Effects of Slab Temperature Profiles on Use of Falling Weight Deflectometer Data to Monitor Joint Performance and Detect Voids

Julie M. Vandenbossche

The primary objective of this research effort is to determine if temperature gradients affect the ability to use falling weight deflectometer (FWD) testing to monitor pavement joint performance and detect voids under the corners of the slab. A field investigation was performed at the Minnesota Road Research System test facility to meet this objective. It was found that gradients can have an effect on the results of FWD testing for rigid pavements. Although the load transfer efficiencies measured for doweled slabs were not found to be affected by slab temperature or temperature gradients (even when poor joint performance was exhibited), load transfer efficiencies measured for the undoweled slabs were greatly influenced by the presence of a gradient. There even appears to be a higher correlation between the gradient present at the time of testing and the average temperature of the slab for the pavement designs and environmental conditions included in this study. Gradients present at the time of testing also affect the ability to detect voids beneath the slab. Large positive gradients produced negative void parameters (indicating that a void was not present), whereas large negative gradients produced large positive void parameters (indicating that a void was present). This study found that loss of support under the slab could be identified even when the joints were locked if gradients were not present. On the basis of these findings, it is important to determine the complete temperature profile throughout the depth of the slab at the time of testing so that this information can be considered when the deflection data are interpreted.

With the aging of the highway infrastructure, pavement engineers must find a reliable means to assess the condition of their pavement networks. The most commonly used method to assess the condition of a portland cement concrete (PCC) pavement is falling weight deflectometer (FWD) testing. FWD testing is commonly performed to characterize joint performance and to determine if voids are present under the corners of the slab. Pavement structural design parameters and climatic conditions both affect the pavement response to applied loads (including FWD test loads). Temperature factors must be considered from two different perspectives: uniform changes in temperature and gradients that exist through the thickness of the slab. The primary focus of this research effort is to determine if temperature gradients affect the ability to use FWD testing to monitor joint performance and detect voids under the corners of the slab. A field investigation was performed at the Minnesota Road Research System (MnROAD) test facility to meet this objective. A description of the study and a discussion of the analysis are provided following a brief historical review.

University of Pittsburgh, 934 Benedum Hall, Pittsburgh, PA 15261, jmv@engr.pitt.edu.

Transportation Research Record: Journal of the Transportation Research Board, No. 2005, Transportation Research Board of the National Academies, Washington, D.C., 2007, pp. 75–85.
DOI: 10.3141/2005-09

HISTORICAL REVIEW

A historical review is provided on previous work pertaining to characterizing the effects of the slab temperature profile on use of FWD data to evaluate joint performance and detect voids under the slab. The method used to quantify joint performance and to detect voids throughout this study is established also.

Evaluation of Joint Efficiency

The joint performance for this study was quantified by dividing the deflection under the load plate and 152 mm (6 in.) away from the joint by the deflection measured 152 mm away from the joint on the unloaded side (see Equation 1). The load transfer efficiency (LTE) can be corrected for slab bending by using deflection data collected at midpanel. The LTE is multiplied by the deflection measured under the load at midpanel divided by the deflection 305 mm (12 in.) from the center of the applied load [see Equations 2 and 3 (*1*)]. The correction factor accounts for the fact that deflections are being measured 152 mm away from the joint and not directly at the joint. Khazanovich and Gotlif found the effects of bending to be minimal except in the case of very low slab stiffness [radius of relative stiffness less than 750 mm (30 in.)] (*2*). The radius of relative stiffness ranged between 145 cm and 81 cm (57 in. and 32 in.) for this study. Because of the large range in the radius of relative stiffness values for the test sections evaluated in this study, a correction factor for bending was used when the LTE was calculated (Equation 3).

$$\text{LTE} = \left[\frac{\delta_{ul}}{\delta_l}\right] \times 100\% \qquad (1)$$

where

LTE = deflection LTE (%),
δ_{ul} = deflection on unloaded side 152 mm away from joint or crack when testing in wheelpath adjacent to joint, and
δ_l = deflection on loaded side 152 mm away from joint or crack when testing in wheelpath adjacent to joint.

$$A = \frac{\delta_M}{\delta_{M12}} \tag{2}$$

where

A = correction for pavement stiffness,
δ_M = deflection at midpanel directly under load, and
δ_{M12} = deflection 25 mm (1 in.) from load applied at midpanel.

$$\text{LTE}_{\text{corrected}} = \text{LTE}_\delta \times A \times 100\% \tag{3}$$

where $\text{LTE}_{\text{corrected}}$ is the deflection LTE corrected for pavement stiffness as a percentage.

The LTE of a new undoweled jointed plain concrete pavement typically will range between 70% and 100% (*3*). The LTE of a new doweled pavement will range between 80% and 100%. The Long-Term Pavement Performance (LTPP) database shows that the LTEs of in-service jointed concrete pavements throughout the United States and Canada range from less than 20% to close to or slightly greater than 100% (*2*). (Khazanovich and Gotlif used a finite element analysis to show that LTEs greater than 100% can be achieved when upward curling of the slab or voids is present.) Only approximately 20% of the undoweled joints tested had LTEs greater than 70%, whereas 45% of the doweled joints had LTEs greater than 70%. A deflection LTE less than 70% typically is considered unacceptable. The U.S. Federal Highway Administration suggests that load transfer restoration be performed to prevent further damage to the pavement if any of the following conditions are present (*4*):

- Faulting of individual joints or cracks of 3 mm (0.12 in.) or more,
- Deflection load transfer of less than 70%,
- Differential deflection between approach and leave slab of over 0.25 mm (0.01 in.), or
- Cumulative faulting of joints and cracks of over 525 mm/km (33 in./mile).

Effects of Uniform Temperature Change on LTE

Significant uniform temperature changes in the slab occur as a result of both daily and seasonal temperature oscillations. Temperature-related expansion and contraction of the slab affect the deflections when FWD testing is performed at the edge and corner and in the wheelpath. A uniform temperature increase in the slab will cause the slab to expand. As the slab expands, crack and joint widths decrease. This decrease results in an increase in the apparent stiffness of an undoweled joint or crack and an increase in load transfer potential. Increases in load transfer will decrease the deflections of the loaded slab while increasing the deflections for the unloaded slab.

A uniform reduction in temperature through the slab thickness will cause the slab to contract, resulting in increased joint and crack widths. This effect reduces load transfer potential and increases differential deflections between the loaded and unloaded sides of the slab. Previous research has reported that a small increase in joint opening results in a significant loss in load transfer for an undoweled joint. The LTE for an undoweled joint can decrease to less than 50% when the joint width is greater than 0.9 mm (0.035 in.) (*3*). In 1989 Edwards et al. measured LTEs of 50% in the morning that increased to 90% later in the afternoon (*5*). In 1990, Greer measured seasonal changes in LTE that ranged from 16% to 84% caused by seasonal average slab temperature changes between the winter and spring months (*5*). These effects are not as significant when load transfer devices are present.

