Mechanistic-Empirical Rigid Pavement Design for New York State

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In 1993 New York published a new *Thickness Design Manual for New and Reconstructed Pavements* based on the 1986 AASHTO design guide. The AASHTO equation for rigid pavement performance was calibrated with performance data for 225-mm rigid pavements in New York, and the calibrated equation was then used to design rigid pavements. Because New York does not have experience with thicknesses greater than 225 mm, the modified AASHTO equation could not be verified for thicker pavements. The development of a mechanistic-empirical (M-E) design procedure for verifying the designs presented in the new thickness manual is described. First, a nondimensional fatigue model was established on the basis of New York's past pavement performance, environmental conditions, and traffic loadings. The study was then extended to develop design curves for thicknesses of 225, 250, 275, 300, and 325 mm (5-m slab lengths for 225- to 275-mm thicknesses and 5.5-m slab lengths for 300- to 325-mm thicknesses). Finally, the M-E design curve was compared with the modified AASHTO equation. The results indicate that for thicknesses greater than 275 mm, AASHTO predicts up to 40 percent more equivalent single axle loads than the M-E approach.

In New York State an empirical procedure has been used since early in the 20th century to design highway pavements. It was formalized about two decades ago when it was incorporated into the state's *Highway Design Manual* (*1*). This procedure had evolved from New York's past experience, but was not based on pavement performance data from specific projects and was last revised in 1979. It provides a simplified pavement-thickness selection system based on highway classification, design hourly traffic, and two truck weight categories (heavy or light). It does not consider such other variables as reliability, drainage, soil and material properties, joint load transfer, or failure criteria. The cross section required in designing a rigid pavement carrying truck traffic is 225-mm portland cement concrete over a minimum 300-mm granular subbase course, independent of the number of load repetitions or the magnitude and configuration of the axle loads.

With the 1972 publication of the AASHO *Interim Guide for Design of Pavement Structures* (*2*), the New York State Department of Transportation (NYSDOT) recognized an opportunity to adopt a more rational rigid pavement design procedure. In-house evaluation of the 1972 guide was considered but was set aside without affecting the state's design procedure because of the lack of agreement with New York's experience. In 1986 AASHTO finally published its comprehensive *Guide for Design of Pavement Structures* (*3*). FHWA urged the states to review and adopt all or portions of it. In 1987 the Engineering Research and Development Bureau initiated Research Project 202 (Relating the AASHTO Guide to New York State Procedures) to evaluate the guide, anticipating that this could be a major undertaking. Then in 1989 FHWA issued a pavement policy mandating that each state develop a process for pavement design and type selection that was acceptable to FHWA. Although FHWA would recognize design procedures based on local experience, the AASHTO guide procedure was cited as an example that it would accept. In 1989 the NYSDOT Pavement Management Steering Committee recommended that appropriate portions of the 1986 guide be adopted as a framework for design selections.

In March 1992 the NYSDOT Technical Services Division organized a task force to prepare a departmental pavement design procedure for new and reconstructed pavements by January 1993 on the basis of the 1986 AASHTO guide. In their work the findings of Research Project 202 were to provide a basis for implementation. The AASHTO rigid pavement performance equation was calibrated with performance data for 225-mm New York rigid pavement, and the calibrated equation was then used to predict the number of load repetitions to failure in the state's *Thickness Design Manual for New and Reconstructed Pavements* (*4*). Because New York did not have actual performance data for thicknesses other than 225 mm, performance predicted with the modified AASHTO equation could not be verified for greater thicknesses. To verify that the manual was reasonable, an independent mechanistic-empirical (M-E) fatigue analysis was conducted, relating a nondimensional parameter (stress ratio σ_{cr}/M_r, where σ_{cr} is the critical stress under center slab loading near the free edge and a daytime gradient, and M_r is the concrete modulus of rupture) to load applications to failure. For New York this study and adoption of the new *Thickness Design Manual for New and Reconstructed Pavements* are the first steps in incorporating M-E structural analysis into future design procedures.

EMPIRICAL DESIGN PROCEDURES

Historically, the most important full-scale experimental design study in the United States was the AASHO Road Test in the 1950s at Ottawa, Ill. The 1986 AASHTO guide is based on in-service performance of the Road Test pavements and takes into account numerous design and behavioral factors under test loads, such as soil conditions, material properties, load transfer devices, and axle loading and configuration. The AASHTO approach is empirical and is based on the ability of the pavement to serve traffic over the design period—that is, its functional performance. This is measured by the present serviceability index (PSI), which is a numerical value ranging from 5 (representing the best possible pavement) to 0 (representing no pavement at all). Test data on pavement serviceability were obtained in the Road Test, and for each test section a serviceability-time history was plotted from the start of traffic operation until its PSI dropped to a minimum

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tolerance level—typically 2.5. A decrease in PSI over the serviceability-time history was then used to develop performance equations for flexible and rigid pavements.

