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PTF machine collects data on actual pavement sections

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COVER: The effects of increased truck tire pressures on flexible pavement are a subject of great concern. Experiments to measure the effects of load, tire pressure, and tire type on the response of an asphalt concrete pavement have been conducted at the Pavement Testing Facility (PTF) located at the Turner-Fairbank Highway Research Center in McLean, Virginia.

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Departments

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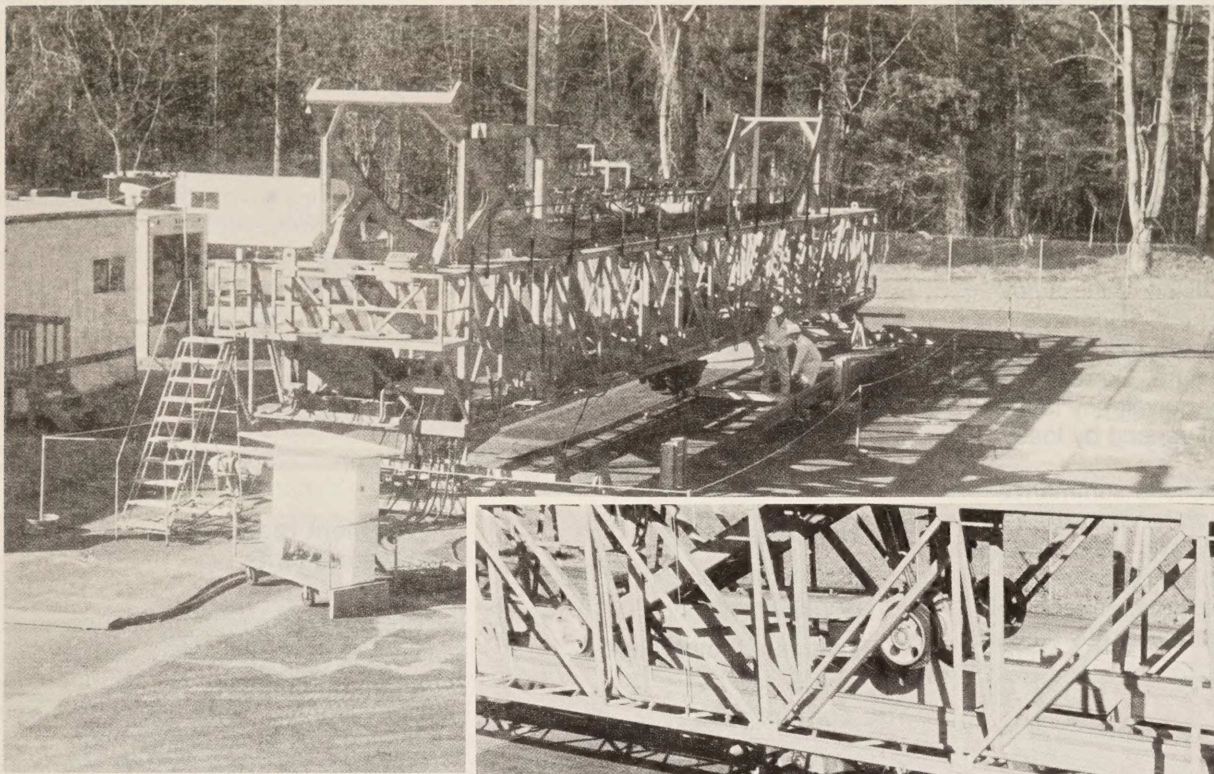

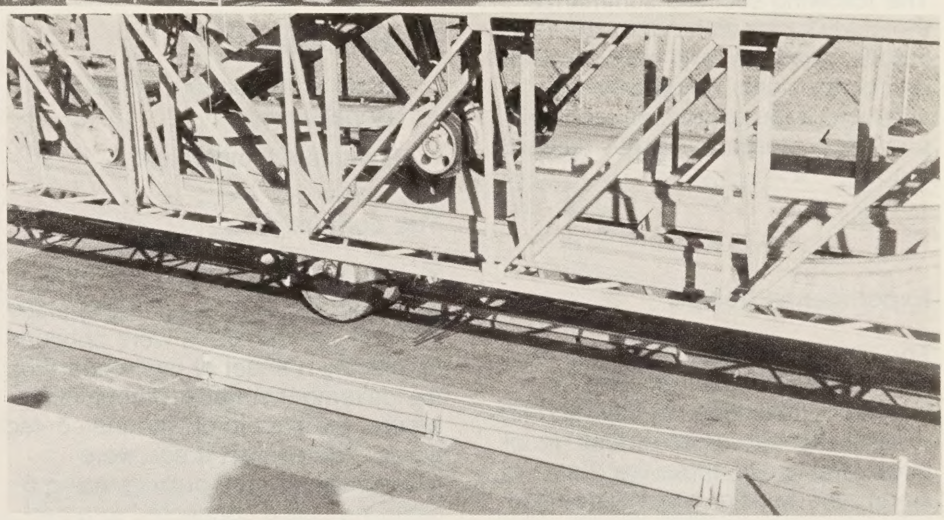

Effect of Load, Tire Pressure, and Tire Type on Flexible Pavement Response

by
Ramon Bonaquist, Charles Churilla, and Deborah Freund

Introduction

The effect of increased truck tire pressures on flexible pavement performance recently has become a subject of great concern. Various researchers have used analytical methods to attribute decreased fatigue life, increased rutting, and accelerated serviceability loss to the effects of increased tire pressure. (*1, 2, 3*)[1] The study summarized in this article investigated these concerns by measuring the effects of load, tire pressure, and tire type on the response of an asphalt concrete pavement. This experiment was conducted at the Federal Highway Administration's (FHWA's) Pavement Testing Facility (PTF), located at the Turner-Fairbank Highway Research Center in McLean, Virginia. Specifically, the researchers used the PTF's Accelerated Loading Facility (ALF), a machine used to simulate truck traffic. ALF models one-half of a single-truck axle with dual tires and can apply loads ranging from 9,400 lb to 22,500 lb (41.8 to 100.1 kN).

The experiment was conducted on Test Lane 2 at the PTF, which consists of a 2-in (50.8 mm) wearing course over 5 in (127.0 mm) of binder and 12 in (304.8 mm) of crushed aggregate base.

[1]Italic numbers in parentheses identify references on page 7.

Pavement responses measured were surface deflection, surface strain, and strain at the bottom of the asphalt layer. Fatigue equivalency factors were developed using an exponential relationship between the number of cycles to failure and the magnitude of the tensile strain at the asphalt layer's bottom.

The basic conclusion of the study was that each of the responses studied was affected more by load than by tire pressure. Correspondingly, the equivalency factors, too, are more influenced by load.

The following sections summarize the study's research approach, test results, analyses, and major findings and conclusions. A complete description of the experiment will be published later in an FHWA research report.

Table 1.—Experimental design

Load (lb)	Radial 76 (psi)	Radial 108 (psi)	Radial 140 (psi)	Bias Ply 76 (psi)	Bias Ply 108 (psi)	Bias Ply 140 (psi)
9,400	X	X	X	X	X	X
14,100	X	X	X	X	X	X
19,000	X	X	X	X	X	X

SI Conversions:
1,000 lbf = 4.448 kN
1 psi = 6.895 kPa

Research Approach

Experimental design

The objective of this study was to measure pavement response for various combinations of load and tire pressure for two types of tires. In all three load levels, three tire pressures, and two tire types were used in the experiment. Table 1 summarizes the experimental design. For each experimental cell, the following data were collected:

- Tire contact area.
- Surface deflection.
- Surface strain.
- Strain at the bottom of the asphalt layer.
- Pavement temperature.

Tire contact area

Two types of dual truck tires, radial and bias ply, were used in this study. The tire contact areas were obtained by placing a sheet of posterboard on the pavement surface, and painting around the tires to outline the contact areas. The contact areas were measured from the outlines using a digitizing board connected to a personal microcomputer. The contact areas were computed by algorithm for the individual tires in each pair. No correction was made for tread area. Figure 1 shows typical digitized traces for radial and bias ply tires.

Pavement response measurements

The pavement responses measured were surface deflection, surface strain, and strain at the bottom of the asphalt layer. Response curves were obtained for each of the measurements by using the PTF computer data acquisition system to monitor instruments at a selected point as the ALF wheel assembly traversed the pavement. Figure 2 shows the ALF dual wheel assembly and the pavement instrumentation.

Surface deflections

Surface deflections were measured with a deflection beam consisting of a linear variable differential transformer (LVDT) mounted at the midpoint of a 15-ft (4.6 m) long rigid

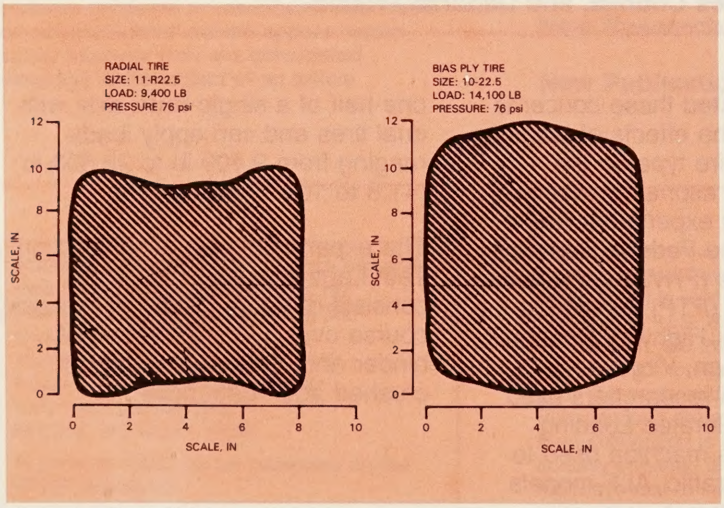

Figure 1.—Typical tire contact areas.

Figure 2.—ALF dual wheel assembly and pavement instrumentation.

reference beam. Figure 3 shows a typical plot of the LVDT signal versus longitudinal wheel position. The maximum deflection indicated by this signal is not the pavement's actual maximum deflection since the beam supports are within the deflection basin of the wheel. Actual pavement deflection can be obtained from figure 3 by adding the theoretical displacement of the beam center point. Since this study was concerned with relative effects, the correction was not applied to the deflection data.

Surface strain

Surface strains were measured with 2-in (50.8 mm) gauge length bonded foil resistance strain gauges. Surface strain gauge locations were selected to provide strain measurements outside the contact area as well as under the sidewall and center of the tire. Typical plots of surface strain versus longitudinal wheel position are shown in figures 4 and 5 for gauges mounted in the longitudinal and transverse directions, respectively.

Strain at the bottom of the asphalt layer

During construction of the PTF pavements, strain gauges were installed at the interface between the asphalt and the crushed aggregate base. Strains at the bottom of the asphalt layer were measured with these gauges. Typical plots of these strains versus longitudinal wheel position are shown in figure 6 for a gauge mounted in the longitudinal direction.

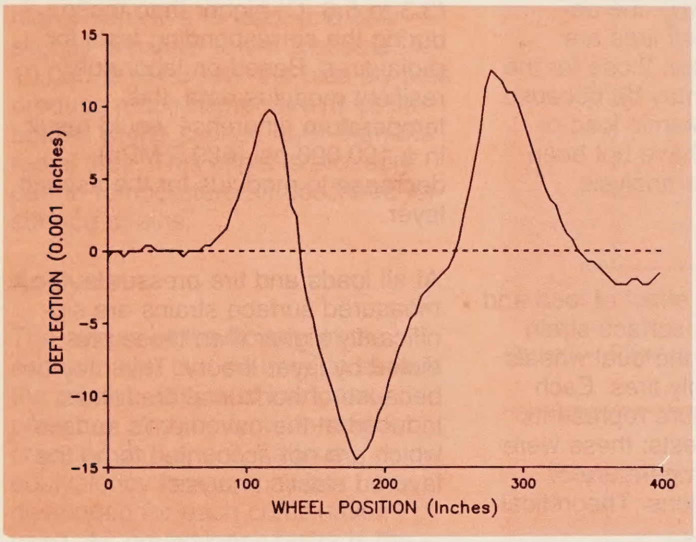

Figure 3.—Typical deflection beam response.

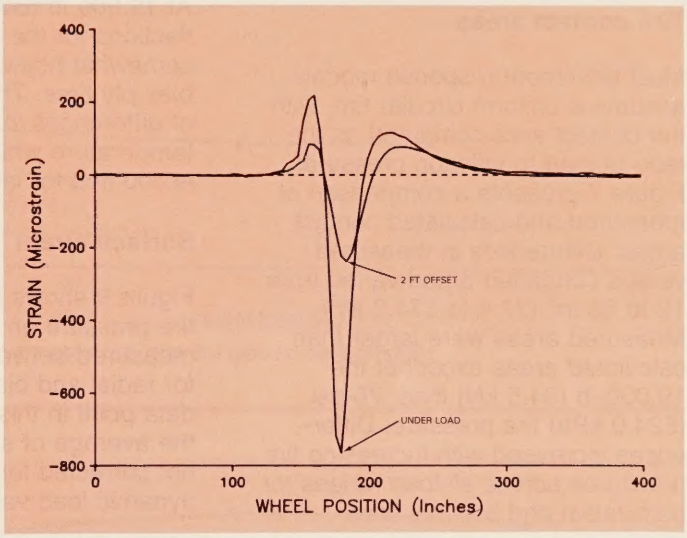

Figure 4.—Typical surface strain response for longitudinal gauges.

