TRANSPORTATION RESEARCH RECORD 1083

Pavement and Bridge Maintenance

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page 1
Change title of paper to

Freeway Modeling and Simulation for Personal Computers: Summary of the KRONOS IV Program

PANOS G. MICHALOPOULOS AND JAWKUAN LIN

Transportation Research Record 1083

page 13
Change title of paper to

Deck Assessment by Radar and Thermography

D. G. MANNING AND F. B. HOLT

Transportation Research Record 1070

page 29
Following page 29, insert Closure to the Discussion by A. M. Ionnides (pp. 27–29):

AUTHOR'S CLOSURE

The Authors appreciate the input of the discussant to this paper. The discussant correctly points out the use of tensile stress under edge loading for design of PCC pavements. Maximum tensile stress predicted by the layered elastic theory is only valid for interior loading condition. In the RPEDD1 computer program, the tensile stress calculated by the layered elastic theory is corrected by multiplying it with a critical stress parameter (C_p) before using it for fatigue life prediction *(1, 2)*. The value of C_p ranges from 1.05 to 1.60 based on the research reported by Seeds et al. *(3)*.

The primary objective of the methodology presented in this paper and related references *(1, 2, 4)* is the prediction of the in situ Young's moduli. It is emphasized that deflection basins measured away from the pavement edge, joints, and other discontinuities should be analyzed by the RPEDD1 and FPEDD1 computer programs. These restrictions lead to the interior loading condition and support the validity of the use of the layered elastic theory.

The information presented by the discussant on a comparison of the layered and plate theories is interesting. The authors would like to present published results of other investigations that reflect somewhat different perspectives. In a paper published in 1969 *(5)*, McCullough presented field verifications of Westergaard's solutions and layered elastic theory predictions. The paper shows that deflections and stresses predicted by the layered theory are higher than those predicted by the Westergaard interior equation. It was found that although the deflections predicted by the layered theory were somewhat higher than measured deflections, its response to the influence of the soil support value was much more representative of the actual condition than is that of the Westergaard equation. Haas and Hudson *(6)* also presented a summary of the field verification of layered elastic theory predictions.

Majidzadeh et al. *(7)* presented an excellent overview of various models for structural response analysis of pavements. The RISC program for Structural Analysis was used in the OAR procedure for rigid pavement rehabilitation design. The RISC program couples the finite element method for slab analysis with a three-layered elastic foundation. Their comparison of layered elastic theory and the RISC program shows that for the interior loading condition the responses from the two models are in agreement.

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1. W. Uddin, A. H. Meyer, W. R. Hudson, and K. H. Stokoe II. Project Level Structural Evaluation of Pavements Based on Dynamic Deflections. In *Transportation Research Record 1007*, TRB, National Research Council, Washington, D.C., 1985, pp. 37–45.
2. W. Uddin. *A Structural Evaluation Methodology for Pavements Based on Dynamic Deflections.* Ph.D. dissertation. University of Texas at Austin, Dec. 1984.
3. S. B. Seeds, W. R. Hudson, and B. F. McCullough. A Rigid Pavement Rehabilitation Design System. *Proceedings, 2nd International Conference on Rigid Pavement Design*, Purdue University, Lafayette, Ind., April 1981.
4. W. Uddin, A. H. Meyer, and W. R. Hudson. A Flexible Pavement Structural Evaluation System Based on Dynamic Deflections. Presented at the 1985 Annual Meeting of the Association of Asphalt Paving Technologists, San Antonio, Tex., 1985.
5. B. F. McCullough and K. J. Boedecker. Use of Linear-Elastic Layered Theory for the Design of CRCP Overlays. In *Highway Research Record 291*, HRB, National Research Council, Washington, D.C., 1969, pp. 1–13.
6. R. Haas and W. R. Hudson. *Pavement Management Systems.* McGraw Hill, Inc., New York, 1980.
7. K. Majidzadeh, G. J. Ilves, and H. Sklyut. *Mechanistic Design of Rigid Pavements*, Volume 1: *Development of the Design Procedures.* FHWA, U.S. Department of Transportation, 1983.

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page 50
Insert the following note to Figure 4:

Where

ρ	=	the azimuth angle from the driver's eye to a point *P* on the pavement;
θ	=	the elevation angle from the driver's eye to a point *P* on the pavement;
EZ	=	the driver's eye height above the pavement; and
DX, DY, DZ	=	the longitudinal, horizontal, and vertical distance between the headlamp and eye point.

Then

$$
\begin{array}{llll}
EX = EZ/\tan\theta & & HZ = EX - DZ \\
H1^2 = EZ^2 + EX^2 & & HX = EX - DX \\
EY = H1 \tan\rho & & HY = EY - DY \\
H2^2 = H1^2 + EY^2 & & H3^2 = HX^2 + HZ^2
\end{array}
$$

$\alpha = \mathrm{Tan}^{-1}\,(HZ/HX)$, $\beta = \mathrm{Tan}^{-1}\,(HY/H3)$,
$H4^2 = H3^2 + HY^2$

NCHRP Report 286

page 43, Table 5
The prime is missing from $\bar{B}$ in the third row. The third row should read

$\bar{B}'$	39	23	14.5	12

page 55, Table 7
The prime is missing from $\bar{B}$ in the third row. The third row should read

$\bar{B}'$	31	18	11	11

page 55, Table 8
The coefficients given in Table 8 are for redundant load path members. They provide curves corresponding to the lower bound lines given in Figures 63 to 80. The tabulated values given in Table 5 are compatible with these curves. For nonredundant load path members, the corresponding curves are provided by multiplying the coefficients in Table 8 by 0.8.

NCHRP Report 243 and Synthesis of Highway Practice 119

page 44 of NCHRP Report 243 and page 67 of Synthesis 119
Substitute the following revised Figure 45.

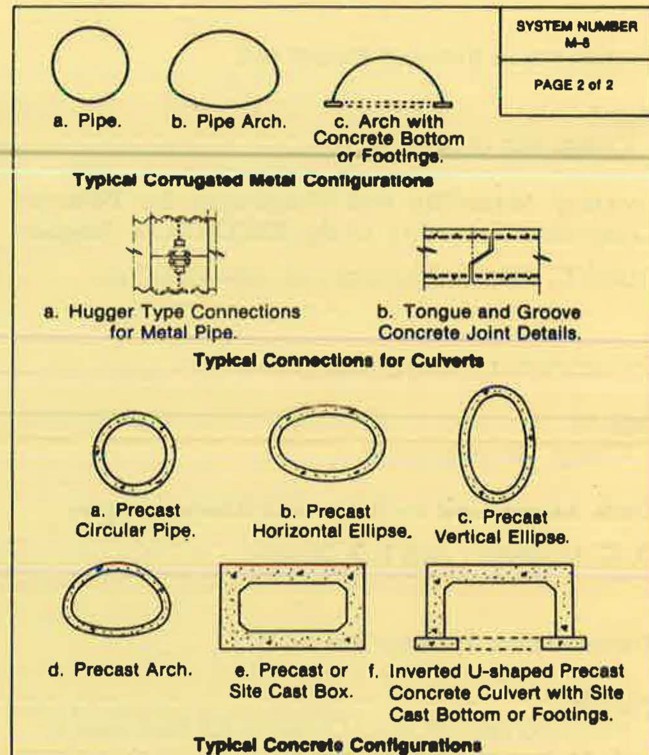

FIGURE 45. Single Culverts.

On line 8, PROMINENT FEATURES, System Number M-8, change "sleeve" to "bell and spigot joint."

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Slab Stabilization of Portland Cement Concrete Pavements

C. M. PEDERSON and L. J. SENKOWSKI

ABSTRACT

The Research and Development Division of the Oklahoma Department of Transportation (ODOT) undertook research to aid in the development of the department's special provisions pertaining to the stabilization of US-69, which had experienced accelerated deterioration and was scheduled for restoration. This effort consisted of determining the proper procedures to effectively stabilize the plain portland cement concrete (PCC) pavement, the quantity of material needed, and a method of verifying the quality of the stabilization operation for this particular highway. A general solution for the stabilization of all pavements is not proposed, but a basis for the method of void identification and the techniques used in pressure grouting is established.

Many of the portland cement concrete (PCC) pavements in Oklahoma are approaching the end of their design life and are showing signs of distress. In the past, a new road would be built or the pavement temporarily upgraded by means of an asphalt overlay; today, there are methods for PCC pavement restoration. This process, known as concrete pavement restoration (CPR), is gaining nationwide attention.

With the advent of the recycling, rehabilitation, resurfacing, and restoration (4R) program, resources were made available to accomplish the much needed work on the Interstate system. The restoration part of the 4R program, particularly the slab stabilization of PCC pavements, is addressed in this paper.

In May 1984, a Demonstration Project 69 (Portland Cement Concrete Pavement Restoration) was done in conjunction with the FHWA Tri-Regional Pavement Rehabilitation Conference in Oklahoma City, Okla. It was observed during the conference that each CPR activity was dependent on the stabilization of the slabs. It was clear that if the slabs were not stabilized properly, other activities such as diamond grinding, joint sealing, and patching may not perform as desired.

Research to develop procedures for slab stabilization began in July 1984. The facts and conclusions drawn from this research are documented in this paper and specifications and procedures are recommended.

BACKGROUND

As a PCC pavement ages, several signs of distress such as faulting and pumping, may be detectable. These symptoms, together, indicate the presence of voids in the supporting layers beneath the pavement. Usually the voids are caused by the infiltration of water, along with the presence of an erodible material in the supporting strata, and the movement of the slabs associated with traffic (1).

The method of slab stabilization, addressed in this paper, is the process by which grouting material is pressure injected beneath a pavement so that it will be laterally distributed into voids that are present to displace either free water, air, or both, that may be located in them. Once the grouting material is injected and allowed to harden, it should provide incompressible, nonsoluble, nonerodible support to the pavement (2).

The Research and Development Division of the Oklahoma Department of Transportation has been investigating the methods of slab stabilization currently in use. A literature review including abstracts revealed that there was little available on the subject of slab stabilization. Requests to several states for specifications on slab stabilization revealed a large variation in requirements from state to state. Therefore, a research effort was outlined and equipment purchased.

The purpose of this study was to determine the pressure grouting criteria needed for the development of the Oklahoma Department of Transportation Special Provisions for Pressure Grouting Portland Cement Concrete Pavement for Federal Aid Project MAF-186(170). The study determined (a) the proper procedures to effectively stabilize the pavement, (b) the quantity of materials needed for a full-scale restoration, and (c) a method of verifying the quality of the stabilization operation. The evaluation and verification of the technique and equipment associated with the Transient Dynamic Response (TDR) method were included, not only for void identification, but also to determine the feasibility of the TDR method as a quality control measure for slab stabilization operations.

LOCATION

The highway selected for the research was a 13-mi section of US-69, south of Muskogee, Oklahoma (Figure 1). This section, constructed in 1978-1979, consists of a 9-in. plain PCC pavement with sawed joints every 15 ft on a 4-in. fine aggregate bituminous base (FABB) with either 0, 6, or 12 in. of lime-treated subgrade. A typical section of the roadway is shown in Figure 2. The road was designed with a predicted average daily traffic (ADT) of 17,300 for 1996. The ADT is currently 9,800 with 21 percent trucks.

Within the first year after construction minor faulting was detected at the joints. During the next few years the faulting increased in magnitude and

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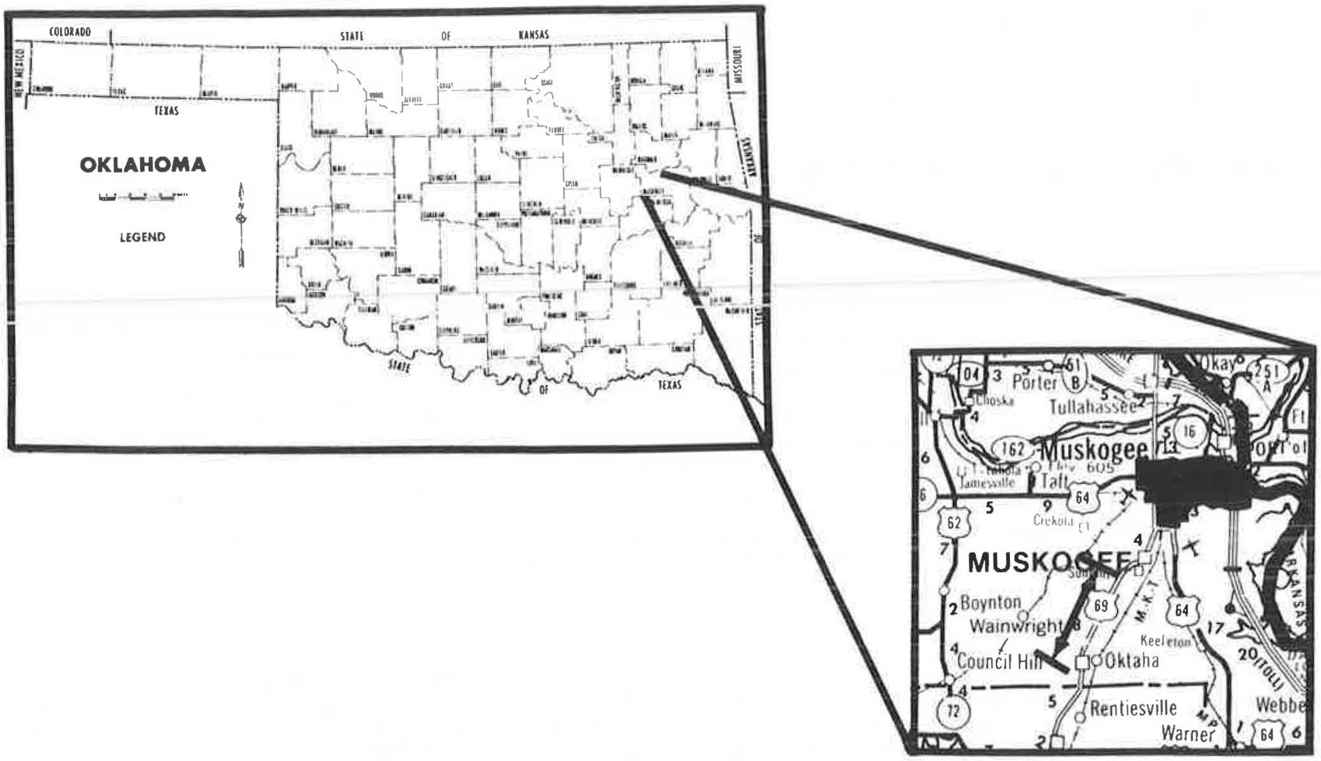

FIGURE 1 Map of location.

pumping became evident. About 4 years after construction diminishing ride quality attracted some concern.

A core investigation was conducted to evaluate the condition of the pavement and the subgrade. The PCC cores showed satisfactory strength, while the subgrade soil condition was defined in the range from poor to very poor. Throughout its life, this segment of highway had been the subject of several investigations. It was scheduled to become Oklahoma's first CPR project to begin in late 1985.

In order to present the findings of the studies, this paper has been organized into six categories: Void Identification, Grouting Material, Injection Holes, Grouting Operation, Quantities, and Quality Control. All slabs tested were in the outside lane. All the voids were identified as being in the base and within 2 in. of the pavement-base interface. For brevity, only the data from Site 1 are included.

VOID IDENTIFICATION

To fill a void in the supporting strata beneath a pavement, it is necessary to know the characteristics of the void. The dimensions, location, and boundaries of the void are important variables that must be investigated to determine what procedures should be taken. The dimensions (area and thickness) of the void determine the quantities of material required for filling the void. The location (void pattern and depth) of the void determines the frequency, location, and depth of the injection holes. These, along with the boundaries (confined or vented, material above, below or in the void, and moisture) of the void, will determine the pumping requirements for the proper injection of the grout.

Many tests are currently available to the engineer for obtaining required information. These tests are classified as either destructive or nondestructive. The destructive methods include coring, lifting of slabs, and excavation of the shoulder. The nondestructive testing (NDT) methods include visual surveys, roughness surveys, fault measurements, deflection measurements, and other sophisticated techniques.

For the current investigation, a new, nondestructive state-of-the-art method of void identification, known as TDR, was used along with other conventional

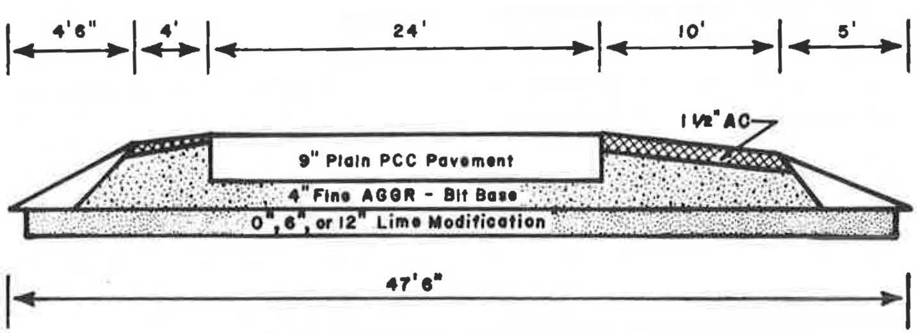

FIGURE 2 Typical section of US-69.

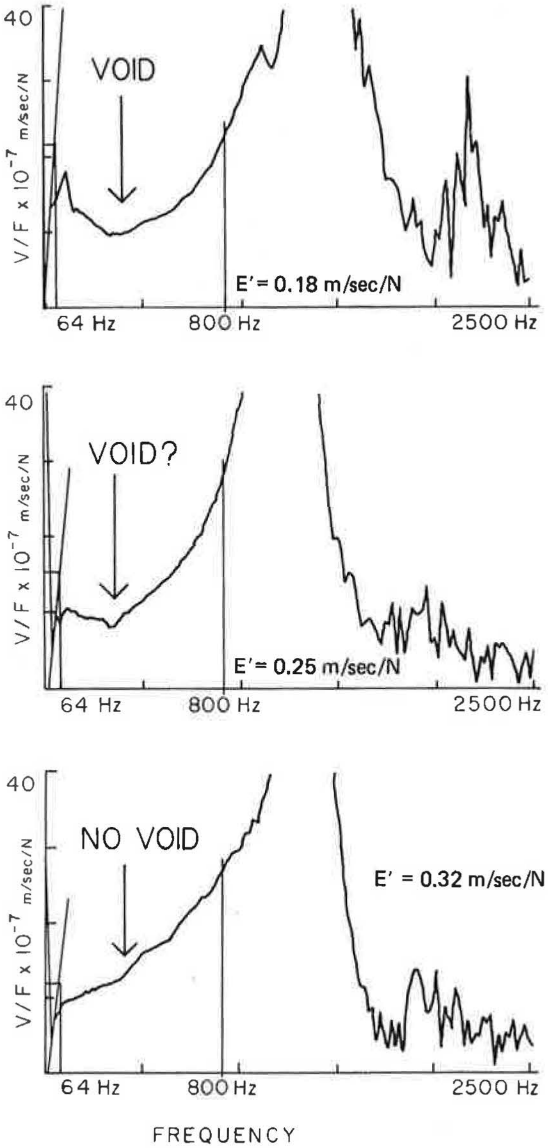

FIGURE 3 Transient dynamic response output.

tools. The basis of the TDR method is the analysis of steady-state vibration by the measurement of mechanical impedance. This method has been proved in the testing of piles in Europe (3).

The TDR method utilizes a twin-channel Spectrum Analyzer, an HP-85 computer, a plotter, a geophone, and a hammer with a built-in force transducer. The test is performed by placing the geophone firmly on the slab surface, and striking the pavement with the instrumented hammer. The distance between the geophone and the point of impact is kept constant throughout the testing. Force and velocity signals are recorded and plotted out as the mechanical admittance versus frequency (v/F) via Fast Fourier Transformation Analysis. The slope of the low frequency (up to 100 Hz) portion of the response curve is the apparent slab flexibility, S. The apparent slab stiffness, E' = 1/S, is expressed in units of meter per second per Newton (m/sec/N). The apparent slab stiffnesses can also be found on the plot. Three plots, as shown in Figure 3, are labeled VOID, VOID?, and NO VOID.