In the AASHTO guide the performance equations were derived for dual-tire single and tandem axle trucks, with a maximum single axle load of 133.5 kN and maximum tandem axle load of 214 kN, for about 10 million 80-kN equivalent single axle loads (ESALs) under the environmental conditions prevailing only at the AASHO site. These equations thus were limited to conditions at Ottawa, Ill., and to those axle configurations and load ranges. As a result of trends in the trucking industry toward heavier gross vehicle weights, pavement problems related to multiple wheel loads have developed. Higher axle loads and varying axle spacings were not considered in the AASHO Road Test—thus, performance for heavier and different axle configurations must be extrapolated from the Road Test performance data outside the boundaries for which the equations were developed. Moreover, the trucking industry has adopted various axle spacings to meet the limitations of vehicle size and weight subsequently imposed by legislative action. These new configurations allow increased load-carrying capacity without violating the law, but they introduce complications in fatigue loading that were not considered in developing the AASHTO guide.

Obviously, problems arise when the functional method is used in predicting pavement performance without experimental data to support its use. To update the AASHTO guide to current conditions, various approaches have been suggested. One is to develop a test track similar to that at Ottawa in the 1950s to test current configurations (loads and numbers of adjacent axles). However, prototype pavement studies involving new axle load configurations would be time consuming and very costly. An alternative approach used nationally for more than 30 years has been reevaluation of AASHO Road Test data in conjunction with information obtained from state satellite tests. Finally, theoretical solutions have also been used; these are based on mechanistic parameters calibrated with pavement fatigue distress.

MECHANISTIC-EMPIRICAL DESIGN PROCEDURES

Pavements are very complicated structures, with performance depending on the interaction of such factors as climate, traffic conditions, support conditions, physical dimensions, and material properties. Developing a purely analytical prediction model thus is also extremely complex. To overcome this difficulty numerous simplifications must be made to predict long-term pavement performance. An M-E procedure is the solution most commonly used. M-E analysis and design methods evolved in the 1970s on the basis of known relationships between material behavior under stressed conditions (stress, strain, deflection) and the corresponding empirical performance of the pavement structure for all important combinations of loading and environmental conditions. The M-E approach is based on the hypothesis that pavement distress is related to states of strain or deflection. Performance models thus can be developed from theoretical computations of these pavement response parameters, calibrated with serviceability-time histories of pavement structures. The M-E approach uses the principles of engineering mechanics to calculate pavement responses and relate them to rates of deterioration. Then, Miner's hypothesis is used to sum the damage caused by traffic and environmental loading.

For rigid pavements two models are used to predict fatigue life: fatigue cracking and faulting. For the fatigue cracking model, the M-E procedure uses critical stresses at the bottom of the concrete slab to estimate total wheel load applications before cracks begin, propagate, and ultimately fracture the pavement. For the faulting model the performance evaluation relates faulting rate to a mechanistic parameter—maximum concrete bearing stress. Fatigue life is the number of stress repetitions, at a magnitude less than its strength, required for the pavement to fail structurally for a given failure criterion. In addition to fatigue cracking, faulting is also considered in determining fatigue life. This is considered because (as shown in NCHRP Project 1-26) faulting is a function of loadings and concrete bearing stress, and thus can be considered a mode of structural failure. New York's new design does not expect any failures because of faulting for the following reasons: (*a*) provisions to ensure a well-drained base (the stabilized base has been shown to provide drainage of 915 m/day), (*b*) sufficient load transfer devices, and (*c*) the added support of full-depth concrete shoulders and widened outer lanes.

NONDIMENSIONAL PERFORMANCE MODEL FOR NEW YORK STATE RIGID PAVEMENTS

The nondimensional performance model for New York State rigid pavements was developed by using pavement response to the combined effects of temperature and traffic loading, calibrated with actual pavement performance and 90 percent reliability. Miner's hypothesis was used to sum the damage caused by traffic and environmental loading. Pavement response (that is, critical stresses) was determined with actual physical dimensions, material properties, soil support, temperature gradients, and traffic from New York pavements.