The finite element method and field measurements have been used to develop several models relating joint width to load transfer. Nishizawa et al. developed the following empirical linear relationship between joint width (ω) in millimeters and percent LTE (*6*):

$$\text{LTE}_\delta = 100 - 25(\omega) \tag{4}$$

Foxworthy and Darter developed a method for predicting load transfer at one temperature for a joint transfer efficiency that was measured at a different temperature (*7*). They defined the relationship between load transfer and air temperature with an S-shaped curve with specific upper and lower bounds (100% and 25% to 30%) and slope or rate at which the LTE approaches the bounds. This relationship is then used to determine the LTE for a particular joint when there is a shift in temperature.

As described earlier, methodologies have been developed to look at the effects of uniform changes in slab temperature on LTE. The effects of temperature gradients on LTE have not been investigated as thoroughly.

Effects of Temperature Gradients on LTE

The change in temperature with slab depth results in a change in joint width with depth and vertical slab movement (*8*). Jeong et. al showed that slab curling can affect the load transfer at a doweled joint when there is dowel looseness. They reported that effective dowel locking due to the upward curvature caused by negative gradients will result in higher LTEs. Khazanovich and Gotlif performed an analytical study to investigate the effects of voids or curling under the slab and determined the effects to be influential. The researchers suggested that correction for these effects on LTE should be incorporated into the bending factor (*2*).

Void Detection

Another primary reason for performing FWD testing on rigid pavements is to detect voids under the slab. Deflections typically are highest when loads are applied to the slab corner. These high deflections can lead to permanent deformation of the underlying support layers, which results in a void under the slab. Applied loads also will pump fines from the base out from under the pavement when excessive moisture, high deflections, and an unstabilized base material are present, resulting in a loss of support under the slab. High slab stresses arise from these areas of reduced support and cause rapid deterioration.

Voids can be detected from the deflection data gathered when FWD testing is done in the corner of the slab directly adjacent to the transverse and lane or shoulder joints. Subsealing should be performed if significant voids are present. The variable corner deflection analysis procedure (*9*) was used to analyze all of the FWD data collected at the corner of the slab during the three collection periods. This procedure consists of plotting the load for three different load levels [27, 40, and 67 kN (6, 9, and 15 kips)] against its corresponding deflection (measured directly under the applied load). A best-fit regression line is then drawn through these three data points, and the *x*-intercept is used as an estimate of the void size, which will be referred to as the void parameter. The void parameter increases as the size of the void present beneath the slab increases. If the regression line drawn through the group of points extends through the *x*-axis near the origin [within 0.05 mm (2 mils)], it is assumed that no void is present. Increased support caused by downward curling of the slab

TABLE 1 Summary of Concrete Test Cell Design Features at MnROAD

Test Section	Cell	Slab Thickness, mm (in.)	Joint Spacing, m (ft)	Lane Widths, Inside–Outside, m (ft)	Dowel Diameter, mm (in.)	Base Type,[a] Thickness, mm (in.)	Edge Drains	Comments
5-year	5	190 (7.5)	6.1 (20)	4.0/4.3 (13/14)	25 (1)	cl4sp, 75 (3) over cl3sp (68)	No	
5-year	6	190 (7.5)	4.6 (15)	4.0/4.3 (13/14)	25 (1)	cl4sp, 125 (5)	No	
5-year	7	190 (7.5)	6.1 (20)	4.0/4.3 (13/14)	25 (1)	PASB[b] 100 (4) over cl4sp, 75 (3)	Yes	
5-year	8	190 (7.5)	4.6 (15)	4.0/4.0/4.3 (13/13/14)	25 (1)	PASB[b] 100 (4) over cl4sp, 75	Yes	3 lanes, transverse steel
5-year	9	190 (7.5)	4.6 (15)	4.0/4.0/4.3 (13/13/14)	25 (1)	PASB,[b] 100 (4) over cl4sp, 75 (3)	Yes	3 lanes, no transverse steel
10-year	10	240 (9.5)	6.1 (20)	3.7/3.7 (12/12)	32 (1.25)	PASB,[b] 100 (4) over cl4sp, (75)	Yes	
10-year	11	240 (9.5)	7.3 (24)	3.7/3.7 (12/12)	32 (1.25)	cl5sp, 125 (5)	No	
10-year	12	240 (9.5)	4.6 (15)	3.7/3.7 (12/12)	32 (1.25)	cl5sp, 125 (5)	Yes	
10-year	13	240 (9.5)	6.1 (20)	3.7/3.7 (12/12)	32 (1.25)	cl5sp, 125 (5)	No	

[a]See Table 2 for the gradations of these materials.
[b]Permeable asphalt stabilized base (PASB).

corners in the presence of a negative gradient is indicated in the graphs by negative void parameters (intercepts that intersect to the left of the *y*-axis).

Traditionally, it has been recommended that void detection testing be performed at a slab temperature of less than 27°C (80°F), when the joint and cracks are not locked up because of thermal expansion and when there are no significant temperature gradients present. The increase in stiffness at the joint as a result of testing when slab temperatures are high has been reported to reduce corner deflections, making it difficult to detect the presence of voids (*1*). Crovetti performed an analytical study showing that a large positive gradient causes the corners of the slab to curl downward, thereby potentially providing a false negative (not detecting a void that is present) when void detection is attempted. Likewise, a false positive (detecting a void that is not present) can be obtained when testing is done in the presence of a negative gradient, which will cause the slab corners to curl upward (*9*). Little work has been performed to verify this analysis with actual field data.

DESCRIPTION OF FIELD STUDY

Test Sections

MnROAD is a densely instrumented pavement test facility constructed adjacent to I-94 approximately 65 km (40 mi) northwest of Minneapolis, Minnesota. This study includes data collected from the nine test cells that constitute the 5-year mainline and 10-year mainline concrete test sections, representing different combinations of slab thicknesses, joint spacings, restraint conditions, and subbase types. The design parameters for each test cell are provided in Table 1. The gradations for each base type referenced in Table 1 are shown in Table 2. All of the test sections have asphalt shoulders. The test sections contain temperature, moisture, and static strain sensors, which are connected to a data acquisition system that collects readings from each sensor every 15 min 24 h a day. The data are then stored in a database for easy retrieval and analysis.

All test sections were constructed originally with dowels and tie bars. Although many concrete pavements are built with load transfer devices, pavements are still constructed without them. It was desired to measure the effects of temperature profiles on the LTE and the detection of voids for slabs restrained by load transfer devices as well as for those not restrained. The dowel and tie bars were cored from the joints surrounding one slab that represented each pavement design 2 years after paving. The released slabs allow comparisons between restrained (doweled) and unrestrained (undoweled) slabs under otherwise identical conditions.

