Measurement of Stress Based Load Transfer Efficiency Using Single Gage Under Moving Multi-axle Carriages at the National Airport Pavement Test Facility

C. Cunliffe, W. McNally, F. Blotta
Department of Civil and Environmental Engineering, Rowan University, Glassboro, NJ

Dr. Y. Mehta & Dr. D. Cleary
Department of Civil and Environmental Engineering, Rowan University, Glassboro, NJ

ABSTRACT: It is currently believed that mechanistic evaluation of rigid pavements using full scale test data is the most appropriate way to study pavement behavior. Current methods outline a single and dual gage approach for evaluating stress-based load transfer efficiency (LTE (S)) at the transverse joints. The single gage method can be very useful for joints where only a single gage is located or where dual gages are located, but only one of the gages is producing accurate responses. Additionally, the installation and monitoring of gages is expensive; therefore, the ability to accurately calculate the stress-based load transfer efficiency based on a single gage will provide an impetus to several resource starved agencies to monitor the performance of the joint without the need to install two gages. Both methods have produced similar estimations of LTE (S) for single axle carriages; however, it is unknown if this approach yields similar results for multi axle carriages. The dual gage approach for measuring LTE(S) has been implemented successfully in prior studies at Rowan University for tandem axle configurations for testing conducted on Construction Cycle 2 (CC2) at the Federal Aviation Administration (FAA) National Airport Pavement Test Facility (NAPTF). In this study LTE(S) was determined based on a single gage for multi-axle carriages. The analysis was conducted on CC2 and CC6 test items. The LTE(S) calculated based on a single gage was then compared with those calculated with the dual gage. This analysis has shown mixed results and limitations regarding the accuracy of the stress-based LTE using a single gage.

KEY WORDS: single strain gage analysis, dual strain gage analysis, load transfer efficiency

1 INTRODUCTION

1.1 Stress Based Load Transfer Efficiency (LTE (S))

When traffic loading is applied near a joint of a jointed concrete pavement, both the loaded slab as well as the adjoining unloaded slab undergo some amount of deflection. A portion of the applied load is transferred to the adjoining unloaded slab through the load transfer mechanisms of a joint such as aggregate interlock and dowels. As a result, the deflections and stresses in the loaded slab may be reduced relative to a slab with free edges. The degree of load transfer is commonly called load transfer efficiency (LTE) and can be defined based on stresses or deflections. The relative reduction in edge stress is termed as load transfer efficiency LTE (S). Equation 1 represents the mathematical definitions of LTE (S).

$$LTE(S) = \frac{\sigma_{unloaded}}{(\sigma_{loaded}+\sigma_{unloaded})} = \frac{\varepsilon_{unloaded}}{(\varepsilon_{loaded}+\varepsilon_{unloaded})} \quad (1)$$

Where, $\sigma_{unloaded}$ and σ_{loaded} are slab bending stresses while $\varepsilon_{unloaded}$ and ε_{loaded} are corresponding strains on unloaded and loaded slabs, assuming linear behavior, respectively.

LTE (S) is considered as a measure of joint behavior and it plays an important role in pavement evaluation and design. LTE (S) can be evaluated in the field from slabs in which strain gages have been embedded at the joint. Current methods outline single and dual gage approaches for evaluating stress-based load transfer efficiency (LTE (S)) (Brill et al., 2001). The single gage method can be very useful for joints where only a single gage is located or where paired gages are located, but only one of the gages is producing accurate responses. Additionally, the installation and monitoring of gages is expensive; therefore, the ability to accurately calculate the stress-based load transfer efficiency based on a single gage will provide value to a set of gages that would otherwise be inaccurate or unused..