Past Performance of New York Rigid Pavements

NYSDOT periodically collects pavement condition data for all sections of its highway network. These data consist of sufficiency ratings and dominant distress features that together are used to classify uniform highway sections. The rating scale is 1 to 10—1 for a pavement having a very high distress frequency and severity and 10 for a pavement with no distress. To compare the AASHTO design to New York's design, it was necessary to approximate the PSI by dividing the New York sufficiency rating by 2.

In the present study the actual fatigue life of rigid pavements in New York was assumed to be the number of cumulative 80-kN ESAL applications from the time that the pavement was opened to traffic to the time that it reached a sufficiency rating of 5. This was done by examining the state's *Highway Sufficiency Ratings* (*5*) to determine the cumulative 80-kN ESALs at a sufficiency rating of 5 or when the pavements were overlaid. The next step was to examine contract plans to determine when the pavements were placed and the details of their cross sections. The pavements considered had a 225-mm concrete layer on a 300-mm granular subbase. The failure criterion used in selecting these pavements was fatigue cracking; pavements that failed because of faulting or D-cracking were not included, both of these problems having been previously considered in New York and the suggested solutions implemented.

Next, pavement condition surveys were examined to determine the sufficiency rating at the time of overlay for each pavement that had been overlaid before reaching a sufficiency rating of 5. The surveys showed that about 60 percent of the pavements were overlaid at a sufficiency rating of 6. The next step was to calculate the additional traffic that the pavement would have carried to reduce the sufficiency rating from 6 to 5. This additional traffic was estimated for the average time that a rigid pavement took to go from a condition rating of 6 to one of 5. It was found that an average of 3 years or about 3 million ESALs are required to produce a drop of 1 in the sufficiency rating. The additional time and traffic were then added to each of the performance curves. Because all the data were for pavements with similar cross sections and the number of load applications to failure ranged from 21 million to 54 million ESALs, a representative fatigue life had to be chosen.

In the present study reliability was defined as the probability that the service rating would remain above 5 during the design life. It was decided that the number of load applications to failure would be based on a 90 percent design reliability. The first attempt to determine this life was to try to fit the data to a normal distribution, but it was found that the normal distribution did not fit well, and a distribution-free confidence interval for percentiles (*6*) was used instead. By this method it was found that 24.4 million ESALs was the fatigue life giving the desired reliability. In Figure 1 the actual number of ESALs on rigid pavements in New York is shown and their performance is compared with 90 percent reliability to the AASHTO rigid pavement prediction equation with 50 and 90 percent reliability. It also shows that the AASHTO prediction equation with 50 percent reliability correlates well with performance at 90 percent reliability.

Pavement Response

The Illi-Con program (*7*) was used to analyze the combined effects of temperature (curling stresses) and traffic loading. This is a data base program based on output from the Illi-Slab finite-element program (*8*). Critical stresses found by using Illi-Con were checked with those from Illi-Slab to ensure equivalence. The comparison showed that they were about equal (within ± 4 percent), which confirmed that Illi-Con was adequate to calculate the stresses resulting from thermal and traffic loads for various slab thicknesses. Illi-Con was used instead of Illi-Slab because of its ease of data input and the speed with which it generated results. Input parameters to determine critical stresses in New York rigid pavements were physical properties, temperature gradients, and traffic.

Physical Properties

The previous New York State pavement design consisted of a single cross section for interstate pavements with the following design values: dowel bars were solid 28-mm-diameter steel bars with a modulus of elasticity of 200 GPa and spacing beginning 150 mm from the pavement edge and then every 300 mm on center across the transverse joint. The rigid pavement had a modulus of elasticity of 25 GPa, a thickness of 225 mm, a coefficient of thermal expansion of 1.0×10^{-5} (1/°C), a lane width of 3.6 m, and 3-m-wide asphalt shoulders.

Because of its high modulus and rigidity, rigid pavement behaves as a rigid body—that is, a wheel load is distributed over a relatively large area of soil. Rigid pavement flexural capacity is provided primarily by flexural strength of the concrete—its modulus of rupture (M_r). In New York concrete modulus of rupture has been estimated on the basis of the results of split-cylinder tensile testing on two experimental projects—in 1965 on Route 23 between Catskill and Cairo and in 1975 on I-88 near Otego (*9*). Testing on core centers was performed in accord with AASHTO Method T-198, and M_r was estimated by using Hammit's equation (*10*). The results indicated that the mean M_r is 5.2 MPa, with a standard deviation of 572 kPa and a 90 percent lower confidence limit of 4.25 MPa. Also, the NYSDOT Materials Bureau tested concrete beams for 28-day flexural strength and found that it was approximately equal to 15 percent of concrete compressive strength. For the present study M_r was estimated by using a factor of 15 percent of the compressive strength. For Class C concrete with a compressive strength of 27.58 MPa, a value of 4.14 MPa was used for M_r.