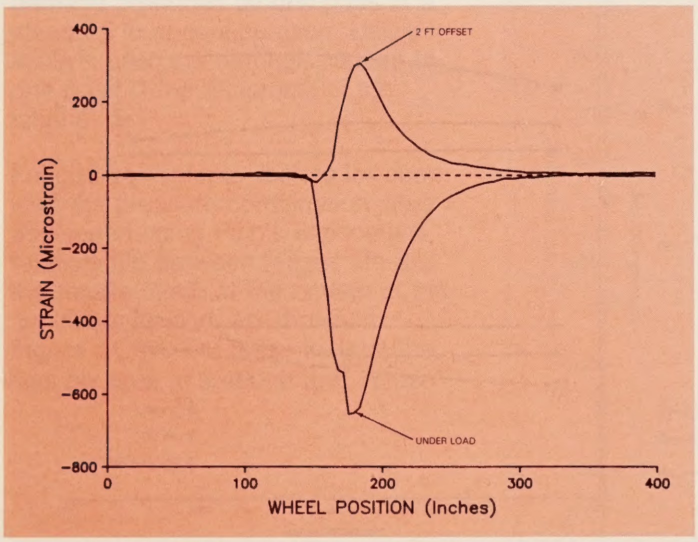

Figure 5.—Typical surface strain response for transverse gauges.

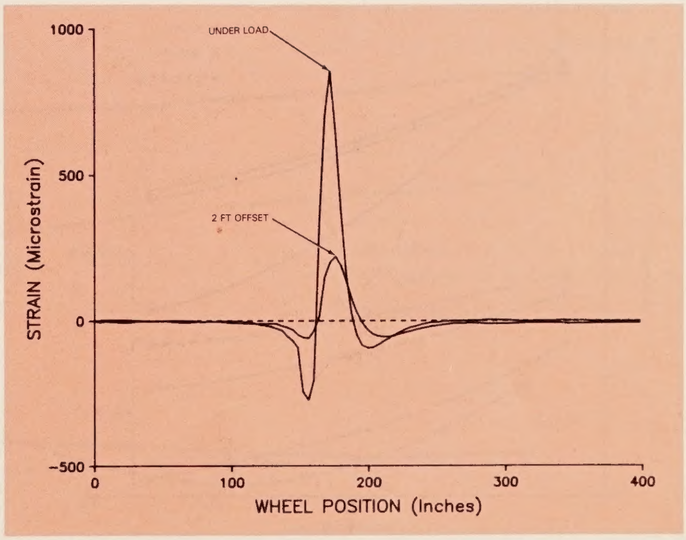

Figure 6.—Typical response for longitudinal strain at the bottom of the asphalt concrete.

Pavement temperatures

For each pavement response measurement, the temperature of the asphalt layers was measured with the thermocouples installed at the pavement surface and 2-in (50.8 mm) increments in the pavement adjacent to the response instrumentation. These temperature measurements provide a thermal profile for the asphalt layer which can be used for temperature adjustment of the pavement response measurements.

Test Results

Tire contact areas

Most pavement response models assume a uniform circular tire, with the contact area computed as the ratio of load to inflation pressure. Figure 7 presents a comparison of measured and calculated contact areas. Differences in measured versus computed areas varied from 12 to 58 in^2 (77.4 to 374.2 M^2). Measured areas were larger than calculated areas except at the 19,000-lb (84.5 kN) load, 76-psi (524.0 kPa) tire pressure. Differences increased with increasing tire pressures across all load ranges for both radial and bias ply tires.

Surface deflection

Figure 8 presents the effect of load and tire pressure on surface deflection for radial and bias ply tires. Each data point represents the average of six tests. Theoretical pavement surface deflections calculated from layer theory are also shown in figure 8.

The effects of load and tire pressure on surface deflection agree well with those predicted by layer theory. Both show the surface deflection 27 in (685.8 mm) from the center of the dual wheels to be highly sensitive to load and insensitive to tire pressure. At 19,000 lb (84.5 kN), the deflections for the radial tires are somewhat higher than those for the bias ply tires. This may be because of differences in dynamic load or temperature which have not been accounted for in this analysis.

Surface strain

Figure 9 shows the effect of load and tire pressure on the surface strain measured between the dual wheels for radial and bias ply tires. Each data point in this figure represents the average of six tests; these were not corrected for temperature or dynamic load variations. Theoretical surface strains from the sensitivity analysis are also shown.

At the instrumentation locations, the theoretical surface strains are insensitive to tire pressure, and are affected only slightly by load. The measured surface strains, however, show a different pattern. They are relatively insensitive to tire pressure but highly sensitive to load. At the 19,000-lb (84.5 kN) level, the bias ply tires produce much higher surface strains than the radial tires. This effect is probably due to temperature, not tire type. The average pavement temperatures during these tests were 6 to 10 °F (3.3 to 5.6 °C) higher than those during the corresponding tests for radial tires. Based on laboratory resilient modulus data, this temperature difference would result in a 100,000-psi (689.5 MPa) decrease in modulus for the asphalt layer.

At all loads and tire pressures, the measured surface strains are significantly higher than those predicted by layer theory. This may be because of horizontal tire forces induced at the pavement's surface which are not accounted for in the layered elastic analysis.

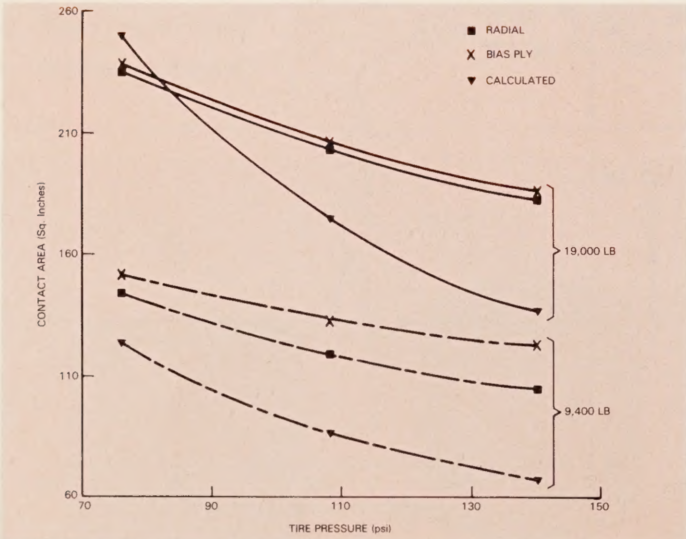

Figure 7.—Comparison of measured and calculated contact areas.

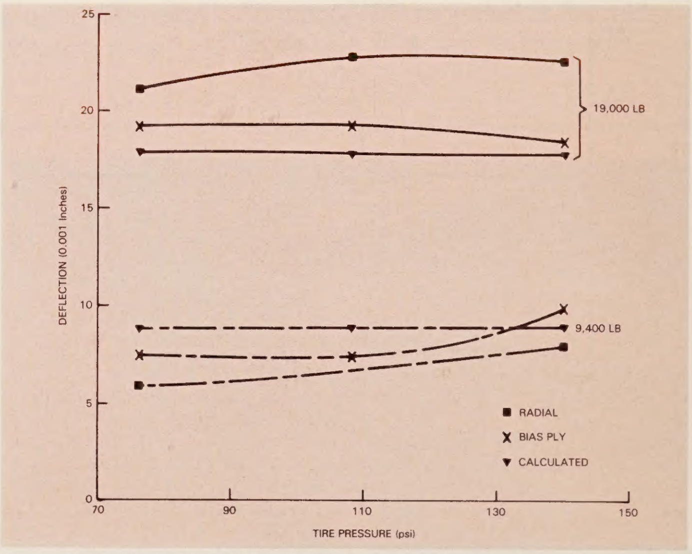

Figure 8.—Effect of load, tire pressure, and tire type on surface deflection.

Strain at the bottom of the asphalt layer

Figure 10 shows the effect of load and tire pressure on the strain at the bottom of the asphalt layer for radial and bias ply tires. Again, each data point represents the average of six tests which have not been corrected for temperature or dynamic load variations. Theoretical strains from the sensitivity analysis also are shown.

The effects of load and tire pressure agree well with those predicted by layer theory. Both show the strain at the asphalt layer's bottom to be highly sensitive to load, and only slightly sensitive to tire pressure. At 19,000 lb (84.5 kN), the bias ply tires produce much higher strains at the bottom of the asphalt layer than the radial tires. This effect is probably due to temperature as described for surface strains.

Analysis

The strains at the bottom of the asphalt layer can be used to assess the relative effects of load, tire pressure, and tire type on fatigue cracking for Lane 2. Fatigue equivalency factors may be developed for each combination used. An equivalency factor is the damage produced by one pass of any load configuration divided by the damage produced by one pass of a standard load configuration. Using Miner's Law, the damage caused by one pass is the reciprocal of the fatigue life.

Fatigue equivalency factors for each load-tire pressure combination were developed using Finn's exponential relationship between fatigue life and the tensile strain at the bottom of the asphalt layer and are shown in Figure 11. (*4*) For these factors, the bias ply tires at 9,400 lb and 76 psi

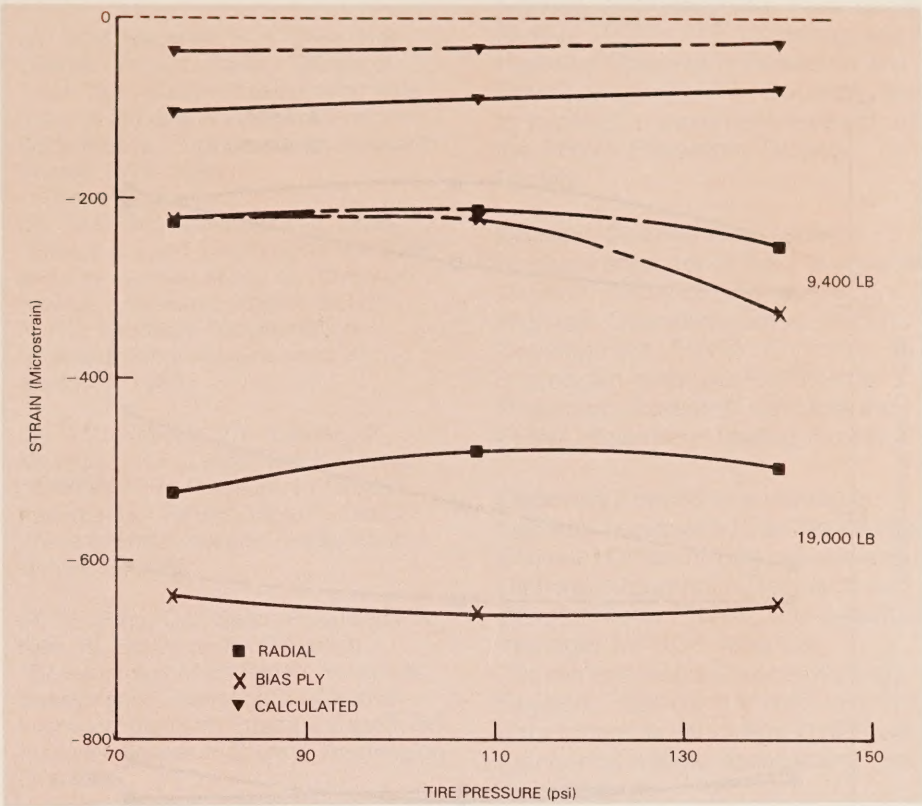

Figure 9.—Effect of load, tire pressure, and tire type on surface strain.