1.2 CC2 at National Airport Pavement Test Facility (NAPTF)

Full-scale testing is conducted on rigid pavements by the FAA at the NAPTF, located at the William J. Hughes Technical Center, Atlantic City International Airport, New Jersey (USA). This study focuses on rigid pavement tests performed during Construction Cycles 2 and 6 (CC2 & CC6). The test items of CC2, which were tested upon from April to December of 2004, consisted of three rigid pavements constructed on granular conventional base (MRC), on grade (MRG) and on stabilized Econocrete base (MRS). The pavement classification used at the NAPTF features a three letter acronym. The first letter signifies the strength of the subgrade, low strength (L), medium strength (M), or high strength (H). The second letter signifies whether the test item is rigid (R) or flexible (F). The third letter designates the base type as mentioned. A medium strength subgrade of CBR value 7 was measured for MRC in the field. Each test item section was 75 feet (22.9m) long and 60 feet (18.3m) wide, comprised of 20 slabs of size 15 feet by 15 feet (4.6m by 4.6m). The slab thickness was 12 inches (30.5cm). Figure 1 represents the plan and sectional view of the test items. The slabs were designed with the inner lanes connected with steel dowels on all four sides. The slabs in the outer lanes were doweled on three sides, leaving free outer edges. Concrete strain gages were installed at various locations, including locations on each side of joints, to measure the strains. Traffic loading was applied by the National Airport Pavement Test Vehicle (NAPTV) which is programmed for controlled aircraft wander simulation. The basic wander pattern consisted of 66 discrete tracks centered on the outside edge of the inside slab, approximating a normal traffic distribution (3). However, a modified wander pattern was used for MRC-N only, so that no wheel loads were applied directly to the outside row of slabs as seen in Figure 1; it is seen that the coverage pattern on the north section of MRC does not extend to the outer slabs of the test section. Both the north and south test sections were loaded with a dual tandem carriage configuration (1). A wheel load of 55000 lbs. (243.7kN) was used producing a tire pressure of about 220 psi (1516.9kPa). The strain gages analyzed were within 3 inches (7.6m) of the transverse joint on either side.

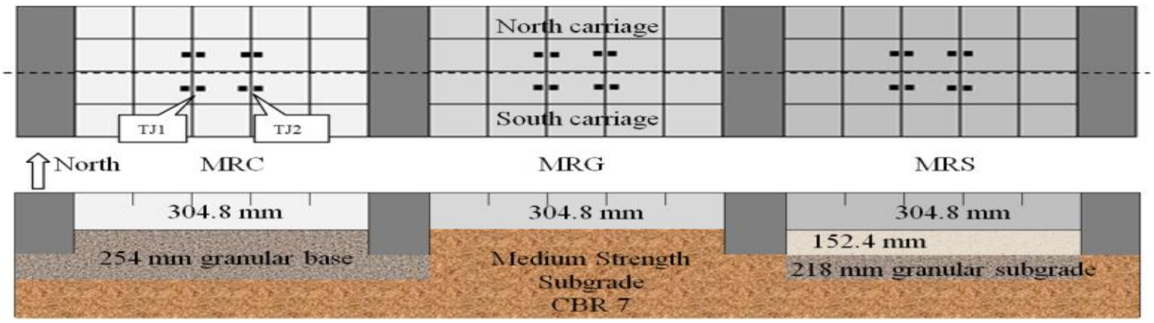

Figure1: Plan and sectional view of CC2 test items with position of concrete strain gages used in this study (*Cunliffe et al., 2012*)

1.3 CC6 at NAPTF

The test items of CC6, which were loaded from April 2011 until April 2012, consisted of six pavement sections. Sections with three different flexural strengths were used and these were placed over bituminous and Econocrete bases. The compositions of the sections can be seen in Figure 2, where the low flexural strength (MOR ≈ 500 psi (3447.4kPa)) sections are MRS-1, the medium flexural strength (MOR ≈ 750 psi (5171.1kPa)) sections are MRS-2 and the high flexural strength (MOR ≈ 1000 psi (6894.7kPa)) sections are MRS-3. The North section of CC6 consists of PCC slab over a bituminous base whereas the South section consists of PCC over an Econocrete base. The layout of the pavement sections can also be found in Figure 2.

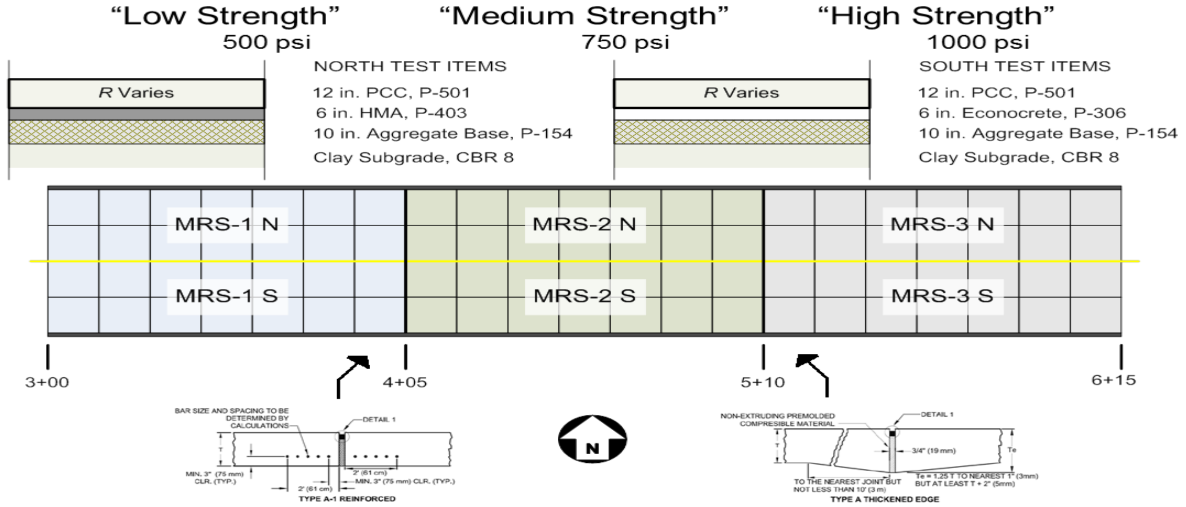

Figure2: The Facilities Testing Site with the Composition of the Different Test Sections, the Flexural Strengths of the Different Test Sections and the Two Different Transition Types (Blotta et al., 2012)