FWD Testing

A Dynatest 8000 FWD was used for all of the FWD testing performed. FWD data were collected on three different occasions (September, April, and October) in an attempt to quantify seasonal effects. Each test period required a minimum of 24-h data collection to

TABLE 2 Aggregate Gradations in Percent Passing for MnROAD Base Materials

Sieve Size	Base Material: cl3sp	cl4sp	cl5sp	PASB
37.5-mm (1½-in.)	—	100	—	—
31.5-mm (1¼-in.)	—	—	—	100
25.0-mm (1-in.)	—	95–100	100	95–100
19.0-mm (¾-in.)	—	90–100	90–100	85–98
12.5-mm (½-in.)	100	—	—	—
9.50-mm (⅜-in.)	95–100	80–95	70–85	50–80
4.75-mm (No. 4)	85–100	70–85	55–70	20–50
2.00-mm (No. 10)	65–90	55–70	35–55	0–20
850-µm (No. 20)	—	—	—	0–8
425-µm (No. 40)	30–50	15–30	15–30	0–5
75-µm (No. 200)	8–15	5–10	3–8	0–3

NOTE: Special crushing requirements (sp):
cl3sp and cl4sp: crushed/fractured particles are not allowed.
cl5sp: 10%–15% crushed/fractured particles are required.

capture diurnal effects by determining data for each test cell under zero, negative, and positive gradient conditions.

ANALYSIS OF DATA

Joint Lock-Up Temperatures

Typically, it is recommended that FWD testing not be performed when temperatures are greater than 27°C (80°F), because the joints will be locked and a true measure of the ability of the joint to transfer load cannot be made (*1*). Instead of using this arbitrary temperature, static strain sensors were used to determine the temperature at which the slab expands to the point at which the joints lock. The test sections include vibrating wire strain gauges oriented in the longitudinal direction. The sensors used for this analysis are located 45 cm (18 in.) from the lane–shoulder joint and 45 cm (18 in.) from the longitudinal joint on both the approach and leave sides of the slab. Strain was calculated as follows:

$$\mu\epsilon_s = (R_1 - R_0) + (T_1 - T_0)CF_1 \tag{5}$$

where

$\mu\epsilon_s$ = static strain (microstrain),
R_0 = strain reading when no gradient is present just prior to R_1 (microstrain),
R_1 = strain reading at time of interest (microstrain),
T_0 = temperature at time of initial strain reading (°C),
T_1 = temperature at time of interest (°C), and
CF_1 = thermal coefficient of steel used for vibrating wire in strain gauge (microstrain/°C).

As previously stated, strain and temperature measurements were collected at 15-min intervals. The static strains were plotted against temperature, as shown in Figure 1, throughout a 24-h period. As temperature increased, the strain increased up to the temperature at which the joint locked. In Figure 1*a*, the rate of increasing strain with increasing temperature begins to decrease at approximately 10°C (50°F), indicating that the joint was starting to lock up. The temperature continues to increase above 10°C (50°F), with smaller increases in strain. Figure 1*b* shows static strain plotted against temperature for a 24-h period in which the temperature of the slab did not increase to the point at which the joints locked. In Figure 1*b*, the strain continues to increase approximately linearly with increasing strain because the temperature was not sufficiently high to lock the joint.

Strain was plotted against temperature for each day testing was performed to determine when the joints were locked. It was determined from these plots that none of the joints locked up during the September data collection period. The joints for the 6.1-m (20-ft) panels in both the 5- and 10-year sections (Cells 5 and 10, respectively) locked at times during the April collection period. Cell 5 and Cell 6 also had locked joints at times during the October testing.

The joint lock-up temperatures were influenced by rain events. For example, during the April data collection period, the joints in

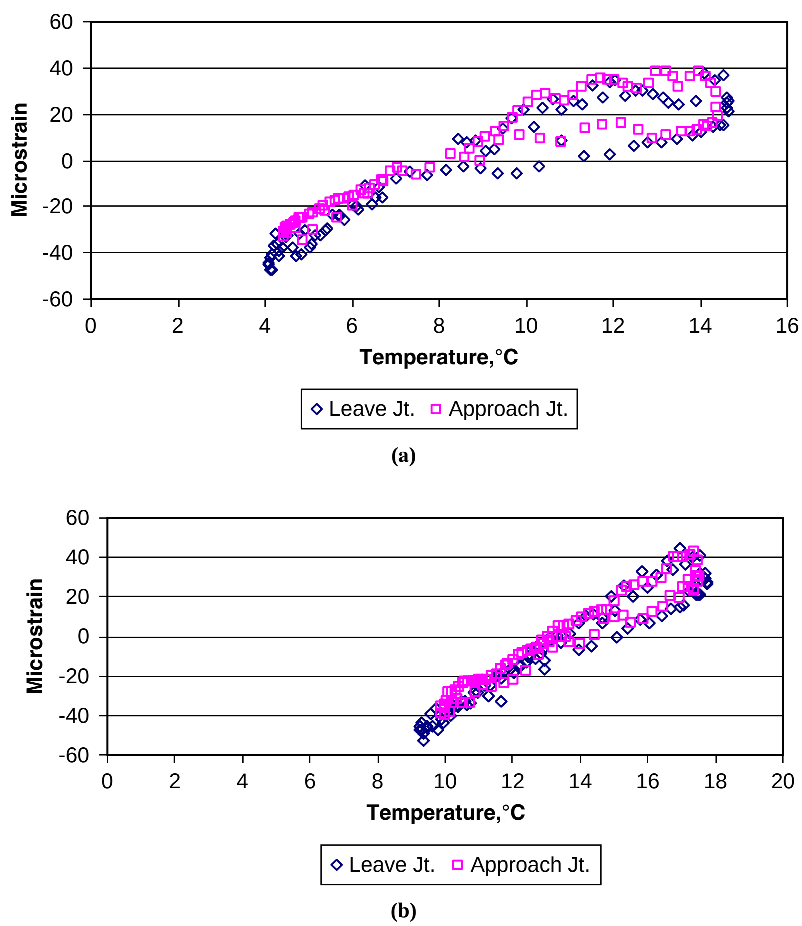


FIGURE 1 Determining occurrence of joint lock-up: (*a*) strain measurements when joint did lock up and (*b*) strain measurements when joint did not lock up.

