Effect of Frozen Support and Tridem Axles on Concrete Pavement Performance

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ABSTRACT

A field program of strain and deflection measurements was conducted by the Construction Technology Laboratories for the Minnesota Department of Transportation. The objective of the program was to evaluate the effects of frozen support, tied-concrete shoulders, and tridem-axle loading on concrete pavement performance. The effects of frozen support and tridem-axle loading are presented. Field measurements were obtained during October 1982 and February 1983 at five pavement project sites located on I-90 in Minnesota. Measurements included edge and corner deflections and edge strains. Loadings applied were a 20-kip single axle, a 34-kip tandem axle, a 42-kip tandem axle, and a 42-kip tridem axle. Theoretical analysis was also conducted by using a finite-element program. Study results indicate that pavement deflections and strains are greatly reduced during winter months when the support is frozen. Based on analysis of these results, it is concluded that the effect of axle loads applied during the winter can be considered to be only one-seventh as damaging as the same loads applied during the fall. Study results also indicate that for application to the AASHTO thickness design procedure, tridem axles can be considered as equivalent to a single axle weighing about 50 percent of the tridem axles and to tandem axles weighing about 80 percent of the tridem axles. Traffic equivalence factors are presented for tridem axles on concrete pavements.

A field program of strain and deflection measurements was conducted by the Construction Technology Laboratories for the Minnesota Department of Transportation (MnDOT). The objective of the measurement program was to evaluate the effect of frozen support, tied-concrete shoulders, and tridem-axle loading on concrete pavement performance. The results of the investigation of the effect of frozen support and tridem axles on concrete pavement performance are presented (1,2). Results of the tied-concrete shoulder study are given elsewhere (3).

Minnesota's current concrete pavement design procedure does not consider climatic effects. When the base, subbase, and subgrade are frozen, pavement strains and deflections due to load are smaller. Therefore, traffic-induced damage during winter months is greatly reduced. Because concrete pavement design procedures consider repeated application of traffic loading and fatigue damage, it should be possible to take advantage of the frozen support conditions in the design of concrete pavements.

Minnesota's current design procedure does not account for the effect of tridem-axle loading on pavements either. Increases in the amount of truck traffic and vehicle gross weight have led to increased need for highway maintenance. To increase trucking productivity and minimize the detrimental effects of heavier axle loading, the trucking industry is rapidly adopting the use of tridem axles in lieu of tandem axles. The rationale behind this concept is that on a gross weight basis, the tridem axles are less damaging to pavements than equally loaded tandem axles.

BACKGROUND

One of the most widely used procedures for thickness design of concrete pavements is the AASHTO Interim Guide for Design of Pavement Structures (4). The AASHTO guide is based on results of the AASHTO Road Test supplemented by existing design procedures and available theory. The AASHTO Road Test site was located about 80 miles southwest of Chicago on right-of-way that is now part of Interstate 80 near Ottawa, Illinois. Test traffic began operation in November 1958 and ended on November 30, 1961. The final axle load count was 1,114,000.

Because MnDOT has adopted the AASHTO procedure as a basis for design of concrete pavements, presentations in this paper will be referenced to the AASHTO design procedure.

Effect of Frozen Support

The AASHTO Road Test design equation for concrete pavements contained in the AASHTO guide does not provide for variations in pavement life that may result from changes in environment and weather as compared with that for the road test location. Although a regional factor is used for design of flexible pavements in the AASHTO guide, no such factor is considered in the design of concrete pavements to account for regional effects.

Effect of Tridem-Axle Loading

Most concrete pavement thickness design procedures consider the effect of mixed truck traffic. Some procedures consider the effect of different axle loads directly, as in the case of the Portland Cement Association design procedure (5). In other procedures, such as that contained in the AASHTO Interim Guide (4), mixed truck traffic is converted to a common denominator, which is an 18-kip single-axle load (SAL).

The AASHTO procedure provides for conversion of mixed traffic to an equivalent number of 18-kip SALs by use of traffic equivalence factors. However, the procedure does not contain traffic equivalence factors for tridem-axle loads nor does it contain any other provisions to consider the effect of tridem-axle loads.

Because of the increasing use of tridem axles by the trucking industry, several agencies have been studying ways to incorporate the effect of tridem-axle loads in their thickness design procedures. A study was conducted at the Pennsylvania Transportation Research Facility to develop load equivalency factors for tridem-axle loadings on flexible pavements (6). In this study experimental pavements were subjected to approximately 55,000 repetitions of a 76-kip tridem-axle load. Study results were combined with theoretical analysis to develop equivalency factors for a range of tridem-axle loading.

In another study reported by Treybig (7) an attempt was made to relate theoretically computed concrete pavement response parameters to the AASHTO traffic equivalence factors for SALs and tandem-axle loads (TALs). However, no successful correlations were developed.

RESEARCH OBJECTIVES

The study presented in this paper was sponsored by MnDOT to compare measured pavement responses for SALs, TALs, and tridem-axle loads at five pavement sites. Field testing at these sites was conducted during October 1982 and February 1983. In this report results of field testing, analysis of results, and recommendations to incorporate study results in Minnesota's thickness design procedure for concrete pavements are presented.

Objectives of the study were as follows:

1. To measure load-induced strains and deflections in pavement sections during fall and winter periods,
2. To analyze test results to establish the effects of frozen support on concrete pavement performance, and
3. To analyze test results to establish the effects of tridem-axle loading on concrete pavement performance.

PAVEMENT TEST SECTIONS

Field measurements were obtained at five pavement project sites in Minnesota. Projects 1, 2, and 3 were included in a 1976 field study on concrete shoulders and lane widening (8). A brief description of each project follows:

Project 1: Designation State Project 2280-30 (TH-90) is a roadway 27 ft wide consisting of an inside lane 15 ft wide and an outside lane 12 ft wide with an outside tied keyed concrete shoulder 10 ft wide. Shoulders are tied at 30-in. spacing by using 30-in.-long No. 5 tie bars. Shoulder thickness is 6 in. The pavement is plain concrete slabs 9 in. thick with skewed joints at a repeated random spacing of 13, 16, 14, and 19 ft. Subgrade at the site was classified as silty clay to clay loam and had a gravel subbase 5 in. thick over it. Dowel bars were placed only in the 12-ft-wide outside traffic lane. Dowels are No. 8 round bars, spaced at 12 in. on centers; the first dowel is located 6 in. inward from the pavement edge. Panels selected for test are located at stations 538+65 and 540+10.

Project 2: Designation State Project 2280-30 (TH-90) is a roadway 27 ft wide and an outside tied keyed concrete shoulder 10 ft wide. Dowel size and location are the same as those for project 1. Pavement thickness is 8 in. Subgrade at the site was classified as silty clay to clay loam and had a gravel subbase 6 in. thick over it. The modulus of subgrade reaction was reported to be 270 pci. Panels selected for test are located at stations 520+55 and 521+81.

Project 3: Designation State Project 2280-31 (TH-90) is a roadway 27 ft wide with an inside lane 15 ft wide and an outside lane 12 ft wide. The pavement is reinforced concrete slabs 9 in. thick with skewed joints at a spacing of 27 ft. Subgrade at the site was classified as clay loam to silty clay loam to sandy clay loam. A gravel subbase 5 in. thick was used. Dowel bars were placed only in the 12-ft main-line pavement portion of both traffic lanes. Dowels are No. 8 round bars, spaced 12 in. on centers. Panels selected for test are located at stations 985+53 and 987+11.

Project 4: Designation State Project 4680-27 (TH-90) is a roadway 24 ft wide with a 12-ft inside and a 12-ft outside lane. The pavement is reinforced concrete slabs 9 in. thick with skewed joints at a spacing of 27 ft. Subgrade at the site was clay loam with an AASHTO classification of A-6. Modulus of subgrade reaction was reported to be 300 pci. A gravel subbase 6 in. thick was used. Dowel bars were placed in both the outside and inside lanes. Dowels are No. 8 round bars, spaced 12 in. on centers. Panels selected for test are located at stations 1329+52 and 1330+59.

Project 5: Designation State Project 7380-53 and 8680-57 (TH-94) is a roadway 24 ft wide with a 12-ft-wide inside lane and 12-ft-wide outside lane. The pavement is reinforced concrete slabs 9 in. thick with skewed joints at a spacing of 27 ft. Subgrade at the site was coarse sand with an AASHTO classification of A-1-b. Modulus of subgrade reaction was reported to be 700 pci. A gravel subbase 5 in. thick was used. Dowel bars were placed in both the outside and inside lanes. Dowels are No. 8 round bars, spaced 12 in. on centers. Panels selected for test are located at stations 507+93 and 509+28.

Projects 1, 2, 3, and 4 are located on I-90 between Albert Lea and Fairmont, Minnesota. Project 5 is located on I-94 near Clearwater, Minnesota.