The above mentioned six test items were constructed over a medium strength sub-grade (CBR 7-8) with a 10 inch (25.4cm) thick crushed stone aggregate sub-base. On the North side, the stabilized base consists of 6 inches (15.2cm) of P-403 hot-mix asphalt, while on the South side the stabilized base is 6 inches (15.2cm) of P-306 Econocrete. As with CC2, the slab size is 15 feet by 15 feet (4.6m by 4.6m) and the joints are doweled on all four sides. A wheel load of 45000 lbs. (200.2kN) was used. The strain gages analyzed were within 3 inches (7.6cm) of the transverse joint on either side. The trafficking history and NAPTV wheel load is presented in Table 1.

Table1: CC6 Trafficking History (Brill, 2012)

Dates	Wander Pattern	Wheel Load, lbs. (kN)	Passes		
			MRS-1	MRS-2	MRS-3
7/8/11-8/15/11	*	44,000 (195.7)	6,790	0	0
8/30/11-12/20/11	1-238	45,000 (200.2)	15,708	15,708	15,708
12/27/11-2/29/12	239-405	52,000 (231.3)	0	11,022	11,022
2/29/12-3/30/12	406-508	52,000 (231.3)	0	6,798	0
		70,000 (311.4)	0	0	6,798
3/30/12-4/16/12	509-558	70,000 (311.4)	0	3,300	3,300
Total Passes:			22,498	36,828	36,828

*Preliminary traffic tests (zero wander) on MRS-1N only

2 LTE (S) FROM DUAL AND SINGLE GAGE

2.1 Dual Gage

Load transfer efficiency is calculated using data collected by strain gages on the top and bottom of the pavement. There are two methods for calculating the load transfer efficiency. The first method requires sensor readings from strain gages on each side of the joint, where the loaded strain value is the peak strain on the loaded slab and the unloaded strain is the corresponding strain on the unloaded slab. This method is known as dual gage analysis. Figure 3 gives an example of two corresponding transverse gage readings (EG-147 and EG-151 located on MRS-3 of CC6) plotted with time. More information on the location of gages can be found in Blotta et al (2012). The graph depicts the time period in which the load was applied above the sensor, prior to that there was no change in strains.

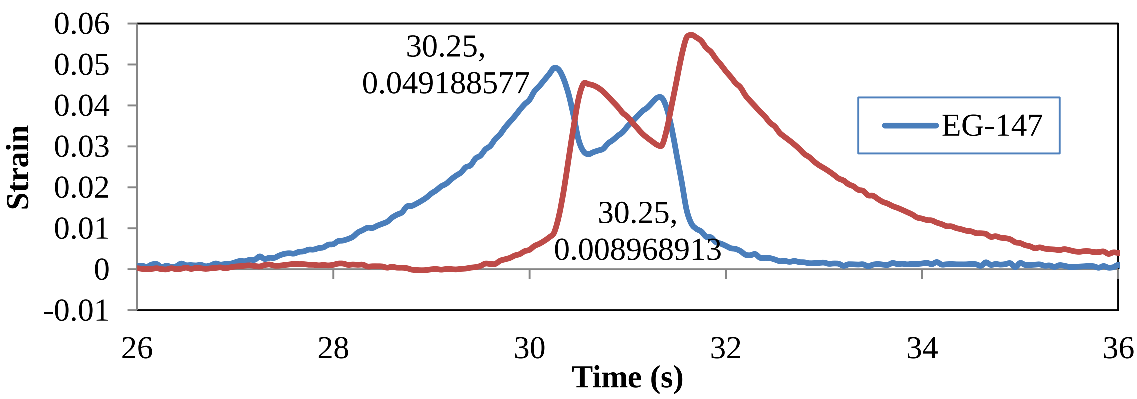

Figure3: Typical Strain Profile for Dual Gage Analysis

The two outside peaks represent the two positions where the load is only being applied to one slab. The peak values for the loaded slab and the corresponding value for the unloaded slab is determined and LTE(S) is calculated using Equation 2, also shown below.

$$LTE(S) = \frac{\varepsilon_{unloaded}}{(\varepsilon_{loaded}+\varepsilon_{unloaded})} = \frac{.008968913}{.049188577+.008968913} = .15 \quad (2)$$

Where the unloaded strain, $\varepsilon_{unloaded}$, at 30.25 seconds into the pass is .00897 and the peak loaded strain, ε_{loaded}, at 30.25 seconds is .0492.

