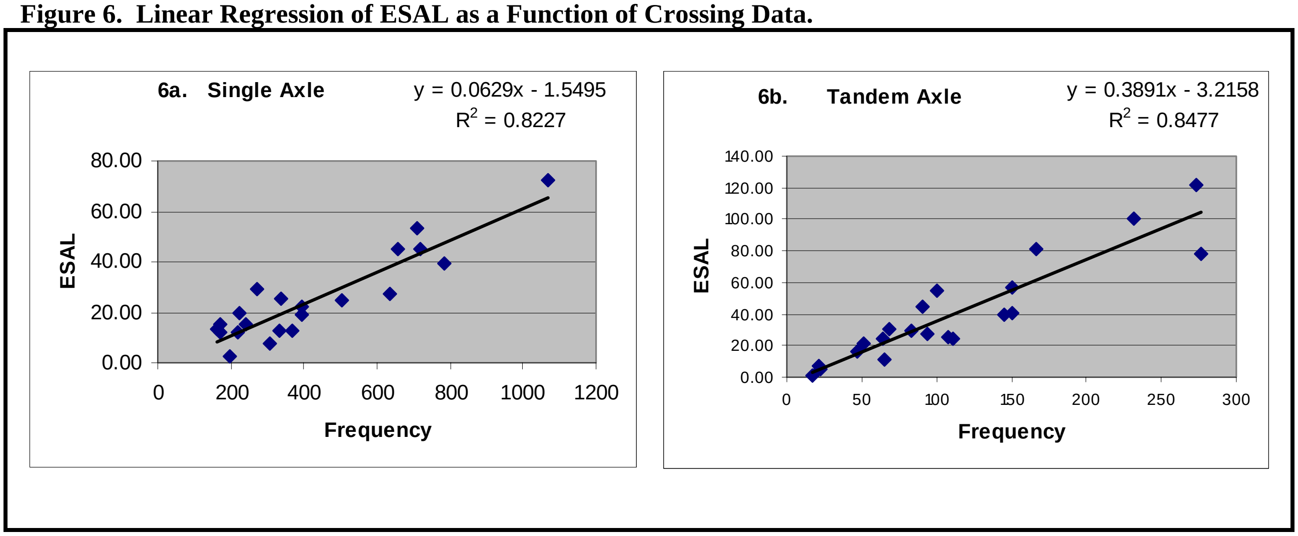
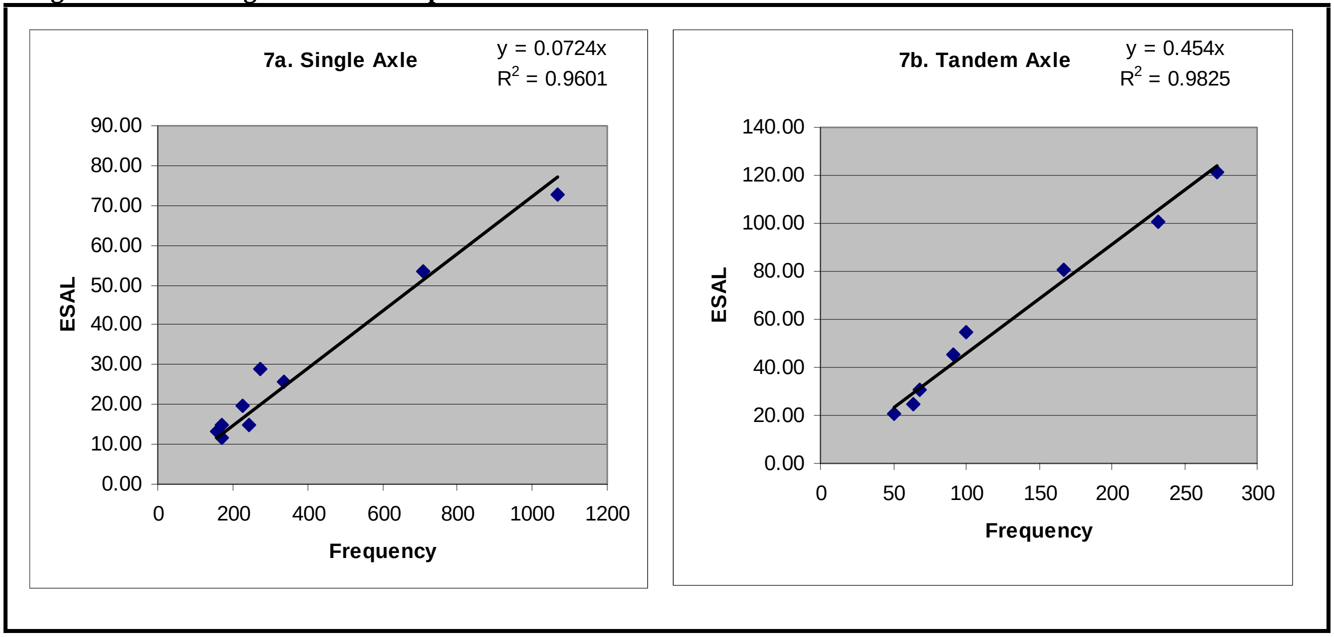