Temperature Gradients

Thermal gradients between the top and bottom on a rigid pavement slab are the cause of slab curling and also determine the direction and size of this curling. When temperatures at the top and bottom of a slab are equal, it is flat (level). A positive gradient (the top warmer than the bottom) causes the free edge to curl downward, and a negative gradient (the top cooler than the bottom) will cause an upward curl. The greater these gradients, the more the slabs curl. Large stresses are created when downward curl is restrained by the weight of the slab itself. As observed by the Department of Civil Engineering of the University of Illinois at Urbana-Champaign (*7*):

> Because the warping stresses occur early in the concrete aging cycle, and the stresses are long term, it follows that much of the warping stress is relieved by concrete creep. Thus the residual stress due to warping is highly speculative

For this reason warping stresses were not included in this study.

New York has been collecting temperature gradients on two roads (I-87 and Route 29) as part of Research Project 188 (Effect of Overlays on Faulted Concrete Pavements) (*11*). Temperatures were gathered to determine how a bituminous overlay affects stresses within an overlaid concrete slab (the test section) in comparison with the stresses and pressures within a concrete slab without an overlay (the control section). Data collected for the control section are discussed here. Temperatures were collected for about 2 years for periods of 2 to 4 days each month. Ambient (air) temperatures were also recorded by thermocouples situated on two 0.8-m wooden stakes and covered with small wooden squares to prevent exposure to direct sunlight.

Information from analyses of these data included temperature variations for various depths within the slab, temperature differences between various depths, and temperature differentials (gra-

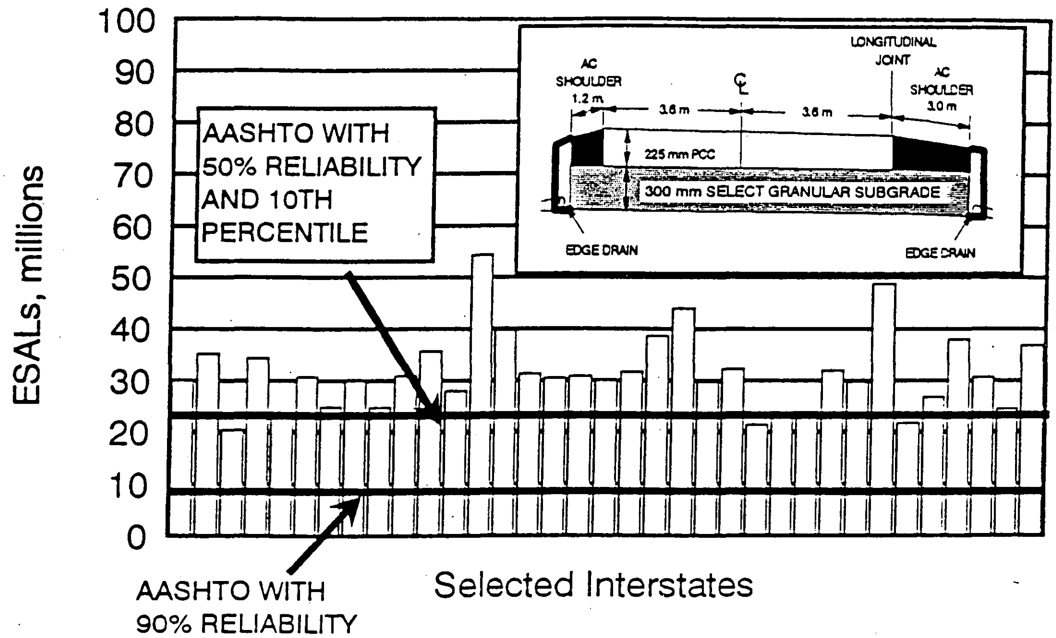

FIGURE 1 ESALs to failure (inset shows former cross-section).

dients) through the slabs. Thermal gradients were computed by subtracting the temperature reading at a 200-mm depth from that at the slab surface.

Shown in Figure 2 are typical seasonal variations of thermal gradients over a 24-hr period, indicating that positive gradients occur during daylight hours and negative gradients occur at night. Frequency of various gradients over an entire year are shown in Figure 3. That study (*11*) indicated that the greatest variation in thermal gradient occurs during hot months, when the daily air temperature also varies the most. It showed that the greatest thermal gradients also occurred during the summer months. For the combined effects of temperature and loading, the critical condition occurred with a daytime temperature gradient and design truck loading.