2.2 Single Gage

Dynamic LTE(S) is typically determined from paired strain gages, in some cases there may be only one gage. Brill et al. (2001) developed a procedure to calculate LTE(S) using a single gage (2). Figure 4 depicts a typical strain plot for single gage analysis. It shows the peak strain, peak time (T_p), estimated unloaded strain and estimated unloaded time (T_q). In Figure 4, T_p = 30.55 seconds and T_q = 30.25 seconds. The modified procedure for calculating unloaded strain and LTE(S) is explained below.

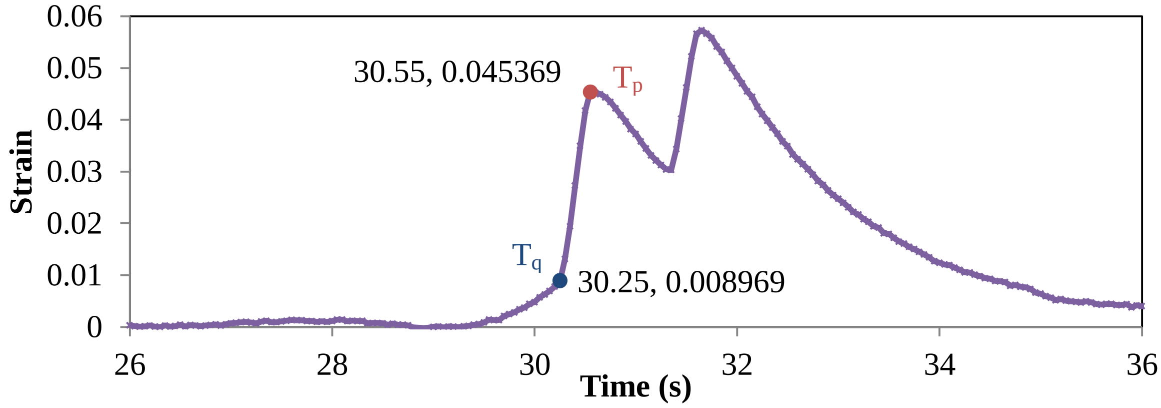

Figure 4: Typical Strain Profile for Single Gage Analysis

LTE(S) can be determined by predicting when the load is no longer on the slab edge. In Figure 4, the peak point T_p acts as the loaded strain, but the unloaded strain needs to be determined. Before the peak point is reached, the trend is both linear and quadratic. The point where the linear and quadratic portions meet is denoted as T_q and the time between T_q and T_p is denoted as Δt. By plotting all the strain values between points $T_q - 3\Delta t$ and T_q the quadratic equation $y = c_1x^2 + c_2x + b$ can be found. Then by plotting all the strain values between $T_q + 1$ and $T_p - 1$ the linear equation $y = mx + b$ can be found. The actual unloaded strain time T^*, is the instance when the quadratic trend meets the linear trend. Finally, to calculate LTE the strain at T^* must be divided by the sum of the strains at T^* and T_p. Figure 5 shows an example plot of the linear equation and quadratic equation.