The existence of support for a particular point can be determined by the continuous increase of the curve in the 0 to 800 Hz range, as illustrated by NO VOID. A void is identified by any down slope in this region, as illustrated by VOID. The magnitude of the void, that is the thickness, can be correlated to the peak-to-valley difference. The larger the difference the thicker the void. A small difference, as illustrated in VOID? generally denotes a delamination.

Through a series of tests on a given slab, information can be assembled to yield a map of the void pattern. The more points tested, the more accurate the void pattern. A map of a typical void pattern is shown in Figure 4.

In order to establish the reliability of the TDR method of void investigation, three sites (good, average, poor) were chosen. Selection was based on previous work in which it was determined that the level of faulting correlated with the Mays Ride Meter data on US-69. Each site consisted of 90 consecutive slabs. Within each site, nine slabs were selected for repeat testing to verify the consistency of the TDR method, and 15 slabs were selected to be stabilized and retested. Also, a slab was removed within each site, based on the data that was provided, and several cores were taken to verify the results.

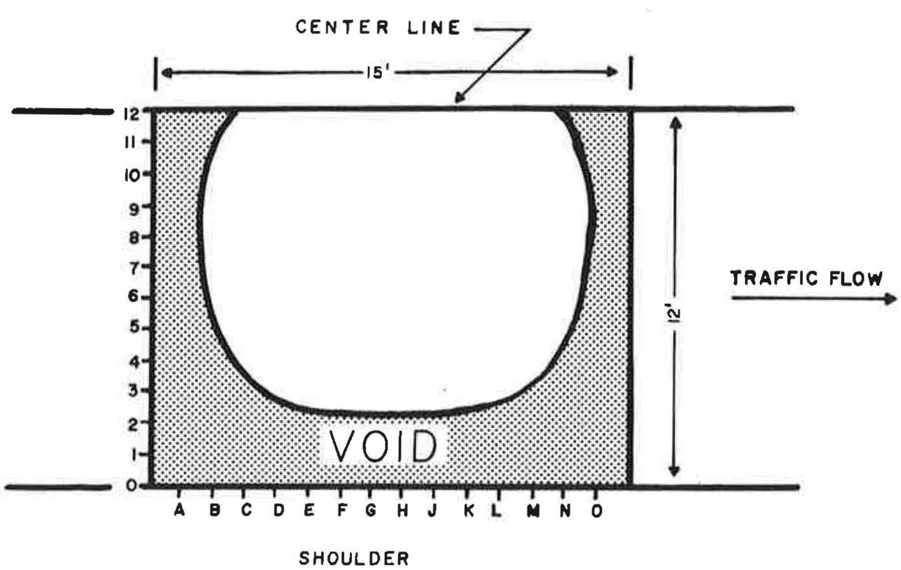

FIGURE 4 Map of void pattern with grid system.

The data presented is limited to Site 1. This site was characterized, from previous work, as average. All 90 slabs were tested and mapped. The following day nine slabs were retested. The same slabs were tested again the third day. The stiffness values of the repeat testing for Points B3 and O3 on the slabs are shown in Figure 5. The area of the voids did not change significantly during the repeat testing.

Fifteen stabilized slabs were also checked. The area of the voids before grouting and after grouting is plotted in Figure 6. The results indicated that the area of voids decreased with stabilization. However, subsequent testing indicated that the voids reappeared when traffic resumed. This may be attributed to the pounding action of traffic across the faulted joints; the pavement was not subjected to diamond grinding.

Before and after stabilization, the slabs were subjected to deflection measurements according to industry standards (1,4-6). A truck with a single-axle load of 18 kip was used for both static and dynamic loading and two dial gages were used to measure the deflections. All deflections recorded with the 18-kip loading method indicated that voids were not present. In fact, most of the readings were comparable to readings taken on a new pavement before traffic exposure (in the range of 5-thousandths to 10-thousandths of an inch). However, the TDR method identified voids under these slabs. The voids accepted grout, and cores confirmed that the voids existed as mapped.

Another approach was to remove one unstabilized slab from each of the three sites. It was found that the TDR method was not only useful in finding and mapping the voids, but delamination in the base as small as 1/16-in. thick could also be located. Although the TDR method identified the area of the void, the thickness of the void could only be determined through destructive means, and, therefore, the volume of the void could not be determined. Also, multiple layers of voids were not identified.

Slab testing became a nighttime operation when the daytime slab temperature exceeded 75°F causing slab lockup.

GROUTING MATERIAL

The material used in the investigation was the standard mix (2,4-7), that is one part (by volume) Portland Cement Type 1, three parts (by volume) Class C fly ash, and sufficient water to achieve fluidity. The material was purchased premixed in 80-lb sacks. No additives were used.

The fluidity of the grout was monitored with the use of a flow cone in accordance with ASTM C 939-81. Generally, the efflux time of the grout ranged from 10 to 12 sec, compared to water at 8 sec.

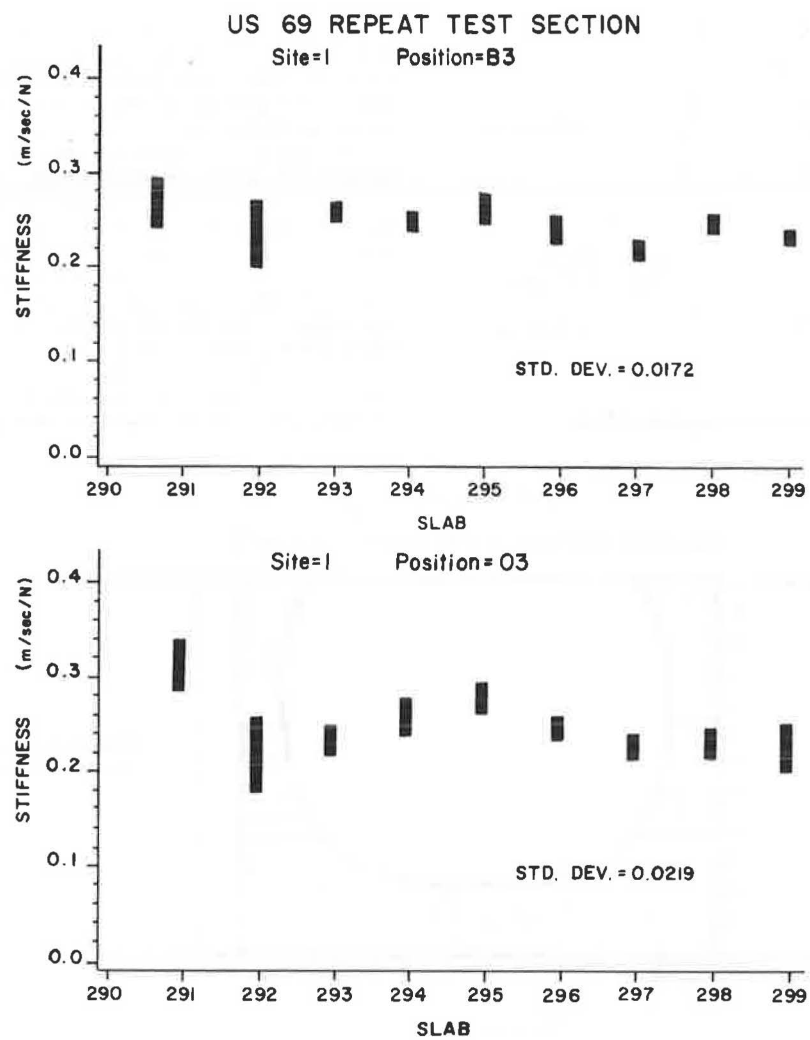

FIGURE 5 Plots of repeat testing.

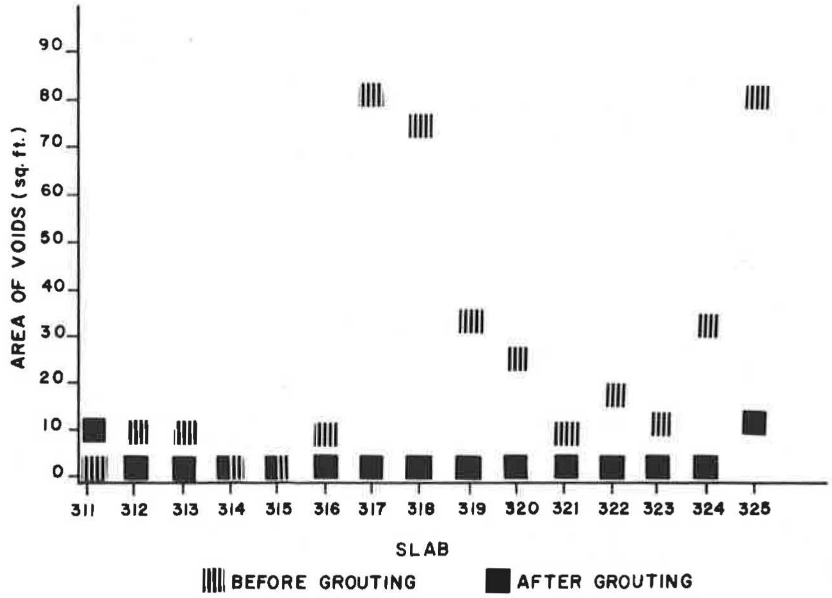

FIGURE 6 Area of voids (before and after grouting).

The mix design used in the field was 13.2 gal of water for a four-sack mix (w/c = 0.34). A laboratory report on this design is shown as follows:

- Mix Design Based on Absolute Volume
  1 part cement (AASHTO-M-85) + 3 parts Class C fly ash (ASTM-C-618 C), 51 percent
  Water, 49 percent
- Compressive Strength Averages for Three Molded 2-in. Cubes (AASHTO-T-106)
  1-day testing 141 psi
  3-day testing 203 psi
  7-day testing 811 psi

  Flow of mixture, 20.2 sec (ASTM C-939)
  Shrinkage or expansion, -7.9 percent (ASTM D-472)
  Water retentively 60 ml, 172 sec (ASTM-C-941)
  Initial set, 180 min (AASHTO-T-131)

The set time in the field did not correspond to that in the laboratory. In many cases the grout appeared not to have hardened for several days. This was evident in cores that were taken, as well as from the slow hardening of discarded grout that was piled off the shoulder.

INJECTION HOLES

To fill a void supporting strata beneath a pavement, it is necessary to provide injection holes to access the void. The diameter, frequency, location, and depth of the injection holes are important variables that must be considered for effective PCC pavement stabilization.

The diameter of the injection holes and the thickness of the void determines the cross-sectional area of the void that is exposed to the infiltration of the grouting material. If the diameter of the injection hole is reduced by one-half, then the cross-sectional area of the void is also reduced by one-half and vice versa.

The frequency and location of the injection holes are subject to the ability of the grout to disperse through the voids. If the voids are large (thickness at the injection hole greater than 0.20 in.), then the grout is more likely to be distributed far from the injection hole, and perhaps communicate up through other injection holes or up through the joints. In this case, a few injection holes might be all that are needed. However, if the voids are in the early stages, there may be a need for many holes. It is important to keep the number of injection holes at a minimum while providing adequate filling of the voids.

The location is also dependent on the borders of the slab. The injection holes are usually located at least 18 in. from any edge (1,5). The depth of the injection holes determines which voids are subject to the infiltration of grouting material. If a void is lower than the bottom of the injection hole, it is unlikely that the grout will penetrate to that level. If there are several layers of voids in contact with the injection hole, the grout may only flow in the channel of least resistance. Therefore, repeated grouting of a particular location may be necessary to ensure that all the voids are filled.

Ideally, to stabilize a PCC pavement the injection hole should be designed to expose the maximum cross-sectional area of the void to the infiltration of grouting material; the frequency, depth, and location of the injection holes should provide for proper distribution of the grouting material.

The method of drilling injection holes has been improved since the days of mud jacking. However, limiting the weight of the hammer and the down pressure applied, has not prevented the occurrence of the bottom breakout of the slab known as cone failure. This failure is the conical shattering of the bottom 2 in. of the slab within a 6-in radius of the injection hole. Cone failure still occurs to a large degree, although less frequently. To avoid this phenomenon on the US-69 project, the injection holes were cored. It takes more time to core holes than to drill them with a rock bit. However, coring equipment is available that can compete in terms of expedience as well as economy.

GROUTING OPERATION

Several factors lead to the success of the pressure grouting phase. Assuming the voids have been identified, the injection holes properly selected and placed, and the proper grouting material available, the pressure grouting sequence begins.

As mentioned earlier, the cross-sectional area of the void exposed to the infiltration of grout must be maximized. Some states have already implemented the flushing of injection holes with either air, water, or both. The US-69 investigation supports this practice. No problems were encountered when injecting grout into holes that were flushed with water. Flushing the holes with water removes debris that could potentially block the grouting channel. Also, by coring the injection holes, fragments of concrete associated with the blocking of the grouting channel were eliminated, thereby eliminating the high pressures associated with the initial injection of the grout.

The pressure at which the grout is injected plays an important role in slab stabilization because the uplift of the slab should be held to a minimum. Theoretically, for a 12- by 15-ft free-standing slab of 9-in. PCC, an applied pressure of 1 psi is all that is needed to lift it. Additional pressure in the field is necessary only to overcome the frictional and mechanical, or both, restraints. For this investigation, there was no need to exceed 60 psi.

The flow rate at which the grouting material is injected controls not only the rate of distribution but also the pressure buildup. The grouting material must be pumped in a manner that will allow time for continuous distribution throughout the void. If the grouting material is pumped at too high a flow rate, it may not have enough time to penetrate and the pressure will begin to build up. If the grouting material is pumped at too low a flow rate, the flow may not be continuous causing blockage and pressure buildup. A flow rate of 7 gal/min produced the desired distribution of grouting material during the US-69 investigation.

Another important aspect of pressure grouting is the displacement of the free water and air. If the void is confined, water or air cannot escape. Because water is incompressible, there is the potential that an air pocket will be formed in the grout matrix when the water dissipates. However, if seepage of grout is noted in the joints or in adjacent holes, the possibility of pockets being formed is minimal because the specific gravity of grout is greater than that of water or air.

Pumping at the injection hole was stopped if (a) the pumping pressure exceeded 60 psi (b) excessive grout was observed seeping up through the joints, or (c) if the slab was raised more than 0.035 in. The low pressure and moderate flow rate allowed for greater control, and good distribution of the grout, thereby preventing excessive raising of the slab.

QUANTITIES

The quantities of grouting material required for a full-scale restoration project on US-69 were based on the volume of grout used to stabilize various slabs. It was found that the average slab required less than 2 ft^3 of material for stabilization.

QUALITY CONTROL

Verification of the grouting operation has been a most difficult task. Following the guidelines relating to proof rolling, it was found that the 18-kip loading method can be meaningless because of the low magnitude of the deflections. On the other hand, the TDR method proved successful.

The TDR method yields two quantitative results, stiffness and void area. When an unstable slab has been stabilized, stiffness increases and the void area decreases. However, the TDR method is labor intensive, costly, and subject to environmental conditions.

The most critical factors in a successful stabilization project are the inspector and the contractor who should have a thorough working knowledge of the entire process, because their level of understanding determines the quality of the job (8).

SUMMARY

The effectiveness of slab stabilization depends on the identification of the voids, the grouting material, the design of the injection holes, the techniques of grouting, and the competence of the workers; the TDR method can map void patterns and measure the relative stiffness of PCC slabs; and finally, note that the 18-kip loading method of void detection can be meaningless.

RECOMMENDATIONS

The following recommendations are associated with the special provisions relating to the pressure grouting of US-69.

1. Core the injection holes,
2. Flush the injection holes with water before grouting,
3. Limit maximum pressure to 60 psi,
4. Specify the flow rate at 7 gal/min,
5. Do not specify repeat grouting,
6. Estimate the quantities based on 2 ft^3 of material per slab, and
7. Do not specify proof rolling.

Other general recommendations:

1. Further investigation is needed to verify and refine the foregoing recommendations on a project-by-project basis,
2. The TDR method should be further investigated as a quality control method for slab stabilization, and
3. The effects and timing of each CPR activity on the stabilization of PCC slabs should be investigated.

ACKNOWLEDGMENTS

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Portland Cement Concrete Pavement Performance as Influenced by Sealed and Unsealed Contraction Joints

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ABSTRACT

In the 1950s and 1960s, the contraction joints in portions of several Wisconsin pavements were purposely left unfilled in an effort to determine the effect joint filling (and routine refilling) had on subsequent pavement performance. After 11 to 19 years of observation it was determined that the initial filling or refilling of contraction joints (40- to 100-ft spacings) had no beneficial effect on overall pavement performance. In 1974 a carefully designed joint and sealant study began with the objectives of evaluating the effect of joint spacing and joint sealing or nonsealing on total pavement performance, and evaluating joint sealants. This study was conducted on a new 9-in. jointed reinforced concrete pavement on a well-drained subgrade, employed five joint sealants, and considered four joint spacings (20, 40, 60, and 80 in.). A total of 22 test sections were evaluated, including eight control sections in which the joints were left unsealed and 14 test sections in which the joints were sealed (the joints in these test sections were resealed to maintain a sealed system for 10 years). Based on 10 years of monitoring total pavement performance (considering summer and winter ride, pavement distress and material integrity), it was found that some sealants served well for 10 years, short joint spacings gave the best pavement performance, and the pavement with unsealed joints had better performance than the pavement with sealed joints. It was concluded that there may be conditions and circumstances that do not justify the cost of sealing PCC pavement joints.

In order to understand the origin of the subject of this paper, it is helpful to review past experience in Wisconsin relating to sealing of joints in portland cement concrete (PCC) pavements. In 1953 a jointed plain concrete pavement (JPCP), with a 40-ft contraction joint spacing of 0.25-in. wide joints, was built on US-151 in two contiguous counties (Lafayette and Iowa) in the southern part of the state. In Wisconsin the counties perform the maintenance work, and, at that time, PCC pavement joint resealing was, or was not, routinely performed in each county based on a number of factors. In both counties the joints and cracks were sealed (actually filled) with an asphalt-based sealant at the time of construction; but in Iowa County they were routinely resealed (refilled) to prevent the intrusion of incompressibles and water, whereas there was no reseal-

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ing in Lafayette County. After 11 years of service and based on pavement performance factors (faulting, cracking, spalling, patching, etc.), S.T. Banaszak, Wisconsin Department of Transportation (WisDOT), concluded: "It is quite apparent that the omission of the joint sealer resulted in better overall pavement performance than that of the sealed joints." (Unpublished data.) Clearly, the sealed joints were not sealed but only filled, yielding a partially sealed condition; however, this study indicated that keeping some of the water and incompressibles out of the joint was of no benefit to overall pavement performance.

Based on the above experience and that of several other pavements on which sealing at the time of construction had inadvertently been omitted, several iconoclastic engineers propounded what at the time was, and to many still is, an outrageous question: Is it actually necessary to seal the contraction joints in PCC pavement? (1). Their curiosity prompted a more systematic investigation.

BACKGROUND

Past Experience in Wisconsin

In 1958 several test sections were placed in the southbound lanes on US-41 in Washington County. This jointed reinforced concrete pavement (JRCP) was 9-in. thick, rested on a 6-in. gravel or crushed stone base course, and had 9 in. of granular subbase material. The subgrade soil had a K value of about 250 pci. The joints, which contain load-transfer dowels, were sawed 0.25-in. wide at 100-ft intervals and filled with hot-poured sealant conforming to ASTM D 1190. One experimental section had sealed joints, one had alternately sealed and unsealed joints, and another section had all unsealed joints.

The performance of the two types of joints (initially sealed and unsealed) was monitored via biannual visual inspections. By 1966 the investigators were reporting that the condition of the unsealed joints was "far superior" to the sealed joints. Specifically, the unsealed joints exhibited less corner cracking and spalling than their sealed counterparts.

Because of the previous report, the investigation was expanded, and in 1966 a second, larger experimental project was started on WI-78 in northern Columbia County. This 4-mi segment of pavement was similar to US-41 in design features, except that contraction joints were spaced at 80 instead of 100 ft. Like US-41, WI-78 was a four-lane divided highway. The pavement structure, designed for a subgrade K value of 300 pci, consisted of a 9-in. JRCP, over 6 in. of base course, and 6 in. of select excavation. The joints in the southbound pavement were sealed with a hot-poured sealant (ASTM D 1190), whereas the northbound pavement joints were left unsealed. It was also decided in 1966 to expand the objectives of the studies on US-41 and WI-78, and, what had begun as a study of joint performance became a study of pavement performance.