Two test sites were selected at each project. At each site, both inside and outside lanes were instrumented and monitored to evaluate pavement response. At some of the sites for projects 1, 2, and 3, the panels tested in 1976 were retested. Care was taken to assure that the sites selected were representative of the project.

INSTRUMENTATION

All pavement test sections were instrumented to measure load-induced strains and deflections. In addition pavement temperature and slab curl were monitored. Curl is a change in the vertical profile of the slab resulting from changes in the slab temperature.

Strain gage and deflectometer locations for projects 1 and 2 test sections are shown in Figure 1. Instrumentation locations were similar for projects 3, 4, and 5. These locations were selected to obtain the maximum values of strain and deflection for the different load positions. Curl measurements were made at deflectometer locations. Concrete temperatures were measured in instrumented test blocks placed in the subbase adjacent to the pavement.

Load Strains

Concrete strains were measured with electrical-resistance strain gages 4 in. long cemented to the pavement surface. Gages were placed at the free edge, shoulder edge, transverse joints, and joint corners and in the interior. Gage positions and loading locations shown in Figure 1 are referred to

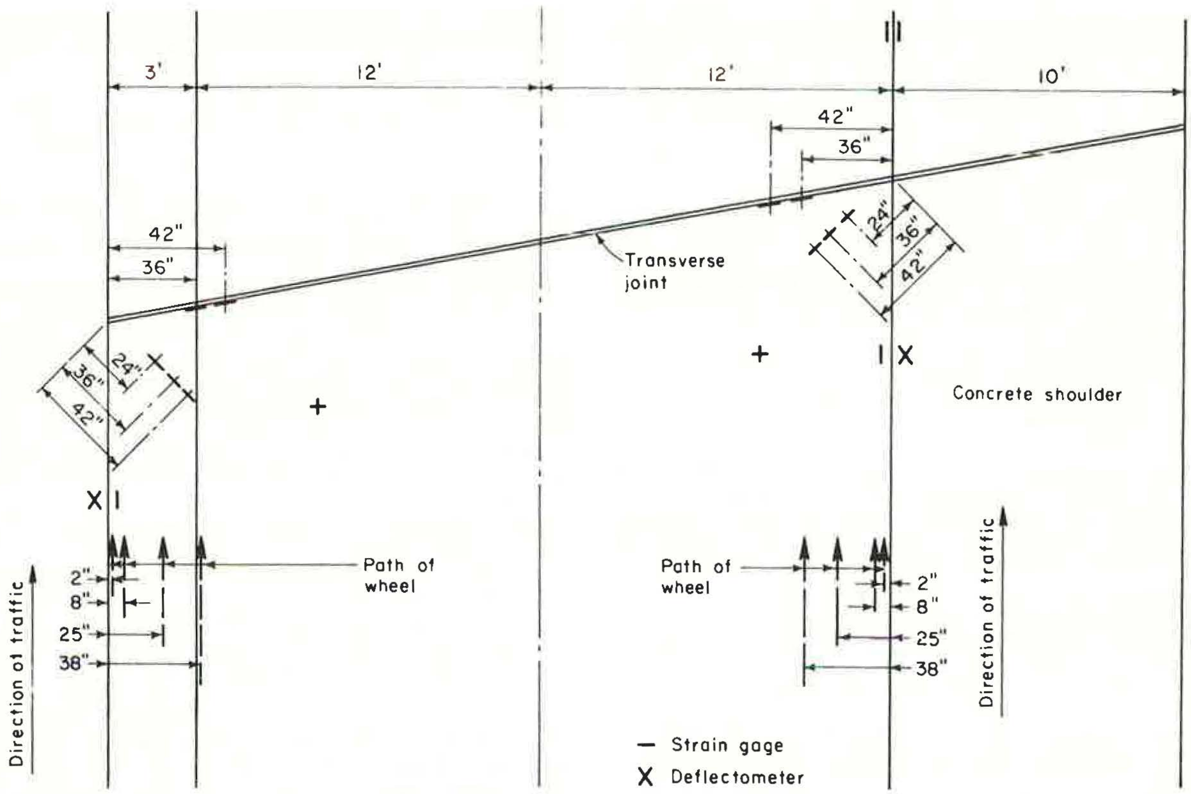

FIGURE 1 Instrumentation layout for projects 1 and 2.

in subsequent discussions. All gages were placed in recessed grooves to protect them from direct application of wheel loads.

Load Deflections

Load deflections were measured with resistance-bridge deflectometers bolted to the pavement. Readings were referenced to encased rods driven into the subgrade to a depth of 6 ft. Construction details of the deflectometer are presented in Research and Development Bulletin D83 (9) of the Portland Cement Association.

Curl Measurements

Pavement curl was measured with 0.001-in. indicators placed at the same locations as the deflectometers. The dial indicators were bolted to the pavement and the movement was referenced to encased rods placed in the subgrade. Curl readings were taken approximately once an hour.

Temperature Measurements

Changes in pavement temperature were measured with copper-constantan thermocouples embedded in concrete blocks. The laboratory-cast blocks were 1 ft square and 8 or 9 in. thick. Thermocouples were located 0.125, 0.50, 1, 2, 4, and 6 in. from the top and 0.125 in. from the bottom surfaces. Temperature blocks were placed in the subbase adjacent to the highway at least 12 hr before testing. Air temperature was monitored with a thermocouple shaded from the direct sun.

Monitoring Equipment

Data were monitored and recorded with equipment carried in the Construction Technology Laboratories' field instrumentation van. Strain and deflection data were recorded with a high-speed computer-based data acquisition system. Twenty-two channels of instrumentation were monitored and recorded simultaneously for each vehicle loading. Computer programs were written to monitor, record, and tabulate all field data. Temperature data were recorded with a 24-channel continuously monitoring temperature recorder. All monitoring and recording instrumentation was calibrated before testing.

TEST PROCEDURES

Strain and deflection data were recorded for 20-kip SALs, 34-kip and 42-kip TALs, and 42-kip tridem-axle loads. Loading was applied with the two semitrailers shown in Figure 2. One truck applied the 20-kip SALs and 34-kip TALs. The other truck applied the 42-kip TALs and 42-kip tridem-axle loadings. Trucks used were supplied by MnDOT. Before testing, axle weights were checked and loads were adjusted to obtain uniform distribution to the wheels.

The effects of axle weight and load location on strains and deflections were recorded with the trucks moving at creep speed along the wheel paths shown in Figure 1. Tire placements varied from 2 to 38 in. from the pavement edge. All wheel-path measurements were from the pavement edge to the outside edge of the tire sidewall at its maximum width. In addition, pavement curl and temperature data were obtained periodically during the day.

Inside- and outside-lane test slabs at each project site were tested on the same day. Primary read-

FIGURE 2 Trucks used: 18-kip SAL and 34-kip TAL (top), 42-kip TAL and 42-kip tridem-axle load (bottom).

ings were taken on both inside and outside lanes between approximately 11:30 a.m. and 2:00 p.m. In addition, readings were also taken on one lane before 11:30 a.m. and on the other lane after 2:00 p.m. Specific testing times were governed primarily by weather and traffic control requirements.

DATA ANALYSIS

In this section a comparison is presented of pavement responses measured under 20-kip SALs, 34-kip and 42-kip TALs, and 42-kip tridem-axle loadings during October 1982 and February 1983. Pavement responses compared are edge and corner deflections and edge strains. In addition, results of theoretical analysis are also presented to compare pavement responses under the four different axle loads. Wheel configurations and spacings for the four axle loads used during the field testing correspond to those shown in Figure 3.

Curling and Warping Effects

Soon after concrete has been placed, drying shrinkage of the concrete begins. Drying shrinkage in a slab on grade occurs at a faster rate at the slab surface than at the slab bottom. In addition, because the subgrade and subbase may remain wet, the slab bottom remains relatively moist. Thus, total shrinkage at the bottom is less than that at the top. This differential in shrinkage results in a lifting of the slab from the subbase at edges and corners. Movements of this type resulting from moisture differentials are referred to as warping. Warping leaves slabs unsupported for distances of as much as 4 to 5 ft at slab corners and 2 to 3 ft at slab edges. Warping is almost never recoverable.

In addition to warping, a slab on grade is also subjected to curling. Curling is the change in the slab profile due to temperature differential between slab top and bottom. Curling is a daily phenomenon. Slabs curl up during the night and curl down during the midday. Thus, curling deformation is additive to warping during the night and reduces the warping effect during the midday. It is believed by many engineers that the warping effect is almost never cancelled out by daytime curling and that some loss of support always exists under the slab even for hot days.