Critical tensile stresses were then taken from the output of the Illi-Con program (*7*) and analyzed. Input files for the program were also created for critical loading and temperature conditions. Other analyses were performed to determine which variables directly affected the critical stress. All analyses showed that daytime gradient was a determining factor for maximum tensile stress. This was expected, because the nighttime gradient would produce compressive stresses opposing the tensile stresses produced by the load.

Traffic

Vehicle weight and traffic profile information is necessary to calibrate the fatigue model accurately. To determine hourly traffic

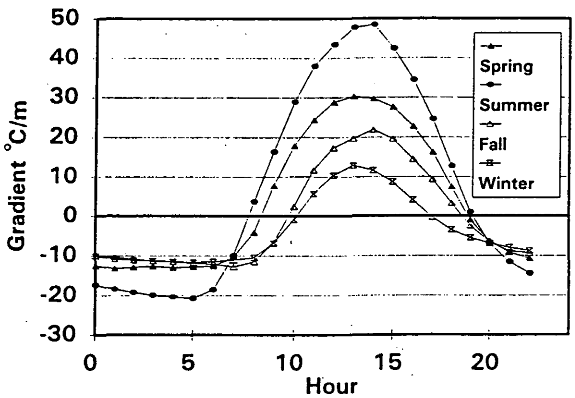

FIGURE 2 Thermal gradient versus time of day.

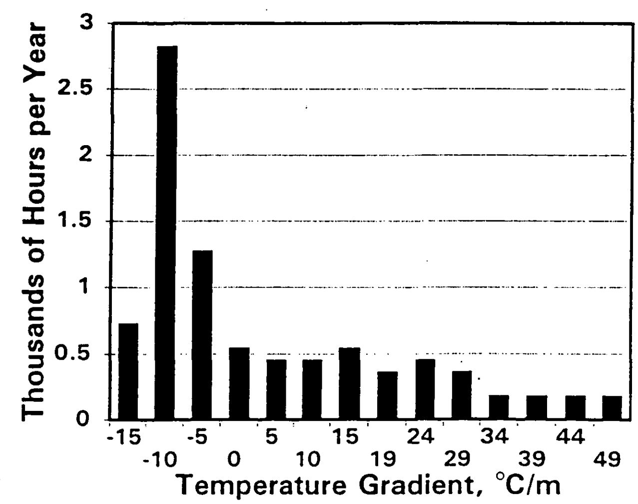

FIGURE 3 Frequency versus gradient.

profile, the percentage of truck traffic must be determined throughout the day. The values presented in Figure 4 are taken from *New York State Truck Weight and Vehicle Classification Reports* (*12*), converting the number of trucks each hour on a typical Interstate to the total percentage of trucks. The next step was to select what design trucks and axle loadings produced the greatest cumulative damage—that is, the highest combination of weights and load repetitions. It was found that the most numerous and heaviest trucks were 3S-2 tractor semitrailers (five-axle trucks). This was chosen as the design truck, whose loads would be converted to ESALs as a basis for traffic predictions. Figure 4 is a profile of the hourly percentage of traffic on a typical day for five-axle trucks. This profile was necessary because damage caused by a truck depends on both its weight and the thermal gradient at which the load is applied. These factors are combined later in this paper. The traffic data analyzed to determine the profile and weight measurements were based on more than 25,000 heavy vehicles.

It was necessary to develop a representative method to convert from load applications to ESALs for two reasons: (a) traffic predictions generally are given in ESALs, and (b) the fatigue model used is based on load applications, not ESALs. One pass of the design truck consists of a single 35-kN axle and two 160-kN tandem axles (a five-axle truck with 355-kN gross vehicle weight). Next, load equivalency factors (LEFs) were determined by using the tables provided in the 1986 AASHTO guide. LEFs were used only for the comparison of traffic between the M-E model and the AASHTO model. In addition, ESALs were the only available measure of performance for existing sections and had to be converted to design truck loading for the M-E model. In that model traffic was accounted for by the number of passes of the design truck, not ESALs. The design truck was found to account for 80 percent of all traffic, and only 5 percent of the trucks were heavier (most of those were trucks with more than five axles). After the LEFs were found, the total number of ESALs for the design truck was determined by multiplying the LEF for each axle load by the percentage of time that axle would cross a given point; that is, 33.3 percent for a single 35-kN axle and 66.7 percent for a 160-kN tandem axle (Table 1). The number of ESALs per truck for the design truck was found to be 4.86. This was used to convert the number of passes of the design truck with a total fatigue of 1 to ESALs.