In 1977, based on evaluation criteria such as pavement distress and ride and materials integrity (one pavement was 19 and the other was 11 years old), it was concluded that the inclusion or omission of a joint sealant at the time of construction did not exert a significant influence on pavement performance (1).

The three aforementioned studies were not well-designed research projects and all had one deficiency, that is, the joints were neither truly sealed nor could they be with the joint spacing, joint shape factor, and sealants used. Thus, although these studies did clearly indicate that keeping some of the water and incompressibles out of the joint was of no benefit, they did not answer the real question concerning the cost-benefit of truly sealed contraction joints.

Experience of Others

Until the midtwentieth century little progress had been made in the art of joint and sealant design since the first apparent use of a sealant in road construction in seventh- or eighth-century B.C. Mesopotamia where asphalt block or asphalt brick pavements were built, the asphalt serving as a sealant and cement (2,3). Until then, most sealants were merely asphaltic-based compounds and little or no attention was paid to allowable sealant extensions, sealant strains, or joint shape factors. However, in the 1950s joint and sealant design began to be subjected to scientific investigation, the preeminent contribution being made by Egon Tons in the late 1950s (4,5). Such investigation, coupled with the development of new and promising sealants, helped usher in an era in which there was at least the potential for designing a joint that would remain sealed.

By the early 1970s there was a tremendous volume of research and information on PCC pavement joint sealing; however, the vast bulk of this research was on joint or sealant performance. There appeared to be a dearth of information available on overall pavement performance as influenced by joint sealing, the emphasis being placed on the secondary issue of sealant and joint performance. The benefit of keeping incompressibles and water out of joints appears to have been accepted in general, with few or no qualifications as to pavement type, subgrade characteristics, environmental conditions, or material properties.

Origin of Pavement Performance Study

Although the findings of the studies in Wisconsin were provocative, they were not conclusive. The searches for information from other agencies on total pavement performance (as influenced by joint sealing, or lack thereof) was even less conclusive because most research was devoted to only sealant or joint performance. This lack of information was enough to justify a carefully controlled study of the subject in the state. However, the great advances in joint sealing theory and sealing materials, coupled with the research by other agencies on the benefits of close contraction joint spacings, compelled the state in 1973 to begin a study of pavement performance as influenced by sealed and unsealed contraction joints at various spacings.

OBJECTIVES

The a priori arguments concerning joints and joint sealing were set aside to take a fresh look at pavement performance. The objectives of the study were to evaluate (for a period of 10 years and possibly longer):

1. The effect of joint sealing, or lack thereof, on total pavement performance;
2. The effect of joint spacing on total pavement performance; and
3. The performance of various joint sealants.

STUDY DESIGN

Highway Factors

The pavement test area is a 9-in., slip-formed, mesh-reinforced, dowelled PCC pavement. The test area is on US-51 from Wausau to Merrill in Marathon County (north-central portion of the state). US-51 is a four-lane divided highway with an average daily traffic (ADT) of 7,000 (11 percent heavy vehicles). The subgrade is a sandy glacial outwash material. The 8-in. base course, concrete coarse aggregate and shoulders are a crushed gravel (primarily igneous material). No shoulder pavement was placed on the crushed gravel shoulders. Wisconsin's crushed gravel for base and shoulders is well graded and is not considered to be free draining (maximum size is 0.75-in. with a P200 of 3 to 10 percent for base material and 8 to 15 percent for shoulders).

Winter maintenance for snow and ice control consists primarily of plowing and chemical application; few, if any, abrasives have been used on this section of highway. This portion of the state receives 30 in. of rainfall annually and temperatures commonly range from 100°F to -40°F.

Research Features

Four contraction joint spacings of 20, 40, 60, and 80 ft were used for this study. At the time, a 20-ft contraction joint spacing was considered a practical minimum; however, within 3 years the state routinely used a shorter joint spacing. All joints were cut transversely with respect to the centerline.

The intent of the study was to use one or two of the most promising sealants from three sealant groups (thermoplastic, chemical setting, and preformed sealants). Based on contact with other states and information from a literature search by the Transportation Research Information Service, four sealants, together with the standard sealant, were selected for inclusion in the study.

1. Rubberized asphalt (Federal Specification SS-S-1401),
2. Coal tar-based polyvinyl chloride (ASTM D 3406),
3. Two-component cold-pour polysulfide (Federal Specification SS-S-00195A),
4. Preformed neoprene compression seal (ASTM D2628), and
5. Standard sealant (ASTM D1190).

Two test sections were used for each combination of joint spacing and sealant (including no sealant in the joints). Therefore, a total of 22 test sections (each a nominal 1,000 ft in length) were used (Figure 1 and Table 1). In 14 test sections all the joints were sealed and in 8 test sections all the joints were left unsealed. Test sections were randomly placed; no attempt was made to place all sealed test sections next to one another and all unsealed test sections next to one another. It was recognized that the performance of one test section may influence the performance of a contiguous test section. Therefore, the use of two test sections for each combination of joint spacing and sealant provided a replica test section to help determine if one test section influenced another.

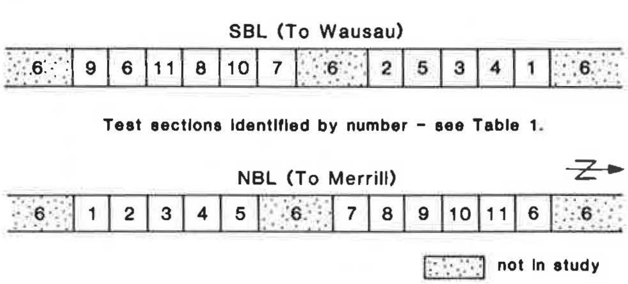

FIGURE 1 Schematic drawing of test section layout.

TABLE 1 Characteristics of the Various Test Sections

Test Section	Sealant	Joint Spacing (ft)	Sealant Depth to Width Ratio	Original Joint Width (in.)
1	Polyvinyl chloride	20	2.7	3/8
2	Preformed compression	40	NA[a]	3/8
3	None	40	10	1/4
4	Polysulfide	20	1.2	5/8
5	None	80	10	1/4
6	Standard	80	10	1/4
7	Polysulfide	40	0.6	1-1/4
8	None	20	10	1/4
9	None	60	10	1/4
10	Preformed compression	60	NA[a]	1/2
11	Rubberized asphalt	20	1.0	7/8

Note: See Figure 1 for schematic drawing of test sections.
[a]NA = not applicable.

The design of the joint width for the joints in each test section (except the unsealed and standard joints that were 0.25-in. wide) was based on data from Wisconsin (6), the American Concrete Institute (7), sealant manufacturers (8), and other states' agencies (9). The sealant shape factor (depth to width ratio) was based on manufacturer recommendations and design aids (10) (Table 1).

The centerline joint was sealed with either a rubberized asphalt or polyvinyl chloride sealant, and the centerline joint was only sealed in the test sections that contained a sealant in the transverse joints. More detailed information on the study design and layout is available elsewhere (11,12).

Pavement Performance Indicators

The pavement performance indicators for this study are as follows:

1. Three pavement performance indicators were documented annually for 10 years.
   a. Ride was measured summer and winter on a scale from 0 to 5 according to the Wisconsin Roadmeter, a response-type road roughness measuring system.
   b. Pavement distress (faulting, blowups, cracking, joint spalling, etc.) was measured each fall.
   c. Sealant performance was determined each winter.
2. Thousands of joint movement readings were taken at various temperatures throughout the year for several years to determine joint movement as a function of joint spacing and temperature.
3. Cores were taken when the pavement was 10-years old to observe subsurface distress at joints.

COSTS

To estimate cost-effectiveness, the cost for a pavement with unsealed joints must be compared to a similar pavement with sealed joints, that is, the costs to maintain the joints in a sealed condition must also be included. Thus, the costs to create and maintain a sealed joint system must be offset by an equivalent increase in pavement performance and life in order for joint sealing to be cost-effective.

The costs (including the pavement, initial saw cut, reinforcement, and dowel bars) for the pavement with unsealed joints averaged $7.28/yd². The additional costs to create a sealed joint system (including the second saw cut to create an appropriate joint shape factor, backing material, cleaning and sealing) ranged from 8.2 to 22.5 percent of the costs for the pavement with unsealed joints. When the costs for maintaining the joints in a sealed condition for 10 years are added, the pavement with sealed joint systems costs up to 45 percent more than a similar unsealed pavement.

As a point of interest, the cost for sealing joints was not so much related to the sealant cost as it was to the extension range of the sealant. For example, the relatively inexpensive rubberized asphalt had a low extension range and accordingly required sawing a wide sealant reservoir; therefore, considering all costs, the use of this sealant resulted in higher costs than the expensive sealants that had a larger extension range and required less saw cutting.

RESULTS

The following results are based on 10 years of data collection.

Joint Movement

The average joint opening in inches expected from the warmest to coldest temperature was found to be 0.0448 + 0.0044 x joint spacing in feet. Therefore, for a 40-ft joint spacing the average joint will move 0.22 in. from maximum closure to maximum opening.

Further information on joint movement studies and resulting joint design aids is available elsewhere (11).

Maintenance of Seals

Criteria for distinguishing a sealed from an unsealed joint system was derived from a percentage of sealant failure (20 percent). If less than 20 percent of the linear feet of sealant in a test section had failed, the test section was considered sealed. Whenever 20 percent or more of the linear feet of sealant in a test section revealed a sealant failure that allowed water or incompressibles into the joint, the entire test section was considered unsealed, and was resealed in kind as soon as possible.

The preformed neoprene compression seals, in the test sections with a 60-ft joint spacing, failed very early in the study and no attempt was made to reseal these joints. In all other test sections, if resealing was required more than three times in 10 years attempts to maintain a sealed joint were abandoned. Accordingly, the test sections with the 60- and 80-ft joint spacings were not, or could not, be kept sealed and attempts to reseal the joints were abandoned. Therefore, the test sections with 60- and 80-ft joint spacings were not kept sealed but existed in a partially sealed condition for most of the 10 years. The test sections with the 20- and 40-ft joint spacings could be, and were, kept sealed for the entire 10 years.

Resealing joints with poured sealants was accomplished by removing the old sealant, cleaning the concrete sidewall, and placing the new sealant. The preformed neoprene compression seals failed in 4 years in the test sections with a 40-ft joint spacing. The joints with the preformed neoprene compression seals were resealed by removing the old seal, resawing the joint to a greater width, and then installing a new preformed neoprene compression seal.

The unsealed joints were not blown or flushed; incompressibles were allowed to remain in place. The centerline joints that were sealed remained sealed throughout the study.

Sealant Performance

From this study, it was found that sealants can exhibit the lives given in Table 2 (based on the 20 percent failure criteria), if properly placed in an adequately designed joint. For some sealants the life has been extrapolated from present conditions. There is a range in life for the preformed neoprene compression and polysulfide sealants. The shorter life for the polysulfide sealant is for the 40-ft joint spacing and the longer life is for the 20-ft joint spacing. Better joint design and material specification (data derived from this study) has lengthened the anticipated life for preformed neoprene compression seals in a 40-ft joint spacing from 4 to 10 or more years.

TABLE 2 Life of Properly Placed Sealants

Sealant	Life (yr)
Rubberized asphalt	5
Polyvinyl chloride	10
Preformed compression[a]	4-10+
Polysulfide	8-12+

[a]For the 40-ft joint spacing.

Pavement Distress

Pavement distress was determined from all observed forms of distress. No faulting was observed; the joints had positive load transfer devices. In Wisconsin, pavement blowups often begin before 10 years of age; however, no blowups were observed in any of the 22 test sections. Because the pavement in this study has had a history free of blowups, the results and conclusions of this study are to be understood in the context of these observed conditions. The two significant pavement distress types observed were joint spalling and panel cracking (transverse, longitudinal, and diagonal).

Joint Spalling

Any joint distress that developed within the first year of the pavement's life was considered to result from factors other than those relating to joint spacing and sealing. To compensate for such factors, the change in joint distress from 1 to 10 years will be considered (Figure 2). Recall that the joints were only partially sealed in the sections with a 60- and 80-ft joint spacing. The spalls did not predominate in any one area along a joint (such as in the corner areas), but were fairly well distributed. In most cases the joint spalling was slight to moderate in severity (partial depth and less than 4-in. wide). Therefore, the extent of spalling is the primary concern. It is immediately clear that (a) for

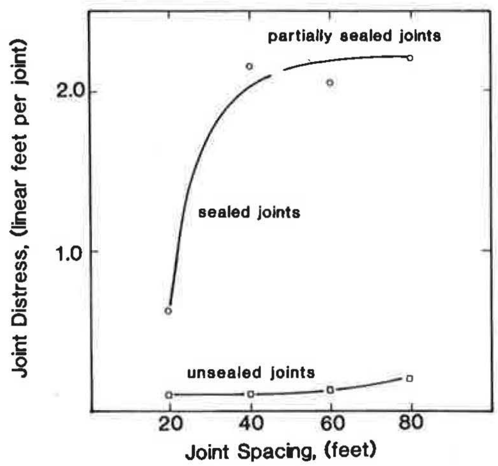

FIGURE 2 Joint distress versus joint spacing.

the sections with sealed joints, there was a large increase in joint distress with joint spacings above 20 ft and (b) the unsealed joints had much less spalling, regardless of joint spacing, than the sealed or partially sealed joints.

An increase in joint distress with joint spacing is understandable, but the small amount of joint distress for the unsealed sections (0.25-in. wide joints), regardless of joint spacing, is unexpected. Some of the joint distress in the sealed sections is due to the resealing operations that occasionally caused small spalls; however, the superior performance of the unsealed sections appears inexplicable.

Panel Cracking

Panel cracking refers to all forms of full-depth cracking within a panel (area bounded by transverse joints and pavement edges). The panel cracks were not corner breaks typical of those due to lack of support or pumping. The panel cracks were primarily transverse cracks with a small amount of longitudinal and diagonal cracking. All forms of panel cracking per test section were summed (Figure 3) and the results indicate that (a) there was a dramatic increase in cracking for panel lengths over 40 ft, and (b) there was more cracking in the sealed and partially sealed sections than in the unsealed sections.

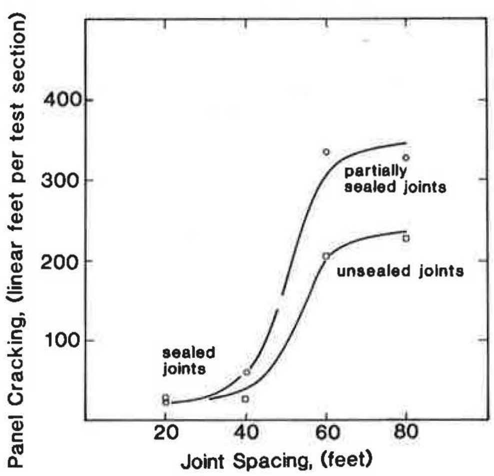

FIGURE 3 Panel cracking versus joint spacing.

Although the difference between the amount of panel cracking in sealed and unsealed sections is not considered significant, the difference in cracking between the partially sealed and the unsealed sections of the same joint spacing is significant. The large increase in panel cracking for the larger joint spacings, regardless of whether joints are sealed or not, is of real concern (Figure 3). Because most of the panel cracking occurred within the first year of the pavement's life, it is believed to result from factors other than loading. Such panel cracking apparently results from the base's frictional restraint to concrete movement, for example, from thermal contraction and concrete shrinkage. The base's frictional restraint increases with increasing panel lengths, causing more cracking in the longer panels. The large amount of panel cracking in the longer panels is common and is one of the reasons for the use of shorter joint spacings.

The results shown in Figures 2 and 3 indicate a clear preference for the use of short joint spacings and unsealed joints. Even if there were indications that sealed and unsealed sections behave more similarly than data suggest, they would not explain the lack of superior performance of the sealed sections, which is necessary to justify sealing costs.

Change in Ride

Considering all test sections, the change in ride from the 1-year old pavement to the 10-year old pavement was approximately 0.5 psi (pavement serviceability index), indicating that (a) sealed, partially sealed, and the unsealed sections decreased in ride a similar amount; (b) summer and winter rides were similar; and (c) joint spacing had little or no effect on the change in ride.

It was believed, before this study, that the infiltration of water in the unsealed joints would cause a much rougher winter ride in the unsealed test sections than in the sealed sections. This did not prove to be true.

Coring

PCC joint repair projects on other highways have revealed a cone of disintegrated concrete beneath most joints. It is believed this cone is partly due to compressive forces that tend to concentrate in the lower portion of the joint--such concentration being the result of incompressibles in the joint, especially the lower portion. In the pavement project described in this paper, the unsealed joints have been filled with incompressibles (except the upper one to 3 in. in the wheel paths) from shortly after the time of construction. Cores taken at joints at 10-years old indicate no distress beneath any joints regardless of joint spacing or sealing. Although this distress may become significant in the future, at this point there is no difference in material integrity as a result of joint sealing or joint spacing.

CONCLUSION

After 10 years of observation, the following conclusions are justified for this study. Recall that the pavement under consideration had positive load transfer at the joints, was on a well-drained subgrade, had a blowup-free history, was in a northern climatic zone and had gravel shoulders.

1. The pavement with unsealed joints performed better than the pavement with sealed joints,

2. The pavement with shorter joint spacings performed better than the pavement with the longer joint spacings,

3. There are sealants that can keep joints effectively sealed for 10 years when placed in a properly designed joint, and

4. Contraction joint sealing costs cannot be justified.

SUMMARY

When total pavement performance is considered, the results from 10 years of experience on US-51 indicate that shorter joint spacings (e.g., 20 ft) lead to better pavement performance than longer joint spacings, which was an expected result supported by other agencies. However, the conclusion that pavements with unsealed joints performed better than those with sealed joints, is provocative.

Arguments may be made to show that sealed and unsealed test sections behave more similarly than the data suggest. However, such efforts could only prove, at best, equality of performance, which does not sufficiently justify the cost of sealing over nonsealing. The entire costs for maintaining a sealed pavement for 10 years, from sawing a joint reservoir and sealing it to resealing the joint when needed, amounted to as much as 45 percent more than the cost for a similar unsealed pavement. Therefore, to justify this cost, there would have to be (a) a much greater serviceability (ride) during the pavement's life, (b) much less maintenance, or (c) a significant increase in pavement life. At this time, there is no basis for such justifications.

The results of this study correspond to the precursory studies made in the 1950s and 1960s. Today, WisDOT routinely uses a joint spacing of 20 ft or less and is conducting other sealed versus unsealed joint studies because the efficacy of joint sealing is in question. These other studies were necessitated because the present study had limitations and the following questions still remain unanswered:

1. Although joint sealing appears not to be beneficial for dowelled contraction joints, is the same true for nondowelled joints for which there is greater opportunity for pumping and faulting?

2. Although joint sealing appears nonbeneficial on a well-drained subgrade, would it be beneficial on a heavy, poorly-drained soil?

3. Is joint sealing justified where blowups are more prevalent?

A true assessment of joint sealing must be based on total pavement performance, not just sealant and joint performance. This study clearly indicates there are situations for which joint sealing may not be justified. Even if pavement performance can be enhanced and pavement life prolonged by joint sealing and resealing, the cost-benefit of such operations has to be evaluated. Considering the costs for all sealing operations, a pavement would have to ride better, require less maintenance, or its life would have to be extended many years to make sealing a sound investment.

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The Development of Deck Assessment by Radar and Thermography

D. G. MANNING and F. B. HOLT

ABSTRACT

A systematic approach to bridge deck rehabilitation requires considerable data on the condition of decks. In the past, data were collected using the traditional methods of visual inspection supplemented by physical testing and coring. Such methods proved tedious, expensive, and of limited accuracy. Research studies were undertaken to develop methods for the rapid and automatic collection of data on the condition of bridge decks, resulting in the deck assessment by radar and thermography (DART) system. As the acronym implies, DART utilizes two basic systems: impulse radar and infrared thermography. A prototype vehicle was equipped with both the radar and thermography equipment. The vehicle is driven slowly across the bridge deck and data are collected and stored on magnetic tape. Programs were written that would then retrieve and automatically process the data to produce a scaled plan of the bridge showing the location and type of deterioration present. DART can be used on exposed concrete decks and on concrete bridge decks covered with a bituminous wearing course.