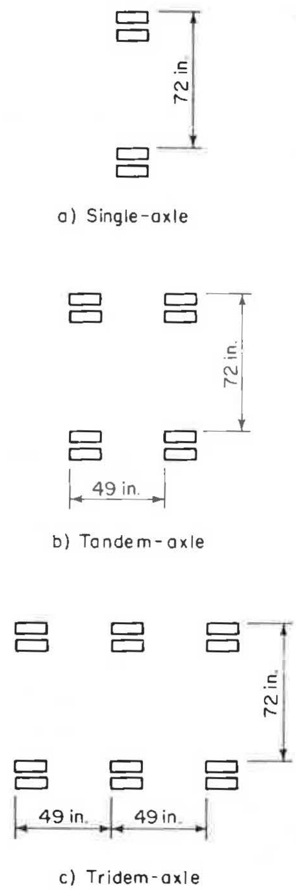

FIGURE 3 Axle configurations.

Because of curling effects, the measured deflections under load along a slab edge or a slab corner are greatly affected by the time of testing. Measured slab strains are also affected by time of testing but at a lower level. Therefore, great care needs to be exercised in interpreting deflection and strain measurements if they are made at different times of a day or on different days. The usual procedure in reporting deflection measurements at a given location is to correct the measurements with respect to a reference time. The reference time is generally selected to be the time when the slab top and bottom temperatures are equal.

As discussed, temperature and curl measurements were made at each of the five test sites considered in this study. At each test site, pavement responses under load were generally measured at two different times, usually within a span of 3 hr around noon.

Figure 4 shows the variation with time of the air temperature, corner curl, and corner deflection under a 20-kip SAL at each of the five sites.

It is seen that although slabs at each site exhibit pronounced curling, the deflections under load were not greatly influenced by the time of testing between approximately 11:00 a.m. and 2:00 p.m. Similar trends were obtained for edge curl and deflections and edge strain. This is because the slabs have curled to their most downward profiles and change from these profiles is gradual with respect to time, as shown in Figure 4. Therefore, no temperature corrections were applied to these readings. The measurements reported in this paper are the averages of the readings for the two test times and correspond to the period when each slab being tested was near its maximum downward curl.

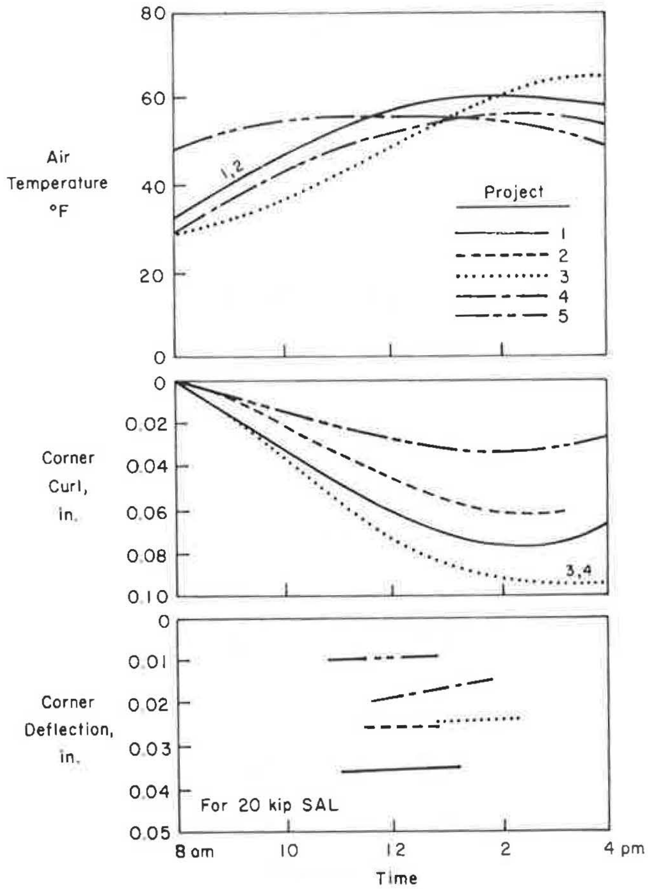

FIGURE 4 Variation of air temperature, corner curl, and deflection with time.

Summary of Data

Load tests were conducted during October 1982 when air temperatures at midday were about 55°F and during February 1983 when air temperatures at midday were about 20 to 30°F.

Pavement response measured at each of the five sites is given in Table 1. Edge and corner deflections and edge strains measured during October 1982 and February 1983 at inside and outside lanes for each of the four axle loadings are listed. Each data point is an average of four readings made up of data taken at two different times at each of the two replicate sections at each project location.

EFFECT OF FROZEN SUPPORT

This section considers the effect of frozen support on concrete pavement performance. The February measurements are shown as a percentage of the October measurements in Figures 5, 6, and 7 for edge deflection, corner deflection, and edge strain, respectively. (Axles are defined as follows in Figures 5-7: axle 1, 20-kip SAL; axle 2, 34-kip TAL; axle 3, 42-kip TAL; axle 4, 42-kip tridem-axle load. Lane I is the inside lane; lane O the outside lane. N denotes lack of reliable data.)

Measured Edge Deflections

As shown in Figure 5, edge deflections measured during February generally ranged from 15 to 25 percent of edge deflections measured during October. Under the 20-kip SAL, edge deflections ranged from 0.007 in. at project 5 to 0.021 in. at project 1 during October and from 0.001 in. at project 5 to 0.004 in. at project 1 during February.

It should be noted that at project 2, edge deflections measured along the outside lane do not show any variation with different axle loads. This is believed to be because of malfunctioning of the deflectometers at this location.

Measured Corner Deflections

As shown in Figure 6, corner deflections measured during February generally ranged from 5 to 15 percent of corner deflections measured during October. Under the 20-kip SAL, corner deflections ranged from 0.010 in. at project 5 to 0.035 in. at project 1 during October and from 0.001 in. at Project 5 to 0.004 in. at project 1 during February.

Measured Edge Strains

As shown in Figure 7, edge strains measured during February generally ranged from 20 to 60 percent of edge strains measured during October. Under the 20-kip SAL, edge strains ranged from 19 x 10^{-6} at the inside lane of project 5 to 35 x 10^{-6} at the inside lane of project 1 during October. Edge strains under the 20-kip SAL ranged from 9 x 10^{-6} at the inside lane of project 5 to 18 x 10^{-6} at the inside lane of project 1 during February.

Theoretical Considerations

Analysis was conducted to determine the effect of the subbase and subgrade support on pavement response. A finite-element program, JSLAB, developed by Construction Technology Laboratories for FHWA was used (10). The program can analyze a large number of jointed slabs. Joints can be modeled as doweled, aggregate interlock, or keyed. Load input is in terms of wheel loads at any location on the slabs. Loss of support, variable support, or material properties can be considered. In the program subbase and subgrade support is characterized by the modulus of subgrade support. Thus, the effect of a frozen support can be considered by using a high value for the modulus of subgrade reaction.

The analysis was conducted for a concrete pavement 9 in. thick with and without a tied shoulder and with dowel bars at transverse joints. For the case of a tied shoulder, a slab 6 in. thick was used. Values used for the modulus of subgrade reaction were 100, 150, 250, 1,000, and 2,000 pci. Calculated corner deflections, edge deflections, and edge stresses are listed in Tables 2-4. For both corner and edge loadings, tire placements were 2 in. inward from the edge.

The calculations verify that although a stiffer subbase and subgrade support will produce a large reduction in slab deflections, the corresponding decrease in slab edge stresses is not so large. For example, edge deflection for a support value of 2,000 pci is reduced to about 25 to 35 percent of that for a support value of 250 pci. However, edge stress for a support value of 2,000 pci is reduced to only about 50 to 70 percent of that for a support value of 250 pci. These calculations and field measurements indicate that during winter months, the support value under a concrete pavement can be expected to exceed 1,000 pci. For this condition, edge