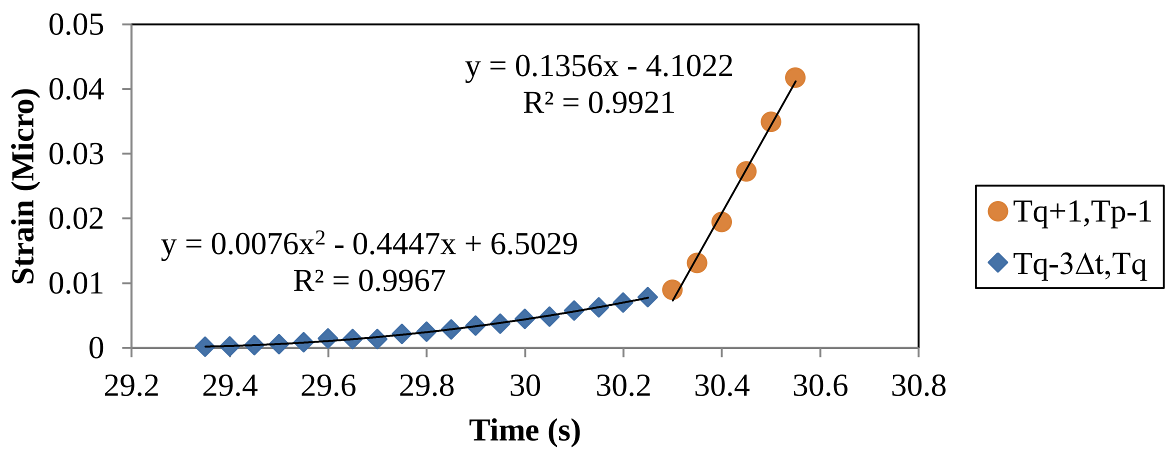

Figure 5: Linear and Quadratic Plots for Single Gage Analysis

The time period T_q and T_p can be found using the single gage plot and by subtracting T_q from $T_p$the Δt value can be determined to plug into the quadratic and linear plots.

$$Tq = 30.25s, \quad Tp = 30.55s, \quad \Delta\, t = 0.3s, \quad 3\Delta t = 0.9s$$

By setting the two equations equal to each other, the actual unloaded strain time T^* and the actual Δt can be calculated.

$$0.0076x^2 - 0.4447x + 6.5029 = 0.1356x - 4.1022 \quad (3)$$

$$T^* = x = 30.3101s, actual\ \Delta t = 0.24 \quad (4)$$

Then plugging the T^* value into both the linear and quadratic equations, the unloaded strains are calculated.

$$0.0076(30.3101)^2 - 0.4447(30.3101) + 6.5029 = 0.00868 \quad (5)$$

$$0.1356(30.3101) - 4.1022 = 0.00868 \quad (6)$$

Using the calculated unloaded strain and the peak strain from Figure 3 the LTE of the gage is calculated.

$$\frac{0.00868}{0.045369 + 0.00868} * 100\% = 16.06\%$$

LTE can only be calculated for one position using the single gage analysis method, because the two peaks depicted in the plot of the single gage analysis are either position 1 and 3 or position 2 and 4. Position 2 and 3 are peaks when the load is applied to both the approach and departure slab, which is why those positions are not analyzed for LTE(S).

3 TIME LAG

Raw strain responses for CC2 can be obtained from the online database located on the FAA's NAPTF website (http://www.airporttech.tc.faa.gov/naptf/). However, accessing this database requires some knowledge of structured query language (SQL) commands. A user with sufficient knowledge of SQL language can obtain, with the proper command prompt, raw strain gage data for CC2 that is time stamped. Table 2 shows detailed information for gages CSG-5 and CSG-7 including transverse and longitudinal position as well as peak strains for each gage being considered during passes 6415 and 6416.

Table 2: Detailed strain response for CSG-5 and CSG-7 for passes 6415 and 6416

Pass Direction	**Pass 6415**		**Pass 6416**	
Gage	**CSG-5**	**CSG-7**	**CSG-7**	**CSG-5**
Transverse Position, ft. (m)	-10 (3.0)	-10 (3.0)	-10 (3.0)	-10 (3.0)
Longitudinal Position, ft. (m)	354.75 (108.1)	355.25 (108.3)	355.25 (108.3)	354.75 (108.1)
First Peak (1st wheel)	9.9	10.2	14.9	15.25
Second Peak (2nd wheel)	11.15	11.5	16.2	16.55

From Table 2, we can calculate the theoretical speed of the test vehicle from a single gage. For CSG-5 the theoretical speed of the test vehicle can be calculated as seen below:

$$Speed = \frac{Axle\ Spacing}{(Time\ of\ Second\ Peak - Time\ of\ First\ Peak)} \quad (7)$$

$$Speed = \frac{4.75ft}{(11.15s - 9.9s)} = \frac{3.80ft}{s} = 2.59mph\ (4.17kph)$$

(8)

The resulting speed corresponds closely with the actual speed of the test vehicle of 2.5mph (4.02kph). When the same analysis is conducted for other gages or passes the results are in the 2.5±0.1mph (4.02±0.16kph) range. The same can be done for peaks between paired gages across a joint based upon the spacing of the gages and the time between peaks as seen for CSG-5 and CSG-7 below:

$$Speed = \frac{Gage\ Spacing}{([Time\ of\ First\ Peak\ CSG{-}7]-[Time\ of\ First\ Peak\ CSG{-}5])} \tag{9}$$

$$Speed = \frac{0.5ft}{(10.2s-9.9s)} = \frac{1.67ft}{s} = 1.14mph \tag{10}$$

From this example it can be seen that the vehicle appears to be traveling slower than actual carriage speed. When the same is conducted for other cases of consecutive peaks the results are in the 1.5±0.1mph (2.41±0.16kph) range. This would seem to indicate that the peak responses from consecutive gages do not necessarily occur as the center of the axle passes over the gage. The time lag for this example would be calculated as .136s. The general range for time lag on CC2 has been calculated to be between .3-.35s. The time lag calculated for CC6 would be .114s, while the general range for CC6 time lag values is around .25s.