Figure 6. Linear Regression of ESAL as a Function of Crossing Data.



In order to identify a conservative number for use in the field, another regression analysis was performed forcing a zero intercept and using only the data points above the regression line from Figure 6 (deleting those points below it). The final curves are shown as Figure 7.

Figure 7 Regression for Simple ESAL Estimation



Both this conservative equation and the original regression equations were checked for fit by entering the individual station data in the equation and calculating the residuals. These equations provide acceptable ESAL values for pavement design purposes. This produced the equation

Total Estimated ESAL = 0.072 x number of single axle crossings
+ 0.45 x number of tandem axle crossings. **Equation 6**

Where Single Axle Crossings are all the single axles on trucks (class 5 or better) crossing the Classification counter in a given time period and tandem Axle Crossings are all the tandem (or tridem) axles on trucks (class 5 or better) crossing the Classification counter in the same time period.

For example, assume a daily traffic level of 300 trucks 100 S-9 and 200 S-6 trucks. These trucks have combined 700 single axle crossings and 200 tandem axle crossings and no tridem axles. The daily ESAL's for that site would be estimated as

ESAL's = 0.072*700+0.46*200 = 154

CHECK OF EQUATION

This simple equation based on axle crossings was checked against similar efforts in North Carolina. S-S. Wu[16] has developed a simplified procedure for determining ESAL's depending on the truck mix. Wu divided the truck population into (1) single unit trucks with single axles, (2) single unit trucks with tandem axles and (3) all other semi-trailer trucks. He then develops an average ESAL that would result from each of those truck types. Using his data from the North Carolina roads and the truck mix above, his approach would result in 148 ESAL's. To further examine the relationship with the North Carolina approach several truck mixes were evaluated by each method. Table 6 presents that comparison.

Table 6 Comparison of Axle Crossing Approach with Truck Model of Wu.

	Traffic Mix Approach WU				Classification Axle Count Walters & Whitford			
Case	Single Unit Single Axles	Single Unit Tandem axles	Semi Trailers Tandem Axles	ESAL Estimate from Wu	Single Axle Crossings	Tandem Axle Crossing	ESAL Estimate by this paper	Difference
Multiplier	0.23	0.62	0.6		0.072	0.454		
Suburban	365	85	135	218	950	355	228	5%
Urban	500	60	70	194	1130	200	171	13%
Rural	190	40	120	141	540	280	165	15%
Both	400	140	180	287	1120	500	306	6%
Light Urban	140	15	50	72	345	115	77	7%

CONCLUSION

All of the WIM technologies will perform within the FHWA suggested accuracy needed for the ESAL determination and the data reduction approach is consistent with the level of acceptable errors. Highway stress can be adequately estimated from classification counts of truck axle type as indicated in equation 6.