TABLE 1 Measured Pavement Response at Projects 1 Through 5

Parameter	20-kip SAL Fall	20-kip SAL Winter	34-kip TAL Fall	34-kip TAL Winter	42-kip TAL Fall	42-kip TAL Winter	42-kip Tridem-Axle Load Fall	42-kip Tridem-Axle Load Winter
Project 1								
Inside lane								
Edge deflection (in.)	0.021	0.004	0.034	0.006	0.038	0.008	0.034	0.007
Corner deflection (in.)	0.035	0.004	0.044	0.006	0.051	0.007	0.044	0.006
Edge strain (x 10^{-6})	35	18	30	23	32	17	17	10
Outside lane								
Edge deflection (in.)	0.019	0.004	0.026	0.006	0.029	0.007	0.027	0.007
Corner deflection (in.)	0.030	0.003	0.034	0.005	0.037	0.005	0.031	0.005
Edge strain (x 10^{-6})	30	18	24	18	27	14	19	12
Project 2								
Inside lane								
Edge deflection (in.)	0.016	0.003	0.026	0.004	0.027	0.005	0.023	0.005
Corner deflection (in.)	0.026	0.002	0.036	0.004	0.034	0.004	0.030	0.004
Edge strain (x 10^{-6})	35	12	32	13	33	17	18	9
Outside lane								
Edge deflection (in.)	0.007	0.002	0.007	0.003	0.007	0.003	0.009	0.003
Corner deflection (in.)	0.021	0.003	0.021	0.004	0.025	0.003	0.019	0.002
Edge strain (x 10^{-6})	33	11	31	9	38	9	20	5
Project 3								
Inside lane								
Edge deflection (in.)	0.013	0.002	0.022	0.004	0.025	0.004	0.020	0.004
Corner deflection (in.)	0.024	0.002	0.030	0.003	0.032	0.003	0.026	0.003
Edge strain (x 10^{-6})	33	–	28	–	30	–	18	–
Outside lane								
Edge deflection (in.)	0.015	0.003	0.021	0.003	0.025	0.004	0.023	0.004
Corner deflection (in.)	0.036	0.002	0.040	0.002	0.040	0.002	0.034	0.002
Edge strain (x 10^{-6})	18	–	23	–	24	–	16	–
Project 4								
Inside lane								
Edge deflection (in.)	0.013	0.002	0.020	0.002	0.020	0.002	0.018	0.002
Corner deflection (in.)	0.017	0.002	0.022	0.002	0.024	0.002	0.019	0.001
Edge strain (x 10^{-6})	31	13	27	13	27	–	17	–
Outside lane								
Edge deflection (in.)	0.013	0.002	0.018	0.002	0.021	0.002	0.019	0.002
Corner deflection (in.)	0.022	0.002	0.026	0.002	0.027	0.002	0.024	0.001
Edge strain (x 10^{-6})	–	13	–	13	–	–	–	–
Project 5								
Inside lane								
Edge deflection (in.)	0.007	0.001	0.010	0.002	0.009	0.002	0.007	0.002
Corner deflection (in.)	0.010	0.001	0.011	0.001	0.010	0.002	0.008	0.002
Edge strain (x 10^{-6})	19	9	19	10	20	3	14	–
Outside lane								
Edge deflection (in.)	0.007	0.002	0.008	0.002	0.008	0.002	0.007	0.002
Corner deflection (in.)	0.013	0.002	0.013	0.003	0.012	0.002	0.010	0.003
Edge strain (x 10^{-6})	31	14	23	12	26	6	17	2

Note: Inside-lane measurements were taken along the edge of the 3-ft lane widening. Outside-lane measurements were taken along the joint with tied shoulder. Fall measurements were obtained during October 1982; winter measurements were obtained during February 1983.

and corner deflections for a 34-kip TAL would be less than 0.004 in. and edge stresses for a 20-kip SAL would be less than 150 psi.

It should be noted that deflection values measured during October were much higher than calculated deflection values, even when a modulus of subgrade reaction of 150 pci was used. Modulus of subgrade reaction values at the five locations were reported to be in excess of 250 pci. The reason for the anomaly in measured and computed deflection values is that the theoretical analysis was conducted for the case of full support under the pavement slabs. In practice, there is always some loss of support along slab edges. This support loss results in higher measured slab deflections.

Analysis of Results

As indicated, it is clear that concrete pavement responses for the case of frozen support were much smaller compared with those obtained when the support was not frozen. The greatly improved deflection response is considered to be caused by the frozen subgrade and subbase and also a lower level of slab warping. Slab warping is lower during winter months because of the higher moisture content at the surface of the concrete slab. The effect of less slab warping is less loss of support along slab edges. From the field testing conducted at the five project locations, the following values indicate the improvement in pavement response during February as

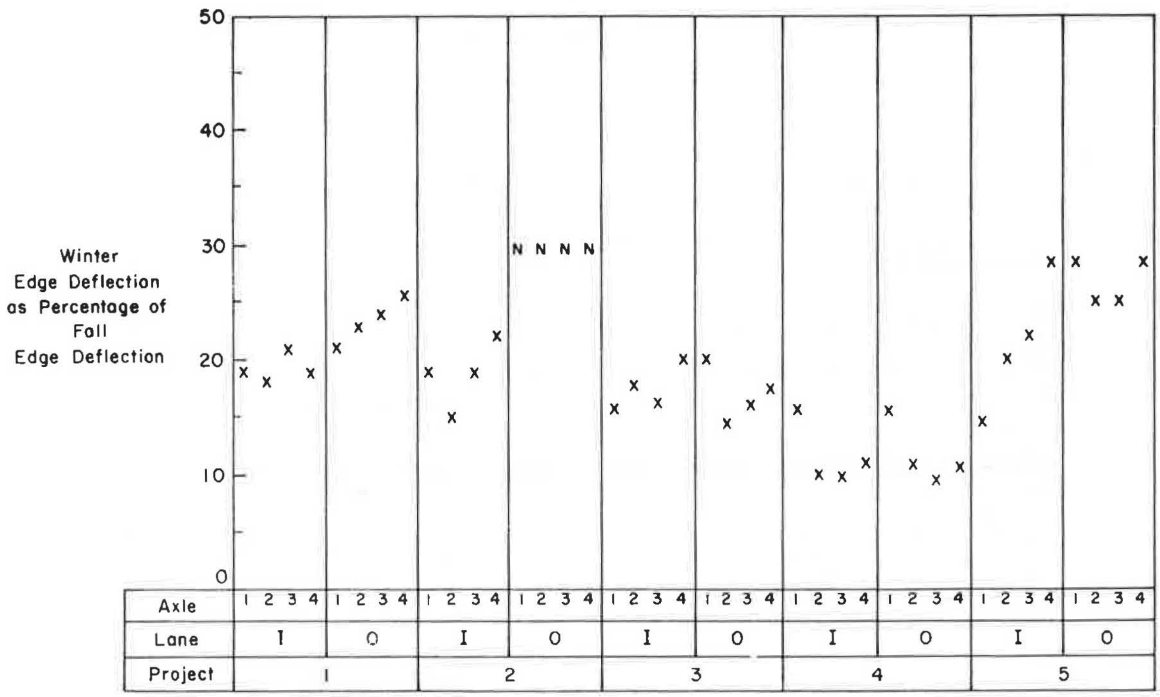

FIGURE 5 Comparison of winter and fall edge deflections.

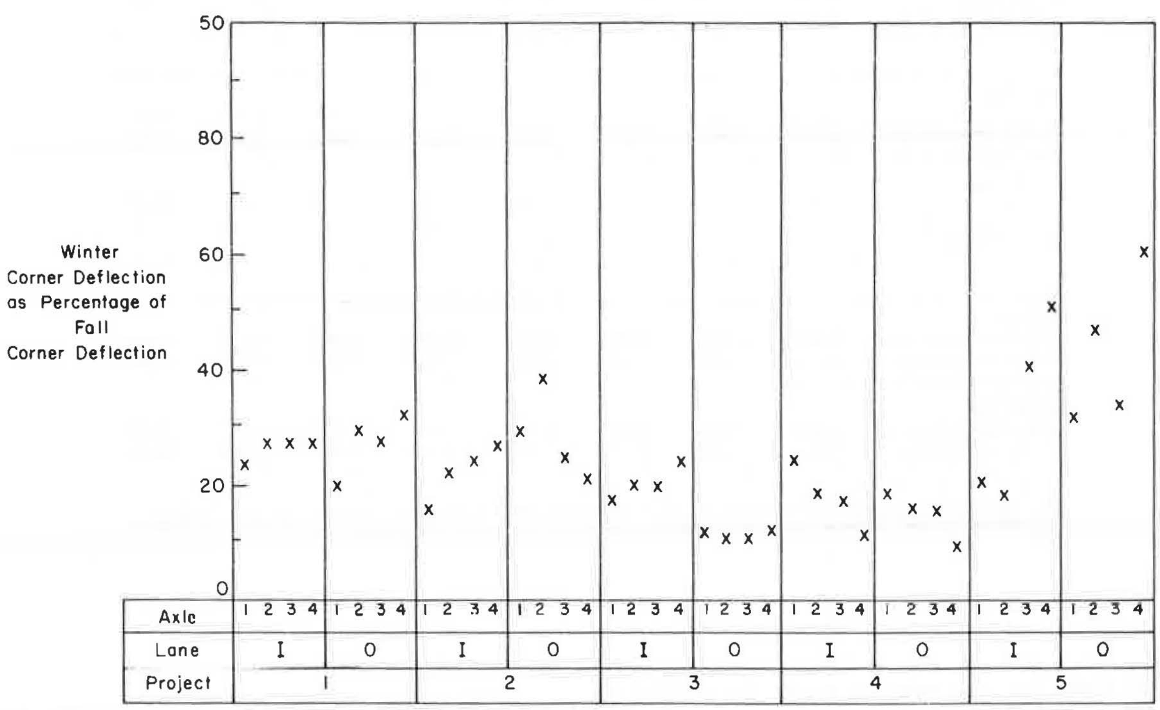

FIGURE 6 Comparison of winter and fall corner deflections.

compared with that in October (A = value calculated for k of 2,000 pci as percentage of value for k of 250 pci; B = measured value during February as percentage of October measurement):

A (%)	Pavement Response	B (%)
25-30	Corner deflection	15
25-35	Edge deflection	25
50-70	Edge stress	60

For consideration of the effects of frozen support in the thickness design for concrete pavements, it is recommended that 60 percent be used as the maximum level of improvement in pavement response from fall to winter. This recognizes that deflections as well as stresses are important in assessing pavement performance.