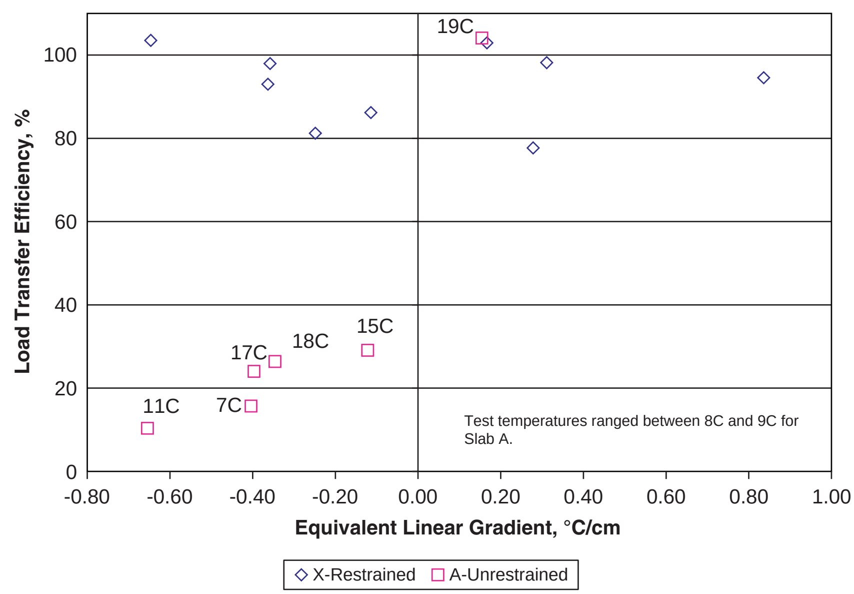
Cell 5 were found to lock up at a temperature that was 3°C (37°F) higher than the lock-up temperature after a particular rain event. The length of the slab and the type of base also influenced the joint lock-up temperature. The joints for the longer slabs [6.1 m (20 ft)] on a stabilized base locked at temperatures 3°C (37°F) higher than the joints for the shorter slabs [4.6 m (15 ft)] on a granular base. A more complete description of this analysis and all of the supporting graphs may be found elsewhere (*10*).

All FWD testing for this study was performed when slab temperatures were lower than 22°C (80°F) and most testing was performed when temperatures were below 21°C (70°F). The strain measurements indicate that the joints for these test sections lock up when the ambient temperature is between 27°C and 24°C (80°F and 75°F) (*10*).

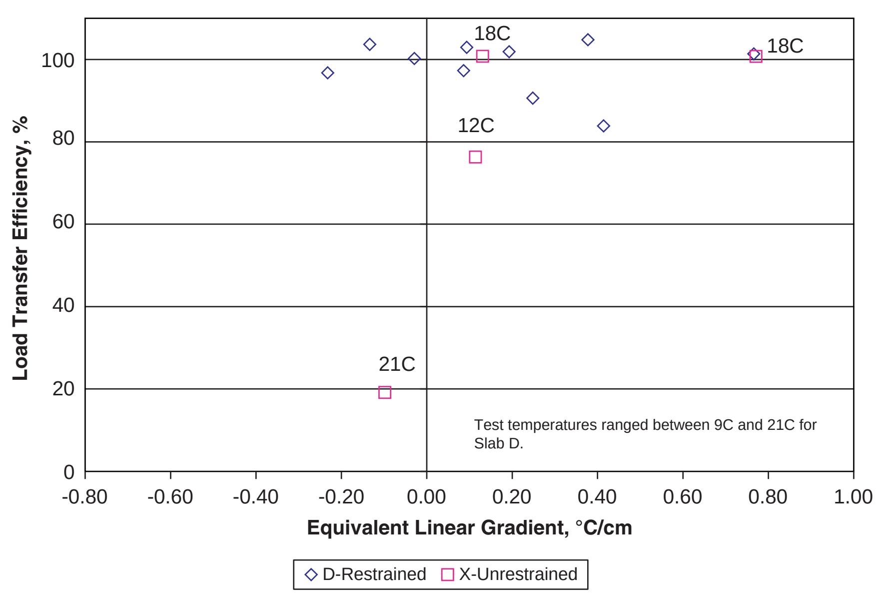
Analysis of LTE

LTEs were calculated for joints in each test section by using Equation 3 and deflections measured in the wheelpath on the approach side and adjacent to the transverse joint. The LTE used represents the average value from three 40-kN (9-kip) drops. The load range for all tests was between 39.0 and 44.8 kN (8,700 and 10,100 lb).

The calculated LTEs plotted against the equivalent linear gradient (*11*) are shown in Figures 2 through 5. The average slab temperature at the time of testing is provided next to each data point recorded for the unrestrained slabs. All plots contain LTEs measured for slabs with and without restraint (dowels and tie bars) for each pavement design. Solid data point markers would indicate that testing was performed when the joint was locked.

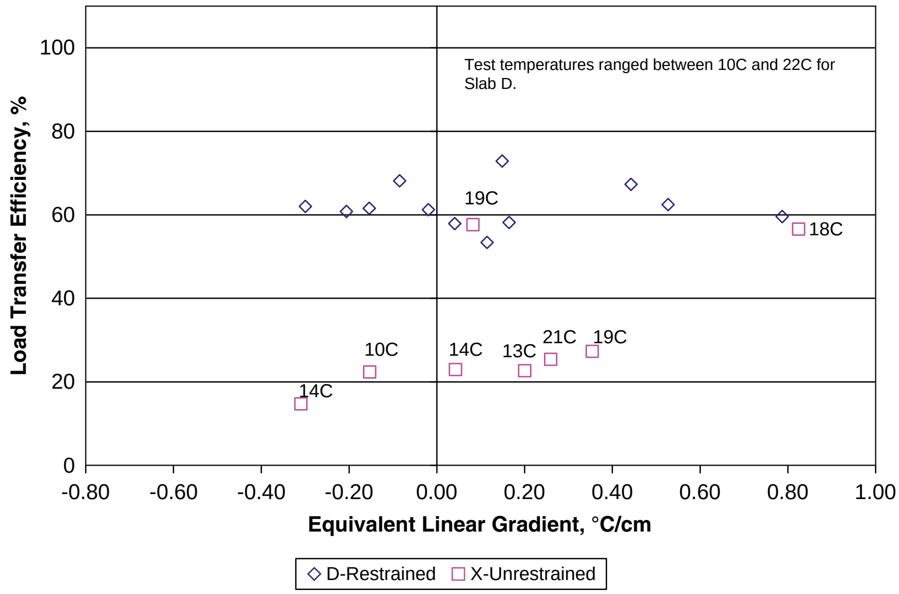


(a)

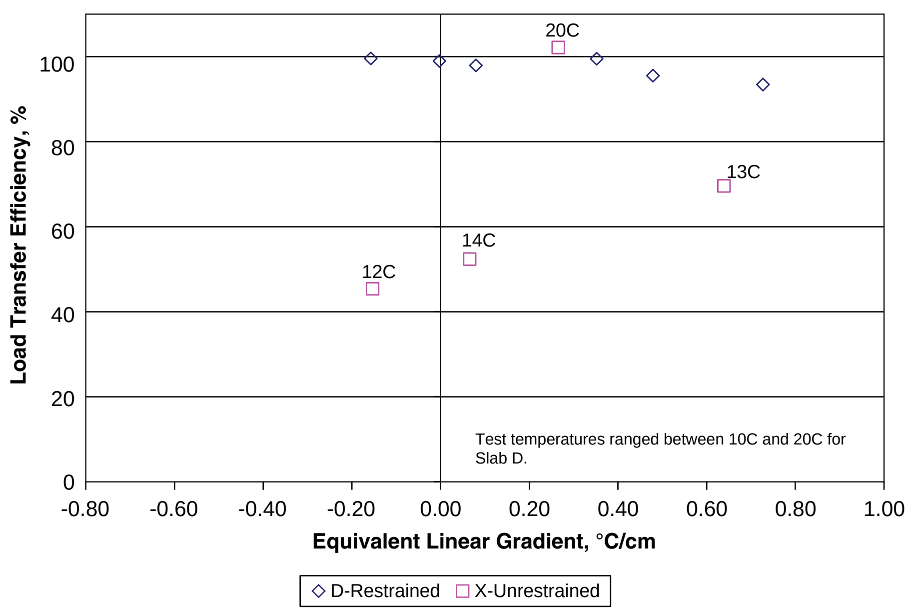


(b)