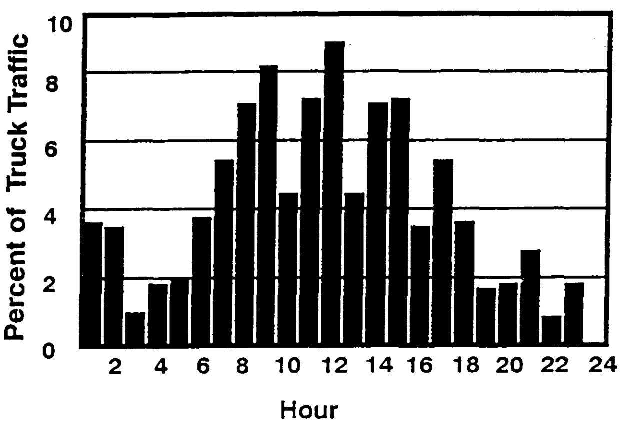

FIGURE 4 Hourly five-axle truck traffic on typical day.

TABLE 1 ESALs per truck and anticipated load applications

Total Axles	Load/Axle, kN	Mixed Traffic	LEF	ESALs/Truck
1 (Single)	35.0	33.33	0.032	0.032
2 (Tandem)	160.0	66.67	2.430	4.860
			=	4.892

Model Formulation

Various fatigue models were examined to determine if they could match actual New York rigid pavement performance; that is, for a 225-mm concrete slab with 300-mm granular base, asphalt shoulders, and traffic, material properties, and environmental conditions representative of New York pavements, the model would have to predict about 24.4 million ESALs. No model that produced results comparable to actual performance was found. Those examined were by Liddle, Vesic, and Hudson and Scrivener, discussed by Treybig et al. (*13*), and by Darter (*14*). Finally, a model was selected and calibrated to match the actual data—that is, the fatigue curve had the same shape but a different location. The model chosen was that developed by Hudson and Scrivener (*13*), because it was the only one that could be calibrated to fit New York's performance. To further improve the M-E procedure, New York has planned projects to establish data requirements, collect the appropriate pavement performance data for new pavements, improve or develop better fatigue models, and integrate the new data and models into its performance model (*15*).

Determination of Anticipated Total Axle Load Applications

The 24.4 million ESALs for the standard cross-section were converted to the total applications of the design truck and to the total number of 160-kN tandem axle applications, as shown in Table 2. Then, the anticipated load applications at each gradient were determined by finding the product of the percentage of traffic occurring at a given gradient and the total 160-kN tandem axle applications.

Determination of Total Permissible Axle Load Applications

Critical stress and stress ratios (σ_{cr}/M_r) were computed for each load-gradient combination. These ratios were used to determine the permissible (noncalibrated) load applications by using the Hudson and Scrivener model (*13*). The anticipated load applications at each load-gradient combination were divided by the noncalibrated permissible load applications. A curve of stress ratios (σ_{cr}/M_r) versus noncalibrated permissible load applications was plotted, and total damage was computed by using Miner's hypothesis. The nondimensional curve was then translated until the total accumulated damage was equal to 1. At this point the calibrated Hudson and Scrivener model became the New York rigid pavement model. The end product was Figure 5—a nondimensional curve for rigid pavement design in New York. Figure 5 was developed by plotting the number of permissible load applications against the stress ratios (σ_{cr}/M_r).

TABLE 2 Calibrated fatigue analysis (24.4 million ESALs/4.96 million 160-kN tandem axle loads) for 225-mm pavement thickness

Temperature Gradient, deg C/m	% Traffic At Gradient	Anticipated Total Applications	Critical Stress, kPa	Stress Ratio, σ_{cr}/M_r	Permissible Load Applications	Damage
0	10.31	510,872	1627	0.39	21,455,099	0.02
10	5.00	247,755	2089	0.51	7,252,832	0.03
15	7.13	353,299	2303	0.56	4,752,356	0.07
19	6.41	317,622	2496	0.60	3,350,992	0.09
24	6.23	308,703	2682	0.65	2,452,378	0.13
29	5.95	294,829	2854	0.69	1,871,488	0.16
34	2.76	136,761	3006	0.73	1,494,849	0.09
39	2.08	102,947	3158	0.76	1,207,292	0.09
44	3.22	159,760	3289	0.80	1,012,043	0.16
49	2.76	136,693	3413	0.83	861,752	0.16
					Total	1.00

Note: M_r = 4137 kPa.