Because pavement damage or loss in serviceability is a function of axle load magnitude and number of load repetitions, it can be concluded that a given

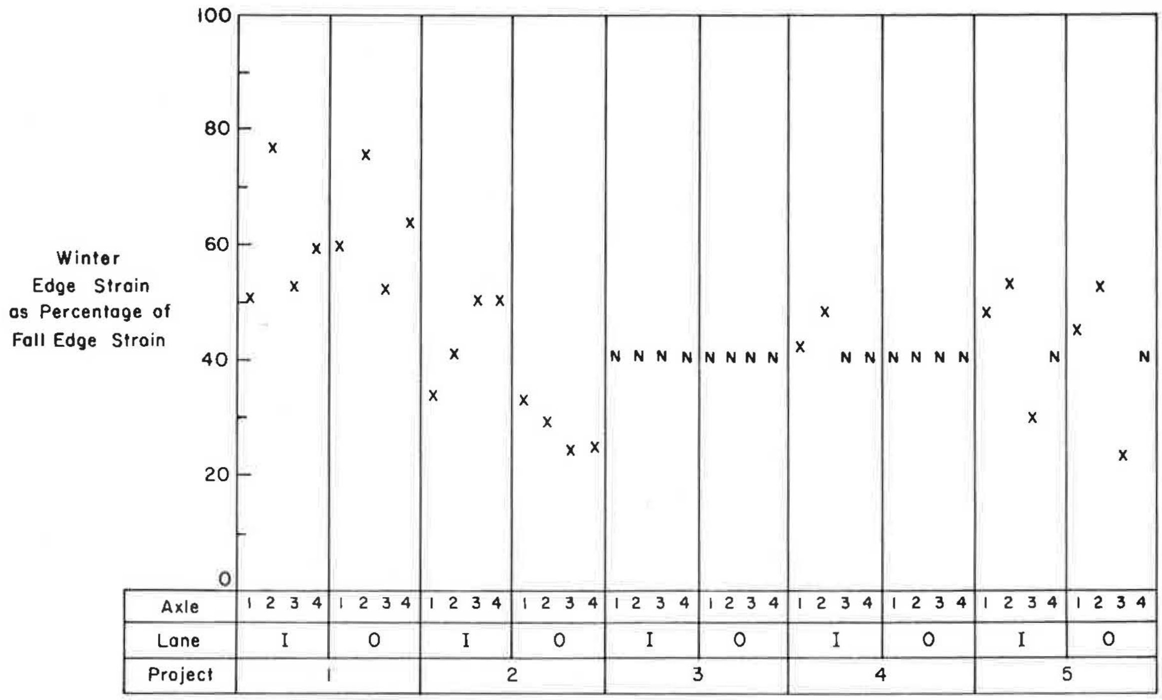

FIGURE 7 Comparison of winter and fall edge strains.

TABLE 2 Calculated Pavement Response: Corner Deflection

		Corner Deflection (in.)			
Shoulder Type	k (pci)	20-kip SAL	34-kip TAL	42-kip TAL	42-kip Tridem-Axle Load
Tied	100	0.025	0.026	0.032	0.026
	150	0.018	0.019	0.023	0.019
	250	0.013	0.013	0.016	0.012
	1,000	0.006	0.005	0.006	0.004
	2,000	0.004	0.004	0.004	0.003
None	100	0.035	0.040	0.050	0.040
	150	0.026	0.030	0.037	0.028
	250	0.019	0.020	0.025	0.019
	1,000	0.008	0.007	0.009	0.007

TABLE 3 Calculated Pavement Response: Edge Deflection

		Edge Deflection (in.)			
Shoulder Type	k (pci)	20-kip SAL	34-kip TAL	42-kip TAL	42-kip Tridem-Axle Load
Tied	100	0.015	0.022	0.027	0.022
	150	0.012	0.016	0.020	0.016
	250	0.008	0.011	0.014	0.011
	1,000	0.004	0.004	0.005	0.004
	2,000	0.003	0.003	0.004	0.003
None	100	0.024	0.035	0.043	0.036
	150	0.018	0.025	0.031	0.026
	250	0.012	0.017	0.021	0.017
	1,000	0.005	0.006	0.007	0.005

axle load would produce less damage or loss of serviceability during the winter as compared with that in the fall. If a linear relationship is assumed between magnitude of axle load and pavement response, an axle load (P) applied during the winter is equivalent to an axle load (0.6P) applied during the fall.

TABLE 4 Calculated Pavement Response: Edge Stress

		Edge Stress (psi)			
Shoulder Type	k (pci)	20-kip SAL	34-kip TAL	42-kip TAL	42-kip Tridem-Axle Load
Tied	100	236	180	222	114
	150	218	160	198	98
	250	199	139	172	81
	1,000	157	102	126	55
	2,000	138	85	103	42
None	100	286	230	284	152
	150	263	203	250	128
	250	236	172	212	103
	1,000	178	116	143	63

Application to AASHTO Design Procedure

The AASHTO Interim Guide uses the concept of traffic equivalence factors for converting mixed traffic to an equivalent number of 18-kip SALs. The equivalence factors, when multiplied by the number of axle loads within a given weight category, give the number of 18-kip SALs that have an equivalent effect on the performance of the pavement.

Traffic equivalence factors for concrete pavements are given in Table 5 for SALs and TALs. It may be seen that for a pavement 9 in. thick, a 30-kip SAL is 8.28 times as damaging as an 18-kip SAL. However, based on measured pavement response, a 30-kip SAL applied during a winter month can be considered to be only as damaging as an 18-kip SAL applied during the fall. Thus, a 30-kip SAL applied during the winter months is only 1/8.28, that is, 0.12 times as damaging as a 30-kip SAL applied during the fall. Applying this logic to different slab thicknesses and other axle loads, it is found that the damaging effect of a given SAL or TAL applied in the winter is about one-seventh to one-ninth of that for the same axle load applied during the fall.

For design purposes it is recommended that the

TABLE 5 Traffic Equivalence Factors for Single and Tandem Axles

Axle Load		Slab Thickness D (in.)						
Kips	kN	6	7	8	9	10	11	12
Single Axle								
2	8.9	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
4	17.8	0.003	0.002	0.002	0.002	0.002	0.002	0.002
6	26.7	0.01	0.01	0.01	0.01	0.01	0.01	0.01
8	35.6	0.04	0.04	0.03	0.03	0.03	0.03	0.03
10	44.5	0.10	0.09	0.08	0.08	0.08	0.08	0.08
12	53.4	0.20	0.19	0.18	0.18	0.18	0.17	0.17
14	62.3	0.38	0.36	0.35	0.34	0.34	0.34	0.34
16	71.2	0.63	0.62	0.61	0.60	0.60	0.60	0.60
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	89.0	1.51	1.52	1.55	1.57	1.58	1.58	1.59
22	97.9	2.21	2.20	2.28	2.34	2.38	2.40	2.41
24	106.8	3.16	3.10	3.23	3.36	3.45	3.50	3.53
26	115.7	4.41	4.26	4.42	4.67	4.85	4.95	5.01
28	124.6	6.05	5.76	5.92	6.29	6.61	6.81	6.92
30	133.4	8.16	7.67	7.79	8.28	8.79	9.14	9.34
32	142.3	10.81	10.06	10.10	10.70	11.43	11.99	12.35
34	151.2	14.12	13.04	12.34	13.62	14.59	15.43	16.01
36	160.1	18.20	16.69	16.41	17.12	18.33	19.52	20.39
38	169.0	23.15	21.14	20.61	21.31	22.74	24.31	25.58
40	177.9	29.11	26.49	25.65	26.29	27.91	29.90	31.64
Tandem Axles								
10	44.5	0.01	0.01	0.01	0.01	0.01	0.01	0.01
12	53.4	0.03	0.03	0.03	0.03	0.03	0.03	0.03
14	62.3	0.06	0.05	0.05	0.05	0.05	0.05	0.05
16	71.2	0.10	0.09	0.08	0.08	0.08	0.08	0.08
18	80.1	0.16	0.14	0.14	0.13	0.13	0.13	0.13
20	89.0	0.23	0.22	0.21	0.21	0.20	0.20	0.20
22	97.9	0.34	0.32	0.31	0.31	0.30	0.30	0.30
24	106.8	0.48	0.46	0.45	0.44	0.44	0.44	0.44
26	115.7	0.64	0.64	0.63	0.62	0.62	0.62	0.62
28	124.6	0.85	0.85	0.85	0.85	0.85	0.85	0.85
30	133.4	1.11	1.12	1.13	1.14	1.14	1.14	1.14
32	142.3	1.43	1.44	1.47	1.49	1.50	1.51	1.51
34	151.2	1.82	1.82	1.87	1.92	1.95	1.96	1.97
36	160.1	2.29	2.27	2.35	2.43	2.48	2.51	2.52
38	169.0	2.85	2.80	2.91	3.04	3.12	3.16	3.18
40	177.9	3.52	3.42	3.55	3.74	3.87	3.94	3.98
42	186.8	4.32	4.16	4.30	4.55	4.74	4.86	4.91
44	195.7	5.26	5.01	5.16	5.48	5.75	5.92	6.01
46	204.6	6.36	6.01	6.14	6.53	6.90	7.14	7.28
48	213.5	7.64	7.16	7.27	7.73	8.21	8.55	8.75

Note: Terminal pavement serviceability index (p_t) = 2.5.

damaging effect of an axle load applied during the winter be considered to be one-seventh of that for the same axle load applied during the fall. Thus, only one-seventh of the equivalent 18-kip SALs applied during the winter months needs to be considered for thickness design. If traffic is considered to be uniformly distributed over the 12-month period and if only one-seventh of the winter period traffic is considered applicable, only 79 percent of the total design value of the equivalent 18-kip SALs needs to be considered for thickness design.