FIGURE 2 Relationship between load transfer and gradients for 190-mm (7.5-in.) slabs on granular base: (*a*) Cell 5, slab length = 6.1 m (21 ft), and (*b*) Cell 6, slab length = 4.6 m (15 ft).



(a)



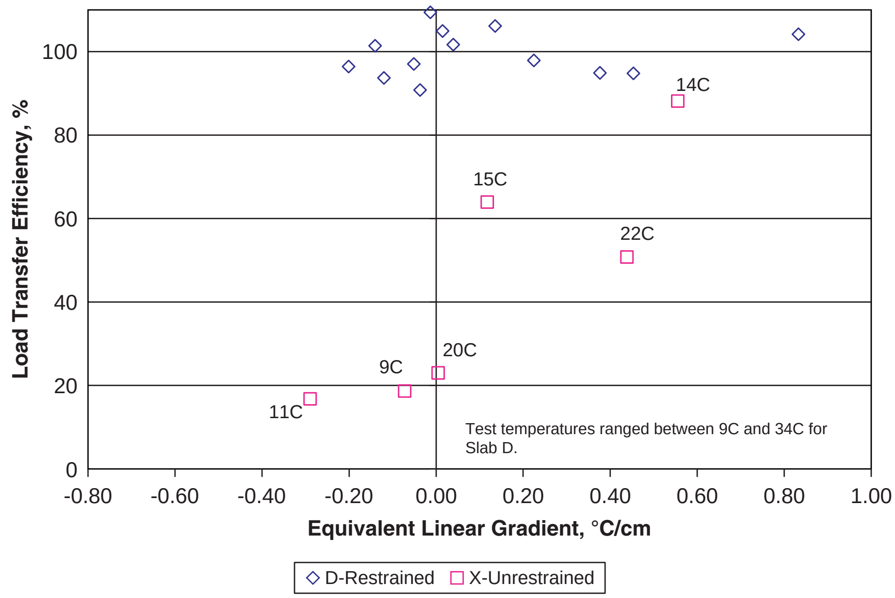
(b)

FIGURE 3 Relationship between load transfer and gradients for 190-mm (7.5-in.) slabs on asphalt-stabilized base: (*a*) Cell 7, slab length = 6.1 m (21 ft), and (*b*) Cell 8, slab length = 4.6 m (15 ft).

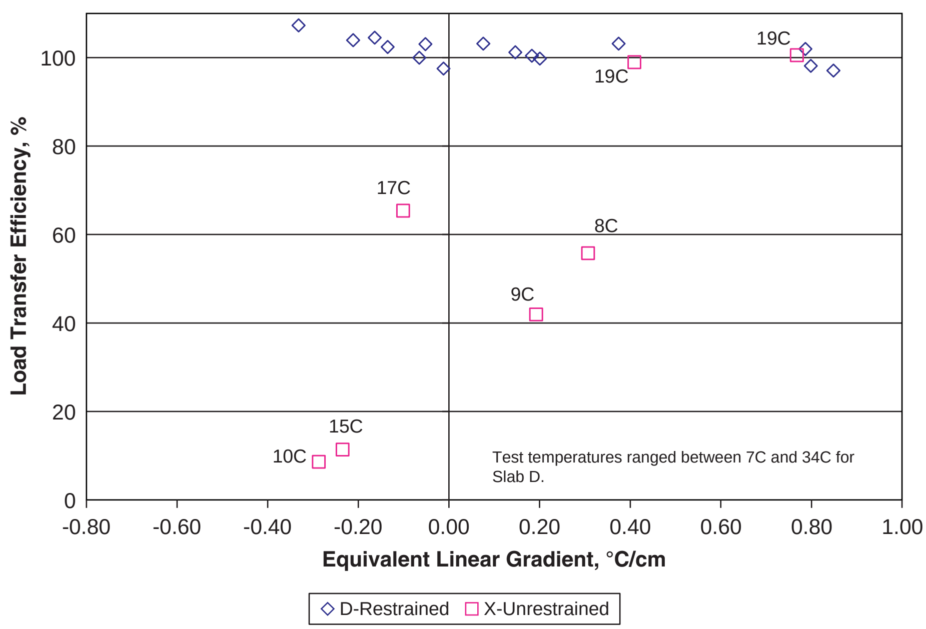
The measured LTE was influenced by the gradient present during testing in the unrestrained slabs. Some of the slabs did not exhibit a strong correlation with the equivalent linear gradient during the September data collection period, but the correlation strengthened during the subsequent April and October data collection periods. As previously mentioned, the dowels and tie bars in the unrestrained sections were cored just prior to testing in September. The unrestrained slabs may not have had sufficient time or vehicle loadings to break free and become truly unrestrained prior to the data collection period in September. For this reason, the September deflection data were excluded from the analysis for the unrestrained slabs.

The results of this analysis show that LTEs measured for doweled slabs are not affected by temperature gradients or slab temperature, regardless of the dowel size [25-, 32-, and 38-mm (1-, 1.25-, and 1.5-in.) dowels were considered]. This finding held true even for Cell 7, which contained a doweled slab with a low LTE (60%) (see Figure 3*a*). (Cell 7 was the only cell with a restrained slab having an LTE below the acceptable level.) Even with this low LTE, the LTE did not fluctuate when different gradients were present at the time of testing.

The load transfer measured for the unrestrained slabs is highly dependent on the temperature gradient present at the time of test-



(a)



(b)

FIGURE 4 Relationship between load transfer and gradients for 240-mm (9.5-in.) slabs with 6.1-m (20-ft) joint spacings: (*a*) Cell 10, asphalt-stabilized base, and (*b*) Cell 13, granular base.

ing. The LTE increases as the gradient increases (downward slab curvature) and decreases when negative gradients (upward slab curvature) are present. Positive gradients develop in the afternoon when the temperature of the slab is higher. The higher temperatures would tend to increase the LTE because the cracks at the joints would close as the slab expands. However, Figures 2 through 5 show that when testing is performed on a slab at times when the average temperature of the slab is the same but the gradients are different, the LTEs will be different. For example, in Figure 2*a* three separate tests were performed when the average slab temperature ranged between 17°C (63°F) and 19°C (66°F), and yet the LTE for these three tests ranged between 25% and 100%. The largest positive gradient corresponds with the largest LTE and the largest negative gradient corresponds with the lowest LTE. Figure 5 shows that when the average slab temperature is 20°C (68°F) with no gradient present, the LTE is lower by approximately 20% compared with that when the slab temperature is 11°C (52°F) and a large gradient is present. Similar trends can be found in all of the graphs.