DEVELOPMENT OF DESIGN CURVES

The nondimensional curve (Figure 5) was used as a fatigue model to predict probable fatigue lives for the pavement cross sections, slab lengths, shoulders, and materials properties used in the *Thickness Design Manual for New and Constructed Pavements* (*4*). First, the stress ratio was computed for each load-gradient combination. Second, the anticipated load applications were calculated. Damage at each gradient was then determined by dividing the anticipated applications by the allowable applications. Finally, total damage was computed by summing the damage at each gradient. To determine ESALs to failure, an arbitrary ESAL total was selected, and the damage owing to this total was then calculated. If this sum differed from 1.0, then total ESALs were divided by total damage, resulting in the ESALs to failure for the given pavement thicknesses. Predicted performance by both the M-E model and the manual (*4*) is indicated in Table 3. The design curve used in the manual (*4*) and that developed by using the M-E model is shown in Figure 6. This graph suggests that the *Thickness Design Manual* is more conservative for thicknesses of less than 275 mm and is less conservative for greater thicknesses.

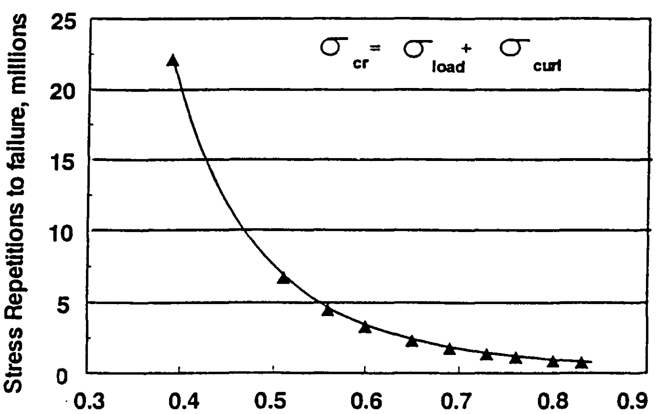

FIGURE 5 Stress repetitions versus stress ratio.

CONCLUSIONS

1. The study described in this paper has demonstrated that it is feasible and practical to develop M-E pavement design procedures, even with limited performance data, as in New York.

2. A nondimensional performance model was developed by using pavement response to the combined effects of temperature and traffic loading, and reliability was calibrated with actual life. For this paper New York's actual environmental data, materials properties, and traffic data were used to develop this nondimensional performance model.

3. Nondimensional models and M-E design procedures can be developed and adopted by other states and agencies, provided that the necessary data are available. These concepts in developing new pavement design procedures, however, are more broadly based (use of mechanistic parameters allows application of the M-E model to any pavement thickness in which the critical stresses are within the range of those used in calibrating the model) than the traditional empirical approach, which is based on full-scale road tests.

4. The pavement community is moving toward M-E design procedures, and state agencies cannot wait 30 or more years to collect new data to develop those procedures. The present study provides a step toward a true M-E process that will be recalibrated when more data become available.

5. The M-E and AASHTO models (after both were calibrated with the same performance data) predicted different performances for identical input values. In the present study the M-E model predicts more ESALs than AASHTO for pavement thicknesses under 275 mm and fewer ESALs for thicker pavements.

TABLE 3 Design results

Pavement Thickness, mm	ESALs to Failure, millions	
	M-E Analysis	Thickness Design Manual
225	91.5	50
250	126.0	90
275	172.0	165
300	216.0	290