However, it should be noted that the current AASHTO design procedure already has built into it the effect of frozen support, because the AASHTO Road Test was conducted over a period of two winters. Study results presented in this report can be implemented into the AASHTO design procedure if the difference in severity and duration of winter conditions between Ottawa, Illinois, and the state of Minnesota can be established.

Application to Other Design Procedures

Results of this study have direct application to design procedures that are based on considerations of stresses or deflections or both under each axle-load group of mixed traffic. For example, the Portland Cement Association thickness design for concrete pavements is based on fatigue consumed under mixed traffic (7). In this procedure fatigue consumption is computed for each axle-load group and summed to determine total fatigue consumption during the design period.

To apply study results to such a procedure, fatigue consumption would be determined separately for winter periods and for nonwinter periods. For nonwinter periods the conventional procedure would be used. For winter periods fatigue consumption computation would incorporate use of a stiff support.

EFFECT OF TRIDEM-AXLE LOADING

In this section the effect of tridem-axle loading is considered. Although measurements were obtained during October 1982 and February 1983, only the October 1982 measurements are discussed in this section. Because of the frozen support, measured deflections during February 1983 were low for each axle type. The measurements listed in Table 1 for October 1982 are shown as a percentage of the 42-kip tridem-axle load measurements in Figures 8, 9, and 10 for edge deflection, corner deflection, and edge strain, respectively. (Axles are defined as follows in Figures 8-10: axle 1, 20-kip SAL; axle 2, 34-kip TAL; axle 3, 42-kip TAL; axle 4, 42-kip tridem-axle load. Lane I is the inside lane; lane O, the outside lane. N denotes lack of reliable data.)

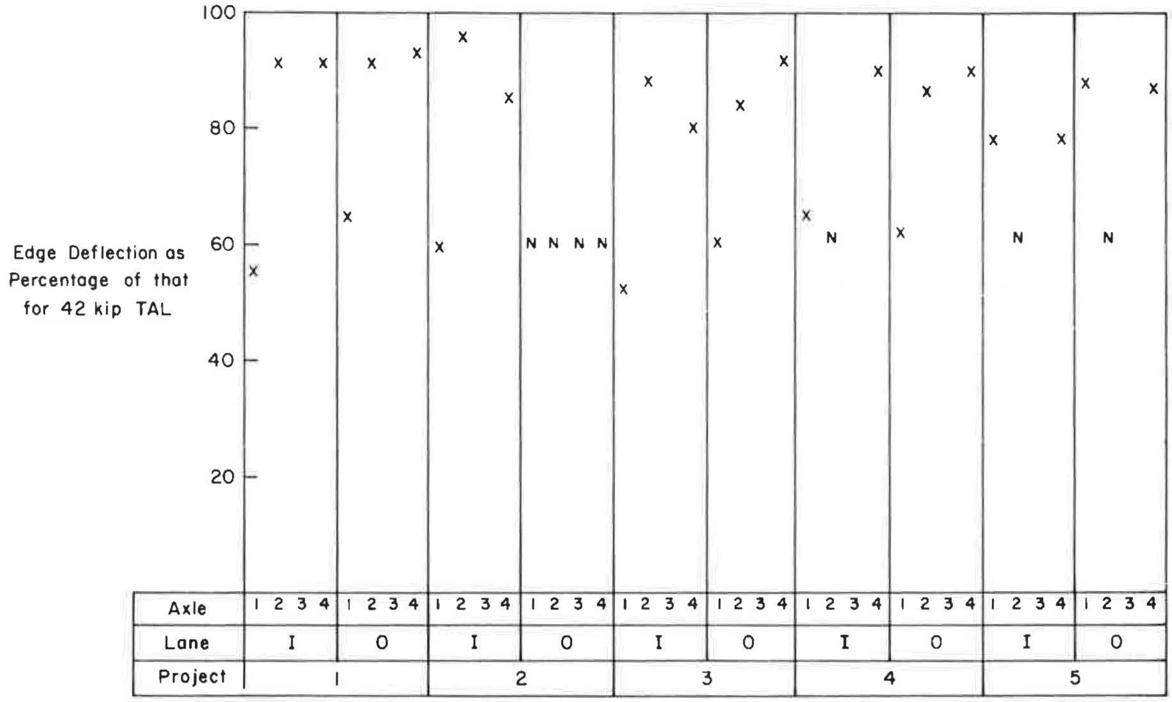

FIGURE 8 Edge deflections as percentage of those for 42-kip TAL.

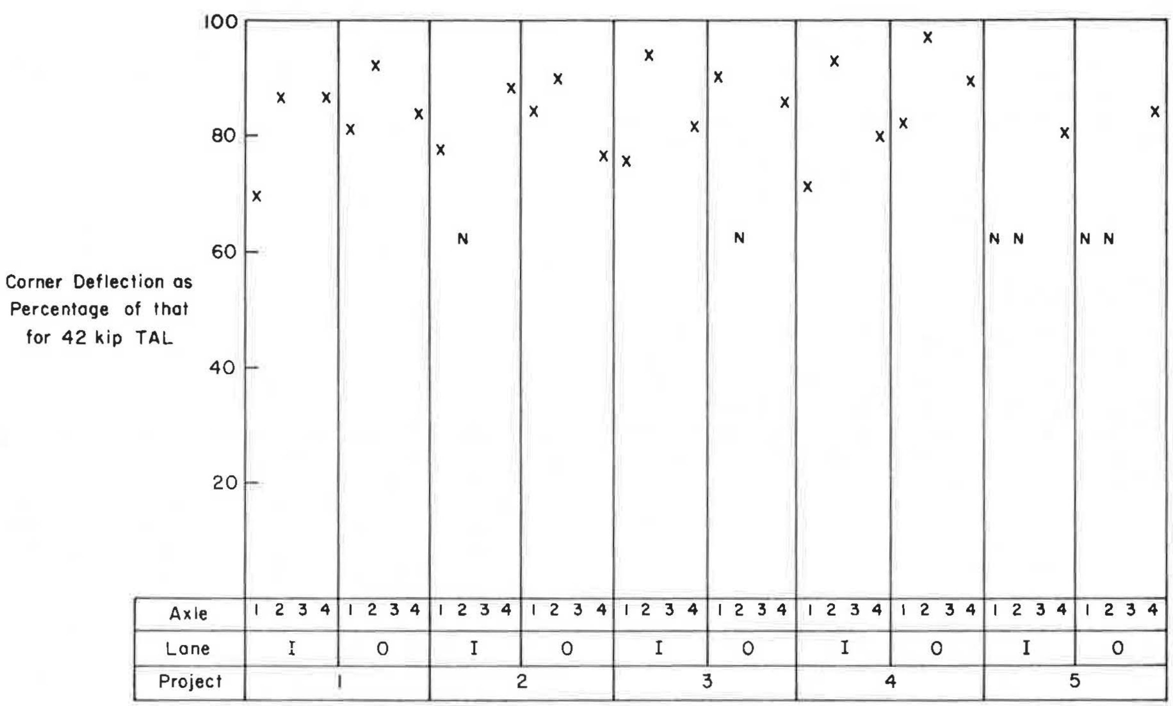

FIGURE 9 Corner deflections as percentage of those for 42-kip TAL.

Measured Edge Deflections

As shown in Figure 8, edge deflections measured during the fall period under the 42-kip tridem-axle loads ranged from 78 to 93 percent of those for the 42-kip TALs. At 7 of the 10 sections, edge deflections under the tridem-axle loads were less than 90 percent of those under the 42-kip TALs. As a comparison, the 34-kip TALs produced edge deflections between 84 to 96 percent of those for the 42-kip TALs. Theoretically, the 34-kip TALs should produce edge deflections about 80 percent of those produced under the 42-kip TALs.