Load transfer restoration is suggested when the load transfer drops below 70%. The load transfer of all of the unrestrained slabs dropped below this unacceptable level during the data collection period in

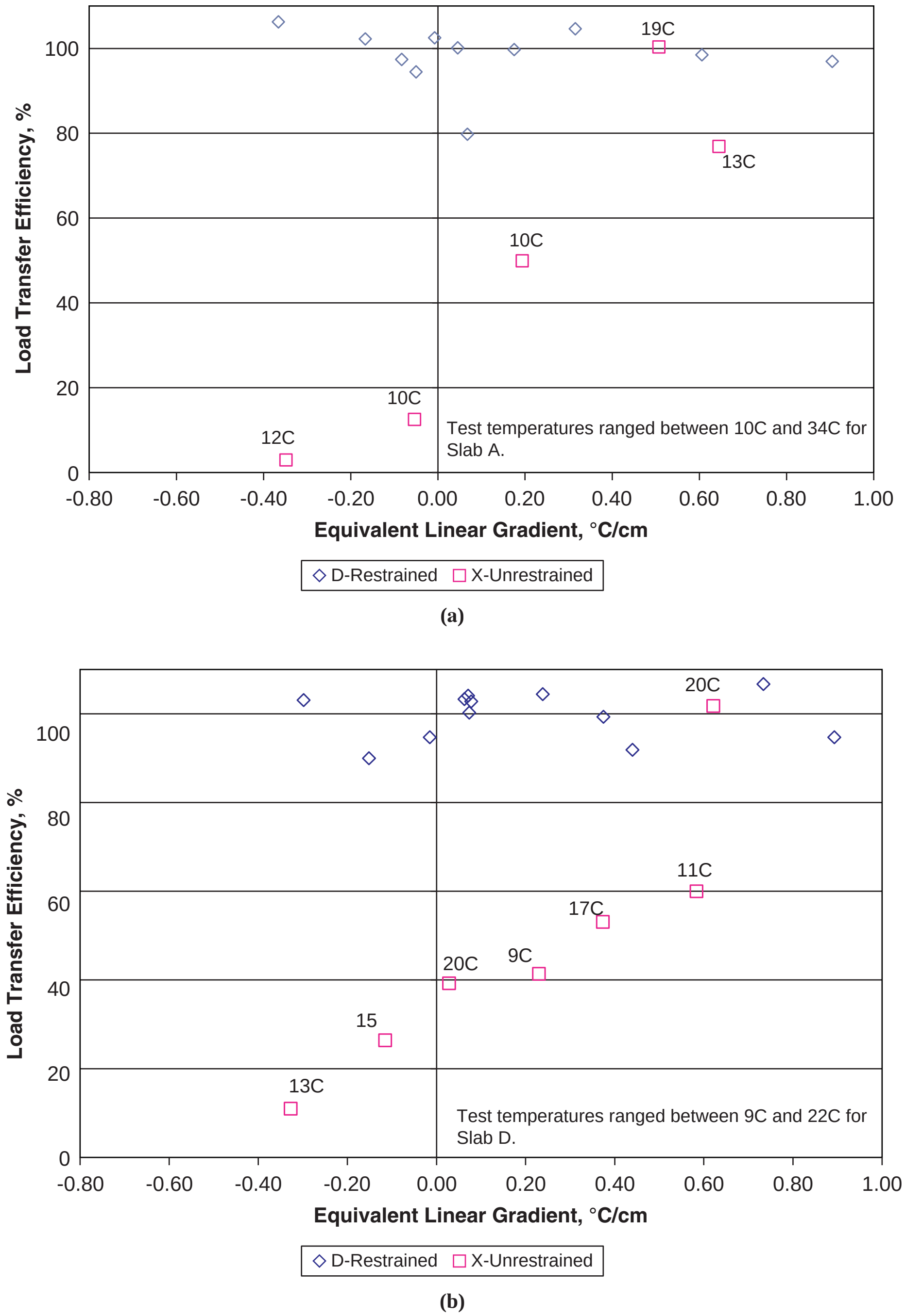


FIGURE 5 Relationship between load transfer and gradients for 240-mm (9.5-in.) slabs with 4.6-m (15-ft) joint spacings: (*a*) Cell 11, asphalt-stabilized base, and (*b*) Cell 12, granular base.

April. This finding emphasizes the need for load transfer devices for long-term performance. The graphs indicate that some of the joints would exhibit an acceptable level of LTE if the testing was performed when a large positive gradient was present, but throughout the majority of the day the LTE would be below the acceptable level. Therefore, one can conclude that undoweled pavements in a climate similar to that found in Minnesota will have low load transfer capabilities for a large portion of each day (and in most cases throughout the day) regardless of the aggregate interlock surface texture available at the slab face for load transfer.

The design variables evaluated include joint spacing, base type, and slab thickness. Figure 2 shows the relationship between LTE and the equivalent linear gradient for the thinner slabs [190 mm (7.5 in.)] placed on a granular base for both 6.1-m (20-ft) and 4.6-m (15-ft) joint spacings. Figures 4 and 5 represent the thicker slabs [240 mm (9.5 ft)] with 6.1-m (20-ft) and 4.6-m (15-ft) joint spacings, respectively, on both granular and asphalt-stabilized bases. Evidence was insufficient to show that joint spacing, base type, or slab thickness significantly influenced the relationship between the slab temperature conditions and the LTE for the range of variables considered.

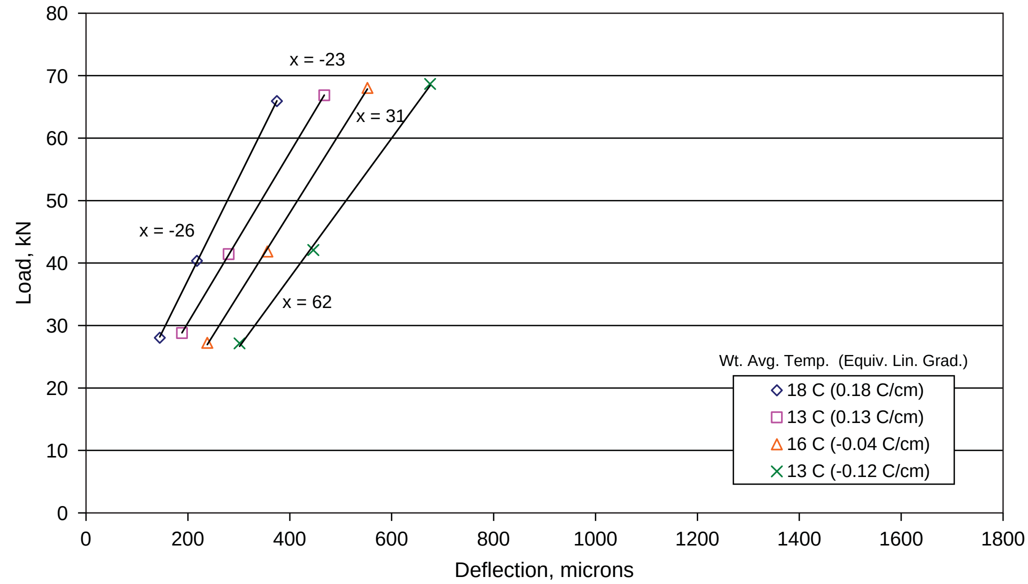


FIGURE 6 Deflections and loads measured in September for unrestrained slab in Cell 5.