The development of a systematic approach to bridge deck rehabilitation (1) created the need for considerable data on the condition of bridge decks in Ontario. Defects and deterioration need to be located for two principal reasons: (a) to establish priorities for rehabilitation, and (b) to determine the method of rehabilitation and prepare the contract documents.

For the first step, it is sufficient to determine only the approximate extent of any deterioration. The information is used to develop the future rehabilitation program and has traditionally been collected through a visual inspection together with a limited amount of physical testing. However, much of the deterioration can be hidden and go undetected. Second, an accurate measurement of the size and location of each type of deterioration is required. This information not only affects the selection of the method of rehabilitation, but also the quantities to be included in the repair contract. In the past, the data were collected through a detailed condition survey of the deck (2). Existing procedures require a thorough visual inspection supplemented by physical testing that includes a chain-drag survey and measurement of electrical potentials. Cores are taken and tested for chloride content, air-void analysis, strength, and sometimes, a petrographic analysis is made. The testing is expensive, and costs vary with deck size and location with an average of (Canadian) $12/m^2$ or about (Canadian) $6,000 for a typical bridge deck.

Despite this systematic approach, the information is sometimes of limited accuracy. This is especially true when the deck has a bituminous surfacing. Even though sections of the surfacing are removed at selected locations in the course of a detailed condition survey, it is difficult to determine the condition of an asphalt-covered concrete deck slab with any degree of confidence. A further disadvantage of the traditional methods of investigation is that the survey of a bridge deck usually takes a few days, and this results in a major disruption of traffic flow.

Research studies were undertaken to investigate improved methods of detecting defects in exposed concrete (3) and asphalt-covered bridge decks (4). The culmination of these studies was the development of the deck assessment by radar and thermography (DART) system. The results of the research studies are summarized in this paper and the prototype DART unit is described.

TYPES OF DETERIORATION

The most serious form of deterioration is that caused by corrosion of embedded reinforcement. As the reinforcing steel corrodes, it expands and creates a crack or subsurface fracture plane in the concrete at or just above the level of the reinforcement (Figure 1). The fracture plane, or delamination, may be localized or extend over a substantial area, especially if the concrete cover to the reinforcement is small. It is not uncommon for more than one delamination to occur on different planes between the concrete surface and the reinforcing steel. Delaminations are not visible on the concrete surface. However, if repairs are not made, the delaminations progress to open spalls and, with continued corrosion, eventually affect the structural integrity of the deck. Spalls on exposed concrete decks seriously impair the riding quality of the deck.

Scaling, which is the breakdown of the cement-paste matrix, is also a serious problem wherever it occurs. The disintegration of the concrete, which is caused by the freezing of concrete critically saturated with water, begins at the surface and gradually progresses so that the full depth of a deck slab may be affected. In Ontario, scaling most commonly occurs in older, asphalt-covered deck slabs built without the benefit of air entrainment or a waterproofing membrane.

On asphalt-covered decks, bond failure may occur

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FIGURE 1 Corrosion-induced delamination in a concrete core.

between the concrete deck slab and the bituminous surfacing. Debonding can result in moisture being trapped on the surface of the concrete and, where thin surfacings are used, can lead to failure of the bituminous surfacing. Although debonding is not as serious as either delamination or scaling, it can be confused with these two phenomena in surveys and, consequently, it is important to be able to identify and define debonded areas.

Cracking is the most common defect in concrete. However, with the exception of delaminations, cracks are usually easy to identify and were, therefore, not included in the research studies.

INVESTIGATIONS ON EXPOSED CONCRETE DECKS

The first studies were conducted on exposed concrete decks during the period 1977-1979 and were designed specifically to investigate methods of detecting delamination.

Most methods, including the use of a hammer or a chain, rely on the fact that a delaminated area produces a characteristic dull sound when the surface of the concrete is struck. These methods are tedious and depend on the skill of the operator. They can be difficult to use when a bridge deck is only partially closed to traffic and there is noise from vehicles in adjacent lanes. A machine was developed to eliminate the subjective judgment of the operator (5). It consists of three basic components: a tapping device, a sonic receiver, and a system of signal interpretation. However, the machine has only limited accuracy (3).

The detection of delaminations by infrared thermography is based on the difference in surface temperature that exists between delaminated and sound concrete under certain atmospheric conditions. The delaminations interrupt the transfer of heat into and out of the deck. Consequently, in periods of

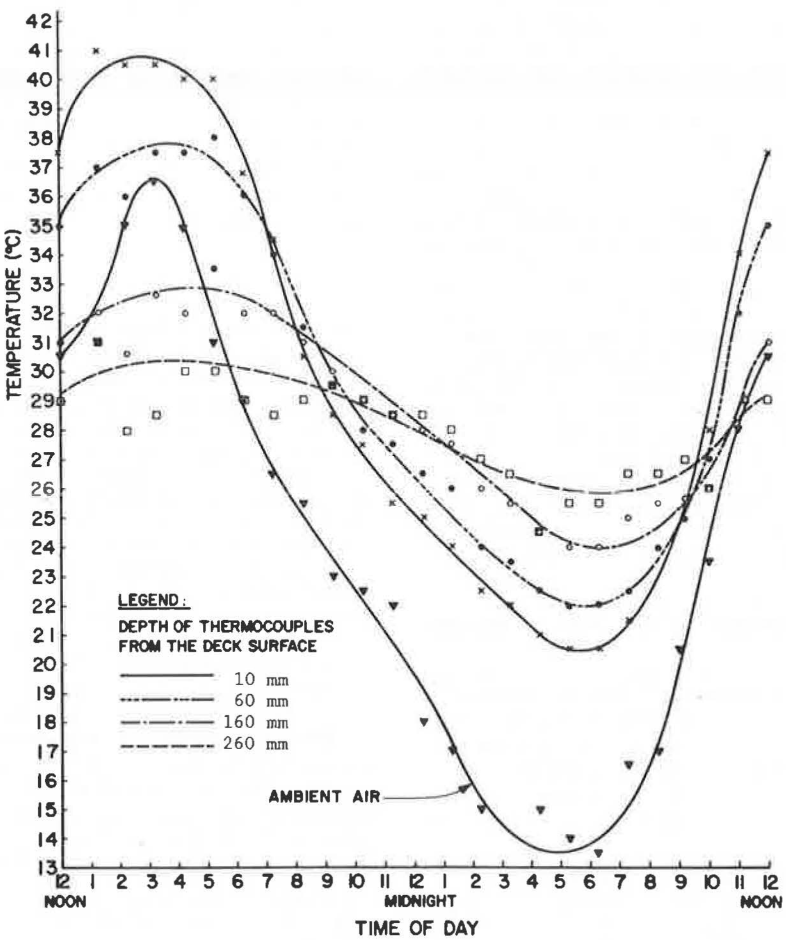

FIGURE 2 Typical diurnal temperature distribution in a thick slab deck in summer.

heating, the delaminated concrete heats more rapidly than surrounding areas of sound concrete and a difference in surface temperature develops. The reverse situation occurs during periods of cooling, usually during the night. Figure 2 shows a typical temperature variation during a 24-hr period in the top 240 mm of a thick slab deck with an exposed concrete surface on a summer day under clear skies. A substantial temperature gradient was recorded in the top 65 mm of the deck where delaminations occur most frequently. During the hottest part of the day, the concrete temperature decreased with the depth below the deck surface and at night the situation was reversed. Similar temperature distributions have been recorded on thin slab decks in both summer and winter, although the temperature gradient is less in winter.

Figure 3 shows the temperature variation in sound and delaminated concrete measured by thermocouples installed 6 mm below the surface. During the test period the difference in surface temperature between the solid and the delaminated concrete reached 3°C.

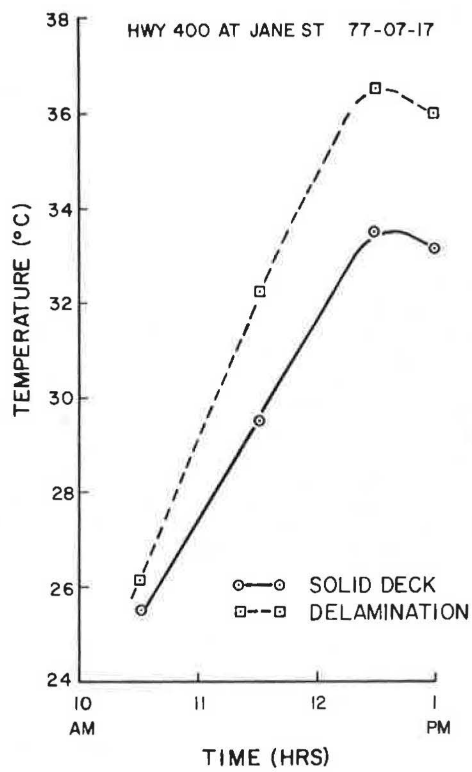

FIGURE 3 Difference in temperature between solid and delaminated concrete on a thin slab deck.

Infrared detection systems are used for the remote measurement of the surface temperature of an object. A typical system consists of an infrared sensitive scanner, a display monitor, and a power source. The typical temperature differences shown in Figure 2 are well within the capabilities of many scanners with a sensitivity of better than 0.2°C. The image from the scanner is displayed on a cathode ray tube and indicates the surface temperature of the object being viewed in a continuous range of gray tones from black to white. During daytime hours, when the delaminated areas are hotter than the surrounding solid deck surface, they appear as white areas against a dark background. A permanent record of the image can be made using an instant camera or a video recorder. Color monitors are also available but are not well suited to bridge deck applications because of the difficulty of recognizing the physical features on the deck (e.g., oil spots, debris in the curb area, or pavement markings) in the image. Furthermore, the types of deterioration present are generally not associated with a single color level.

The first series of tests was made at ground level using targets to locate the position of the image (Figure 4). Although the delaminations were easily identified, the method was impractical because of the limited field of view and the difficulty of constructing a plan of the deck from photographs taken at an oblique angle.

FIGURE 4 Infrared thermovision equipment being operated at ground level.

Airborne testing was undertaken using a scanner mounted in a helicopter. Although this method had the advantage of not requiring lane closures, the quality of the image was substantially reduced. The use of the helicopter was also complicated by the requirement to obtain a waiver of air regulations to fly at low altitude over the bridge decks. The best compromise between these two extremes was the use of a vehicle-mounted scanner, which resulted in an acceptable field of view and good definition of the delaminations.

INVESTIGATIONS ON ASPHALT-COVERED DECKS

During the period 1980-1982, the work on exposed concrete bridge decks was extended to a much more detailed evaluation of methods for investigating the condition of asphalt-covered decks. The objective of this research was to identify reliable methods of defining the type and extent of defects and deterioration. It was also desirable that the methods be rapid, inexpensive, noncontact, and capable of having the data transcribed to a scale plan of the bridge deck using automated equipment.

A test site was created by selecting a typical bridge that exhibited corrosion-induced distress. This deck was surveyed and then paved with two 40-mm thick lifts of bituminous surfacing without first making repairs. Areas of scaling and debonding were simulated before paving (4). Consequently, the locations and type of deterioration were known and the capabilities of different test methods could be evaluated under controlled conditions.

Eight tests methods were investigated (4) and the results obtained are summarized in Table 1. The most promising techniques were found to be infrared thermography and impulse radar.

Thermography

Several commercial infrared systems were tested at ground level from a boom truck and, in some cases, from a helicopter. This work confirmed that the truck was the most practical platform in terms of accuracy, cost, and speed. The optimum height above the deck was in the 4- to 6-m-range to provide the best definition of delaminated areas with the least interference from reflected radiation, and to enable the full width of the traffic lane to be investigated during a single pass.

Temperature measurements showed that a delamination in the concrete deck slab produced a difference in the surface temperature of the bituminous surfacing during periods of heating. As with the exposed concrete decks, this occurs because the delamination interferes with heat flow through the deck and a higher surface temperature is associated with areas of delamination. However, the maximum difference in surface temperature recorded was 2°C at 2 p.m. and the time of day when delaminations could be identified was limited to 11 a.m. to 6 p.m. The ability to detect delaminations on asphalt-covered decks was also found to be much more sensitive to atmospheric conditions such as wind, humidity, and cloud cover than on exposed concrete decks.

Despite the dependence on weather conditions, the results using thermography to detect delaminations were very encouraging. The system detected more than 90 percent of the known delaminations, some of which were less than 150 mm in diameter. Debonding was not detected because it did not produce a thermal discontinuity. The inability to detect scaling was ascribed to the fact that the areas of simulated scaling were all adjacent to the curbs and any difference in surface temperature was masked by the difference in emissivity between the asphalt deck surface and the concrete curbs. The major technical problem to be overcome was identified as the production of scaled hard copy from the image stored on videotape.

Radar

The use of low-power, high-resolution, ground-penetrating radar for detecting deterioration in concrete bridge decks is a relatively new technique first reported in 1977 (6). The equipment consists of a monostatic antenna, a control console containing a transmitter and receiver, and an oscilloscope. A 1-nanosecond pulse of low-power, radio-frequency energy is directed into the bridge deck. A portion of the energy is reflected from each interface between different materials including the asphalt-concrete interface, the surface of reinforcing bars, and air-filled or water-filled voids associated with defects such as delamination, scaling, and debonding. These echoes are received by the antenna and displayed on the oscilloscope.

For the purposes of the investigation described here, the equipment was mounted on a cart pushed by hand along the deck. The waveforms were recorded at grid points using an instant camera. Figure 5a shows a typical radar signal for a sound portion of the deck and Figure 5b shows the signal for a section known to be delaminated. In areas where the concrete is deteriorated or the character of the interface changes, the amplitude and time of the echo also change.

Using a simple visual assessment of the waveforms, 51 percent of the grid points located over delaminations and the simulated scaling were identified. The areas of debonding were not identified and false indications of delamination at several grid points were made. Despite these results, the radar was found to have considerable potential if the interpretation of the waveforms could be improved. The hand-operated cart was rather crude and it was apparent that the next step in the development of the radar system should be vehicle-mounted equipment in which the signal is recorded continuously on magnetic tape for off-line processing, using software specifically developed for the purpose. A useful feature of the radar is that it permits an accurate measurement of the thickness of the asphalt surfacing because of the well-defined echoes from the surface of the asphalt and from the asphalt-concrete interface. Except for the presence of moisture on the deck, the radar is independent of constraints by the weather.

THE DART SYSTEM

Following the completion of the research studies described earlier, development work concentrated on the construction of a prototype unit and automated data processing techniques. The prototype unit was to have the following operational characteristics: (a) be self contained, (b) produce real-time results, and (c) be simple to operate and offer safeguards to ensure that data are useable before the unit leaves the bridge.

TABLE 1 Summary of Results of Procedures Evaluated on Asphalt-Covered Decks

Test Procedure	Evaluation Summary
Chain drag	Only a small percentage of delaminations identified, but no false results. Independent of weather and inexpensive. Useful screening device in conjunction with other procedures.
Sonic reflection	Very low accuracy.
Ultrasonic transmission	Impractical.
Microseismic refraction	Identified anomalies but interpretation difficult. Procedure is very slow.
Resistivity	Results did not correlate with area of deterioration.
Potential survey	Useful indication of corrosion activity. Does not identify other forms of deterioration. Requires drilling through asphalt.
Radar	Good correlation with known deterioration. Many false results but accuracy could be improved by better methods of signal interpretation. Offers potential for rapid, noncontact procedure independent of weather.
Thermography	Excellent correlation with areas of deterioration with no false results. Main disadvantage is dependence on weather.

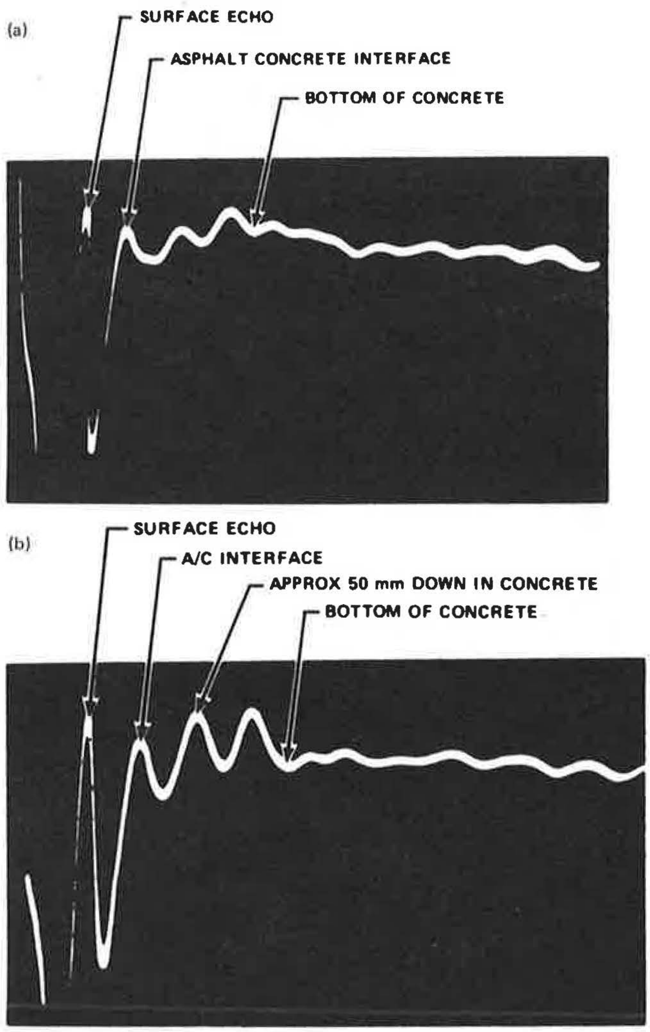

FIGURE 5 Typical radar waveforms: (a) radar output for sound concrete, and (b) radar output for delaminated concrete.

Prototype Unit

The prototype unit was designed to incorporate both the radar system and the infrared system with their respective peripherals. The vehicle dedicated to the system was equipped as follows:

1. A rail was attached to the front of the vehicle so that the radar antenna could be positioned 150 mm over the deck anywhere within the width of the vehicle. Note that the antenna is normally mounted 150 mm over the deck surface and can be mounted as high as 600 mm over the deck. The 150-mm position gives a 300-mm wide survey of the surface, combined with good penetration and resolution of the radar. The rail also allows for the future addition of antennas to scan additional areas of the deck at a single pass.
2. A hydraulically lifted rack for mounting the infrared scanner approximately 5 m above the deck surface was added to the front of the vehicle. A standard video camera to view the real-life condition of the pavement surface can also be mounted on the rack. The rack also ensures the repeatability of the height and angle of view of the scanner and video camera.
3. Vibration-controlled racks were mounted in the vehicle. Although the radar system and the infrared scanner were designed for use in the field, the peripheral equipment and the computer were not. In order to protect the equipment, instrumentation racks were installed in the vehicle. A series of vibration tests was carried out to evaluate the vibration characteristics of the vehicle. Isolation mounts were installed for each of the racks to minimize the vibrations being transferred to the equipment.
4. Two air-conditioning units were added to the vehicle to help keep the environment stable and prevent the breakdown, as a result of excessive heat, of the electronic equipment.
5. A fifth wheel was added to the vehicle. The fifth wheel gives accurate information on the speed and the distance traveled. An interface was built to produce distance pulses to the recording devices for both the radar and the infrared systems.
6. A trailer was built to house a generator to supply electrical power to the instrumentation on board the vehicle. In addition to the generator, the trailer can also carry such equipment as safety signs and a core drill.

Figure 6 shows the exterior of the vehicle with the radar antenna and the infrared scanner in position. A schematic representation of the infrared and radar components is shown in Figure 7.