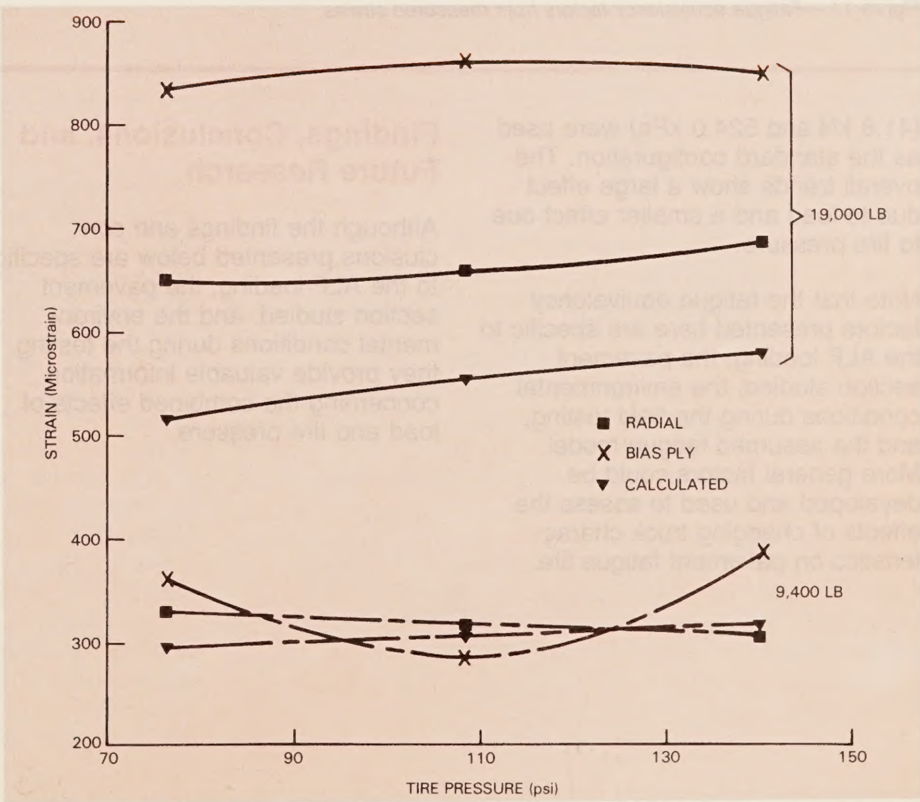

Figure 10.—Effect of load, tire pressure, and tire type on strain at the bottom of the asphalt concrete.

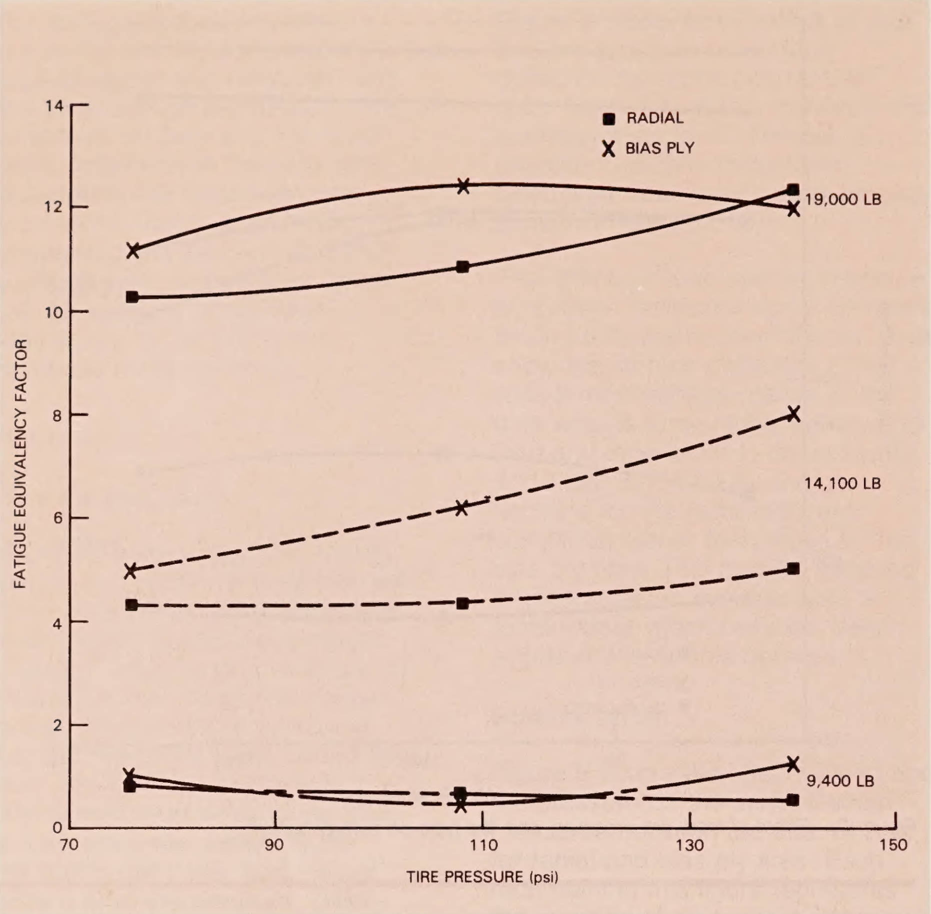

Figure 11.—Fatigue equivalency factors from measured strains.

(41.8 kN and 524.0 kPa) were used as the standard configuration. The overall trends show a large effect due to load and a smaller effect due to tire pressure.

Note that the fatigue equivalency factors presented here are specific to the ALF loading, the pavement section studied, the environmental conditions during the field testing, and the assumed fatigue model. More general factors could be developed and used to assess the effects of changing truck characteristics on pavement fatigue life.

Findings, Conclusions, and Future Research

Although the findings and conclusions presented below are specific to the ALF loading, the pavement section studied, and the environmental conditions during the testing, they provide valuable information concerning the combined effects of load and tire pressure.

Findings

- The measured tire contact areas were significantly larger than contact areas calculated as the ratio of load to tire pressure except at high loads and low tire pressure.
- The measured pavement responses—surface deflection, surface strain, and strain at the bottom of the asphalt layer—were affected by both load and tire pressure. For the loads and tire pressures used, load had a greater effect than tire pressure.
- Although layer theory underestimated many of the measured pavement responses, the measured effects of load and tire pressure were in general agreement with those predicted by the theory.
- For tests conducted at approximately the same temperature, the measured pavement responses were similar for radial and bias ply tires.

Conclusions

Specific conclusions concerning the relative effects of load, tire pressure, and tire type on rutting, and serviceability loss cannot be drawn from the data here. The measured pavement responses were not indicators of rutting potential, and an evaluation of serviceability loss requires performance data under each combination of load, tire pressure, and tire type.

The relative effects of load and tire pressure on pavement fatigue may be investigated using fatigue equivalency factors which account for both load and tire pressure. Such factors were developed in this study using an exponential relationship between the number of cycles to failure and the magnitude of the tensile strain at the bottom of the asphalt layer. Since this strain was affected more by load than by tire pressure, the equivalency factors are influenced more by load. Doubling the load from 9,400 to 19,000 lb (41.8 to 184.5 kN) increased predicted damage 1,000 percent, while doubling the tire pressure from 76 to 140 psi (524.0 to 965.3 kPa) increased predicted damage only 20 percent. From these fatigue factors, it can be concluded that, for the pavement section studied, the effect of increasing tire pressure from 76 to 140 psi (524.0 to 965.3 kPa) is equivalent to an axle load increase of approximately 2,000 lb (8.9 kN). This equivalency is valid for both radial and bias ply tires.

Future research

Finally, the authors would like to conclude with a brief discussion of future research activities concerning tire types and pressures planned for the PTF. The study summarized here was repeated on Lane 1 (5 in (127.0 mm) of asphalt over 5 in (127.0 mm) of crushed aggregate base) and the data is currently being analyzed. Information from these two studies will be combined with performance data for PTF sections trafficked to failure with different tire pressures to produce an FHWA research report assessing the impact of increased tire pressure on pavement response and performance. In 1989, the ALF will be modified to allow testing with various types and sizes of single truck tires. The next phase of research at the PTF will be directed at assessing the impact of single tires—conventional, low profile, and wide base—on pavement performance.

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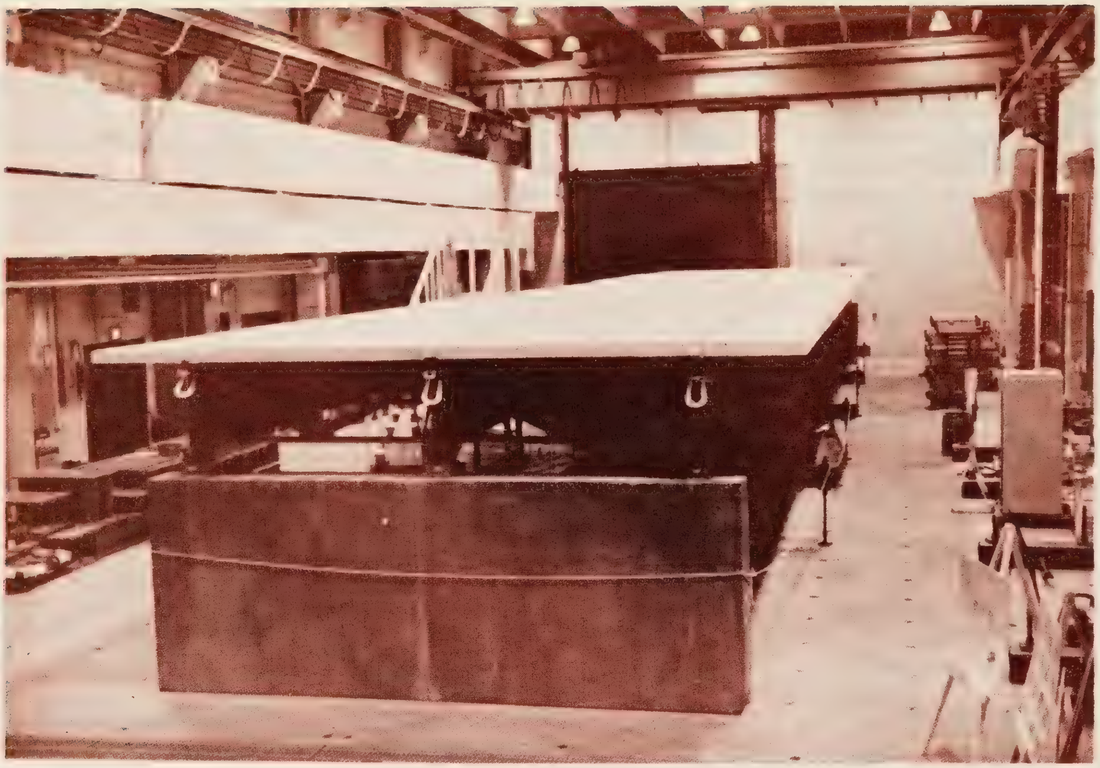


Structural Modeling for Autostress Design by Loading Through a Precast Deck

by
Lloyd R. Cayes

Introduction

In 1986, the American Association of State Highway and Transportation Officials (AASHTO) adopted a guide specification for designing rolled beam bridges based on the autostress concept. (*1*)[1] Such use of the autostress concept enables the designer to tap some of the substantial post-yielding strength present in composite continuous steel beam bridges. This is accomplished by considering the "shakedown" phenomenon at the AASHTO overload level and the plastic mechanism strength at maximum load.

The first two bridges designed according to the new AASHTO specifications were built in New York and Tennessee. Both cost substantially less than bridges designed by conventional procedures. (*2*)

Recently completed and ongoing research efforts are aimed at developing design methods that will permit designers to apply the autostress concept to plate girder bridges, and to incorporate these concepts into the AASHTO bridge specifications. (*3, 4, 5*) The latest of these research projects is a cooperative investigation by the Federal Highway Administration (FHWA) and the American Iron and Steel Institute (AISI). The project entails the design, erection, and testing of a 0.4-scale model of a two-span continuous highway bridge. The design, carried out by industry personnel, was based on the latest autostress research results and modular concrete deck concepts. Bridge components were produced in two commercial shops and erected by the steel fabricator in the the Structures Laboratory at the Turner-Fairbank Highway Research Center at McLean, Virginia. The testing phase is now in progress and is the responsibility of FHWA staff. Test results will be interpreted and reported by FHWA, University of Maryland, and industry personnel.

[1]Italic numbers in parentheses identify references on page 12.

Two main thrusts of this research study were planned. The main thrust of the steel-girder research is the extension of the autostress-design concepts to the design of steel plate-girder bridges.

Plate girder bridges are by far the single most widely used form of steel bridge construction, accounting for approximately 90 percent of the steel bridge market. The autostress research is aimed at maintaining simplicity of completed steel structures to minimize total cost including fabrication. Implementation of autostress concepts results in much lower steel fabrication costs and in the elimination of structural details with undesirable fatigue characteristics. Autostress design procedures already have been shown to prove demonstrated economies in actual short-span steel bridge design. Fabrication cost savings of up to 10 and 15 percent are possible over steel bridges designed using traditional methods. Maintenance costs also may be less since the bridge is less susceptible to fatigue damage.

The main thrust of the modular concrete deck portion of the research involves the extended use of higher strength concretes and modular deck panels. The primary emphasis of the deck research is aimed at:

- Better use of higher strength concretes.
- Increased use of transverse prestressing to improve durability of bridge decks.
- Increased use of longitudinal post-tensioning of bridge decks to provide greater capacities at lower costs.
- More continued use of the modular bridge deck concept with emphasis on new construction and rehabilitation applications, including strengthening and widening.
- Additional use of clustered shear connectors to increase composite action of bridges.