Analysis of Void Detection

Deflection and load data collected for the unrestrained slab in Cell 5 are shown in Figures 6 through 8. In each graph, the *x*-intercept calculated for each data set is provided adjacent to its corresponding data series, and the legend contains the average slab temperature and the temperature gradient at the time testing was performed for each data series.

In comparing temperature gradients present at the time the data were collected with the calculated void parameter (the *x*-intercept) in Figures 6 through 8, a good correlation is found between the magnitude of the void parameter and the size of the gradient. Large positive gradients produce negative void parameters (indicating that a void is not present), whereas large negative gradients produce large positive void parameters (indicating that a void is present). These findings clearly show the effect of gradients on void detection. In Figures 6 and 8, voids are not detected while the joint is locked. In this scenario, positive gradients are present in all cases. Therefore, the slab is curled downward, and a false negative is possible. Data from other cells did indicate voids to be present when the joints were locked and negative gradients were present. From these results, the loss of support under the slab can be identified even when the joints are locked.

Graphs similar to Figures 6 through 8 were developed for each cell to confirm the relationship between void parameter and temperature gradient. Graphs of this type for all of the test cells may be found elsewhere (*11*). Once this relationship was confirmed, the equivalent linear gradient was plotted against the void parameter for each cell and each data collection period for further evaluation. An example of these graphs can be found in Figures 9*a* and *b*; the

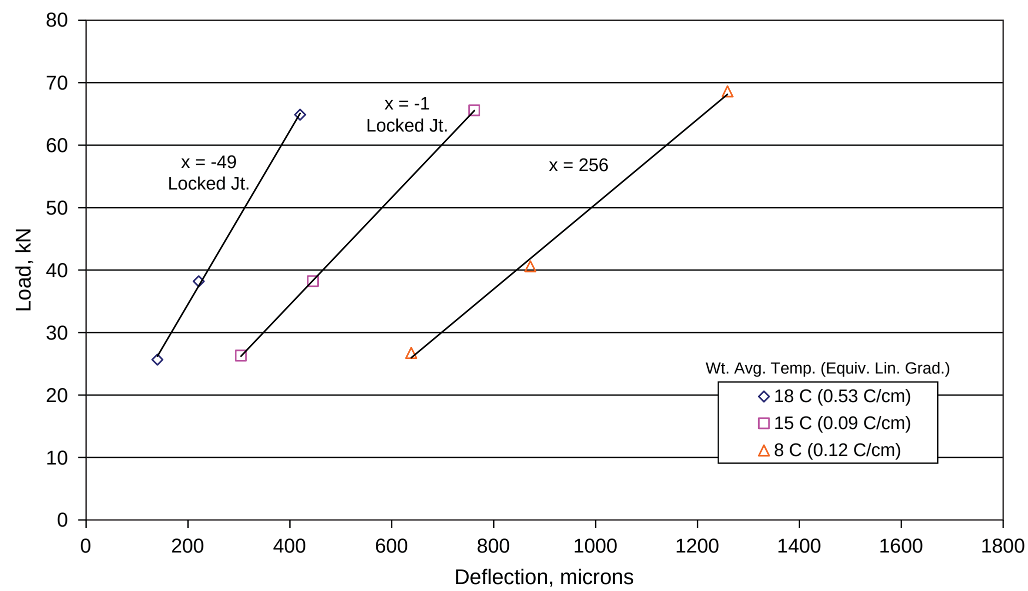


FIGURE 7 Deflections and loads measured in April for unrestrained slab in Cell 5.

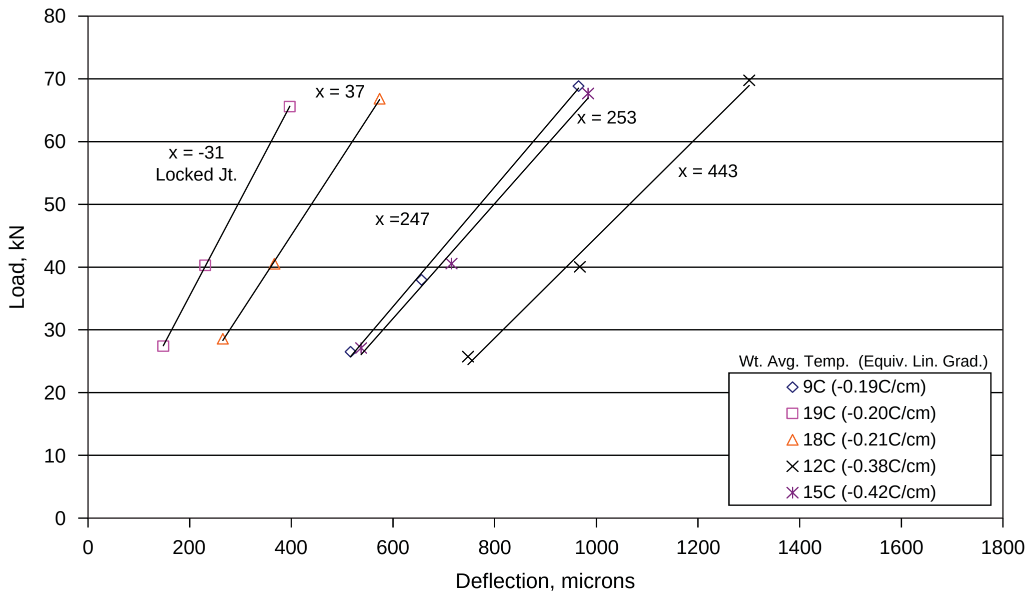
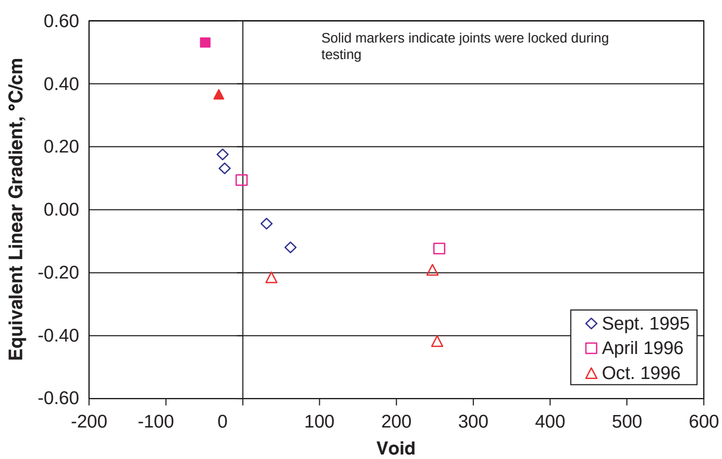
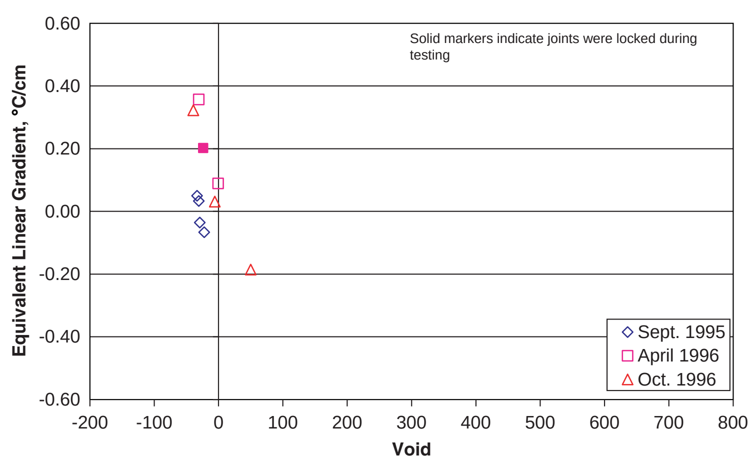