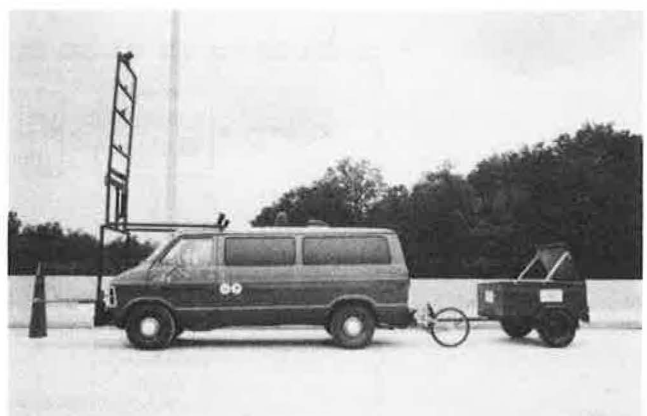

FIGURE 6 DART system with radar antenna and infrared camera mounted on the front of the vehicle.

Radar

The radar signal passes from the control unit inside the vehicle through the transmitter/receiver mounted on the front of the vehicle, through the antenna, and into the bridge deck. The return echoes are received by the antenna and returned to the control unit inside the vehicle. The waveform is simultaneously displayed on an oscilloscope and recorded on a seven-channel FM tape recorder. The operator can monitor the signal from the control unit and from the tape recorder, thus permitting a check of the quality of the signal before and after recording. As well as recording the raw waveform, the trigger pulse for the system, a gated version of the waveform, and three sets of distance information are recorded for analysis using a microcomputer. The radar itself is battery powered and has an operating time of approximately 6 hr on a charge.

The output of the fifth-wheel distance device is put through an interface to produce three sets of distance pulses; the raw 214 counts per revolution,

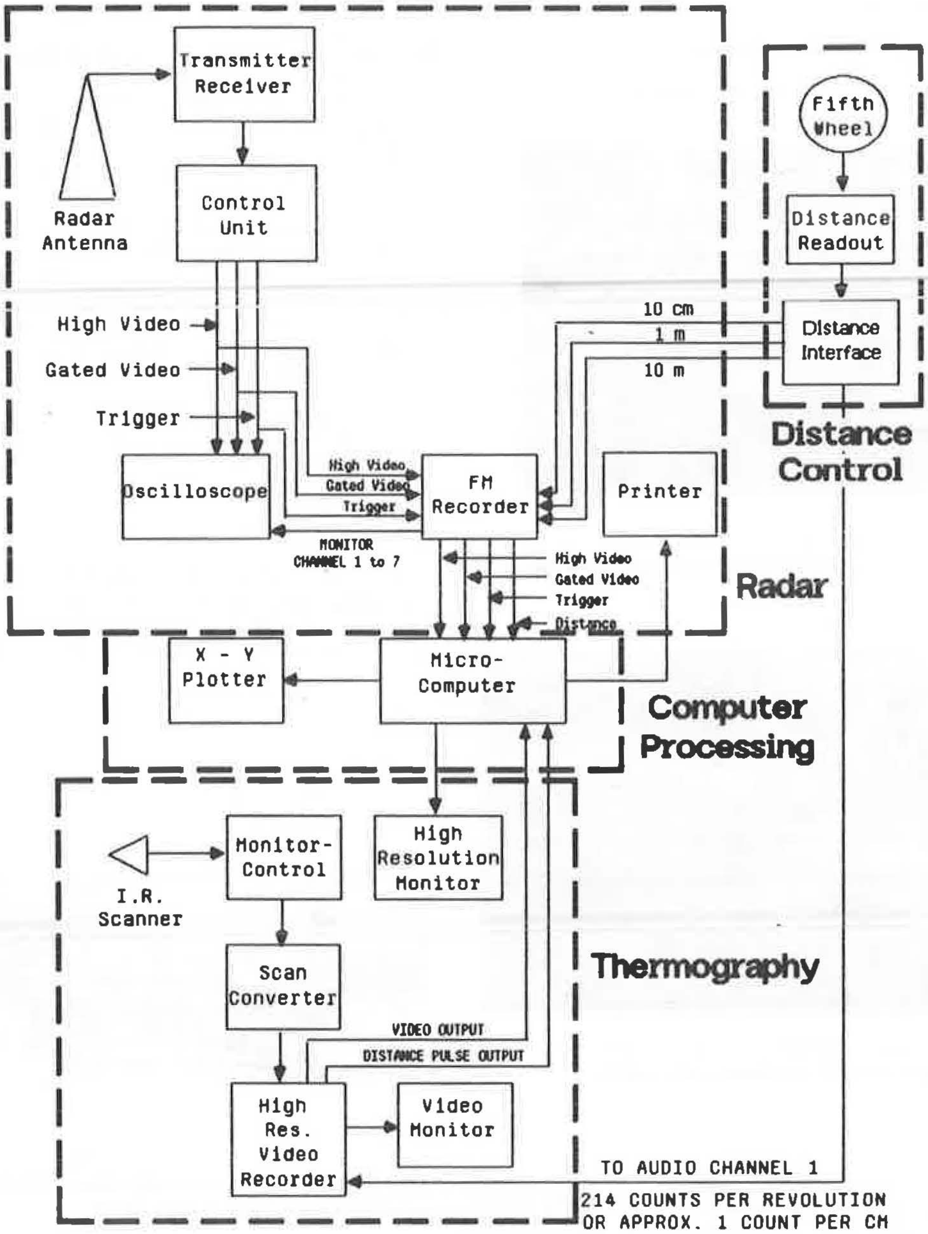

FIGURE 7 Schematic representation of the DART system.

a pulse every meter, and a pulse every 10 m. Each of these is used in the processing of the radar data for various program options.

Initially, the radar system is mounted with the antenna positioned within 1 m of the curb or the right-hand edge of the lane. A second pass is made with the antenna positioned between the wheel tracks, and a third pass is made with the antenna positioned adjacent to the left-hand edge of the lane. The maximum speed for the collection of data is 5 km/hr.

The monitoring checks built into the system allow the operator to verify the data collected before leaving the bridge. Should the operator observe problems with the data or observe significant deterioration in the bridge deck, a pass can be repeated or additional passes can be made over the deck.

Infrared System

The infrared system used on the DART prototype senses in the 3.5- to 5.6-μm range using an indium antomonide (InSb) detector. It has a sensitivity of approximately 0.1°C. Using a 20 degree lens, the system is capable of scanning an entire traffic lane width in a single pass. The scanner is connected through the scanner control unit to a proprietary interface unit to convert the scanner signal to a standard North American Television Standard Color (NTSC) video signal. The output of the interface is then fed to a high resolution videotape recorder.

The operator monitors the raw infrared image on the scanner control unit, on a monitor from the interface output, and from the output side of the video recorder. This enables the operator to ensure that the quality of the data collected is adequate and that the scanner is functioning properly.

The output of the fifth wheel is recorded on one of the audio channels on the video recorder. This enables the microcomputer to process the data and identify the longitudinal position on the deck. Data can be collected at speeds up to 6 km/hr, which is compatible with the maximum speed for collection of the radar data.

A standard color video camera mounted on the rack below the infrared scanner is used to record the visual defects in the deck surface. This information is recorded on a separate video recorder, along with the distance pulses, so that surface defects also visible to the infrared scanner can be interpreted properly during the analysis of the data.

Data Analysis Using Computer Processing

As noted, one of the main objectives of the DART system was to have the ability to produce an accurate and rapid evaluation of the deck condition. During the development phases of the work, it was apparent that the use of a computer would enable the operator to make rapid judgments as to the condition of the deck. In addition, it was apparent that the massive amounts of data produced by the radar system would require the use of computer processing to produce timely and accurate results. Two research contracts with McMaster University in Hamilton, Ontario, resulted in the development of software that processes the radar waveforms automatically (7). The program retrieves the following data from the radar waveforms: (a) location of delaminations, (b) location of debonded areas, (c) location of scaled areas, and (d) thickness of the asphalt overlays. Each of these is presented in the computer printout in the form of a deck plan with the type and extent of the deterioration along each grid line. The thickness measurements are tabulated and also averaged for each grid line, and an average for the entire deck is given.

The initial programs were written for use on a mainframe computer. Although this satisfied the goal of computer processing, there was a need for inexpensive data processing and real-time processing in the field. The second of the McMaster contracts involved the use of a microcomputer and investigated the effects of parameters not included in the first study. For this purpose, a microcomputer was equipped with a 10-megabyte hard disk, 640-K RAM, a single floppy disk drive, and a high-speed analog-to-digital interface. The original software was reconfigured to run on the microcomputer and the processing improved to the point that, with the computer on board the vehicle, some real-time processing was achieved.

Software has also been developed to utilize the microcomputer to analyze the infrared image. First, the videotape is digitized and then sampled so that the oblique angle to view and other distortions of the image are eliminated. Defects are identified through a combination of operator and machine interpretation. The software produces a scaled image of the deck showing the areas of delamination and scaling. Debonding can only be predicted with certainty if it covers large areas of the deck. If the debonded areas are small in area, then their infrared signature is very similar to that of delaminations.

The DART system represents a significant investment. The value of the major items is summarized next; however, because of duties, fluctuations in exchange rates, and volatility in the price of electronic components, the following costs (in 1985 U.S. dollars) should be regarded as approximate: radar $40,000, thermal scanner and converter $40,000, microcomputer and peripherals $16,000 (including extra circuit boards), FM recorder $10,000, industrial VCR $4,000, fifth wheel $5,000, oscilloscope $1,400, and generator $1,300. Adding the cost of the vehicle, trailer, external and interior racks, and numerous smaller items such as cables, filters, and interface devices would make the replacement cost of the fully equipped DART vehicle approximately U.S. $135,000. It should be noted that the Ontario Ministry of Transportation and Communications did not purchase the thermal scanner and converter, but rented the equipment during the summer months for (Canadian) $2,400 per month. In addition, the cost of developing the software was (Canadian) $62,000 for the radar, and (Canadian) $20,000 for processing the thermal images.

Summary

The DART system combines the use of radar and infrared thermography to take advantage of the complementary nature of the two technologies. It also provides a useful check on the validity of the data from two different techniques.

The advantages of the radar system are that it is virtually independent of weather conditions, is well suited to use on asphalt-covered decks, and can identify some defects undetectable by infrared thermography. It is less well suited to use on exposed concrete decks because of interference between the waveform reflected from the deck surface and any delamination just below the surface. This is not a problem on most asphalt-covered decks in Ontario because the standard asphalt thickness is 80 mm. The major disadvantage of radar is that it only produces data along the grid lines traversed by the antenna and, unless several passes are made, areas of deterioration can be missed. Conversely, thermography can be used wherever differences in surface temperature exist to produce information on the entire deck surface and not just along grid lines. Its major disadvantage is dependence on the weather, especially for asphalt-covered decks. Clearly, the two systems complement each other extremely well.

FUTURE RESEARCH

The next stage in the development of this system is to operate the prototype unit under field conditions in 1986 to assess its accuracy and reliability. Initially, it will be used to investigate a large number of bridge decks in order to assess priorities for rehabilitation. The unit may be particularly well suited to this application because only limited accuracy of the data is required. As experience is gained in the use of the equipment, it is expected that a number of the activities now included in detailed condition surveys can be eliminated. The potential exists to carry out detailed surveys using the DART system supplemented by a small amount of physical testing and sampling. This is expected to lead to improvements in the accuracy of the data, reduction in survey costs, and less disruption to traffic.

Other studies will be undertaken to identify other applications for the DART system such as the condition of joints in pavements, voids under bridge approach slabs, and the moisture content and degree of consolidation in subgrade materials.

ACKNOWLEDGMENT

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Considerations for Administering Underwater Contracts

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ABSTRACT

The objective of this study was to identify the considerations for administering an underwater inspection program to be conducted by contractors. Issues include identifying and assigning a priority to structures for periodic inspection, establishing inspection procedures, selecting a contractor, formatting the contract, and estimating contract costs.

National bridge inspection standards require that all bridges located on public roads be inspected at least once every 2 years. The inspections are to be conducted in accordance with the AASHTO standards stated in the Manual for Maintenance Inspection of Bridges (1). In general, highway and transportation departments nationwide comply with these standards; however, many states do not have a program for routinely conducting underwater inspections (2). The Virginia Department of Highways and Transportation is attempting to strengthen its underwater inspection program through the efficient use of contractors.

The objective of the research reported here was to identify those aspects of underwater inspections that are necessary for an efficiently administered underwater inspection program, and can be specifically stated in a contract.

Meetings and interviews were conducted with personnel responsible for bridge inspections in the Virginia Department of Highways and Transportation, other states and federal agencies, and with contractors. The issues identified for consideration in administering contracts for underwater inspections are discussed here in a general manner, and it is anticipated that they will be modified, specifically by traffic engineers, structural engineers, economists, and those experienced in bridge inspections.

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IDENTIFYING AND ASSIGNING A PRIORITY TO STRUCTURES FOR UNDERWATER INSPECTIONS

Based on information available on their maps, many states appear to have responsibility for more bridges with substructures underwater than can be inspected in a short time; therefore a system of assigning priorities to upgrade inspection programs to include structures underwater is needed. The system would not be used to decide what bridges would be inspected, but to determine the order in which all bridges would be inspected during a given time period. Some of the variables that appear to be essential to such a system are discussed.

Risk Assessment

The importance of risk assessment stems from the need to provide safety for the users of the structures. Although the safety of all structures is important, those that would pose the greatest risk to the public in the event of failure must be distinguished from those that present less risk. Probable risk to the public is evaluated from traffic volume. Assuming the worst case, such as bridge failure, the greater injury probably would be sustained by the users of bridges with high volumes of traffic. Therefore, if traffic volumes were the only element to be considered, these bridges are of more concern than those with low volumes.

Structural Data

Although little historical information exists for accuracy, inspection priority is assigned by considering the types and conditions of structural elements. The elements considered are construction materials, quality of construction, foundation type, structure age (or remaining life), and moveable versus stationary spans discussed next.

Construction Materials

Depending on the type of water in which the substructure rests, the priority of inspection is affected by the type of materials involved (concrete, wood, or steel). For example, wooden structures in salt water are vulnerable to borer attack, concrete is susceptible to leaching of chemicals in the soil at the mud line (such as high sulfur), and steel would be subject to oxidation.

Quality of Construction

Engineering judgment is essential in rating the quality of construction. If this cannot be determined from data recorded when the structure was built, information from inspections of the superstructure can be used.

Foundation Type

Pilings constructed on rock foundations are not as adversely affected by scour as are friction piles. Friction piles would be weighted higher in the priority ranking than bearing piles, especially when scour is likely.

Structure Age

A life expectancy of 50 years has been arbitrarily assigned to bridge structures; thus, older structures should receive a higher priority.

Movable Versus Stationary Spans

The added risk of damage by boat and ship traffic under movable spans indicates a need to assign them a higher priority than stationary spans. An added risk is the turbulence from propellers of large vessels, which may cause "necking," a form of deterioration in sections of a pier.

Environmental Factors

The environment in which a bridge is located affects the demand for inspection. Weather, velocity of water flow, and water chemistry are variables that should be considered.

Over a period of time, cycles of freezing and thawing temperatures could result in significant damage to a substructure. Bridges in areas of the state where water commonly freezes in the winter should be assigned a higher priority than those in areas where temperatures rarely drop below freezing.

A substructure is more adversely affected by a rapidly moving stream than by calm water. Problems are also more likely to occur in areas of frequent flooding. The substructure would be vulnerable to undercutting by high velocity flows, cracking from large debris moving rapidly in flood waters, and scour.

Substructures are more adversely affected by salt water than by fresh water. Because the probability of spalling, corrosion from electrolysis, and so on, is greater in salt water, the structures there should be assigned a higher priority.

Economic Considerations

In addition to the safety of the traveling public, the protection of capital investments is a high priority. The inspection of bridge substructures facilitates preventive maintenance involving relatively inexpensive rehabilitation procedures that include costs of replacing the structure, repair, and detour length, and costly reconstruction is avoided. Economic considerations include any losses incurred by the public resulting from a structure being out of service.

When deferred maintenance necessitates replacement, structures with a high replacement cost obviously would be assigned a higher priority for inspections than structures with a lower replacement cost.

The cost of repair is slightly different from replacement cost. Considering only underwater operations, the resources needed to repair or reconstruct a structure having a moderate replacement value could be more costly than those needed to repair a structure with a higher replacement value. For example, repairs to a two-lane stationary span over very deep water would be extremely expensive because of the requirements for highly trained personnel and special equipment. In contrast, where the substructure of a bridge carrying a multilane highway is partly in shallow water, repairs may be less costly. Consequently, the former situation justifies more frequent inspections to detect minor distress and prevent the development of major problems.

The length of the detour required is important in a case in which a bridge is out of service. If the bridge provides the only reasonable route of travel to a given location, then it would be weighted higher than if it were one of several in the area.

Structural Evaluation

The service and maintenance history of a structure is important in assessing the need for immediate inspection. In many cases there are no documented underwater inspection files for the structures, although information from inspections of the superstructures is available and can be used in determining priorities.

DEVELOPING INSPECTION PROCEDURES

Levels of Inspection

Levels of inspection are used by the U.S. Navy and most underwater contractors to generally define the work of an inspection.

• A Level I inspection is a basic inspection (a swim-by) and does not entail cleaning or detailed measurements. The objective is to gather data based on observations (visual, photographic, or videotaped). The Level I inspection should follow the as-built plans of the structure with the intention of detecting obvious major damage or deterioration due to overstress, corrosion, or extensive biological growth or attack. This level of inspection is intended to be part of an initial evaluation of the exterior surface of piers, pilings, footings, and so forth.

• A Level II inspection obtains more information than is provided by the Level I and may involve cleaning and simple measurements using calipers, measuring scales, and probes or ice picks to estimate the depth of cracks or other damage. At times, more sophisticated measurements are required at Level II. For example, if simple measurements indicate a potential problem, a few detailed measurements may help to confirm this indication.

• A Level III inspection is highly detailed. Nondestructive techniques (such as coring), material sampling, and in-place surface hardness testing may be required. Commonly, the Level III inspection will require cleaning preparatory to conducting tests, and obtaining photographic or video representations.

Contractor Tasks

The types of tasks to be performed by a contractor conducting an initial inspection and a follow-up inspection are described. More detailed inspection procedures are given in the North Carolina Department of Transportation Underwater Inspection General Operations Procedures and Safe Practice Manual, compiled by the Bridge Division of the North Carolina Department of Transportation (3).

Initial Inspection

Initial inspections are usually slated for bridges or groups of bridges that have documentation of previous inspections. A Level I inspection would be conducted to note any obvious defects such as extensive spalling or scour. (Follow-up inspections would be scheduled where necessary.) At this level of inspection, a group of structures could be quickly evaluated to establish baseline data.

Three areas of the structure should be observed in a swim-by inspection: (a) the area around the mean water level to detect damage from cycles of freezing and thawing or from boat collisions; (b) the areas from the mean water level down to the mud line, at every 10-ft interval and around the circumference of the pier; and (c) the area at the mud line. The data from a mud line inspection would include condition of footing, extent of scour, the amount of debris collected around the pier, the condition of underground cables, and, if appropriate, soil samples from the mud line for chemical analysis.

Follow-Up Inspections

Follow-up inspections will be either Level I or Level II, depending on the results of any previous inspections. The purpose of a Level II inspection would be to gain detailed data. Usually, this involves light cleaning with steel brushes or scrapers and photographic or video documentation. The use of a computer program would facilitate the evaluation of structures and the scheduling of future inspections.

When inspections indicate possibly serious damage, cleaning and testing may require use of a water blaster with water applied to the structure under pressures ranging from 6,000 to 15,000 psi. At 6,000 psi, the jet would clean marine growth, and pressures near 15,000 psi would reveal loose or damaged material. Pressures above 15,000 psi could cause damage to strong concrete. It is important that the contracting parties agree with and document pressures used.

Color video is desirable for inspections when damage is suspected or when an initial inspection has indicated potential damage, or for documentation for reference. The use of color video enables an engineer on the superstructure to observe conditions below the water. In many circumstances, a diver who becomes "task fixated" will see only what is directly in front and miss obvious details. With the aid of color video, an engineer on the surface can communicate with and guide a diver. The video film can be retained for analysis and documentation.

Sampling

Inspections are necessary to provide the data for making decisions that will protect the users of structures and an agency's capital investments. Inspecting the entire portion of the structure underwater would provide the most reliable data; however, because of limited resources, total inspections are not always possible. The problem is to develop a valid sampling model for inspections of bridge substructures underwater.