An additional thrust of the research is the study of the distribution of live loading to the individual girders. The results from this project will be instrumental in developing more realistic live-load lateral distribution factors that can be incorporated in the AASHTO design specifications for steel I-girder bridges. It has been demonstrated in numerous studies that more realistic live-load lateral distribution factors can result in additional cost saving in bridge design.

Design Approach

The prototype bridge was designed using load factor design (LFD) methods, as modified by autostress and plastic mechanism concepts. (*6*) In LFD, structural bridge members are proportioned for multiples of design loads to meet specified structural performance requirements at three load levels—service load, overload, and maximum load.

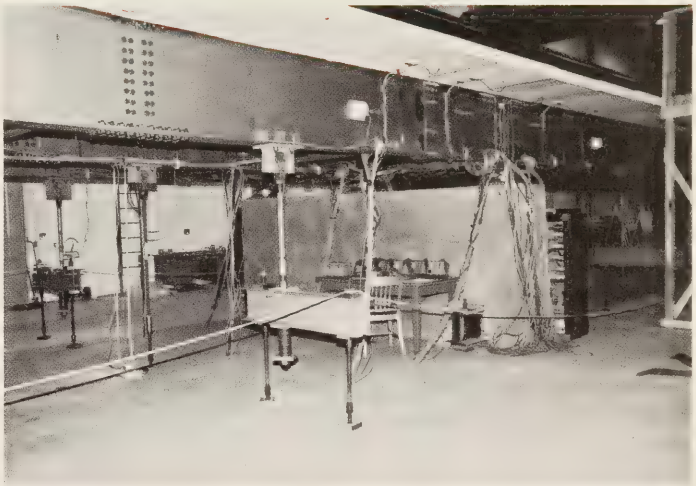
In addition to these LFD requirements, four new limit-state criteria were considered in the bridge design:

(1) Cambering for automoments.

(2) Cambering for differential creep and shrinkage effects.

(3) Shakedown at overloads.

(4) Maximum load based on formation of mechanism.



Center pier, instrumentation, and dead load fixtures under model bridge.

Items 1 and 3 were derived from the autostress concept, in recognition of continuous steel members' ability to adjust automatically for the effects of yielding occurring at the overload level. Bending moments at these higher loads are redistributed by the structure to lower-stressed sections. By taking advantage of this inherent ability, the designer can use uniform steel members in continuous spans along the entire bridge length or between field splices. The benefits include lower fabrication costs and elimination of structural details with undesirable fatigue characteristics. The autostress concept eliminates the need for a limiting stress in negative bending since continuous bridges are allowed to undergo controlled plastic deformations at interior piers. These deformations stabilize after a few passages of the overload vehicle and are small enough not to have an adverse effect on the smoothness of the ride over the bridge.

Item 2 is seldom considered in general bridge design practice; it was used in this study because of the project's research nature. The chosen method of approach is well known. (*7*)

Finally, item 4 was based on plastic theories for evaluating the strength of continuous structures. (*8*) Use of such theories represents an improvement in evaluating the maximum load.

Model Bridge Design

Design of the model bridge began with a full-scale prototype. This prototype was designed initially by LFD and subsequently modified according to the autostress concept. (*9*) The bridge was designed for AASHTO HS20 live load plus the alternate military loading specified for bridges on the Interstate System.

The prototype bridge is a three-girder, two-span continuous structure with equal spans of 140 ft (42.7 m) (figure 1). The superstructure consists of three parallel flange 68-in (1.73 m) deep steel plate girders spaced transversely at 17 ft (5.2 m). The overall deck width is 48 ft (14.6 m). The deck overhangs 7 ft (2.1 m) beyond the exterior girders; this arrangement provides good lateral load balance among the three girders. Steel for the girders was assumed to have 50 ksi (344.7 MPa) nominal yield strength. The deck was assumed to consist of 10-in (254 mm) full-thickness precast concrete panels 8 ft (2.4 m) wide and 48 ft (14.6 m) long, pretensioned at the fabrication yard transverse to the bridge roadway, and post-tensioned in the longitudinal direction of the bridge after erection on the supporting structure. The design strength of concrete for the panels was 6,000 psi (41.3 MPa).

Model bridge dimensions were determined by the physical limitations of the FHWA laboratory, fabrication techniques, and availability of plate material. Based on these limitations, a 0.4-scale factor was selected. Design of the model bridge was largely a process of applying the scale factor to the prototype bridge components, and rounding to practical dimensions and detailing.

The model bridge has two 56-ft (17.1 m) spans and girder spacing of 6 ft 9 in (2.1 m). A typical cross section is shown in figure 2. Each plate girder is approximately 28 in deep (711.4 mm). The top flange is 1/4 in by 5 5/8 in (6.3 by 142.9 mm). The web is 1/4 in by 27 3/16 in (6.3 by 690.3 mm). The bottom flange is 9/16 in by 8 in (14.3 by 203.2 mm) except near the girder ends where the flange thickness decreases to 5/16 in (7.9 mm).

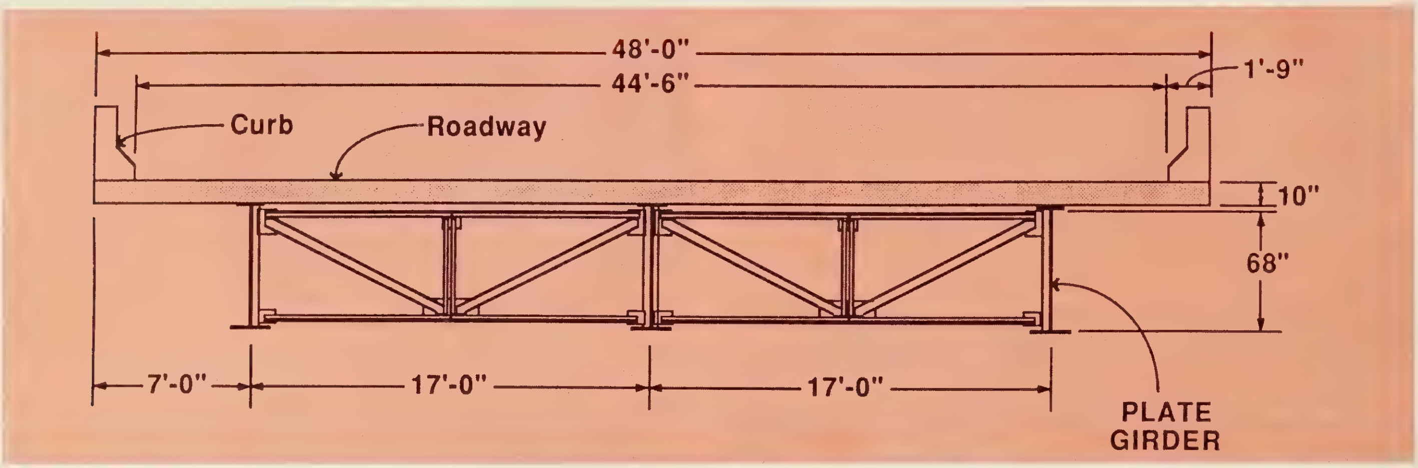


Figure 1.—Cross section of prototype bridge.

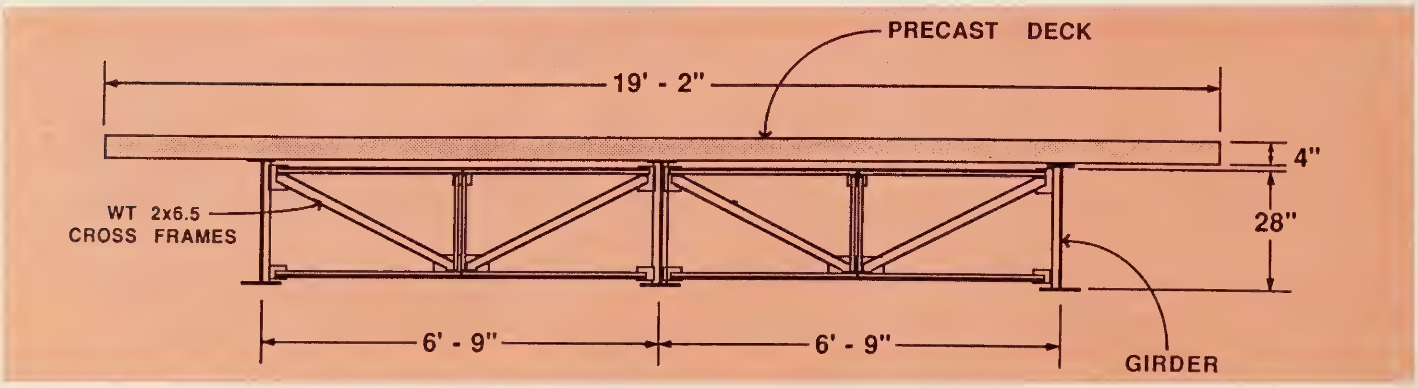


Figure 2.—Cross section of model bridge.

Fabrication and Erection

The steel plate girders, cross frames, and bearing assemblies were fabricated by Atlas Iron and Machine Works (AIMW) in their shop at Gainesville, Virginia. The precast deck panels were cast by Shockey Brothers, Inc., in their yard at Winchester, Virginia. The piers, steel framework, and deck panels were erected by AIMW in the FHWA laboratory at McLean, Virginia; and the slab composed of the precast deck panels was post-tensioned by VSL Corporation of Springfield, Virginia.

Instrumentation

Instrumentation of the model bridge was designed to monitor the test specimen's behavior at critical locations during the test program. Following erection of the steel framing, instrumentation was installed to measure response during the remaining phases of bridge construction and during testing. Primarily, the instrumentation was designed to monitor loads, reactions, deflections, and strains.

Applied loads and bridge reactions are measured with load cells. Deflections of the bridge are monitored by deflection gauges attached to the bottom flanges at selected locations and by taking elevation profiles on top of the deck. Strain measurements on both the steel girders and the concrete deck elements are monitored using electrical resistance and Whittemore strain gauges. A variety of Whittemore gauge lines are used to monitor strains in the deck panels and across panel-to-panel joints.

After installing the instrumentation and erecting the load fixtures, the compensatory dead load (DL1) was applied to the steel girders. The objective of this loading was to eliminate the difference in dead load stresses between the prototype and the model bridge.

Additional compensatory dead load (DL2) was applied to the bridge following erection of the bridge deck. This load was designed to simulate the composite dead loads that would be present on the bridge as a result of curbs, parapets, and a future wearing surface.

The bridge test plan summary shown below encompasses a series of test phases from initial construction to failure of the bridge and its components.

Summary of Model Bridge Test Plan

Task A: Erect steel girders and cross frames. Install and check steel girder instrumentation.

Task B: Apply compensatory noncomposite dead load (DL1) to the steel girders.

Task C: Erect precast panels and post-tension longitudinally. Weld shear connectors. Grout panels to steel girders.

Task D: Apply composite dead load (DL2) to the model bridge.

Task E: Determine elastic influence surfaces for pier sections and maximum positive moment sections.

Task F: Determine elastic lateral load distribution to the exterior and interior girders in positive and negative bending at service load.

Task G: Transfer the dead load to the top of the model bridge deck.

Task H: Apply single concentrated loads to study lateral load-distribution behavior for exterior and interior girders in positive and negative bending before shakedown at overload.

Task I: Apply simulated overload truck loads (plus impact) in positive-bending regions before automoment formation.

Task J: Apply simulated overload lane loads (plus impact) to form automoments.

Task K: Re-apply simulated overload truck loads (plus impact) in positive-bending regions to observe shakedown after automoment formation.

Task L: Re-apply single concentrated loads to compare lateral load-distribution behavior for exterior and interior girders in positive and negative bending before and after shakedown at overload. Repeat the test with selected cross frames removed.

Task M: Apply additional dead load (DL3) to approximate the theoretical increase in dead load at maximum load.

Task N: Apply simulated maximum load lane loads (plus impact).

Task O: Apply simulated lane loads past maximum load (plus impact) until the bridge can resist no additional load.

Task P: Perform punching tests on the precast deck panels.

Conclusions

As of this writing, testing of the model bridge is currently under way at the Turner-Fairbank Highway Research Center. The 2-year test program is about 70 percent complete, and should be concluded in early 1989.

The results from this research eventually will have wide implications that will greatly benefit both FHWA and the American public. Besides being one of the largest single bridge research projects ever undertaken, this project is expected to yield far-reaching results that will produce significantly favorable changes in the AASHTO steel and concrete bridge design and construction specifications.

Results of data collected to date generally indicate that the behavior of the bridge agrees with elastic theory.

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(*4*) K.H. Frank and A. Vasseghi, "Ultimate Strength of Composite Steel Plate Girders," American Iron and Steel Institute Project 320A, March 1985.

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(*9*) "Standard Specifications for Highway Bridges," *American Association of State Highway and Transportation Officials,* Washington, DC, 13th Edition, 1983.