4 DUAL VS. SINGLE GAGE ANALYSIS

4.1 CC2

Dual gage and single gage analysis was performed on MRC-N, MRC-S, MRG-S and MRS-S to determine the accuracy of single gage analysis on CC2 LTE(S). Only a select number of passes were analyzed at both the beginning and end of trafficking at each day throughout full duration of trafficking. More information on the gages analyzed, exact gage locations and load magnitude can be found in Cunliffe et al. (2012). Figure 6 shows the comparison between dual and single gage results for only MRC-N and MRC-S. Only MRC-N and MRC-S were loaded side by side with a dual tandem configuration, thus yielding the only comparison of North and South gages. It can be seen that most of the data points fall below the line of equality indicating that the single gage analysis would tend to under predict joint LTE (S) and is inherently conservative. The largest error of LTE (S) between single and dual gage analysis for MRC-N was 47% and the largest error for MRC-S was 41%, as seen in Figure 6. The largest error of LTE (S) between single and dual gage analysis on the south side for MRG-S was 52% while MRS-S was 40%.

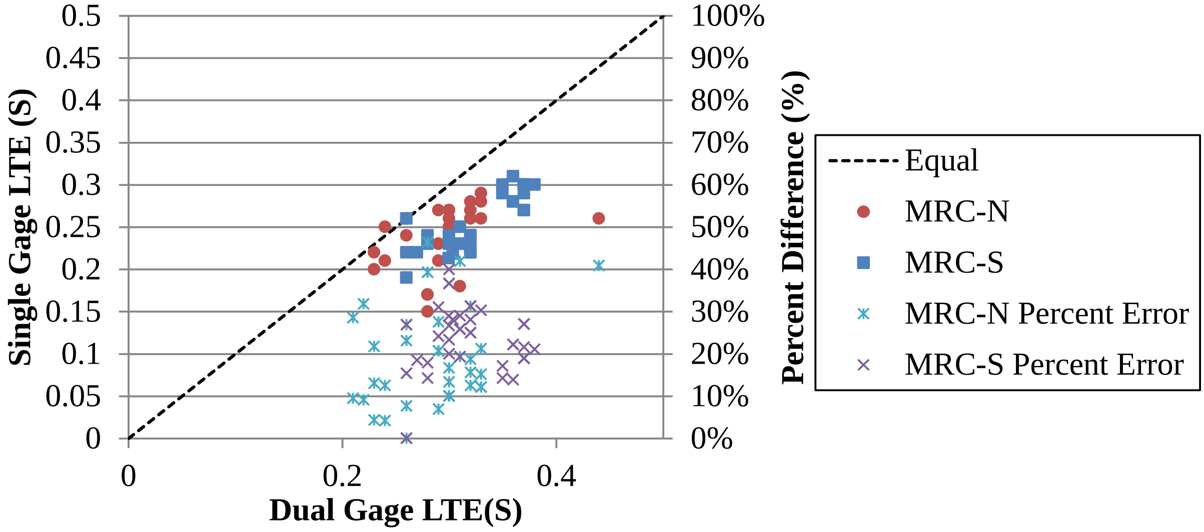

Figure 6: Comparison of Dual Gage LTE(S) and Single Gage LTE(S) for MRC-S and MRC-N

4.2 CC6

To determine the accuracy of single gage analysis on CC6, LTE(S) was calculated using single gage and dual gage analysis on MRS-2 and MRS-3 doweled joints. Only a select number of passes were analyzed at both the beginning and end of trafficking at each day throughout full duration of trafficking. More information on the gages analyzed, exact gage locations and load magnitude can be found in Blotta et al. (2012). Comparisons could not be conducted on MRS-1 as paired gages showed anomalous readings thus rendering LTE (S) determination unreliable. It can be seen that the data fits the line of equality very well indicating that single gage method was very comparable to dual gage method. In fact, the largest percent error between dual gage and single gage was only 6% for MRS-2 and 4.5% for MRS-3 (Figure 7).