There is little literature from research on this subject and no valid sampling formula is available. The main difficulty in developing this formula is that of determining the required size of the sample population. In addition to the variables that relate to the structure, such as age, material, and construction quality, environmental factors that affect the structure must be considered. To determine that all the piers in a given structure are homogeneous enough to constitute a population, at least additional variables of scour, damage from collisions, and freeze-thaw damage must be considered.

The results of a literature search indicate that there is not enough information available to validly state that all piers in a given group are affected in a predictable manner. It is improbable that a population could be defined based on available data.

If sampling is unavoidable, the worst case approach is suggested based on the response to this question, for example: What number of elements in a given structure could be eliminated without the probability of the structure failing? Next, the remaining elements should be inspected.

USE OF A CONTRACTOR

Selecting a Contractor

The regulations governing the use of contractors are spelled out in the Virginia Department of Highways and Transportation's DPM 6.8 (4). Usually, all contracts more than $10,000 must be awarded by competitive sealed bids or by competitive negotiations. Contracts of an emergency nature and single-term contracts of less than $10,000 are exceptions. The important factor in issuing contracts is to ensure that those bidding are qualified to perform the task.

Competitive negotiations appear to be more advantageous than sealed bids for underwater work. Many times, the tasks to be performed can be specifically stated; however, the options available to perform these tasks are not always clear to the contracting agency. In negotiating a proposed contract, the guidance of a potential contractor may increase the quality of the inspection and benefit all parties.

The qualification of an underwater contractor is especially important because the work performed is out of sight. Several factors should be considered when attempting to prequalify potential contractors. The following factors are discussed based on information received from the Naval Facilities Engineering Command located at the Navy Yard in Washington, D.C., and several underwater contracting companies.

Contractor Experience

A contractor experienced in underwater inspections is able to assess existing structural damage and accurately predict potential damage from data obtained. This is especially important on Level I inspections, because the diver is the only one to observe the structure. The diver's ability to describe his observations to a large degree determines whether the engineer in charge of the inspection declares the structure sound or calls for a Level II inspection. Contractors whose primary activity is underwater inspections should be distinguished from those that engage only in underwater construction or salvage. The latter should not be eliminated, but should not be accepted solely on the basis of having performed underwater work.

Contracting firms that routinely conduct bridge inspections employ structural engineers and draftsmen, but subcontract to a diving firm for underwater inspections. Because most highway and transportation agencies have highly qualified structural engineers, for efficiency, they should work directly with the firms that perform the underwater operations.

Personnel Qualifications

In most cases, for their own benefit, contractors engage divers who they believe to be competent. The most important consideration is the diver's experience: the number of divers made, number of hours spent under water, type of training, type of work performed, and recency of work.

Available Equipment

The equipment to be used by the contractor should be stated and the availability of that equipment should be verified before the issuance of a contract. Attention to these details ensures that the contractor and his employees have experience with the equipment and that work will not be delayed because the equipment cannot be obtained.

Establishing the Content of Contracts

From the information gained in the research reported here, the following outline of considerations to be contained in an inspection contract has been developed. Although highway and transportation agencies have a standard contract form, these considerations can be incorporated with little modification.

General Requirements

The general requirements state the objectives of the project. For example, the requirement might be to establish the general condition of all bridge substructures from 2 ft above the water line to the mud line, or to inspect a given location for possible damage resulting from boat or ship collisions. Requirements may also specify the capabilities the contractor must possess to perform underwater inspections, assess damage and deterioration, recommend repair techniques, and estimate repair costs. In addition, the estimated maximum length of time for completion of the project may be stated.

Administrative Procedures

The usual information such as channels of communication, information-reporting schedule, and submittal of vouchers is usually contained in this section of a contract. Especially with underwater contracts, task-oriented conferences between contractors and engineers-in-charge are important. The objective and frequency of these conferences should be stated.

Although the contracting agency should not specify how diving operations will be carried out, it should make a general statement about expected safe-diving practices. For example, it could state that a thorough check of underwater conditions, as well as other conditions pertaining to the proposed work, will be made before all diving operations, and that all diving operations will be conducted in accordance with the best commercial safety standards.

General Criteria

This section contains a brief statement that the contractor is responsible for the quality of submittals, including editing, accuracy of figures, and reproduction.

Study and Analysis

The level of inspection required usually is not explicitly stated but is worded in the form of a guideline. The study and analysis section should include the extent to which the data gathered will be analyzed. In almost all underwater inspections, an analysis must be made by the contractor because in the initial swim-by divers must decide what is significant and what is not. However, repair and cost analyses may not be desired, and this should be stated.

Specifications for on-site reporting should be stated. Some type of daily log should be maintained. Information such as the locations of all observations showing elevations along each pier or pile, water depth referencing mean low water level, and the position of the pier or pile on the bridge should be recorded.

Report Format

The contents and the format of the inspection report are important because the report contains the data to be used in future studies and in scheduling follow-up inspections.

Estimating Costs of Contracts

The calculation of a reasonable estimate of the cost for inspecting a given facility is difficult because of the variables unique to each structure. However, based on cost estimates contained in contracts awarded by the Naval Facilities Engineering Command and discussions with railway agencies and contractors, a daily average manpower rate of about $500 a day can be estimated for one dive team. Variables associated with the size and location of the structure will obviously affect this average.

Cost items that routinely vary from structure to structure are those for overhead, travel or per diem, equipment rental, and transportation. Unexpected variables, such as the need for emergency services and poor weather conditions, may generate additional costs. The extent to which the contracting agency provides bidders with accurate information, such as that on water depth, will determine the accuracy of estimates included in the bids.

ACKNOWLEDGMENT

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The opinions, findings, and conclusions are those of the author and not necessarily those of the sponsoring agencies.

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The Pennsylvania Bridge Maintenance Management System

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ABSTRACT

A Bridge Management Work Group has been organized to develop, as well as to test and implement the concepts and requirements of a total bridge management system (BMS) for Pennsylvania, using highway planning and research (HPR) funding. An electronic data processing (EDP) contractor will develop the software using other sources of funding. The system is scheduled to be fully operational by April 30, 1987. The objectives of the bridge maintenance management portion of BMS are to (a) utilize standardized bridge maintenance activities and costs, (b) store activity needs on a bridge-by-bridge basis, (c) rank activities and assign a priority to bridges for maintenance programming, (d) transfer programmed projects to the maintenance division's programming and scheduling system, and (e) store cost of completed work. The work group has the responsibility for development of a comprehensive system that (a) integrates and utilizes data from the existing structure inventory records system (SIRS) and other data bases, (b) enhances and expands the SIRS data base, (c) systematically evaluates the deficiencies and associated costs, (d) records maintenance and construction-cost history, (e) stores physical attributes of each bridge for the semiautomatic structural analysis to determine load rating, and (f) yields a spectrum of information designed to enable cost-effective management of the bridge system.

A seven-member Bridge Management Task Group was convened by Pennsylvania Secretary of Transportation Thomas D. Larson in 1983-1984 to consider the development of a bridge management system (BMS) for the Commonwealth of Pennsylvania. In its report, the group unamimously agreed that the development of such a system was feasible and a very important and urgently needed tool for better management and engineering of the state's large and antiquated system of bridges (1).

Highway planning and research (HPR) funding was secured for a work group of nine to develop the concepts, technical requirements, pilot test, and guide statewide implementation of a total BMS under Research Project 84-28. This funding covered a 12-month period from August 1, 1984 to July 31, 1985. The work group consists of five Pennsylvania Department of Transportation (PennDOT) employees and four consultants. Richard M. McClure, chairman, Pennsylvania State University; David A. VanHorn, vice-chairman, visiting scientist from Lehigh University; John M. Kruegler, consultant, formerly with FHWA; Oliver J. Weber, consultant, formerly with Bethlehem Steel; Ronald C. Arner, District 3-0 bridge engineer; Hasmukh M. Lathia and Jeffrey J. Mesaric, Fiscal and Systems Management Center; Kantilal R. Patel, Bureau of Bridge and Roadway Technology; and Jonathan D. Oravec, Center for Program Development and Management. Heinz P. Koretzky, chief, Bridge Management Systems Division, Bureau of Bridge and Roadway Technology, served as the project coordinator/manager.

The work group prepared a report that formed the basis for a request for proposal to develop software for BMS (2). The electronic data processing (EDP) contractor is to provide the development, testing, implementation, and training on the use of EDP software. Software development by the EDP contractor is being performed using other sources of funding.

The formulation of a bridge maintenance management subsystem and its integration with PennDOT's maintenance operations and resources information system (MORIS) is an important component of the overall BMS development effort.

HPR funding has been approved for the work group to continue development of BMS under Research Project 84-28A. This funding will cover a 21-month period from August 1, 1985 to April 30, 1987. The complete development of BMS, including all software and implementation is scheduled for completion during this period. At the end of this time, BMS will be operational statewide.

CURRENT SYSTEM

In the past, bridge maintenance has been generally treated as an incidental component of highway work similar to storm sewers, guide rail, and other appurtenances. Although the needs for repairing and preventively maintaining a roadway and associated features are apparent, bridge maintenance needs are more elusive. Potential problems must frequently be sought out by a trained inspector. When found, the repair treatment or, for that matter, its urgency or effect on the structural safety of the bridge, is often not obvious to the highway maintenance manager. Therefore, it is understandable that highway maintenance management systems use obvious and generalized broad activities to describe bridge work. Bridge maintenance activities included in Pennsylvania's

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current highway maintenance management system (HMMS) include the following:

- General maintenance: deck repair, structure (repair), preventive maintenance (cleaning), preventive maintenance (joint sealing), and preventive maintenance (spot painting).
- Betterments and contract maintenance: bridge painting, deck rehabilitation, structural rehabilitation, deck repair, and structure repair.

Although these activities detail the extent of bridge maintenance definition in PennDOT's current maintenance management system, many more activities are available that more definitively describe roadway work.

It is a common perception that the maintenance repair and betterment budget is heavily weighted toward providing a roadway surface that satisfies the public's expectations for riding quality, skid resistance, and year-round utility. Bridge repairs generally result in the expenditure of relatively large sums of money in a small concentrated area. Frequently, the traveling public can detect no significant change in appearance between the original and the repaired facility.

The lack of sufficient data to be able to perceive, quantify, and assign a priority to the maintenance and betterment needs of the overall highway system has in large part resulted in the allocation of funds to those areas where the needs are most visible. This, coupled with past revenue crunches related to the fuel crisis and recessionary periods, has resulted in a large backlog of bridge maintenance and betterment needs. It has also resulted in an ever-increasing magnitude of need on each bridge. In many cases, degradation of the bridge advances to the point that extensive rehabilitation or replacement becomes necessary by the time construction funding is available.

THE PENNSYLVANIA BRIDGE PROBLEM

Pennsylvania has 100 percent of the bridges on the state highway system and about 95 percent of the local (nonstate) highway system that are 20 ft or greater in length on the structure inventory record system (SIRS). Also, 100 percent of the 8- to less than 20-ft long bridges on the state system have been inventoried and recorded. However, few of these 8- to 20-ft span bridges on the local (nonstate) system have been inventoried primarily because there is no federal requirement to do so. There are approximately 52,000 highway bridges in Pennsylvania that are 8 ft or greater in length.

As of November 1985, SIRS has identified more than 7,000 bridges 20 ft or more in length as having federal sufficiency ratings less than 80 and being categorized as structurally deficient or functionally obsolete (3). A structurally deficient bridge is defined as one that has identified structural weaknesses or inadequate waterway. A functionally obsolete bridge is a bridge that has inadequate deck geometry (usually too narrow), is improperly aligned with the roadway, has insufficient vertical clearance, or has inadequate load-carrying capacity to serve today's traffic needs. Those bridges with span lengths 20 ft and greater, and a sufficiency rating less than 80, are generally eligible for federal rehabilitation funds. Those with a sufficiency rating less than 50 are generally eligible for federal replacement funds.

A summary of the bridge situation in Pennsylvania is given in Table 1. The actual number of bridges ≥20-ft long eligible for replacement or rehabilitation exceeds that shown because the inventory for the local system is still in progress.

TABLE 1 Pennsylvania Bridges and Needs

	Number of Bridges			
	Length, >20 ft	ft^2	Length, 8 to <2 ft	ft^2
State system	15,100[a]		9,500	2,360,000
Local and other systems	6,700[a]		Unknown	
Eligible for replacement	4,250	13,900,000	820[b]	262,000
Eligible for rehabilitation	3,100	13,700,000	1,450	409,000

Source: PennDOT's SIRS files, November 2, 1985.
[a]Total 109,900,000 ft^2.
[b]State system.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION ORGANIZATION

PennDOT has decentralized and streamlined its operations. It was reasoned that because the 11 engineering districts are most aware of the needs within their geographic areas, they are in the best position to most directly, efficiently, and responsibly serve the public. The districts are authorized to do so, with the Central Office Bureau of Maintenance administering top-level managerial control and providing policy and procedures, and quality assurance checks for the department.

The Commonwealth's 67 counties are divided among the 11 engineering districts. In each district, the district bridge engineer is the focal point for all bridge activities. This includes responsibility for the ongoing biannual inspection program on all department bridges 8 ft or more in length. Some of these bridges are on former state routes that have been turned back to the municipalities. Because of the large and long-term financial responsibility of a bridge and very limited budgets, most municipalities have not been willing to accept bridge ownership.

A bridge maintenance coordinator working for either the district bridge or district maintenance engineer is responsible for bridge maintenance activities within each district. The coordinator assists in the development of the annual PennDOT force and contract bridge maintenance programs. In addition, he prepares repair sketches and provides technical guidance and quality assurance reviews of the department force work. He is the focal point for communications between PennDOT's District Office and county maintenance offices on bridge maintenance matters. Refer to Figure 1 for a flow diagram of bridge maintenance and minor improvement activities.

EXISTING STRUCTURE INVENTORY RECORDS SYSTEM

PennDOT's current computerized SIRS is an on-line system that has been in use since 1982. Each bridge file has space for recording more than 200 data items including those mandated by FHWA (4,5).

Limited capability exists for defining the maintenance needs of a bridge in the current SIRS. The data are totally inadequate for either costing or programming purposes. The second and third digits of Data Item 182 are available to generally define the type of maintenance work that is needed. Coding is as follows:

- Second digit: Safety improvement, approach improvement, deck improvement, and various combinations of above.

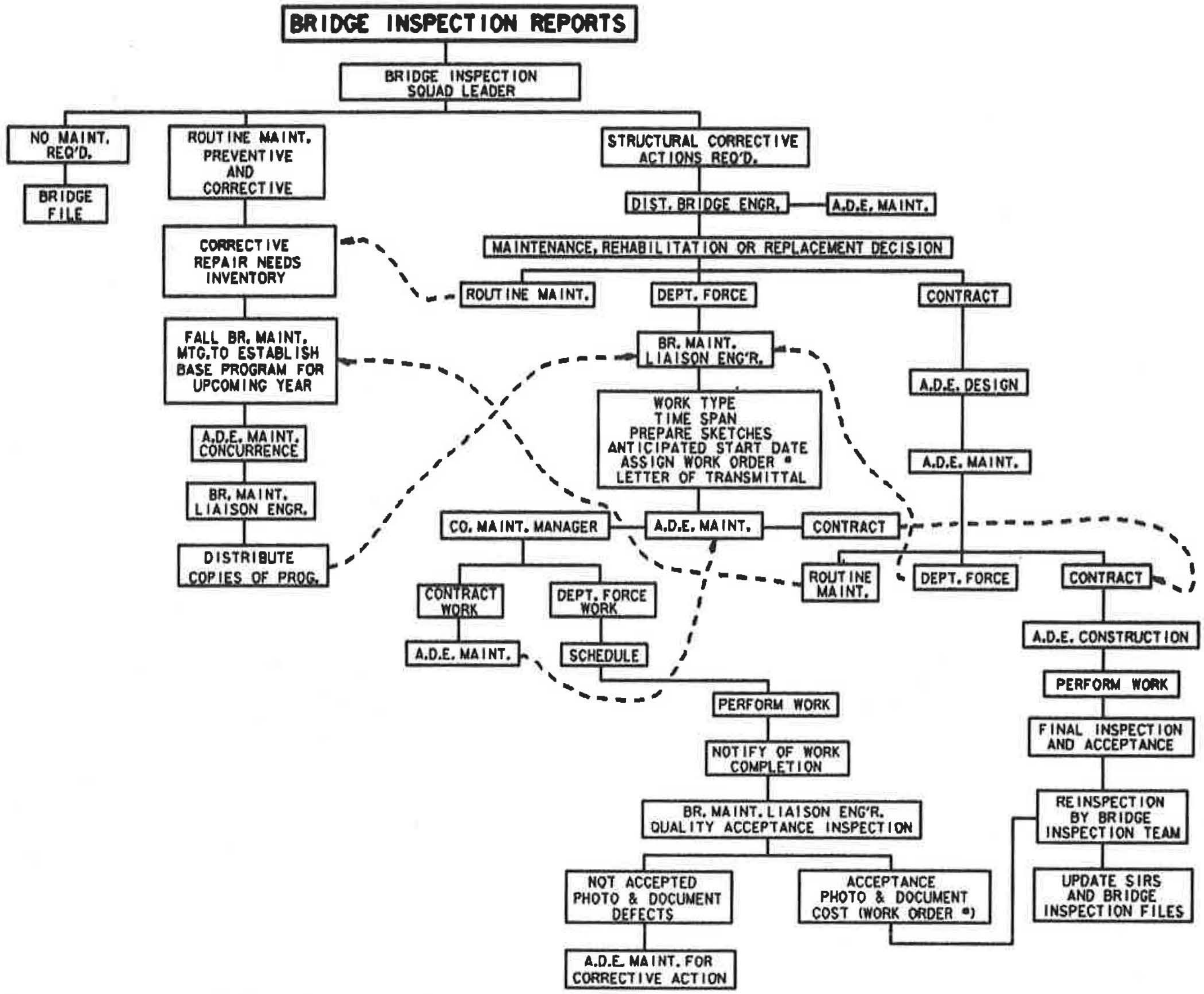

FIGURE 1 Flowchart of bridge maintenance and minor improvements.

• Third digit: Superstructure improvement, substructure improvement, waterway improvement, and various combinations of above.

The priority or urgency of the repair work is coded under Data Item 183. The available coding is as follows:

1. Emergency--within 6 months,
2. Emergency--within 12 months,
3. Priority--within 2 years,
4. Routine structural--can be delayed until funds are available, and
5. Routine nonstructural--can be delayed until programmed.

Because of the inadequacies and severe limitations of SIRS, detailed repair needs inventories must now be maintained manually. Several of the districts have begun storing some of the data on a personal computer. Sorting through the manual listings to select work for implementation by either a contractor or department forces is tedious and time consuming. Besides the inefficiency, there is the chance that structurally important or other urgent repairs will be overlooked.

AUTOMATED MANAGEMENT SYSTEMS

The need to improve the managerial control of its extensive 45,000-mi and 25,000-bridge state highway system, has prompted PennDOT to accelerate development of numerous automated systems. These systems will improve work efficiency and enable the department's declining work force to do more and to make more informed decisions. Electronic data processing development work is now underway on integrated but separate roadway and bridge management systems. Both systems are scheduled to be operational by late 1986. Figures 2, and 3 show the overall roadway and bridge management systems, respectively.

A maintenance management system is also being developed. It will integrate and enhance the existing maintenance planning, equipment, materials and personnel systems. The resulting system will be MORIS, the maintenance operations and resources information system mentioned earlier. More detailed discussion will follow.