It should be noted that at project 2, edge deflections measured along the outside lane do not show any variation with different axle loads. This is believed to be because of malfunctioning of the deflectometers at this location.

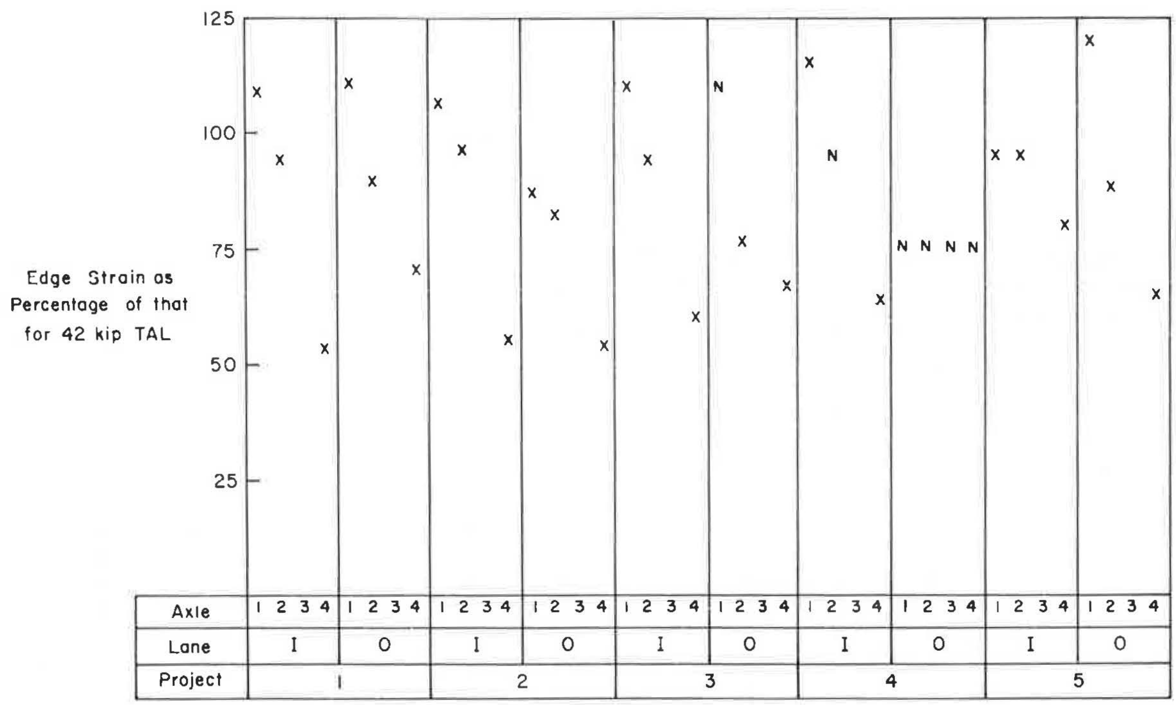

FIGURE 10 Edge strains as percentage of those for 42-kip TAL.

Measured Corner Deflections

As shown in Figure 9, corner deflections measured during the fall period under the 42-kip TALs ranged from 76 to 90 percent of those for the 42-kip TALs. The 34-kip TALs produced corner deflections between 86 and 96 percent of those for the 42-kip TALs. As in the case of edge deflections, theoretically the 34-kip TALs should produce corner deflections about 80 percent of those produced under the 42-kip TALs.

Measured Edge Strains

As shown in Figure 10, edge strains measured during the fall period under the 42-kip tridem-axle loads ranged from 53 to 69 percent of those for the 42-kip TALs. The 34-kip TALs produced edge strains between 82 and 97 percent of those for the 42-kip TALs. Theoretically, edge strain for the 34-kip TALs should be about 80 percent of those for the 42-kip TALs.

Theoretical Considerations

Calculated pavement responses for the different axle loads are given in Tables 2-4. A summary of these calculated results is given in Table 6 as a percentage of values obtained for the 42-kip TALs. As shown in Table 6, calculated slab deflections and strains under the 42-kip tridem-axle loads are much less than those for the 42-kip TALs and in fact are equal to or less than those for the 34-kip TALs. Of the four cases of axle loading considered, the 42-kip TALs resulted in the highest calculated edge and corner deflections and the 20-kip SALs produced the highest calculated edge strains.

When the effects on pavement response of different axle types are compared, the profiles for deflections and strains along the slab edge should also be considered. Figure 11 shows calculated edge deflection profiles for the 20-kip SALs, 34-kip TALs, and the 42-kip tridem-axle loads. As shown,

TABLE 6 Calculated Pavement Response as Percentage of That for 42-Kip TALs

Shoulder Type	Response Type	Percentage of Response by Axle Load: 20-kip SAL	34-kip TAL	42-kip Tridem-Axle Load
Tied	Edge deflection	58	81	81
	Corner deflection	79	81	80
	Edge strain	111	81	49
None	Edge deflection	57	81	83
	Corner deflection	69	81	80
	Edge strain	106	81	51

the shapes of the deflection profiles are similar for the three cases. Figure 12 shows the calculated corner deflection profiles for the tridem-axle loads. The deflection basin length under the tridem-axle loads is almost twice as long as that for the SALs and about 1.5 times as long as that for the TALs.

Profiles for calculated edge strain for the three cases of axle loads are shown in Figure 13. For this case there is a marked difference between the responses under the three different types of axle loads. The SAL exhibits a single peak, the TAL exhibits two peaks, and the tridem-axle load produces three peaks. These peaks are produced under each axle.

Analysis of Results

It has been shown that pavement response under the 42-kip tridem-axle loads is less severe than that for the 42-kip TALs. In fact, the response for the 42-kip tridem-axle loads was equal to or less severe than that for the 34-kip TALs.

According to the AASHTO traffic equivalence factors, presented in Table 5, tandem axles are about

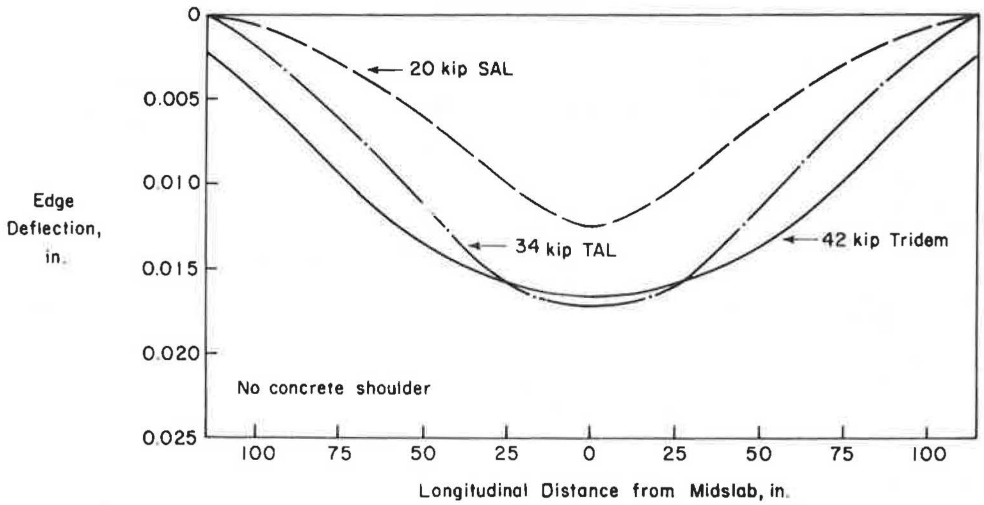

FIGURE 11 Calculated edge deflection profiles.

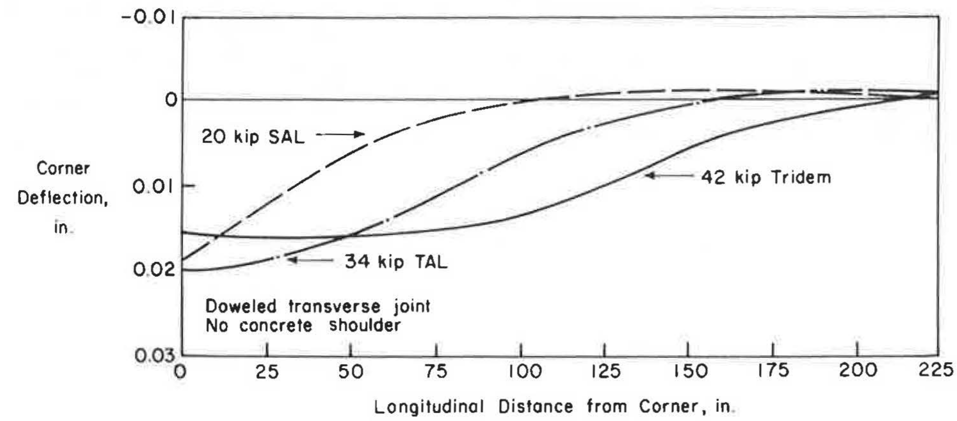

FIGURE 12 Calculated corner deflection profiles.