FIGURE 8 Deflections and loads measured in October for unrestrained slab in Cell 5.



(a)



(b)

FIGURE 9 Relationship between void parameter and equivalent linear gradient present during testing: (*a*) unrestrained slab in Cell 5 and (*b*) restrained slab in Cell 11.

complete set of graphs may be found elsewhere (*11*). These graphs allow a closer examination of the relationship between the equivalent linear gradient present at the time the FWD testing is performed and the magnitude of the size of the void. The void calculated for the unrestrained slabs was highly dependent on the temperature gradient present at the time of testing for all cells. This relationship was not true for all of the restrained slabs in each cell. The void size for the restrained slabs in Cells 7, 9, 10, and 11 did not fluctuate with changes in temperature gradients (see Figure 9*b*), most likely because the slabs were not tested over a sufficiently large range of temperature gradients. Therefore, temperature gradients do appear to influence the ability to use FWD data to detect voids. A correction must be made to the indicated void size to account for the loss of support caused by slab curvature before an estimate of the void size (if a void exists) can be made.

CONCLUSIONS

It was found that temperature gradients can have an effect on the results of FWD testing for rigid pavements. The LTEs measured for doweled slabs are not affected by temperature gradients or slab temperature. Even doweled slabs with a low LTE (60%) were not affected by temperature gradients. LTEs measured for the unrestrained slabs were greatly affected by the presence of a gradient. There even appeared to be a higher correlation between the gradient present at the time of testing and the average temperature of the slab for the pavement designs and environmental conditions included in this study.

Gradients present at the time of testing also affect the ability to detect voids beneath the slab. Large positive gradients produced negative void parameters (indicating that a void is not present), whereas large negative gradients produced large positive void parameters (indicating that a void is present). This study found that the loss of support under the slab can be identified even when the joints are locked if gradients are not present.

This study has shown that temperature gradients do have an effect on the interpretation of FWD data for rigid pavements when joint performance is evaluated and voids are detected. At times, this influence can be greater than that of the average slab temperature. For this reason, it is important to determine the complete temperature profile throughout the depth of slab at the time of testing so that this information can be considered when the deflection data are interpreted.

ACKNOWLEDGMENTS

The author acknowledges the Minnesota Department of Transportation and the Federal Highway Administration for their financial support and extends sincere appreciation to Mark Snyder and Donald Janssen for the opportunity to work on this research effort. The assistance of Jack Herndon, Robert Strommen, Gregory Larson, and the personnel at the MnROAD research facility is greatly appreciated also.

REFERENCES

1. *AASHTO Guide for Design of Pavement Structures.* American Association of State Highway Transportation Officials, Washington, D.C., 1993.
2. Khazanovich, L., and A. Gotlif. *Evaluation of Joint and Crack Load Transfer Final Report.* Report FHWA-RD-02-088. FHWA, U.S. Department of Transportation, Jan. 2002.
3. ERES Consultants. *Techniques for Pavement Rehabilitation: A Training Course,* 3rd ed. FHWA-HI-90-022. National Highway Institute, Arlington Va., Oct. 1987.
4. *Pavement Rehabilitation Manual.* FHWA-ED-88-025. FHWA, U.S. Department of Transportation, Oct. 1990.
5. Armaghani, J. M., J. M. Lybas, M. Tia, and B. E. Ruth. Concrete Pavement Joint Stiffness Evaluation. In *Transportation Research Record 1099,* TRB, National Research Council, Washington, D.C., 1986, pp. 22–36.
6. Nishizawa, T., T. Fukuda, and S. Matsuno. A Refined Model of Doweled Joints for Concrete Pavement Using FEM Analysis. *Proc., 4th International Conference on Concrete Pavement Design and Rehabilitation,* Purdue University, West Lafayette, Ind., April 1989.
7. Foxworthy, P. T., and M. I. Darter. Preliminary Concepts for FWD Testing and Evaluation of Rigid Airfield Pavements. In *Transportation Research Record 1070,* TRB, National Research Council, Washington D.C., 1986, pp. 77–88.
8. Jeong, J.-H., J.-H. Lee, S. Young-Ghan, and D. G.. Zollinger. Effect of Slab Curling on Movement and Load Transfer Capacity of Saw-Cut Joints. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1947,* Transportation Research Board of the National Academies, Washington, D.C., 2006, pp. 69–78.
9. Crovetti, J. A. *Design and Evaluation of Jointed Concrete Pavement Systems Incorporating Open-Graded Permeable Bases.* PhD dissertation. University of Illinois, Urbana-Champaign, 1994.
10. Vandenbossche, J. M. *Interpreting Falling Weight Deflectometer Results for Curled and Warped Portland Cement Concrete Pavements.* PhD dissertation. Department of Civil and Environmental Engineering, University of Minnesota, Minneapolis, 2003.
11. Janssen, D. J., and M. B. Snyder. Temperature Moment Concept for Evaluating Pavement Temperature Data. *Journal of Infrastructure Systems,* ASCE, June 2000.

The Strength and Deformation Characteristics of Pavement Sections Committee sponsored publication of this paper.