BRIDGE MANAGEMENT SYSTEM

The BMS that is now under development will expand the existing SIRS data base, provide a data base for

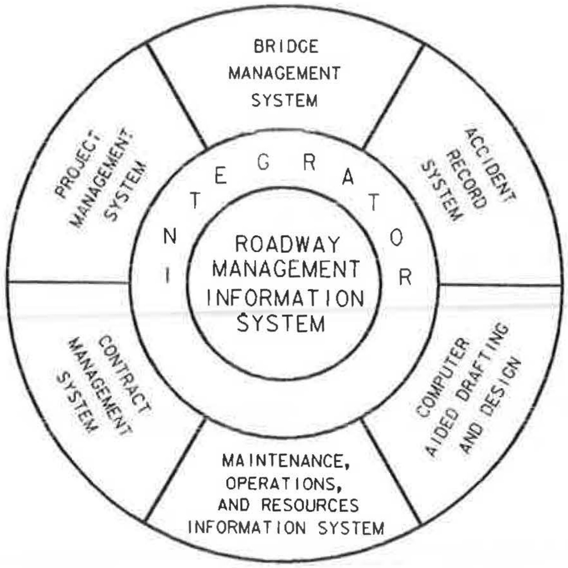

FIGURE 2 Diagram of the roadway management system.

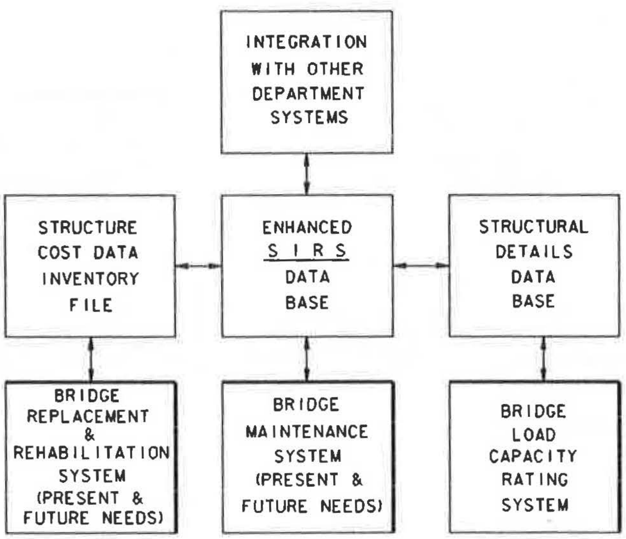

FIGURE 3 Diagram of the bridge management system.

storing structure cost data and automate the estimation of maintenance and rehabilitation or replacement needs based on a generalized scope of work definition by the user. Prioritization models are being developed to assist management in selecting and ranking bridges for maintenance as well as for major improvement.

Integration of BMS with other department systems will enable any data that are common to more than one system to be updated automatically after they are changed in the source system. The project and contract management systems and MORIS will keep BMS updated on the status of planned improvements. This will enable bridge and maintenance managers to coordinate their maintenance efforts consistent with any planned major improvements to the bridge.

BRIDGE MAINTENANCE MANAGEMENT

In formulating the concepts of a BMS, it was readily apparent that the available SIRS data related to maintenance was very general and sketchy. For PennDOT to be able to realistically assess its bridge maintenance requirements on an individual or even on a broad basis, detailed needs must be determined and quantified for each bridge.

A listing of potential bridge-related maintenance activities has been developed in consultation with the districts and the Central Office Bureau of Maintenance. This listing of 9 approach-roadway and 67 bridge-maintenance activities forms the base of the bridge maintenance portion of BMS. It is a comprehensive tabulation of common types of repairs. Activity titles are specific and descriptive. They should give the bridge inspector and the maintenance foreman a descriptive indication of the deficiency and the work that is needed to repair or remove it.

A maintenance needs form has been developed for the bridge inspector as a checkoff type of listing and as the reporting document. When a repairable deficiency is found, the inspector will review the listing, select the proper activity, circle the general location, estimate a quantity, and assign an urgency factor. The coding for the urgency factor will be the same as that currently used in SIRS. It will reflect the inspector's judgment as to how soon the maintenance activity should be completed (Figure 4).

It is anticipated that the bridge maintenance needs data will be collected as a part of the bridge inspection process. Therefore, these data will be entered into BMS's on-line individual bridge files at the same time that the inspection data are updated, that is, promptly after the inspection is completed. Figure 5 shows the general format of the BMS on-line screen where this information will be stored. Once in the computerized system, it can be extracted in any format that is required by bridge and maintenance management to satisfy their particular planning, programming, or other needs. The system is also planned to automatically notify management of any activities that have been coded an 0 (for critical safety deficiency) for their further evaluation and priority implementation.

BRIDGE MAINTENANCE PRIORITIZATION

The maintenance work backlog that exists far exceeds that which the PennDOT can physically and financially handle. Therefore, it is important that guidance be provided to the district and county offices to assist them in selecting the best candidate bridges for maintenance work as well as which activities to perform first. This will help ensure that those deficiencies deemed to be the most critical to the safety of the bridge and hence to its users are brought to the attention of the districts' management

A simple prioritization procedure has been developed. It considers the effect of the most structurally critical maintenance activity need on the bridge, as well as the individual bridge's impact on the road system. The components of the procedure include activity ranking, activity urgency, bridge criticality, and bridge adequacy.

Activity Ranking

The bridge maintenance activities themselves vary in their importance to and effect on the structural integrity of the bridge. Activities such as repairing stringers or repairing abutment underscour would generally be performed on a priority basis, and activities such as applying protective coatings and constructing abutment slopewalls would tend to be deferred.

As a general rule, activities that most directly, D-488FB

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION

STRUCTURE INVENTORY RECORD

BRIDGE INSPECTION REPORT

MAINTENANCE NEEDS

STRUCTURE NO. ____________

INSPECTED BY: ____________

DATE: ____________

		ITEM NO.	LOCATION	UNIT	EST. QUANTITY	PR
APPROACH ROADWAY	PAVEMENT (PATCH/RAISE)	RDPAVMT	N F	S.Y.		
	PAVEMENT RELIEF JT. (REP/REPL)	RDRLFJT	N F	S.Y.		
	SHOULDERS (REPAIR/RECONSTR)	RDSHLDR	N F	S.Y.		
	DRAINAGE-OFF BRIDGE (IMPROVE)	RDDRAIN	N F	EA.		
	GUIDE RAIL-(CONNECT TO BRIDGE)	RDGDERL	N F	EA.		
	LOAD LIMIT SIGNS (REPLACE)	RDLDSGN	N F	EA.		
	CLEARANCE SIGNS (REPLACE)	RDCLSGN	N F	EA.		
	CUT BRUSH TO CLEAR SIGNS	RDBRUSH	N F	EA.		
	APPROACH SLAB (REPLACE)	A744201	N F	S.Y.		
CLEAN/FLUSH	DECK	A743101		E.B.		
	SCUPPER/DOWNSPOUTING	B743101	123450	E.B.		
	BEARING/BEARING SEAT	C743102	123450	E.B.		
	STEEL-HORIZONTAL SURFACES	D743102	123450	E.B.		
DECK	BITUM. DECK W. SURF (REP/REPL)	BITWRGS	123450	S.Y.		
	TIMBER DECK (REP/REPL)	B744301	123450	S.Y.		
	OPEN STEEL GRID (REP/REPL)	C744302	123450	S.Y.		
	CONCRETE DECK (REPAIR)	D744303	123450	S.Y.		
	CONCRETE SIDEWALK (REPAIR)	E744303	123450	S.Y.		
	CONCRETE CURB/PARAPET (REPAIR)	F744303	123450	S.Y.		
DECK JOINTS	RESEAL	A743301	N1230F	L.F.		
	REPAIR/RESEAL	A744101	N1230F	L.F.		
	COMPRESSION SEAL (REP/REHAB)	B744102	N1230F	L.F.		
	MODULAR DAM (REP/REHAB)	C744102	N1230F	L.F.		
	STEEL DAMS (REP/REHAB)	D744102	N1230F	L.F.		
	OTHER TYPES (REP/REHAB)	E744102	N1230F	L.F.		
RAILING	BRIDGE/PARAPET (REP/REPL)	RLGBRPR	N1230F	L.F.		
	STRUCT MOUNT (REP/REPL)	RLGSTRM	N1230F	L.F.		
	PEDESTRIAN (REP/REPL)	RLGPEDN	N1230F	L.F.		
	MEDIAN BARRIER (REP/REPL)	RLGMEDB	123450	L.F.		
DK DRAIN	SCUPPER GRATE (REPLACE)	DRNGRAT	123450	EA.		
	DRAIN/SCUPPER (INSTALL)	B744401	123450	EA.		
	DOWNSPOUTING (REP/REPL)	C744402	N1230F	EA.		
BEARINGS	LUBRICATE	A743501	N1230F	EA.		
	STEEL (REP/REHAB)	A744501	N1230F	EA.		
	STEEL (REPLACE)	B744501	N1230F	EA.		
	EXPANSION (RESET)	C744502	N1230F	EA.		
	PEDESTAL/SEAT (RECONSTRUCT)	D744503	N1230F	EA.		
TIMBER	STRINGER (REP/REPL)	A744601	123450	EA.		
	OTHER MEMBERS (REP/REPL)	B744601	123450	EA.		
STEEL	STRINGER (REP/REPL)	A744602	123450	EA.		
	FLOORBEAM (REP/REPL)	B744602	123450	EA.		
	GIRDER (REPAIR)	C744602	123450	EA.		
	DIAPH/LAT. BRACING (REP/REPL)	D744602	123450	EA.		
RC/PS	STRINGER (REP/REPL)	A744603	123450	EA.		
	DIAPHRAGM (REP/REPL)	B744603	123450	EA.		
	OTHER MEMBERS (REP/REPL)	C744603	123450	EA.		
TRUSS	MEMBER (STRENGTHEN/REP/REPL)	A744701	123450	EA.		
	PORTAL (MODIFY)	B744701	123450	EA.		
	MEMBER (TIGHTEN/FLAMESHORTEN)	C744702	123450	EA.		

		ITEM NO.	LOCATION	UNIT	EST. QUANTITY	PR
PAINTING	SUPERSTRUCTURE - SPOT	A743201	123450	E.B.		
	SUBSTRUCTURE - SPOT	B743201	N1230F	E.B.		
	SUPERSTRUCTURE - FULL	C743201	123450	E.B.		
	SUBSTRUCTURE - FULL	D743201	N1230F	E.B.		
ABUTMENT - WING - PIER, ETC.	BACKWALL (REP/REPL)	A744801	N F	C.Y.		
	ABUTMENTS (REPAIR)	B744802	N F	C.Y.		
	WING (REP/REPL)	C744802	NLRFLR	C.Y.		
	PIERS (REPAIR)	D744802	123450	C.Y.		
	FOOTING (UNDERPIN)	E744803	N1230F	C.Y.		
	MASONRY (REPOINT)	F744804	N1230F	C.Y.		
	ABUT. SLOPEWALL (REP/REPL)	A745101	N F	S.Y.		
	ABUT. SLOPEWALL (CONSTRUCT NEW)	B745102	N F	S.Y.		
	PILE REPAIR	A745901	N1230F	EA.		
EROSION CONTROL	STREAMBED PAVING (REP/CONSTR)	A745301	UPUNDN	C.Y.		
	ROCK PROTECTION	B745301	UPUNDN	C.Y.		
	SCOUR HOLE (BACKFILL)	C745301	UPUNDN	C.Y.		
	STREAM DEFLECTOR (REP/CONSTR)	D745302	UPUNDN	C.Y.		
	VEGETATION/DEBRIS (REMOVE)	ECREMVG	UPUNDN	C.Y.		
	DEPOSITION (REMOVE)	ECREMDP	UPUNDN	C.Y.		
CULVERT	HEADWALL/WINGS (REP/REPL)	A745201	IN OUT	S.Y.		
	APRON/CUTOFF WALL (REP/REPL)	B745202	IN OUT	S.Y.		
	BARREL (REPAIR)	C745203	—	S.Y.		

FOR COMPLETION BY REVIEW ENGINEER

APPLY PROTECTIVE COATING

DECK/PARAPETS/SIDEWALK	A743401	DPS	S.Y.		
SUBSTRUCTURE	B743401	N1230F	S.Y.		

CONSTRUCT TEMPORARY

SUPPORT BENT	A745401	N1230F	EA.		
PIPES	B745401	LT ℄ RT	E.B.		
BRIDGE	C745401	LT ℄ RT	E.B.		

LEGEND

N = NEAR
F = FAR
1,2,3,ETC. = SPAN OR PIER NUMBER
0 = OTHER
NLR = NEAR LEFT OR RIGHT
FLR = FAR LEFT OR RIGHT

UP = UPSTREAM
UN = UNDER
DN = DOWNSTREAM
IN = INLET
OUT = OUTLET
E.B. = EACH BRIDGE (SITE)

PR - PRIORITY CODE

0 - CRITICAL SAFETY DEFICIENCY, PROMPT ACTION REQUIRED (INSPECTOR TO HIGHLIGHT THE DEFICIENCY)
1 - EMERGENCY, WITHIN 6 - MONTHS
2 - EMERGENCY, WITHIN 12 - MONTHS
3 - PRIORITY, WITHIN 2 - YEARS
4 - ROUTINE STRUCTURAL, CAN BE DELAYED UNTIL FUNDS ARE AVAILABLE
5 - ROUTINE NON-STRUCTURAL, CAN BE DELAYED UNTIL PROGRAMMED

FIGURE 4 Maintenance needs reporting form.

Structure Identification

MTCE. DEFICIENCY POINTS

ACTIVITY DESCRIPTION	ITEM NO.	UNIT	EST. QUANT.	EST. COST	ACTIVITY LOCATION	URGENCY	PROGRAM	
							D OR C	FY

NOTES:
EST. - ESTIMATED
EST. COST - AUTOMATED BY SYSTEM USING UNIT COST TABLE
D OR C - D = WORK BY DEPT. FORCE, C = WORK BY CONTRACT
FY - YEAR OF PLANNED ACTIVITY

FIGURE 5 Bridge maintenance activity needs screen.

immediately, and positively affect the continued safety and structural adequacy of the bridge would be performed first, and those that have minimal immediate impacts would tend to be performed later.

The activities have been divided into groups based on their generalized relative importance to the current structural stability of the bridge (Table 2). Repair or replacement of steel stringers, floorbeams, girders, or truss members could be related to existing or potential fatigue damage. If the needs are indeed fatigue related, they are more important and should be given a higher ranking. This determination can be made by comparing these maintenance-activity needs with the type of fatigue-prone member that controls the inventory load rating. SIRS Data Item 178 provides space for recording the controlling member type as well as the related fatigue and load data. If the activity is fatigue related, it will be assigned as Group AF and given additional priority.

Activity Urgency

The severity of a deficiency can be a reason to increase its priority for repair. The urgency factor for each activity need is coded by the District Bridge Inspection Unit. It yields an informed, humanized assessment of how soon the work needs to be completed. As such, it is also a measure of the severity of the deficiency.

The factor is included in the current SIRS; however, it relates only to the very generalized and broad-based maintenance needs definition limitations of the existing system. In BMS, the factor will judgmentally define the promptness of action that is needed for each specific maintenance activity need. The coding legend used is shown in Figure 4.

Bridge Criticality

The importance of a bridge to the road network as well as the effect of the loss of bridge service on traffic is another factor that must be considered in deciding the order in which bridges are to be repaired. That the road system hierarchy realistically defines importance is readily apparent. That is, if a bridge on the Interstate and a bridge on the local access system have similar deficiencies, it is obvious that the Interstate highway bridge would be repaired first. However, the impact of a bridge's closure also needs to be weighed. If the detour length is excessive and hence intolerable, the priority for repair should be raised.

The assessment of the importance of the bridge will be based on the classification of the highway, its average daily traffic (ADT), and the detour length that will be imposed on traffic if the bridge were to be closed. Multiplying the ADT times the detour length results in a partial relative measure of this importance.

The factors to be considered in evaluating the bridge's criticality then are

1. The kind of highway (e.g., Interstate, U.S. numbered highway, state highway, county highway, city, borough street, and township road);
2. The department road network indicator [e.g., priority commercial network (PCN), agri-access network, industrial access (proposed addition)]; and
3. ADT x detour length.

Bridge Adequacy

The capability of the bridge to safely carry the loads that traverse the route and to continue to do so, figure in a manager's decision of whether or not repairs should be implemented. The load capacity rating indicates the current strength of the bridge. It gives no indication of what can be expected in the future. The condition rating of the most critical component of the bridge can be used to generally assess degradation. Refer to the procedure for the

TABLE 2 Maintenance Activity Ranking

Category	Activity	Rank
CLEAN/FLUSH	DECK	E
CLEAN/FLUSH	SCUPPER/DOWNSPOUTING	E
CLEAN/FLUSH	BEARING/BEARING SEAT	E
CLEAN/FLUSH	STEEL-HORIZONTAL SURFACES	E
DECK	BITUM. DECK W. SURF (REP/REPL)	C
DECK	TIMBER DECK (REP/REPL)	B
DECK	OPEN STEEL GRID (REP/REPL)	B
DECK	CONCRETE DECK (REPAIR)	B
DECK	CONCRETE SIDEWALK (REPAIR)	C
DECK	CONCRETE CURB/PARAPET (REPAIR)	C
DECK JOINTS	RESEAL	C
DECK JOINTS	REPAIR/RESEAL	C
DECK JOINTS	COMPRESSION SEAL (REP/REHAB)	C
DECK JOINTS	MODULAR DAM (REP/REHAB)	C
DECK JOINTS	STEEL DAMS (REP/REHAB)	C
DECK JOINTS	OTHER TYPES (REP/REHAB)	C
RAILING	BRIDGE/PARAPET (REP/REPL)	B
RAILING	STRUCT MOUNT (REP/REPL)	B
RAILING	PEDESTRIAN (REP/REPL)	B
RAILING	MEDIAN BARRIER (REP/REPL)	C
DK DRAIN	SCUPPER GRATE (REPLACE)	D
DK DRAIN	DRAIN/SCUPPER (INSTALL)	D
DK DRAIN	DOWNSPOUTING (REP/REPL)	D
BEARINGS	LUBRICATE	E
BEARINGS	STEEL (REP/REHAB)	B
BEARINGS	STEEL (REPLACE)	B
BEARINGS	EXPANSION (RESET)	C
BEARINGS	PEDESTAL/SEAT (RECONSTRUCT)	A
TIMBER	STRINGER (REP/REPL)	A
TIMBER	OTHER MEMBERS (REP/REPL)	B
STEEL	STRINGER (REP/REPL)	A
STEEL	FLOORBEAM (REP/REPL)	A
STEEL	GIRDER (REPAIR)	A
STEEL	DIAPH/LAT. BRACING (REP/REPL)	D
RC/PS	STRINGER (REP/REPL)	A
RC/PS	DIAPHRAGM (REP/REPL)	D
RC/PS	OTHER MEMBERS (REP/REPL)	B
TRUSS	MEMBER (STRENGTHEN/REP/REPL)	A
TRUSS	PORTAL (MODIFY)	D
TRUSS	MEMBER (TIGHTEN/FLAMESHORTEN)	A
PAINTING	SUPERSTRUCTURE - SPOT	E
PAINTING	SUBSTRUCTURE - SPOT	E
PAINTING	SUPERSTRUCTURE - FULL	D
PAINTING	SUBSTRUCTURE - FULL	D
ABUTMENT - WING - PIER, ETC.	BACKWALL (REP/REPL)	B
ABUTMENT - WING - PIER, ETC.	ABUTMENTS (REPAIR)	B
ABUTMENT - WING - PIER, ETC.	WING (REP/REPL)	B
ABUTMENT - WING - PIER, ETC.	PIERS (REPAIR)	B
ABUTMENT - WING - PIER, ETC.	FOOTING (UNDERPIN)	A
ABUTMENT - WING - PIER, ETC.	MASONRY (REPOINT)	C
ABUTMENT - WING - PIER, ETC.	ABUT. SLOPEWALL (REP/REPL)	E
ABUTMENT - WING - PIER, ETC.	ABUT. SLOPEWALL (CONSTRUCT NEW)	E
ABUTMENT - WING - PIER, ETC.	PILE REPAIR	A
EROSION CONTROL	STREAMBED PAVING (REP/CONSTR)	C
EROSION CONTROL	ROCK PROTECTION	C
EROSION CONTROL	SCOUR HOLE (BACKFILL)	C
EROSION CONTROL	STREAM DEFLECTOR (REP/CONSTR)	D
EROSION CONTROL	VEGETATION/DEBRIS (REMOVE)	D
EROSION CONTROL	DEPOSITION (REMOVE)	D
CULVERT	HEADWALL/WINGS (REP/REPL)	B
CULVERT	APRON/CUTOFF WALL (REP/REPL)	C
CULVERT	BARREL (REPAIR)	B
APPLY PROTECTIVE COATING	DECK/PARAPETS/SIDEWALK	E
APPLY PROTECTIVE COATING	SUBSTRUCTURE	E
CONSTRUCT TEMPORARY	SUPPORT BENT	A
CONSTRUCT TEMPORARY	PIPES	B
CONSTRUCT TEMPORARY	BRIDGE	B

LEGEND

A - HIGHEST PRIORITY

E - LOWEST PRIORITY

automated estimation of remaining life given in Table 3. It is based on the summation of the condition ratings for the deck super- and substructures. If any of the ratings are four or less, they individually establish the remaining life (Table 4).