Lloyd R. Cayes is the manager of the Structures Laboratory, Structures Division, Office of Engineering and Highway Operations Research and Development, FHWA. He has been active in bridge research for 25 years and currently is coordinating testing and data reduction for this research study. He completed the FHWA Highway Engineer Training Program in 1960.

Field Evaluation of Calcium Magnesium Acetate During the Winter of 1986–87

by
Brian H. Chollar

Introduction

This article briefly discusses the key findings of the various field deicing studies of Calcium Magnesium Acetate (CMA) conducted in four States during the winter of 1986–87. Findings from three of these studies are also documented in greater detail in: "Field Deicing Tests of High Quality Calcium Magnesium Acetate," (Final Report, Wisconsin DOT, June 1987); "Reduced Salt Experiments," (Highway Maintenance Division, Massachusetts Department of Public Works, 1987); and "A Comparative Field Study of Calcium Magnesium Acetate and Rock Salt During the Winter of 1986–87," (ME–87–16, Ontario Ministry of Transportation and Communications, September 1987).

Wisconsin

Calcium Magnesium Acetate, CMA coated on sand, and CMA mixed with sand were used to deice the two southbound lanes of a 6.6-mi (10.6 km) stretch of a divided highway, Route 14, just south of Madison, Wisconsin. Salt materials were used on the northbound lanes as a control to assess comparative melt characteristics. The CMA deicing was done on this highway in snow storms from November through February; temperatures ranged between 24 °F and 32 °F (−4 and 0 °C) during the storms.

During November, three snow storms occurred. CMA and salt were used in the comparative ratio of 1.6:1 to deice the CMA test and salt control roadways. Deicing was effective on both roadways.

In December, seven snow storms occurred. The weight ratio range of CMA to salt used for deicing during these storms was 1.4 to 1.47:1. The CMA usually melted the snow at a slower rate than salt.

In January, there were a total of seven storms. Three of these extended over 2 days. For the first five storms, CMA and salt were used at a weight ratio of 1.22:1. Deicing was comparable at essentially the same conditions. A one-to-one weight ratio of CMA-sand mixture was used in the sixth storm. A 2.2:1 weight ratio of CMA in this form was used. A one-to-one salt-sand competitive mixture was used for comparison. The sixth storm also involved the use of CMA coated on sand (1:3 parts by weight). In this storm, 20.5 tons (18.6 Mg) of CMA coated on sand and 5 tons of CMA were used. Fourteen tons (12.7 Mg) of salt were also used for a comparative weight ratio of 5:7 CMA to salt to achieve a base pavement.

Two features of CMA spreading have become apparent. The trucks must be covered tightly with a tarpaulin to prevent wind-scatter and loss when the truck is on the road. In addition, a substantial buildup of the CMA on the spreader wheel occurs during application. The latter problem appears to be due to CMA combining with water splashed from the truck wheels. A Teflon™ spreader wheel might correct this problem. Currently, the CMA is chipped from the wheel after each load or two is applied.

One practical problem with the CMA-coated sand has arisen. While a truckload of CMA (about 5 tons (4.5 Mg)) will cover the entire treatment area (14.5 lane mi (23.3 lane km)), a truck load of CMA-coated sand (about 7.5 tons (6.8 Mg)) sometimes does not complete the entire road at adequate treatment levels. In addition, the reduced ice-melting abilities of the 1:3 CMA/sand mixture requires an increased number of loadings, which appears to be a concern for the truck driver.

CMA is an effective deicer, though it is somewhat slower acting than salt. The low density and roundness of the current product contributes to this. In addition, its lightness makes it subject to blowing off the trucks unless covered and to bouncing off the pavement when applied. The final resting place is frequently at the pavement edge. Consideration should be given to producing a more angular CMA product that stays on the pavement and improves its pavement deicing characteristics. The CMA coated sand was heavier and more angular, and had much better adhesion to the treated pavement.

All dry pavement deicers required traffic to create a brine for effective pavement deicing. The test site favored salt because 14 of the 18 snowfalls were in the morning, and the salt lanes had higher commuter traffic at that time.

There were no snowfalls with cold temperatures of 10 to 20 °F (−12 to −6 °C) that winter; thus, there was no opportunity to test CMA in very cold temperatures.

The application of CMA definitely produced a pavement which was darker in color when dry. In a comparison, an adherent coating of the CMA on car windshields and other places was more difficult to remove than salt.

Massachusetts

CMA was used on a 4.8-mi (7.7 km) stretch of Route 138 roadway located in southern Massachusetts, running in a north-south direction through the town of Somerset. This roadway has a considerably wide climatic variation, ranging from a warm and moist extreme at its southern end to a cold and dry extreme at its northern end. Its paving surface is bituminous, and is fairly level. The roadway's speed limit is 30 mi/h (48.3 km/h) and its average daily traffic is 16,600 vehicles. The road's traffic pattern is urban, with many cross streets and traffic lights.

The control roadway was a part of Route 6, located at the south end of the CMA test site, and running east-west for a 4.9-mi (7.9 km) distance. Pure CMA conforming to FHWA-suggested specifications was used. It was stored in bulk in the Department's standard enclosed pole barn storage buildings. Initial depositing of the material in the storage area was very dusty, but the amount of dust was substantially lowered by subsequent stacking of the material with a front end loader.

Spreading of the CMA and salt deicers was accomplished with a hydraulically-driven Torwell spreading unit equipped with a synchrometer which provided a consistent material output with changing vehicle speeds. An application rate of 300 pounds per lane-mi (84.6 kg/km) was used in the spreading of both CMA and salt deicers. The loaded truck and spreading unit were enclosed by canvas tarpaulins.

The first snow storm, with temperatures ranging from 28 to 32 °F, (−2 to 0 °C) lasted 2 hours. One application of CMA was made approximately 30 minutes after the storm started. This initial treatment started to melt the precipitation in approximately 7 to 10 minutes. The salt began melting in the control section almost immediately, producing a brine in 5 minutes. The test roadway went to base and wet pavement approximately 30 minutes after application of the CMA.

The second storm at a temperature of 32 °F (0 °C) blanketed the test roads with 4 in (1.029 cm) of snow in a 10-hour period. The maximum rate of snowfall was 1-in per hour (2.5 cm/hr). CMA was applied about 30 minutes after the storm's onset. Again, it started its noticeable melting within 10 minutes after application. On bridge decks, however, 30 minutes were required for melting. Several other storms at temperatures ranging from 28 to 32 °F (−2 to 0 °C) were used for testing CMA and CMA versus salt. The results were similar to previous tests.

Two similar storms with temperatures varying from 9 to 36 °F (−13 to 2 °C) occurred. Each deposited from 7 to 10 in (17.8 to 25.4 cm) snow on the test sites in a 16-hour period. The initial application of CMA resulted in a noticeable snow-melt within 15 minutes. As the temperature dropped, however, CMA required 30 to 40 minutes to start a noticeable snow-melt. Larger initial applications of CMA had no noticeable effect on this rate of melt. Fortunately, the snow did not pack on the test highway, and remained plowable for the storms' durations. When both storms ended, additional applications of CMA were used to obtain bare and wet pavement.

CMA's melt function appeared to improve with increases of vehicular traffic on the test roads. An apparent reliance on traffic for CMA melting ability was particularly apparent at the low temperature of 15 °F (−9 °C) where no melting is obtained without traffic. Salt, on the other hand, acted slowly but noticeably better than CMA under the same conditions. As soon as there was a rise in temperature above 15 °F (−9 °C) and an appearance of traffic, both CMA and salt deiced their respective test sites to a bare and wet condition in a nearly equal amount of time.

Ontario

The test site for CMA was a 1.5 mi (2.4 km) section of QEW near Beamsville, and included both the freeway and the adjacent service roads. Sections of the freeway and service roads to the east of the site that were geometrically similar to the site were used as the salt control site. The AADT is 38,300 on the freeway portions but less than 500 on the service roads. All road surfaces were bituminous concrete.

CMA or CMA mixed with sand (14% CMA by weight) were used, depending upon weather and pavement conditions. When there was a reasonable expectation of maintaining a bare pavement, pure CMA or salt was applied. The specified application rate for salt was 230 lb per lane-mile (64.8 kg/km). The initial specified application rate for CMA was 1.7 times that of salt, or 391 lb per lane-mile (110.2 kg/km). However, the first few storms showed this CMA rate to be excessive, and new rates of 1.4 to 1.5 times that of salt (322 to 345 lb per lane-mile (90.8 to 97.2 kg/km)) were set for the application of CMA. The application rates of CMA-sand (14% CMA by weight) or salt-sand (10% salt by weight) were 1,415 lb of mix per lane-mile (400 kg/km). The number of deicer applications depended on the conditions, and therefore varied for each deicer used in each storm.

Most storms occurred with temperatures from 23 to 32 °F (−5 to 0 °C), and lasted from 2–3 hours to 3 days. The performance of either CMA or salt was not adversely affected within this temperature range.

The storage and handling characteristics of CMA were comparable to those of salt. Some dusting occurred during loading operations, but did not require face masks for the handlers. Also, tarpaulins were not needed on trucks to prevent CMA from blowing, even on windy days. The amount of dusting of the CMA-sand mixture was greater after a period of prolonged storage—6 weeks or more. On contact with moisture, CMA became sticky and adhered to the spreader hopper, dispensing chute, and spinner. Avoiding this buildup required an effective discharge of CMA by a slowly rotating spinner, spreading CMA evenly and effectively over a 6.56-ft (2 m) wide band along the centerline. Salt-sand and CMA-sand mixtures were spread with the spinner operating at normal speeds.

The trucks and spreading equipment used for dispensing deicers during every snow or sleet storm were washed free of salt or CMA deicer after use. Examination of salt-handling trucks revealed bare spots with rusted metal. Salt spinners were also badly rusted. The trucks and spinners used spreading for CMA had no rust.

CMA-sand and salt-sand mixtures were primarily used on service roads. Through the entire season, the ratio of CMA-sand mixture to the salt-sand mixture was 68:100, or a CMA to salt ratio of 95:100. The ratio of pure CMA to salt for the entire winter was 1.19:1.0. However, the CMA to salt ratio used varied between 1:1 and 2:1, depending on the type of storm experienced. Fewer but larger CMA applications were made than for salt. Also, the time to achieve bare pavement varied according to a storm's intensity. Throughout the test, the performance of CMA and salt as deicers was comparable, exhibiting equivalence of melt within 45 minutes of each other.

A high volume of traffic on CMA- or salt-treated roadways increased the melting rate on roadway precipitation significantly and equally for both deicers. When CMA was applied to a roadway's covering precipitation, it penetrated to the roadway surface and maintained its melting action on any proximate snow or ice. When the roadway was dry, the CMA remained bonded to the pavement awaiting further precipitation. The salt did not bind to the pavement to an equal degree, and was more easily removed from the roadway by traffic, wind, or any subsequent snow plowing.

These findings highlight some differences in the performance of CMA and salt as deicers. Additionally, mention should be made that the CMA was more effective during longer storms of 2 to 3 days' duration. Also, as the season progressed, the required comparative ratio of CMA to salt was decreased. There was evidence of CMA retention in the pavement after one of the storms late in the season. In this case, the imbedded CMA either repelled or did not attract moisture, resulting in no pavement icing.

California

During the winter of 1986–87, field testing of CMA and CMA-coated sand was conducted at three sites: Interstate 80 near Kingvale; State Route 50 near South Lake Tahoe; and State Route 88 near Caples Lake. These areas offered a variety of weather and road conditions. The CMA was purchased and used in 50-lb bags (22.7 kg). The CMA-coated sand was purchased and used in bulk. The site at Interstate 80 used CMA mixed with sand at a rate of 1 to 2 by volume, or 1 to 4 by weight. The site at State Route 88 used the same mix ratio by volume of CMA to cinders. The site at State Route 50 used CMA-coated sand, containing 25 percent by weight CMA. All three sites applied these mixtures at about 500 lbs (140.9 kg/km) per lane-mile. All sites used salt-sand mixtures (25 percent by weight of salt), at an application rate of 200–300 lb/lane-mile (56.4–84.6 kg/km).

The evaluation included the procedures used to handle CMA materials and salt; the operational aspects of deicing, including the effect on equipment; spread pattern and effectiveness of deicing; product deterioration during storage; and the overall acceptance of CMA versus salt by maintenance personnel.

Maintenance personnel at the Interstate 80 site observed that CMA did not deice the roadway as fast as salt during the storms. As soon as the storms were over, CMA roadways became free of snow pack faster than the salt roadway. The salt area had a tendency to refreeze and produce more pack; while the CMA area did not. CMA kept the snow loose (not in a pack) so traffic or snowplows could remove it easily. Observations of traction showed that in the CMA area, under most road and weather conditions, skidding tires would push the snow pack away easily and reach the bare pavement for traction. Skidding tires in the salt area would not break through the snow pack, but instead would glide over these packs, depending upon road and weather conditions.