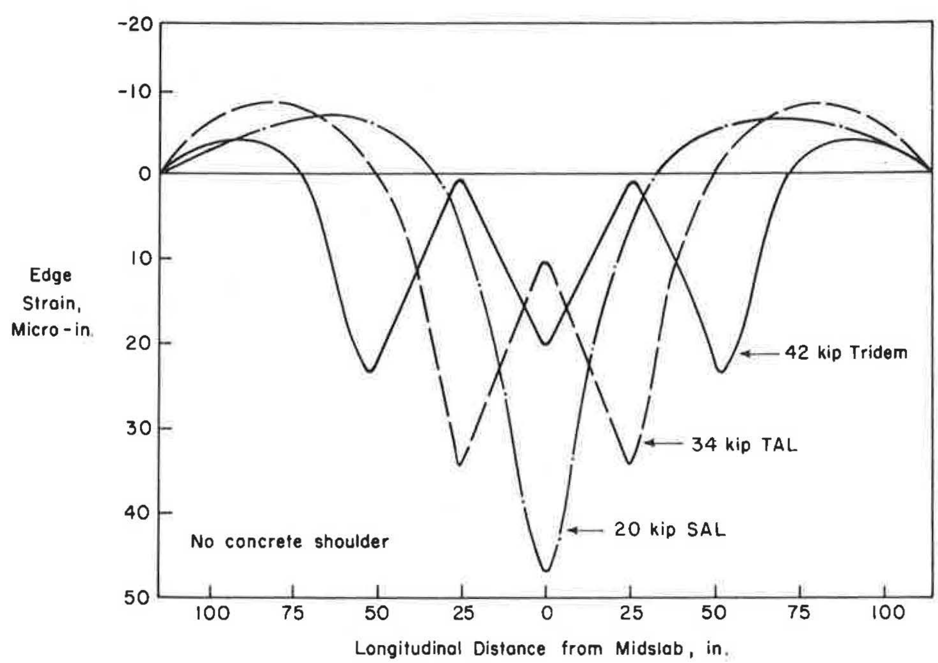

FIGURE 13 Calculated edge strain profiles.

2.30 to 2.50 times as damaging as a single axle weighing half as much as the tandem axles. The ratio of edge deflection under tandem axles to that under a single axle weighing half as much as the tandem axles is about 1.64 based on theoretical analysis and about 1.60 to 1.90 based on field measurements. On the other hand, calculated as well as measured edge strain under tandem axles are less than the edge strains under a single axle weighing half as much as the tandem axles. Thus, it can be seen that the AASHTO traffic equivalence factors give more weight to edge deflection response than any other response parameter when the effects of single and tandem axles are compared.

The ratio of edge deflection under a tridem axle to that under a single axle weighing one-third as much as the tridem axle is about 2.0 based on theoretical analysis and about 2.0 to 2.2 based on field measurements. By extrapolation, it is found that the ratio of edge deflection under a tridem axle to that under a single axle weighing 40 percent as much as the tridem axle is about 1.65 based on theoretical analysis and about 1.65 to 1.80 based on field measurements. Therefore, if proportionality is assumed between deflections and performance, a tridem axle can be considered about 2.30 to 2.50 times as damaging as a single axle weighing 40 percent as much as the tridem axle. As an example, a 50-kip tridem axle would be considered 2.30 to 2.50 times as damaging as a 20-kip single axle.

Based on this reasoning, traffic equivalence factors for tridem axles were developed for concrete pavements. These factors are listed in Table 7 and are considered tentative. The factors were developed by considering a tridem axle to be 2.40 times as damaging as a single axle weighing 40 percent as much as the tridem axle. The factors for each axle-load group were then established by using traffic equivalence factors for a single axle on a slab 9 in. thick that has a terminal serviceability of 2.5. A comparison of traffic equivalence factors for the single axles, tandem axles, and tridem axles is given in Figure 14. The factors for tridem axles presented in Table 7 and Figure 14 are considered applicable to slab thicknesses of 7 through 10 in.

TABLE 7 Traffic Equivalence Factors for Tridem Axles

Tridem-Axle Load (kips)	Traffic Equivalence Factor	Tridem-Axle Load (kips)	Traffic Equivalence Factor
30	0.43	46	2.64
32	0.55	48	3.12
34	0.70	50	3.77
36	0.91	52	4.32
38	1.20	54	5.04
40	1.44	56	6.00
42	1.68	58	7.20
44	2.16	60	8.06

Note: Terminal pavement serviceability index (p_t) = 2.5.

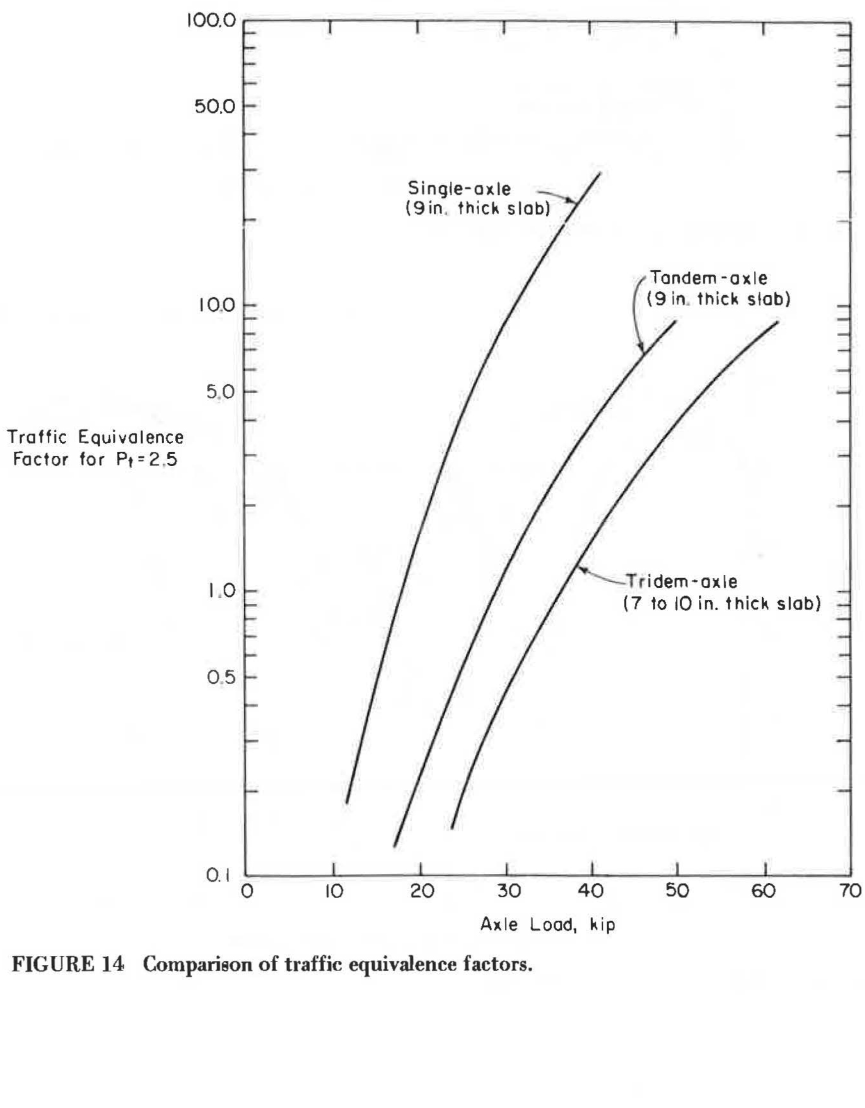

FIGURE 14 Comparison of traffic equivalence factors.

SUMMARY

A field study was conducted to evaluate the effect of frozen support and tridem-axle loading on concrete pavement performance. Pavement deflections and strains were measured during the fall and the winter at five project locations.

Study results indicate that pavement deflections and strains are greatly reduced during winter months when the support is frozen. Based on analysis of these results, it is concluded that the damaging effect of axle loads applied during the winter when the support is frozen can be considered to be only one-seventh as damaging as the same loads applied during the fall.

Study results also indicate that pavement deflections and strains are greatly reduced for a 42-kip tridem-axle loading as compared with those for 42-kip TALs. In fact, measured and calculated corner and edge deflections under 42-kip tridem-axle loadings were almost equal to or less than those for 34-kip TALs. Measured and calculated edge strains for a 42-kip tridem-axle loading were considerably lower than those for a 34-kip TAL.

Based on study results, it is concluded that for application to the AASHTO thickness design procedure, a tridem axle can be considered as equivalent to a single axle weighing about 50 percent of the tridem axle and to tandem axles weighing about 80 percent of the tridem axle. Traffic equivalence factors were developed for tridem axles on concrete pavements. These factors are tentative but may be considered for use in lieu of other field data on the effects of tridem axles on concrete pavement response.

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