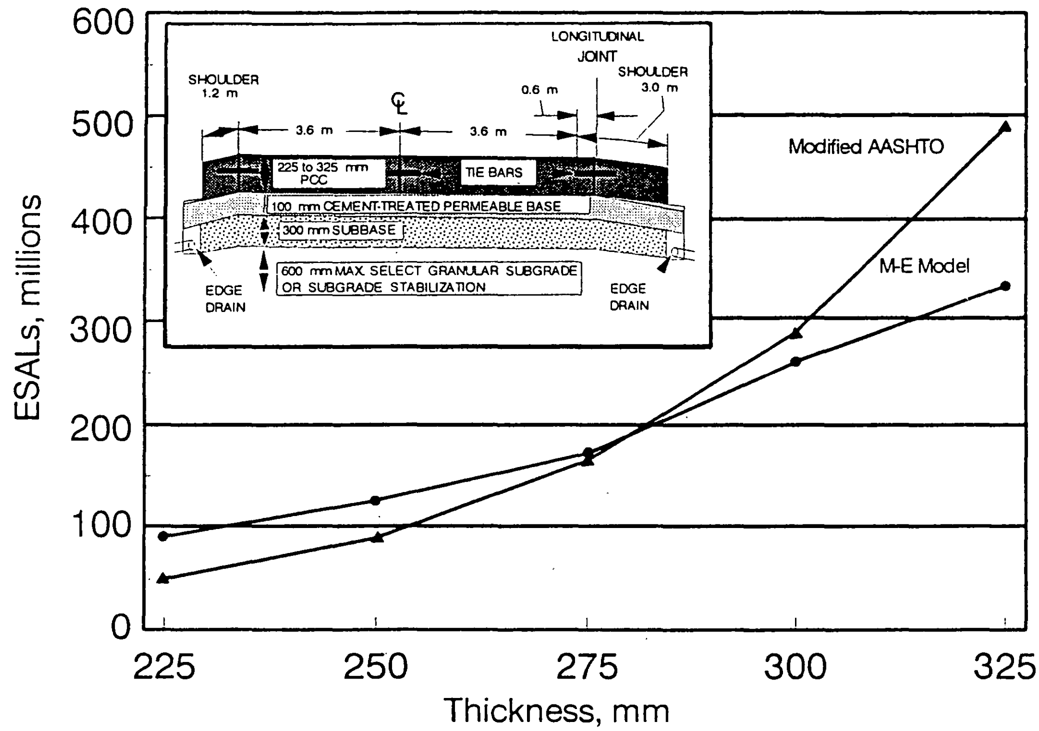

FIGURE 6 Traffic related to rigid pavement thickness (inset shows current cross-section).

REFERENCES

1. Pavement Selection Guide, Table 4–1. In *Highway Design Manual.* Facilities Design Division, New York State Department of Transportation, June 1979.
2. *Interim Guide for Design of Pavement Structures.* AASHO, Washington, D.C., 1972.
3. *Guide for Design of Pavement Structures.* AASHTO, Washington, D.C., 1986.
4. *The New York State Thickness Design Manual for New and Reconstructed Pavements.* Technical Services Division, New York State Department of Transportation, Sept. 1993.
5. *Highway Sufficiency Ratings.* Data Services Bureau, Planning Division, New York State Department of Transportation, published periodically.
6. Hahn, G. J., and W. Q. Meeker. *Statistical Intervals: A Guide for Practitioners.* John Wiley & Sons, 1991.
7. Calibrated Mechanistic Structural Analysis Procedures for Pavements: Phase 2, Vol. 1. Final Report on National Cooperative Highway Research Program Project 1-26. Construction Technology Laboratories, University of Illinois at Urbana-Champaign, Dec. 1992. Unpublished.
8. Tabatabaie, A. M., E. J. Barenberg, and R. E. Smith. Analysis of Load Transfer Systems for Concrete Pavements. In *Longitudinal Joint Systems for Slip-Formed Rigid Pavements,* Vol. II. Report RAA-RD-79-4, II. University of Illinois, Urbana-Champaign, 1979.
9. Smith, K. D., H. T. Yu, E. Owasu-Antwi, D. G. Peshkin, M. I. Darter, and R. Ahmad. *Performance Evaluation of Rigid Pavements—Data Collection and Analysis: Interim Report.* FHWA Contract DTGH61-91-C-00053. ERES Consultants, Savoy, Ill., April 1993.
10. Hammit, G. M., II. Concrete Strength Relationships. Miscellaneous Paper. Waterways Experiment Station, U.S. Army Corps of Engineers, 1974.
11. Morgan, R. L., H. J. Chen, I. Chengalur-Smith, and D. E. McAuliffe. *Overlays on Faulted Concrete Pavements.* Final Report on Research Project 188, in preparation.
12. *New York State Truck Weight and Vehicle Classification Reports.* Data Services Bureau, Planning Division, New York State Department of Transportation, published periodically.
13. Treybig, H. J., B. F. McCullough, P. Smith, and H. VonQuintus. Development of New Design Criteria. In *Overlay Design and Reflection Cracking Analysis for Rigid Pavements,* Vol. I. Report FHWA-RD-77-66. Austin Research Engineers, Inc., Austin, Tex., Aug. 1977.
14. Darter, M. I. Development of Design Procedures. In *Design of Zero-Maintenance Plain Jointed Concrete Pavement,* Vol. 1. Report FHWA-RD-77-111. Department of Civil Engineering, University of Illinois, Urbana-Champaign, June 8, 1977.
15. *Pavement Management Systems Plan Update.* Pavement Management Unit, Office of Operations, New York State Department of Transportation, July 1992.

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