Maintenance crew acceptance of this material at that site was very poor for several reasons. CMA did not deice as fast as salt. Also, CMA crews did not like the CMA odor and its dusting properties. Finally, they were afraid of possible health effects from using it.

Observations of the deicing effects of CMA on sand by the maintenance personnel at the State Route 50 site were similar to those of the personnel at Interstate 80. At the outset, there was an operational problem with the CMA-coated sand material that prohibited any direct comparison studies of it with salt-sand mixtures. An uncovered deicing truck was being used to disperse the CMA material, and moisture from precipitation was falling on the CMA material in the truck, causing the CMA to cake into solid masses and adhere to the truck's metal parts, including the spinner. Maintenance personnel solved this problem by covering the truck and spinner. Afterwards, the CMA material remained dry, flowed evenly, and spread without any problems.

The observations of the deicing effects of CMA-cinders mixture by the maintenance crew at State Route 88 were similar to the those of the other areas. They liked the CMA material because it kept snow pack from forming. They were able to remove loose snow easily using CMA. They also observed that CMA had a residual effect. After using the CMA-cinders mixture for a few storms, they did not have to apply this material as early in successive storms to obtain the same deicing effect and prevent snow pack formation. They were able to remove snow more easily for longer periods of time before having to use more CMA materials. They had no handling problems with the CMA material.

A laboratory study in California to determine the storage properties of CMA materials showed that upon exposure for 1 year indoors at normal humidity, CMA moisture content rose to almost 14 percent of the weight of CMA.

General Conclusions

Most of these field trials tested the comparative deicing effects of both CMA and salt in snowstorms with temperatures ranging from 24 to 32 °F (−4 to 0 °C). The findings indicate that CMA deices roadways as effectively as salt, but requires a longer period of time to accomplish this, depending on conditions. The amount of CMA used and the application rates varied according to existing conditions. Generally, the application rates of CMA for snow storms accompanied by warm temperatures—24 to 32 °F (−4 to 0 °C) —were from 1.4 to 1.6 times that of salt by weight. At these rates fewer applications were needed of CMA than salt. Only 1.1 to 1.2 times more CMA by weight was used than salt for most storms.

Trucks conveying the CMA required protection from precipitation. The spreader was especially susceptible to occlusions of dampened CMA that crusted the spreader and truck, interfering with effective CMA applications.

Dust from the CMA could be an inhalation problem for maintenance personnel. However, this problem can be improved through users of CMA handling the material in bulk, with lesser amounts of powder. Protective masks would be useful in eliminating or minimizing inhalation of fine particles, even when using a dusty CMA.

A benefit of CMA was that roadways treated with this deicer showed signs of maintaining the deicing effect from storm to storm. Another observed benefit was the facile manner in which CMA loosened packed snow for easier removal by snow plows or traffic melting. In this guise of "snowfluffer," it was superior to salt.

Imitating the British practice of mixing urea deicer with sand to counter winds blowing the urea off their bridges, ample supplies of sand coated with CMA, sand mixed with CMA, and sand mixed with salt were furnished for field trials. Comparative use showed that the CMA-sand mixtures or CMA-coated sand showed varying deicing effectiveness when compared with salt-sand mixtures. However, the comparisons were too few to draw any final conclusions on relative effectiveness.

Field tests are continuing in these States during the winter of 1987–88. In addition, five other States and Canadian provinces have started their field evaluations of CMA.

Brian H. Chollar is a research chemist in the Materials Division, Office of Engineering and Highway Operations Research and Development, FHWA. He currently oversees FHWA research in the use and characterization of "Calcium Magnesium Acetate (CMA)." He also manages contracts and conducts staff studies in several areas of materials research including delineation, alternative binder materials, chemistry of asphalts, and additives to asphalts. Before joining FHWA in 1974, Dr. Chollar was a research chemist at NCR Corporation involved with synthesis and analysis of organic materials including liquid crystals, organometallic compounds, and substituted porphyrins.

Recent Research Reports You Should Know About

The following are brief descriptions of selected reports recently published by the Federal Highway Administration, Offices of Research, Development, and Technology (RD&T). The Office of Engineering and Highway Operations Research and Development (R&D) includes the Structures Division, Pavements Division, and Materials Division. The Office of Safety and Traffic Operations R&D includes the Traffic Systems Division, Safety Design Division, and Traffic Safety Research Division. All reports are available from the National Technical Information Service (NTIS). In some cases limited copies of reports are available from the RD&T Report Center.

When ordering from the NTIS, include the PB number (or the report number) and the report title. Address requests to

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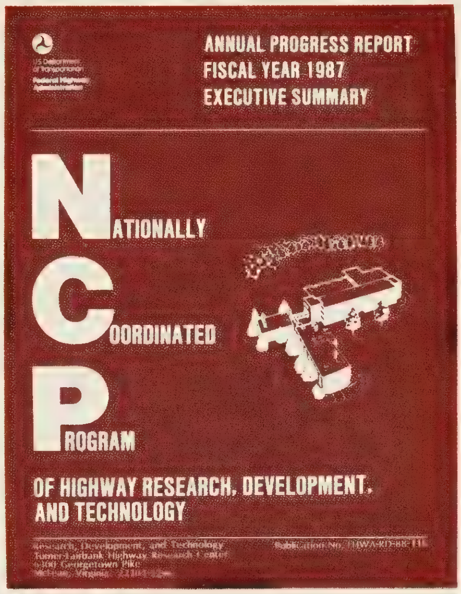

Annual Progress Report Fiscal Year 1987 Executive Summary Nationally Coordinated Program (NCP) of Highway Research, Development, and Technology, Publication No. FHWA–RD–88–116.

by Operations Staff

This executive summary gives an overview of progress being made under the Nationally Coordinated Program (NCP) of Highway Research, Development, and Technology during the period from October 1, 1986, through September 30, 1987.

The Nationally Coordinated Program (NCP) of Highway Research, Development, and Technology, a new management structure, replaced the Federally Coordinated Program (FCP) in fiscal year 1987. The NCP continues the aims of the FCP: To ensure concentration of resources on urgent common problems, to eliminate unnecessary duplication of effort among researchers, and to identify and highlight gaps. It also, for the first time, attempts to provide a complete listing of all highway research activities. Along with the Highway Planning and Research (HP&R) program, the National Cooperative Highway Research Program (NCHRP), and the Federal Highway Administration's contract and staff research programs, the NCP will include work of the Strategic Highway Research Program (SHRP) administered by the National Research Council, FHWA's research on highway transportation policy and planning, and motor carrier transportation, and, to the extent possible, RD&T activities wholly funded by the States.

This report covers technologies for highway design, construction, and operation including the specific categories of: Highway Safety, Traffic Operations, Pavements, Structures, Materials and Operations, Policy and Planning, and Motor Carrier Transportation.

Limited copies of this Executive Summary are available from the RD&T Report Center. The publication also may be purchased from the NTIS: (PB No. 88–157979, Price code: A05).

Effect of Calcium Magnesium Acetate (CMA) on Pavements and Motor Vehicles, Final Report, Report No. FHWA/RD–87/037

by Materials Division

The corrosion and/or deterioration of highway-related and automotive-related materials by chloride-containing deicing chemicals has become a major economical problem in the United States. The Federal Highway Administration has developed CMA as an alternative to these chloride-containing chemicals (specifically, sodium chloride). This report describes the comparative effects of CMA and sodium chloride on various highway-related and automotive-related materials. The results of various exposure techniques, followed by extensive additional testing, indicate that CMA is much less deleterious to highway-related and automotive-related materials than sodium chloride.

Limited copies of this report are available from the RD&T Report Center. The report also may be purchased from the NTIS: (PB No. 88–118351/AS, Price code: A09).

Quantification of Urban Freeway Congestion and Analysis of Remedial Measures, Staff Report, Report No. FHWA/RD–87/052

by Traffic Systems Division

Urban freeway congestion is a serious and growing National problem; one which is receiving increasing attention from local, State, and National officials.

This study quantifies both existing (1984) and expected future (2005) levels of urban freeway congestion in terms of congested travel, motorist delay, excess fuel consumption, and user costs on a National basis. The aggregate impacts of various methods of addressing the problem also are calculated.

The report may be purchased from the NTIS: (PB No. 88–169842, Price code: A03).

Safety Effects of Cross Section Design for Two-Lane Roads, Vol. I, Final Report, Report No. FHWA/RD–87/008

by Safety Design Division

This study was intended to quantify the benefits and costs resulting from lane widening, shoulder widening, shoulder surfacing, sideslope flattening, and roadside improvements. Detailed traffic, accident, and roadway data were collected on approximately 5,000 miles of two-lane roads in seven States. An accident predictive model and statistical tests were used to determine expected accident reductions related to various geometric improvements. Factors found to be most related to reduced accidents were wider lanes and shoulders, improved roadside conditions and flatter sideslopes. Paved shoulders were found to have a marginal safety benefit compared to unpaved shoulders.

Detailed accident analyses also were conducted for roadside features. Factors associated with increased fixed object accidents include higher traffic volumes, greater numbers of roadside objects, and closer distance of roadside objects to the road. Roadside objects associated with high accident severities include culverts, trees, utililty and light poles, bridges, rocks, and earth embankments. Construction cost data from several States were used to develop a cost model for numerous types of roadway and roadside projects.

Limited copies of this report are available from the RD&T Report Center. The report also may be purchased from the NTIS: (PB No. 88–156682/AS, Price code: A10).

Automatic Updating of Traffic Volume Data for Signal Timing Plan Development, Report Nos. Vol. I, FHWA/RD–87/081, and Vol. II, FHWA/RD–87/112

by Traffic Systems Division

A study was undertaken to test the validity of developing new signal timing plans using the TRANSYT–7F program with traffic volume data synthesized from a limited number of detectors that are part of a computerized signal system. This method is called "one and one-half generation signal control" (1.5 GC). Under the 1.5 GC approach, the performance of existing timing plans can be monitored and new timing plans developed without extensive collection of new data in the field.

The study indicates that if no manual data is collected for 2 years, traffic performance will be degraded by only 17 to 20 percent during peak hours and 1 to 28 percent during off-peak hours.

These reports may be purchased from the NTIS: (PB No. 87–229431/AS, Price code: A04; PB No. 87–229548/AS, Price code: A12).

Impact Attenuators - A Current Engineering Evaluation, Final Report, No. FHWA/RD–86/054 and Executive Summary, Report No. FHWA/RD–86/055

by Safety Design Division

This final report describes the impact performance of inertial barrels and GREAT impact attenuator systems using full-scale crash testing with small and large test vehicles. Results of the program showed small-car performance to be generally acceptable when using NCHRP 230 and dummy analysis procedures. The large car produced higher decelerations and in some cases the values exceeded the limits specified. The manufacturer has corrected this problem.

The reports may be purchased from the NTIS.

The Performance of Pile Driving Systems: Inspection Manual, Report No. FHWA/RD–86/160

by Materials Division

This report presents the results of a comprehensive investigation of the performance of pile driving systems. It is an inspection manual for impact hammers and includes pile cushions, helmets, and leads. This report will be of interest to bridge engineers and geotechnical engineers concerned with the inspection of pile driving operations.

Limited copies of this report are available from the RD&T Report Center.

Improving the Dynamic Performance of Multitrailer Vehicles: A Study of Innovative Dollies, Vol. I: Technical Report, No. FHWA/RD–86/161; Vol. II: Appendixes, Report No. FHWA/RD–86/162

by Safety Design Division

This report describes coupling mechanisms for multitrailer combinations which improve their dynamic performance.

The research identified, analyzed, and developed a dolly and trailer hitching hardware that demonstrated reduced rearward amplification (a crack-the-whip type of phenomenon) and eliminated the tendency for second trailer rollover. Both full-scale vehicle tests and computer simulations were conducted to analyze the performance of various dolly types in terms of rearward amplification, roll stablity of the second trailer, and vehicle offtracking.

The results showed that innovative dolly arrangements can provide significant improvements in the dynamic performance of multitrailer combination vehicles. The dolly, developed during this study, uses dual drawbar hitching hardware and allows the tires of the dolly to steer relative to the dolly frame in a controlled manner. The use of such dollies will provide significant safety benefits through the reduction of second trailer rollover accidents.

The reports may be purchased from the NTIS: (Vol. I: PB No. 87–194023, Price code: A11; Vol. II, PB No. 87–194031, Price code: A04).

Railroad-Highway Grade Crossing Signal Visibility Improvement Program, Vol. I: Final Report No. FHWA/RD–86/186, Vol. II: Data and Hardware Details, Report No. FHWA/RD–86/187, and Vol. III: Hardware Users Guide, Report No. FHWA/RD–86/188

by Traffic Systems Division

Studies have shown that a large portion of the signals at public crossings do not meet industry visibility specifications and that a large percentage of drivers involved in accidents at protected crossings report not having noticed warning signals.