REFERENCES

1. Alaska Department of Transportation and Public Facilities, *1995 and 1996 WIM and Classification Data.*
2. American Association of State Highway and Transportation Officials, *AASHTO Guide for the Design of Pavement Structures*, AASHTO, Washington D.C., 1993.
3. Attoh-Okine, N.O and Roddis, W.M.K., *Pavement Thickness Variability and its Effect on Determination of Moduli and Remaining Life,* Transportation Research Report 1449, **Design and Rehabilitation of Pavements,** TRB, Washington D.C.,1994, pp. 39-45,
4. Federal Highway Administration, Highway Performance Monitoring System (HPMS) Field Manual. M5600.1B, Change 1 April 22, 1994 with subsequent updates in 1995 and 1996.
5. Federal Highway Administration, Traffic Monitoring Guide, Third Edition, October 1996.

[16]. Wu, S-S, *Procedure to Estimate Loading from Weigh-in-Motion Data*, 1996, TRB

6. Garber, N.J. and Hoel L.A., Traffic and Highway Engineering, Second Edition, PWS Publishing, Boston, MA, 1996.

7. Gopinath, D, Ben-Akiva, M. and Ramaswamy, R., Modeling *Performance of Highway Pavements,* Transportation Research Report 1449, **Design and Rehabilitation of Pavements,** TRB, Washington D.C., 1994, pp. 1-7.

8. Hall, K.D. and Thompson, M.R., Soil *Property-Based Subgrade Resilient Modulus Estimation for Flexible Pavement Design,* Transportation Research Report 1449, **Design and Rehabilitation of Pavements,** TRB, Washington D.C., 1994, pp. 30-38.

9. Janoo, V.C. and Berg, R.L., *Layer, Modulii Determination During Freeze Thaw Cycle,* Transportation Research Report 1377, **Nondestructive Deflection Testing and Back-Calculations for Pavements**, TRB, Washington D.C., 1992, pp. 26-35.

10. Knoebel, G., *National Traffic Data Acquisition Conference (NATDAC '96) Proceedings Volumes II Albuquerque New Mexico, May 1996, I.* Available through NTIS, Springfield, VA.

11. Lee, W., Bohra, N.C., Altschaeffl, A.G., White, T.D., *"Subgrade Resilient Modulus for Pavement Design and Evaluation*" Final Report, Indiana Joint Highway Research Project, Report FHWA/INDOT/JHRP 92-23, August 1993.

12. McCall, B., WIM Handbook, Center for Iowa Transportation, Iowa State University, Ames, IA, August 1997.

13. Moore, R.K., Clark, G.N. and Plumb, G.N., *Present Serviceability-Roughness Correlation's Using Rating Panel Data,* Transportation Research Report 1117, Pavement **Evaluation and Rehabilitation,** TRB, Washington D.C., 1987, pp. 152-158.

14. National Research Council, Early *Analysis of LTPP General Pavement Studies Data – Executive Summary, SHRP*-P-392, Strategic Highway Research Program, Washington, D.C., April 1994.

15. National Research Council, Evaluation *of the AASHTO Design Equations and Recommended Improvements,* SHRP-P-394, Strategic Highway Research Program, Washington, D.C., April 1994.

16. National Research Council, Sensitivity *Analyses for Selected Pavement Distress,* SHRP-P-393, Strategic Highway Research Program, Washington, D.C., April 1994.

17. Nourledin, A.S., Sharaf, E., Arafah, A. and Al-Sugair, F., Estimation *of Standard Deviation of Predicted Performance of Flexible Pavements Using AASHTO Model,* Transportation Research Report 1449, **Design and Rehabilitation of Pavements**, TRB, Washington D.C., 1994, pp. 46–56.

18. Paterson, W.D.O., *International Roughness Index: Relationship to Other Measures of Roughness and Ride Quality*, Transportation Research Report 1084, **Pavement Roughness and Skid Resistance**, TRB, Washington D.C., 1986, pp. 49-59.

19. Siddharthan, R., Sebaaly, P.E. and, Javaregowda, M. *Influence of Statistical Variation in Falling Weight Deflectometers on Pavement Analysis,* Transportation Research Report 1377, **Nondestructive Deflection Testing and Back-Calculations for Pavements**, TRB, Washington D.C., 1992, pp. 57-66.

20. Wu, S-S., *Procedure to Estimate Loading from Weigh-in-Motion Data*, Transportation Research Report 1536, **Traffic and Pavement Surface Monitoring Issues,** TRB, Washington D.C., 1996, pp. 19-24.