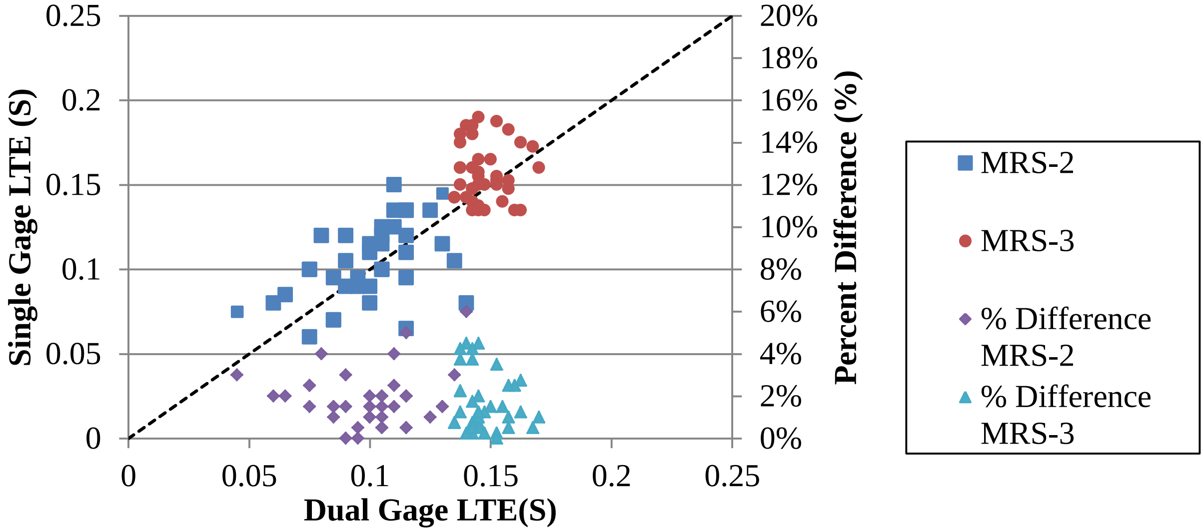

Figure 7: Comparison of Dual Gage LTE(S) and Single Gage LTE(S) for CC6 (Blotta)

4.3. CC2 vs. CC6 Results

As mentioned, there is a distinct difference when LTE (S) single gage approximation method is used for CC2 compared to CC6. In general single gage determination of LTE (S) for CC2 is approximately 20-30% conservative compared to dual gage results; whereas CC6 results were very compatible with all results within 6% for either method. The under estimation of single gage method for CC2 can be attributed to the distinct difference in shape of the CC2 strain gage responses profiles in comparison to CC6. Figure 8 shows the typical strain response for strain gages EG-67 and EG-70 located on the north MRS-2 test section of CC6. It is seen that the approximate point at which the strain response for EG-70 transitions from the quadratic to linear region (T_q) is quite discernible. The stiffness of the MRS-2 PCC and stabilized base layer is so high that the point at which the initiation of load transfer from loaded slab to unloaded slab is quite abrupt. Thus, when conducting regression analysis to find the actual unloaded strain time T^*, it is found that T_q and T^* agree very well resulting in an accurate approximation of LTE (S) from single gage. Figure 9 shows the typical strain response for strain gages CSG-5 and CSG-7 located on the MRC north test section of CC2. In this case, the point at which the CSG-7 strain response transitions from quadratic to linear is less clear. From this type of strain response, when conducting single gage analysis, $T^*$is often found to occur before T_q resulting in a slighltylower approximated unloaded strain which yields an underestimation of joint LTE (S). Because this pavement is on a granular base with significantly lower stiffness compared to stabilized base types, activation of the dowels for load transfer occurs less abruptly.

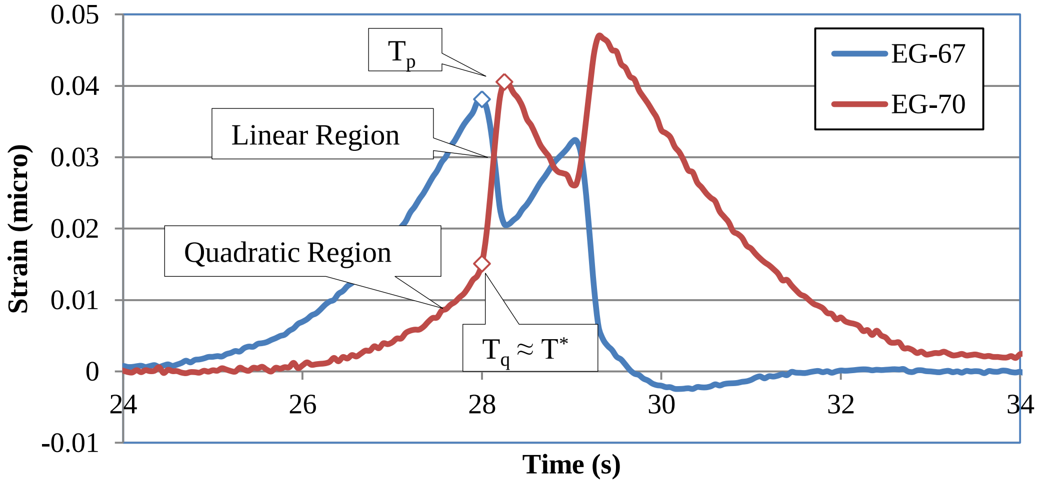

Figure 8: EG-67 & 70 strain response from CC6 MRS-2 north test section (Pass 477)