After assessing driver visibility requirements and incorporating them into a recommended set of visibility specifications, this investigation pursued two approaches to improving signal visibility: Modifications to current signal design, and the development of practical tools useful to the maintenance personnel and signal manufacturers.

The final report, Vol. I, documents the methodology, development, and findings of the study and serves as an informational document. Vol. II provides data and hardware details. The users guide, Vol. III, is a stand-alone handbook for potential users of the project's developments. This document contains details on modifying railroad signals and the development and use of maintenance tools to improve signal visibility. Details are provided on the following outputs:

- Signal alignment tool.
- Signal focusing tool.
- Signal flux meter.
- Tripod lamp mounting bracket.
- Integral lamp/reflector assembly.
- Roundel retainer clip.

Limited copies of volumes I and III are available from the RD&T Report Center. These reports may also be purchased from the NTIS: (Final Report, Vol. I, PB No. 88–130216/AS, Price code: A05; Data and Hardware Details, Vol. II, PB No. 88–130224/AS, Price code: A05; and Hardware Users Guide, Vol. III, PB No. 88–130232/AS, Price code: A05).

ROADSIM Software

by Traffic Safety Research Division

ROADSIM, the two-lane traffic simulation model described in the December 1985 (Vol. 49, No. 3) issue of *Public Roads,* is now available in an IBM PC compatible microcomputer version.

ROADSIM can be used to determine the traffic operational effects of geometric changes and volume changes on two-lane rural highways at a microscopic level. FHWA would like to receive research findings, reports, and recommendations from users of the ROADSIM models, especially the microcomputer version. If you are interested in trying this model on a real-case application, please send two formatted 5 1/4-in diskettes to Juan M. Morales, HSR–30, Traffic Safety Research Division, Federal Highway Administration, 6300 Georgetown Pike, McLean, Virginia 22101–2296. Telephone: (703) 285–2499.

Implementation/User Items "how-to-do-it"

The following are brief descriptions of selected items that have been completed recently by State and Federal highway units in cooperation with the Office of Implementation, Offices of Research, Development, and Technology (RD&T), Federal Highway Administration. Some items by others are included when the items are of special interest to highway agencies. All reports are available from the National Technical Information Service (NTIS).

When ordering from the National Technical Information Service, use PB number and/or the report number with the report title, and address requests to

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Improved Methods and Equipment to Conduct Pavement Distress Surveys, Report No. FHWA–TS–87–213

by Office of Implementation

This report documents the results of an evaluation of selected pavement distress survey methods and devices. Included in the evaluation were a manual mapping method, detailed visual surveys using manual recording and automatic data logging, the PASCO Roadrecon survey vehicle, the GERPHO device, the ARAN survey vehicle, and the Laser RST device. Each method and device was field tested on several flexible, rigid, and composite pavement sections exhibiting a wide range of distress. Evaluations were based on observations during the field testing and analysis of the collected data.

This report will be of interest to those individuals involved with pavement evaluation procedures and equipment.

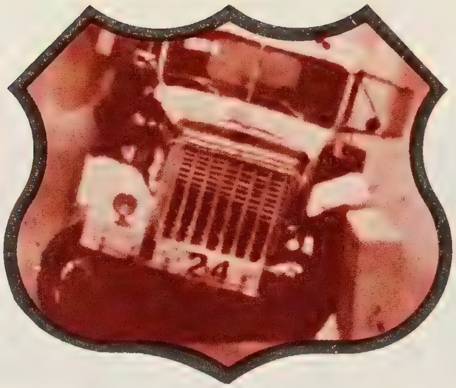
Limited copies of this report are available from the RD&T Report Center. The report also may be purchased from the NTIS: (PB No. 88–119730/AS, Price code: A12).

Guide to Currently Available Pavement Marking Equipment, Report No. FHWA–TS–87–219

by Office of Implementation

This report reviews current pavement marking machinery and materials based on information provided by both users and manufacturers. Machinery is divided into three categories: Walk-behind models, driver-operated models, and large truck-mounted units. Examples of each type of pavement marker are provided and specifications and performance criteria are discussed. Marking materials covered in this report are limited to traffic paints, thermoplastic, epoflex, polyester, and epoxy-based materials.

The report may be purchased from the NTIS: (PB No. 88–120571/AS, Price code: A03).



Application of Mechanics to Roadside Safety, Report No. FHWA–TS–87–228

by Office of Implementation

This report describes the concepts of mechanics—system of units, Newton's second law, rigid body mechanics, friction, momentum, and energy and work—as they relate to roadside safety design. Following the review of the important concepts of mechanics, the report addresses vehicle characteristics, human injury criteria, and details of highway safety hardware design.

The report is written for readers who wish to increase their knowledge of highway appurtenances and their functions. It will be valuable to highway engineers and researchers who are involved with highway safety.

The report may be purchased from the NTIS: (PB No. 88–132980/AS, Price code: A08).

Enhanced Urban Traffic Control System Software

by Office of Implementation

The Federal Highway Administration's (FHWA) long involvement with the development and promotion of the Urban Traffic Control System (UTCS) has successfully been completed.

The UTCS software was initially developed and installed in Washington, D.C. in 1972. The software was translated to FORTRAN in 1973 to facilitate installation on other computers in other cities. A modified version of the FORTRAN software that was installed in Charlotte, North Carolina in early 1978 is known as UTCS-Extended and it, or various software packages based on it, have been installed in a number of U.S. cities.

The FHWA incorporated the proven functions of the previous versions of UTCS with the functions requested by end users and system firms (manufacturers and consultants) into this current release known as the Enhanced UTCS software.

The following reports provide a complete description of the Enhanced UTCS software.

Functional Description, Report No. FHWA–TS–79–228, (PB No. 82–150871, Price code: A04).
Applications Manual, Report No. FHWA–IP–87–10, (PB No. 88–160122, Price code: A05).
Data Base Specification, Report No. FHWA–IP–87–11, (PB No. 88–160130, Price code: A10).
Operator's Manual, Report No. FHWA–IP–87–12, (PB No. 88–160148, Price code: A08).
System Software Specification — Volume I, Report No. FHWA–IP–87–13, (PB No. 88–160155, Price code: A22).
System Software Specification — Volume II, Report No. FHWA–IP–87–14, (PB No. 88–160163, Price code: A99).
System Software Specification — Volume III, Report No. FHWA–IP–87–15, (PB No. 88–160171, Price code: A17).

These reports may be purchased individually from the NTIS or as a set (PB No. 88–160114, Price code: E99).

Copies of the Enhanced UTCS software on two magnetic tapes are available on loan from the Systems and Software Support Branch, Office of Traffic Operations, HTO–23, Federal Highway Administration, Washington, DC, 20590.

Caution—Litigation Ahead: The Road to Effective Risk Management

by Office of Implementation

The purpose of this 25-minute videotape presentation is to motivate State and local highway officials to implement effective risk management programs to improve highway safety and mitigate the risk of tort liabililty.

Highway tort litigation is a growing problem; however, there are mitigating techniques for highway agencies. "An ounce of prevention is worth a ton of litigation," so says the host of a new videotape presentation on highway risk management. The focus of the videotape presentation is not on the problem; rather, it emphasizes and defines a coordinated risk management approach to mitigate the problem. Although there is no guarantee of avoiding litigation, agencies can avoid routine, and even unexpected highway safety deficiencies through good management and a strong commitment to highway safety. This includes a program for effective design, construction, maintenance and operation of all highway facilities.

The videotape presentation is aimed toward State and local highway management officials who are concerned with tort liability problems.

This videotape is available on loan from the RD&T Report Center.

New Research in Progress

The following new research studies reported by FHWA's Office of Research, Development, and Technology are sponsored in whole or in part with Federal highway funds. For further details on a particular study, please note the kind of study at the end of each description and contact the following: Staff and administrative contract research—*Public Roads* magazine; Highway Planning and Research (HP&R)—performing State highway or transportation department; National Cooperative Highway Research Program (NCHRP)—Program Director, National Cooperative Highway Research Program, Transportation Research Board, 2101 Constitution Avenue, NW, Washington, DC 20418.

NCP Category A—Highway Safety

NCP Program A.1: Traffic Control for Safety

Title: Effective Utilization of Street Width. (NCP No. 5A1A0092)
Objective: Determine the relationship between capacity and safety for various lane widths and allocations of given street segments and intersections. Consider velocity/capacity ratio/speed, vehicle type, volume, alignment, roadside development, street classification, and environmental effects.
Performing Organization: Midwest Research Institute, Kansas City, MO 64110
Expected Completion Date: April 1990
Estimated Cost: $160,000 (NCHRP)

NCP Category C—Pavements

NCP Program C.1: Evaluation of Rigid Pavements

Title: Concrete Pavement Design and Rehabilitation. (NCP No. 4C1A2122)
Objective: Evaluate the existing Portland cement concrete pavements in the Phoenix area. Develop rehabilitation strategies and prepare a workplan recommending a concrete pavement management system and a detailed workplan for establishing a functional system.
Performing Organization: ERES Consultants, Inc., Champaign, IL 61820
Funding Agency: Arizona Department of Transportation
Expected Completion Date: March 1989
Estimated Cost: $192,044 (HP&R)

NCP Program C.2: Evaluation of Flexible Pavements

Title: Subgrade Resilient Modulus Evaluation. (NCP No. 4C2B1162)
Objective: Select samples of soil types submitted from 34 field test sites to be lab tested at various moisture and density levels. Make falling weight deflectometer measurements and back compute in situ moduli. Study correlation between in-place and lab remolded modulus.
Performing Organization: Georgia Department of Transportation, Atlanta, GA 30334
Expected Completion Date: April 1990.
Estimated Cost: $136,000 (HP&R)

NCP Program C.4: Management Strategies

Title: Response of Flexible Pavement to Heavy Trucks with High Tire Pressures. (NCP No. 4C4A1092)
Objective: Measure the response of a flexible pavement to a range of loads and tire pressures for bias ply, radial, and super-single tires to include surface deflection and surface strain. Conduct a field survey to determine the distribution of tire types and inflation pressures used by trucks in California.
Performing Organization: Caltrans/Translab, Sacramento, CA 95819
Funding Agency: California Department of Transportation
Expected Completion Date: August 1990
Estimated Cost: $110,000 (HP&R)

NCP Category D—Structures

NCP Program D.1: Design

Title: Pier Wall Ductility. (NCP No. 4D1A2102)
Objective: Study the force-deflection properties of wide, thin R/C pier columns; the effects of failure mode and deformation of wide pier columns due to varying vertical, horizontal, and cross-tie reinforcement, development of design methods and rebar details, and computer models for weak axis bending of such walls. Perform testing of a 15-ft high column.
Performing Organization: University of California, Los Angeles, CA 90024
Funding Agency: California Department of Transportation
Expected Completion Date: November 1990
Estimated Cost: $318,000 (HP&R)

Title: Full-scale, Medium-level Vibration Test at Meloland Road Bridge. (NCP No. 4D1A2112)
Objective: Measure the translation and rotation of bridge abutments and pier footing. Develop a structure/foundation model for dynamic vertical and seismic transverse loads. Evaluate the bridge's performance and assess adequacy of Caltrans' design criteria.
Performing Organization: University of Nevada, Reno, NV 89507
Funding Agency: California Department of Transportation
Expected Completion Date: June 1991
Estimated Cost: $374,000 (HP&R)

Title: Seismic Response of Tieback Walls. (NCP No. 4D1A22142)
Objective: Model and analyze the interaction between soil and tieback walls under seismic loading, using the finite element computer program Flush. Design an instrumentation package for an existing wall and investigate the wall with Flush. Evaluate existing designs, identify conditions for which they may not be safe, and recommend changes in the Washington State Department of Transportation design procedures.
Performing Organization: Washington State University, Pullman, WA 99163
Funding Agency: Washington State Department of Transportation
Expected Completion Date: June 1989
Estimated Cost: $85,000 (HP&R)
(The above information was incorrectly stated in the December 1987 issue of *Public Roads* magazine.)

Title: Evaluation of New Approach Slab Concept. (NCP No. 4D1B3122)
Objective: Evaluate California's concrete approach slab concept and two variations to determine if performance is as intended. Identify problem areas and cost reduction changes which can be made without affecting performance.
Performing Organization: Caltrans, Sacramento, CA 95807
Funding Agency: California Department of Transportation
Expected Completion Date: November 1993
Estimated Cost: $125,000 (HP&R)

NCP Category E—Materials and Operations

NCP Program E.2: Cement and Concrete

Title: Use of Chemical Admixtures for Highway Concrete. (NCP No. 4E2A1102)
Objective: Review, evaluate, and interpret current state-of-the-art practices with respect to the use of chemical admixtures in ordinary concrete. Prepare a manual for current types of admixtures, benefits to be expected from proper utilization, estimated costs, and cautions concerning proper use and possible problems. Conduct basic research in the area of the functioning of admixtures in concrete.
Performing Organization: Purdue University, Lafayette, IN 47907
Funding Agency: Indiana Department of Highways
Expected Completion Date: July 1991
Estimated Cost: $112,150 (HP&R)

NCP Program E.3: Geotechnology

Title: Computer Program for Analysis of Embankments with Tensile Elements. (NCP No. 4E3B0452)
Objective: Develop a generalized, tensile-reinforced computer model by adapting an existing, newly developed slope stability program. Formulate the model to solve problems involving multilayered reinforced slopes, overall bearing capacity, circular failures, and non-circular failures, such as sliding wedges.
Performing Organization: Kentucky Transportation Cabinet, Frankfort, KY 40622
Expected Completion Date: January 1991
Estimated Cost: $110,000 (HP&R)

Title: Geotechnical Engineering Data. (NCP No. 4E3B0462)
Objective: Develop a general overview of the engineering properties and characteristics of Kentucky soils. Data will define the scope of potential construction problems that may occur when Kentucky soils are used in highway embankments and pavement subgrades.
Performing Organization: Kentucky Transportation Cabinet, Frankfort, KY 40622
Expected Completion Date: December 1990
Estimated Cost: $97,820 (HP&R)

Title: Stresses and Deformations in Earth Reinforcement Systems. (NCP No. 4E3B0472)
Objective: Analyze the stresses, deformation, and deflections, both predicted and observed, in reinforced soil walls built by Caltrans. Emphasize structures containing silty and clayey backfills, as few have been built to date.
Performing Organization: Caltrans, Sacramento, CA 95807
Funding Agency: California Department of Transportation
Expected Completion Date: June 1989
Estimated Cost: $106,000 (HP&R)

Title: Load Factor Design Criteria for Highway Structure Foundations. (NCP No. 5E3A0352)
Objective: Develop a framework for applying load factor design to the design of highway structure foundations. Define values for all load and resistance factors including soil-structure interaction and time-dependent behavior. The recommendations will be put in AASHTO format, calibrated against working stress design, and studied for its effect on other parts of the AASHTO specifications.
Performing Organization: Virginia Polytechnic Institute, Blacksburg, VA 24061
Expected Completion Date: December 1989
Estimated Cost: $375,000 (NCHRP)

Title: Recommend Specifications for the Design of Foundations, Retaining Walls, and Substructures. (NCP No. 5E3A0462)
Objective: Develop recommendations for revising sections 4, 5, and 7 of the AASHTO standard specifications for highway bridges. Propose guidelines which reflect the current practice and state of the art in geotechnical engineering and substructure design. Address the design of conventional and alternative retaining structures and foundation elements including, but not limited to, spread footings, piles and drilled shafts.
Performing Organization: STS Consultants, Ltd., Northbrook, IL 60062
Expected Completion Date: June 1989
Estimated Cost: $100,000 (NCHRP)

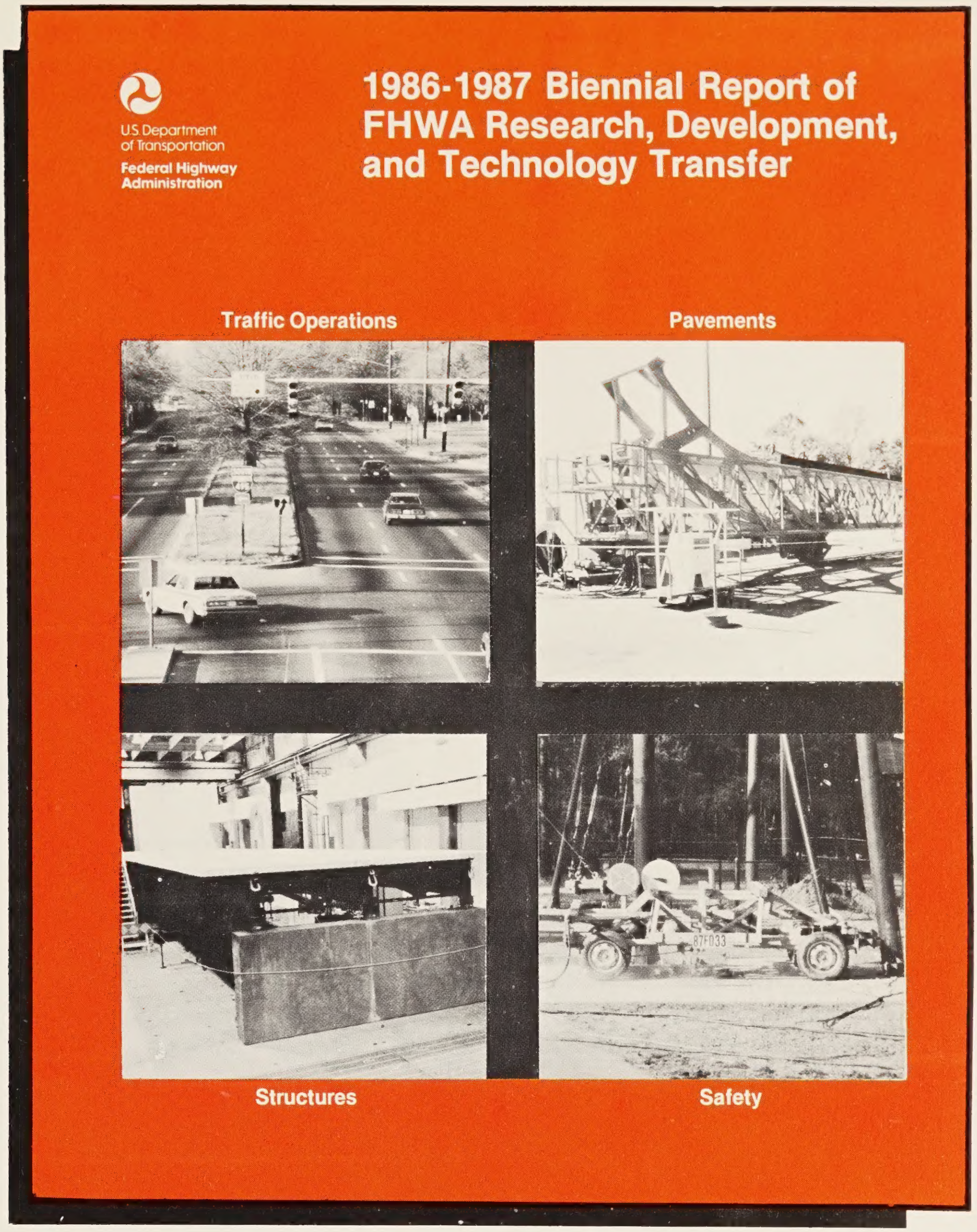
NCP Program E.5: Maintenance Effectiveness

Title: Field Evaluation of Moisture Sensing Devices. (NCP No. 4E5D2122)
Objective: Modify five moisture sensing devices into five automatic irrigation systems in the Sacramento, California area. Investigate watering schedules of irrigation controllers, varying frequency and length of each watering cycle to provide adequate moisture but not cause runoff or result in other undesirable events to occur. Develop general guidelines and implement via video cassettes.
Performing Organization: Caltrans, Sacramento, CA 95807
Funding Agency: California Department of Transportation
Expected Completion Date: September 1989
Estimated Cost: $128,481 (HP&R)

NCP Program E.8: Construction Control and Management

Title: Nuclear Method of Determination of Asphalt Content Corrected for Moisture in Bituminous Mixture. (NCP No. 4E8A6092)
Objective: Assess the suitability of nuclear asphalt content gauges. Develop a method which adequately compensates gauge results for the presence of moisture in bituminous mixtures.
Performing Organization: Pennsylvania Transportation Institute, University Park, PA 16802
Funding Agency: Pennsylvania Department of Transportation
Expected Completion Date: March 1989
Estimated Cost: $74,996 (HP&R)

☆U.S. Government Printing Office: 1988—202-880/80003



New Publication

The Federal Highway Administration has released its **1986–1987 Report of FHWA Research, Development, and Technology Transfer.** This report is a continuation of the series of annual and biennial reports published for fiscal years 1974 through 1985. This biennial report has been revised to reflect an expanded scope; the report now includes highway research, development, and technology transfer activities and accomplishments for all FHWA offices, as well as activities of the Strategic Highway Research Program (SHRP).

The report briefly discusses the scope of the Nationally Coordinated Program (NCP) of Highway Research, Development, and Technology and highlights technical accomplishments in the areas of policy and statistical analysis; pavement management; structural design and hydraulics; highway operations and maintenance; highway design and operation for safety; traffic control and management; and motor carrier safety. Also described are other activities of the Office of Research, Development, and Technology, funding and facilities for conducting FHWA's RD&T programs, and an update of the progress in implementing the SHRP. The report concludes with a listing of the technical reports published in 1986 and 1987, which are available through the National Technical Information Service.

While supplies last, individual copies of the report are available without charge from the Federal Highway Administration, RD&T Report Center, HNR–11, 6300 Georgetown Pike, McLean, Virginia 22101–2296. (Telephone: (703) 285–2144.)

U.S. Department
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**Federal Highway
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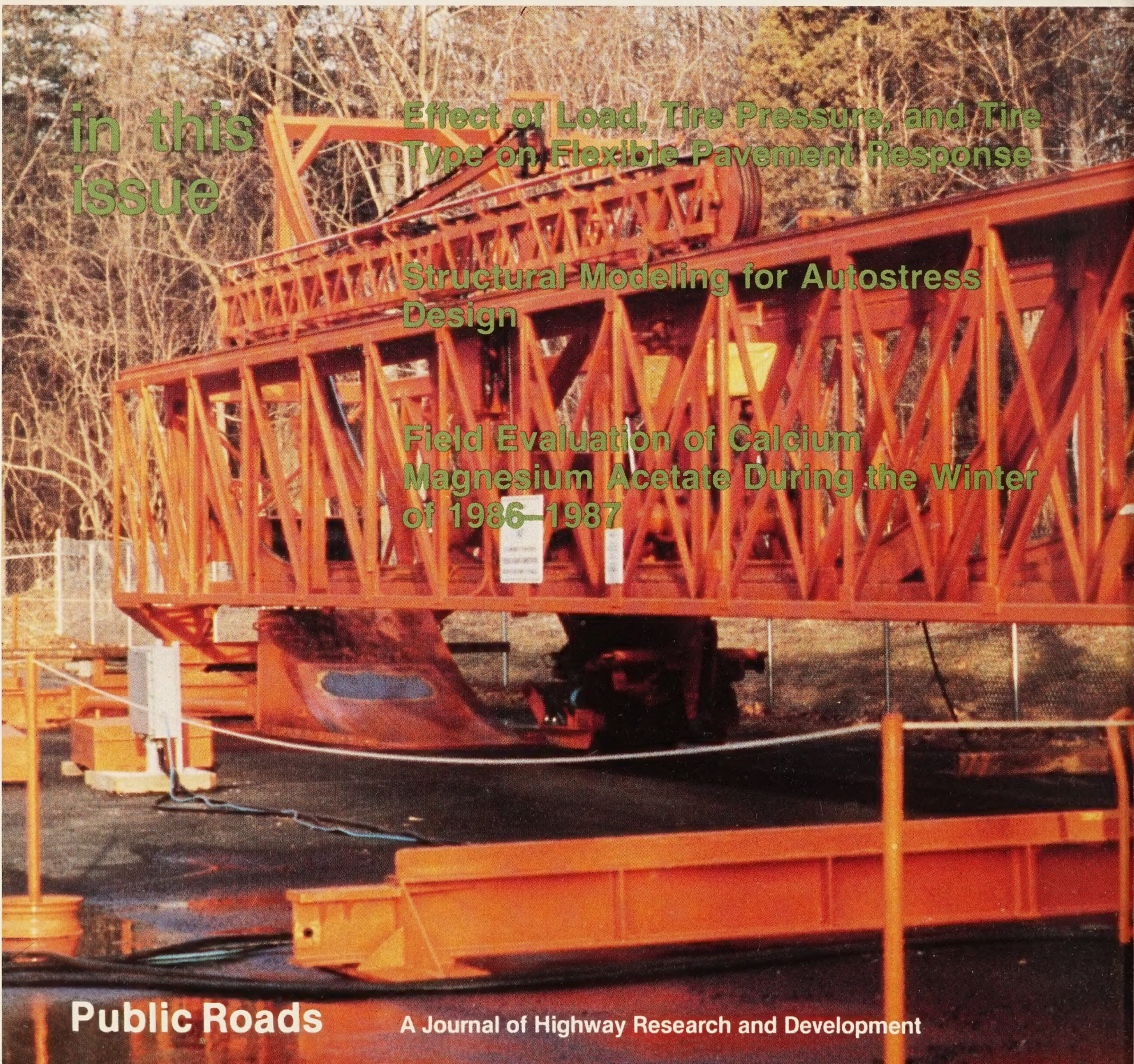

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