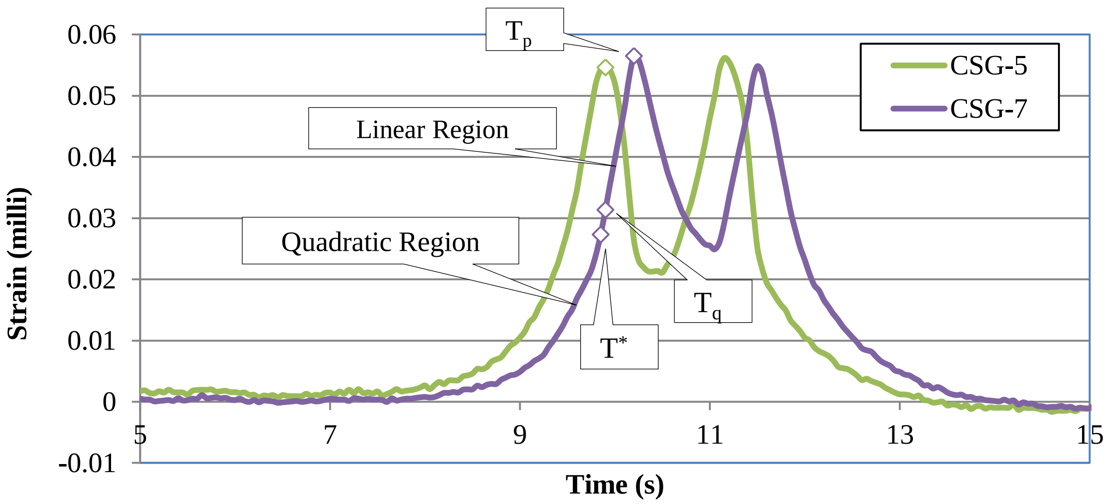

Figure 9: CSG-5 & 7 strain response from CC2 MRC north test section (Pass 6415)

5 SUMMARY AND CONCLUSIONS

5.1 Summary

The summary of findings is as follows:

1. The time lag between peak readings in consecutive gages for CC2 and CC6 seems to vary in the range of 0.30-0.35s and in the range of 0.25-0.30s, respectively.
2. Single gage determination of LTE (S) tended to be in the range of 20-30% lower than dual gage determination of LTE (S) for CC2 but as high as 52% as was the case for MRG-S.
3. Single gage determination of LTE (S) tended to be in the range of 0-4% different than dual gage determination of LTE (S) for CC2 but as high as 6% as was the case for MRS-2.

5.2 Conclusions

The conclusions based on the analysis conducted are:

1. The time lag found for CC2 corresponds with a vehicle speed in the range of 1.5±0.1mph (2.41±0.16kph) while the time lag for CC6 corresponds with a vehicle speed in the range of 1.25±0.1mph (2.41±0.16kph). This seems to indicate that the peak strain in each gage does not occur exactly as the axle (center of load) passes directly over the gage.
2. Single gage determination of LTE (S) for CC2 is considerably lower than that by dual gage determination which indicates that the single gage analysis is inherently conservative, which is not necessarily detrimental for design purposes.
3. Single gage determination of LTE (S) for CC6 is comparable to that by dual gage determination while results for CC2 were on average 25% more conservative than that by dual gage determination. Results would indicate that single gage method can be used successfully for high modulus of rupture pavements on stabilized base types.
4. LTE (S) from single gage method seems to be effected by material properties and pavement structure characteristics.

REFERENCES

Blotta, F., Mehta, Y.A., Cleary, D., Cunliffe, C., and Joshi, A, 2012. *Evaluation of Performance of Dowel and Transition Joints at the National Airport Pavement Testing Facility*, Proceedings of the Transportation Research Board 92nd Annual Meeting.

Brill, D.R., 2000. *Field Verification of a 3D Finite Element Rigid Airport Pavement Model*. Publication DOT/FAA/AR-00/33. DOT, U.S. Department of Transportation.

Brill, D.R., 2012. *Update on CC6.* Presented at Airport Pavement Working Group Meeting, Atlantic City.

Cunliffe, C., Mehta, Y.A., Cleary, D., and Joshi, A, 2012. *A Study to Determine the Impact of Cracking on Load Transfer Efficiency of Rigid Airfield Pavements*, Proceedings of the Transportation Research Board 92nd Annual Meeting

Smith, K. D. and Roesler, J. R., 2003. *Review of Fatigue Models for Concrete Airfield Pavement Design.* ASCE Airfield Pavement Specialty Conference, Las Vegas, NV.