By considering both the current load capacity and the lowest condition rating of the structure's components, a measure of the inadequacy of the bridge can be obtained.

DEFICIENCY POINT ASSIGNMENT

Most of the data that will be needed to define the foregoing components of the prioritization procedure are already in SIRS. The only new items are the maintenance activities themselves and their individually assigned urgency rankings. They are important components of the proposed BMS.

Having defined the major parameters that are to be considered, the relative weights to be assigned to them and their elements must be established. To be consistent with the general philosophy of the rehabilitation or replacement prioritization system (6), a deficiency point concept (7) will also be used for the maintenance activity prioritization system. However, it is readily apparent that the factors and methodology used in each system are quite different. Although it is numerically possible for a single bridge to be assigned in excess of 100 deficiency

TABLE 3 Estimated Remaining Life of Bridges with Condition Ratings More Than 4

Bridges		Culverts	
Sum of Deck, Superstructure and Substructure Condition Ratings	Estimated Remaining Life (yr)	Culvert Condition Rating	Estimated Remaining Life (yr)
27	50	9	50
26	46	8	42
25	42	7	33
24	38	6	25
23	34	5	17
22	30	4	10
21	26	3	5
20	23	2	1
19	20	0,1	0
18	17		
17	14		
16	12		
15	10		
14	8		
13	7		
12	6		
11	5		

TABLE 4 Estimated Remaining Life of Bridges with Condition Ratings Less Than 4

Condition Rating	Estimated Remaining Life (yr)
4	10
3	5
2	1
0, 1	0

points, the deficiency point assignment will be limited to a maximum of 100. The higher the point assignment on a bridge, the higher its priority; 100 points represents total deficiency, and 0 points represents no deficiency.

Table 5 summarizes the four major components of the prioritization system, defines the elements in their makeup, and indicates the initial or trial weights that have been assigned to each. As the procedure is tested, evaluated, and refined the weight assignments could and probably will change.

The maintenance deficiency point assignment for a bridge will be based on the bridge maintenance activity that has the largest sum of deficiency points for activity ranking and urgency. The bridge's deficiency point assignment and the bridge's county ranking for maintenance based on the deficiency point assignment will be recorded on the bridge maintenance activity needs screen. Therefore, when a manager views the subject screen for individual bridges, an immediate indication of the relative priority of the most critical repair need on one bridge compared to another bridge and to the worst possible case (100 deficiency points) is available.

With a deficiency point assignment stored in BMS for every bridge, listings in priority order can be easily generated using the particular parameters desired. To facilitate this reporting, user-friendly preprogrammed report generators with user-defined variables will be developed.

A listing of bridges in priority order to be repaired can be generated for review by the district and county maintenance managers and for their use in developing the annual bridge repair programs. Once programmed, the activity needs screen can be updated to reflect whether work is to be done by department force or contract and the date of implementation scheduled.

MAINTENANCE MANAGEMENT

PennDOT is developing a MORIS to assist its Maintenance Organization to plan, implement, and effectively manage activities. The system combines various existing material, equipment, manpower, and planning subsystems and further enhances their combined capabilities. Figure 6 shows an overview of the system.

It is envisioned that when the BMS is told that a certain activity or activities on specific bridges are programmed for implementation by department forces, a copy of the data will be transmitted to the planning file in MORIS. The maintenance manager can then review and transfer the data to their annual and periodic work plans within MORIS.

MORIS will generate the daily crew payroll form, filling in the bridge location identifier plus the cost function and method (Activity Number) for the work that is to be performed. It should be noted that, initially, only 35 of the 76 maintenance activities identified on the needs reporting form are being assigned cost functions. Therefore, for cost-accounting purposes, activities without a valid cost function will have to be grouped with a similar activity that has an approved cost function.

During the actual implementation of the work, labor, use and cost of equipment, and materials are tracked daily from the crew foreman's payroll. The activity, quantity, and the cost of work performed will be reported to the individual bridge file in BMS on a daily basis. A running total quantity and cost will be maintained until notification is received from MORIS that the activity is completed. The completion date of each activity and the final quantity and its cost will then be kept for historical record purposes. Figure 7 shows the general format of the BMS on-line screen where these data will be stored. Following this, the needs-inventory portion of the bridge file can be automatically updated to eliminate those programmed activities that have been completed.

CONTRACT MAINTENANCE WORK

As part of the overall BMS development, a three-digit code has been developed to detail major improvement (rehabilitation or replacement) needs and work (6). The number inserted in the first digit indicates the type of work to be performed on the deck, the second digit relates to the superstructure, and the third to the substructure. Because this code is being incorporated into PennDOT's contract management system and may be included in the project inventory and program management systems, it will also be used as a general indicator that maintenance type work is to be performed. An R or similar indicator can be placed in the digit corresponding to the bridge component where work is to be done.

Use of the aforementioned code will allow the development and implementation of contract maintenance projects to be easily tracked and BMS to be kept informed of their new status. Although this system will monitor progress of a contract maintenance project, it will not definitively indicate the type or extent of work. To determine this, the user has to manually review the plans, contract document, or the automated structure cost data file. On completion of the work, BMS will be notified that contract maintenance-type work has been completed and

TABLE 5 Maintenance Deficiency Points Assignment

Component	Maximum Deficiency Points	Element	Deficiency Point Assignment
Bridge maintenance activity rank	40	Group AF[a]	40
		A	25
		B	20
		C	15
		D	10
		E	5
Activity urgency factor	25	Code 0	25
		1	20
		2	15
		3	10
		4	5
		5	0
Bridge criticality	25		
Part A: Interstate			5
U.S. numbered highway			4
State highway			3
County highway			2
City, borough street, or township road			1
Part B: PCN			5
PCN or coal haul			5
Agri-access network			3
Industrial access			3
Part C: ADT x detour length			
≥30,000			15
≥15,000 but <30,000			10
≥3,000 but <15,000			5
<3,000			0
Bridge adequacy	25		
Part A: Lowest condition rating			
≤3			15
>3 but ≤4			10
>4 but ≤5			5
>5			0
Part B: Load capacity (individual rating)			
H configuration ≤12 tons			10
>12-19 tons			7
ML 80 configuration >19-30 tons			4
>30 tons			0

[a]AF = Group A, activity that is fatigue prone and controls the inventory rating.

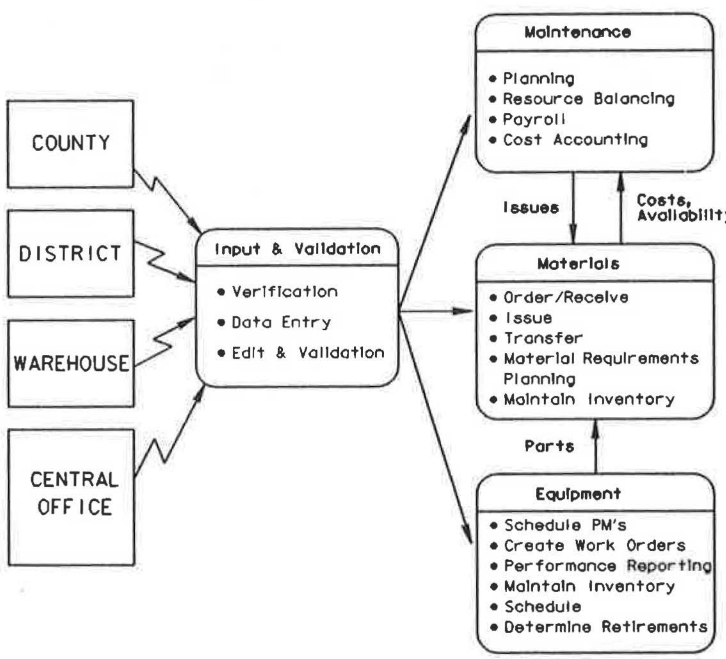

FIGURE 6 Maintenance operations and resources information system overview.

Structure Identification

DATE REPORTED	ITEM NO.	UNIT	ACTUAL QUANTITY	ACTUAL COST

FIGURE 7 Completed bridge maintenance activities screen.

its cost. To construct a more detailed historical record, the work has to be manually matched to activities and quantities, and costs determined.

PREVENTIVE MAINTENANCE AND CYCLIC NEEDS

The extent and frequency of some repetitive and age-associated preventive maintenance, routine maintenance, and minor rehabilitation needs and their effects on service life are conceptually known from generalized experience. However, this work must be blended with the other identified needs to arrive at an overall balanced program.

The repetitive need cycle or bridge age when certain bridge maintenance and minor rehabilitation needs develop can be generalized as given in Table 6. Annualized funding needs can be generated and considered in conjunction with the tabulated existing bridge maintenance needs to allocate maintenance monies.

TABLE 6 Need Cycles for Bridge Maintenance and Minor Rehabilitation

Activity	Frequency (yr)
Structural steel painting	12
Deck rehabilitation[a]	
Bridge	20
Deck, condition rating	≤ 5
Deck joints cleaning and resealing	5
Scuppers cleaning	1
Deck cleaning	1
Breakdown maintenance[b]	2
Timber deck replacement	15

[a] For unprotected decks or uncoated reinforcement.
[b] Carried out on bridges in poor condition until rehabilitation or replacement.

As historic activity and cost completion data are accumulated, the frequencies given in Table 6 can be revised, expanded, and perhaps categorized by highway system, level of maintenance, and so on. In addition, the effects on the service life of the bridge can be assessed, and eventually, life-cycle costing models can be developed.

SUMMARY

Various computerized highway management and information systems are scheduled to be implemented by late 1986. These will give PennDOT comprehensive and powerful tools to improve the effectiveness, efficiency, and depth of its managerial control over the Commonwealth's highway system and will help ensure that the department's limited financial and human resources are used to maximize the benefits to and safety of the traveling public.

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A Comprehensive Bridge Posting Policy

I. R. PAREKH, D. R. GRABER, and R. H. BERGER

ABSTRACT

The posting of bridges for maximum allowable loads should allow for the safe continued use of existing bridges without unnecessary restrictions. The mechanism for determining load postings must minimize risk to the structure and maximize the benefits for the user by considering economic as well as engineering factors. A rational policy is essential to maximize the remaining service life of existing bridges and to protect the bridge owner's capital investment. The method developed here to determine bridge postings will span the gap between current inventory and operating load levels of AASHTO. It will provide more than one stress level for evaluation and be flexible in the decision-making process by balancing risk and safety levels and taking structural redundancy, load variations, permit operations, load enforcement, and level of inspection into account. This method will help raise the load posting level of deficient bridges and at the same time reduce the risk of bridge failure. The method is equally applicable to reinforced concrete, prestressed concrete, structural steel, and timber. Equivalent load factors are identified for use with load factor method (LFM).

The posting of bridges for maximum allowable loads should allow for the safe continued use of existing bridges without unnecessary restrictions. The mechanism for determining load postings must minimize risk to the structure and maximize the benefits to the user by considering economic as well as engineering factors. A rational policy is essential to maximize the remaining service life of existing bridges and to protect the bridge owner's capital investment.

In 1983 Byrd, Tallamy, MacDonald and Lewis, under contract with the Pennsylvania Department of Transportation (PennDOT) began a 21-month study of deficient bridges in Pennsylvania, which resulted in this paper. The objectives of the project were to develop a comprehensive bridge-posting policy, analyze bridges by both working stress and load factor methods, and evaluate the effect of bridge loads.

Specific tasks (Figure 1) were identified: (a) study the economic effect of the proposed posting policy; (b) conduct telephone surveys of several state agencies and all district bridge offices; (c) classify 6,000 deficient bridges by type, location, and traffic characteristics; and (d) compare typical posting values that result, using working stress (WSM) and load factor methods (LFM).

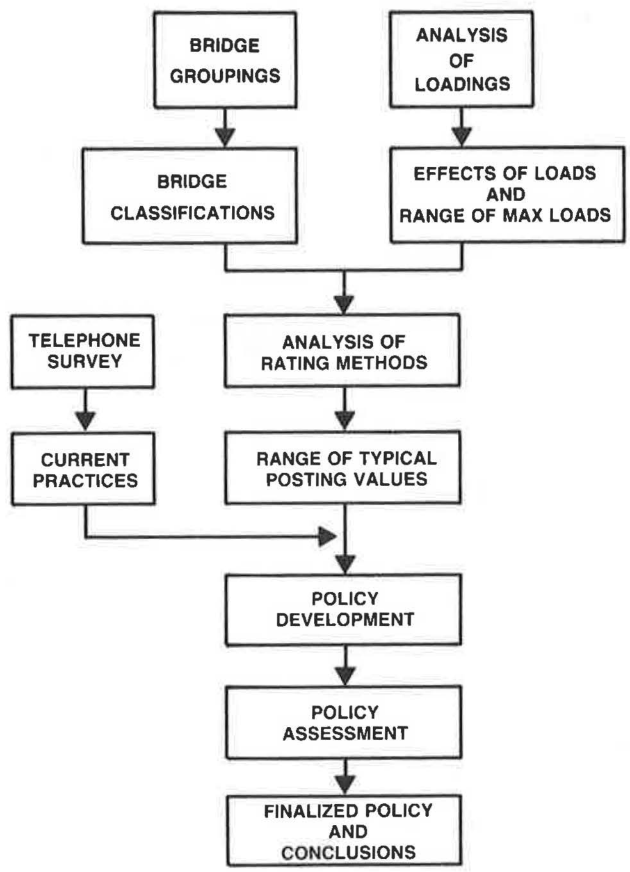


FIGURE 1 Task relationship.

FINDINGS

Telephone Surveys

The telephone surveys revealed that all states rate their bridges in accordance with AASHTO's Manual for Maintenance Inspection of Bridges (1). However, most states do not post their bridges in accordance with these specifications. Variations were found among states, reflecting the nation's geopolitical spectrum.

The telephone survey revealed that several states have their own posting policy. North Carolina's procedure was the most complete example found, consisting of 81 pages defining stresses, rating trucks, interpretation of AASHTO specifications, and specifying signs to be used (2). Illinois' policy defines lanes, stresses, and exceptions. New York's policy is actually a rating memorandum. Pennsylvania allows the use of stresses up to operating rating based on

Byrd, Tallamy, MacDonald and Lewis, A Division of Wilbur Smith and Associates, 2921 Telestar Court, Falls Church, Va. 22042

engineering judgment if (a) the bridge is in good condition, (b) the inspection frequency greatly exceeds the minimum required, (c) the load history of the bridge is closely monitored, and (d) the increase in stress is approved by the chief bridge engineer.

The policy also specifies the use of the AASHTO trucks but substitutes the state's 36.64-ton, four-axle dump truck, the ML-80 (Figure 2).

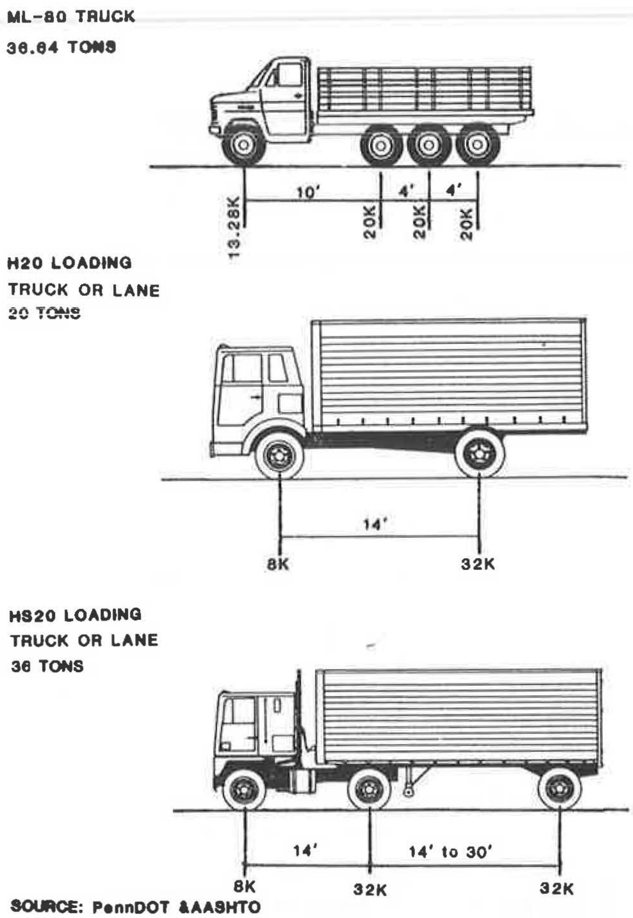


FIGURE 2 Rating vehicles.

Most states do not post for load using fatigue as a criteria because fatigue is more a function of stress cycles than stress range. These states include fatigue in the inspection-rehabilitation process. The surveys also revealed that most states do not issue overweight permits for posted bridges.

Deficient Bridge Classifications

It was decided that the research effort should concentrate on 6,000 structurally deficient bridges as defined by a sufficiency rating of 80.00 or less because these structures are either posted (presently 3,700 bridges) or candidates for posting. Both values include state and local owners. It was shown that the majority of structurally deficient bridges are older than the general population of bridges and are located on rural routes with an average daily traffic of under 2,000 vehicles per day and an average detour length of 5 mi. Five bridge types were selected for concentrated effort. These types, which account for most structurally deficient bridges, are steel I-beam spans, steel through truss spans, steel girder-floorbeam spans, reinforced concrete slab spans, and reinforced concrete T-beam spans. The types also account for most posted structures.

Prestressed concrete does not appear on the foregoing list because of the low number of such bridge types that are structurally deficient. It is speculated that this is because these structures are relatively young, made with dense concrete, and factory cured under a high degree of quality control.

Span ranges were obtained for each of the deficient structure types so that analyses of typical bridges could be performed. Figure 3 shows the span ranges for each type of structure considered.

Bridge Loads

Bridge loadings of various states were analyzed for their effect on structures. The legal loads in California, Michigan, and Pennsylvania were found to be among the highest in the nation. Figure 4 shows the relative effect of various bridge loads. The bending moment without impact was calculated and divided by the moment induced by the HS20-44 design truck. The resultant ratio is plotted versus the span length for which it was calculated. Based on this analysis 10 bridge loadings were selected for concentrated effort.

Comparative Analysis

A comparative analysis of WSM and LFM was performed on 15 sample bridges. For the five typical, structurally deficient bridge types, typical posting values were generated for short, average, and long spans at inventory and operating ratings for the 10 selected bridge loadings. The analysis was designed to compare variables affecting bridge postings. WSM versus LFM, inventory rating versus operating rating, truck configurations, original design load, and the effect of span length were examined. The results of the comparative analysis are summarized in Figure 5, which shows the number of bridges that require posting in each category.

WSM Versus LFM

WSM was used initially to design the nation's deficient bridges. The results of the comparative analysis demonstrate that postings are nearly the same, independent of the rating method used. The difference is that one sample bridge was borderline and would require posting using WSM. It was found during the comparative analysis that LFM was very time consuming. Much additional information is required to perform an LFM posting analysis.

The use of LFM for rating structural steel is not advantageous when the engineer is working with a noncompact section or a compression flange that is fully supported. Secondary stresses induced by beam curvature do not lend themselves well to rating by LFM. However, WSM is easily adapted to the rating task. The short span through truss in the comparative analysis also had the disadvantage of timber stringers that could not be rated using LFM. LFM for rating reinforced concrete is advantageous only for Grade 60 steel. Most deficient bridges were built using Grade 40 reinforcement, however.

Rating Vehicles

Single vehicle postings were controlled by either the H20 loading or by the four-axle dump truck. Com-

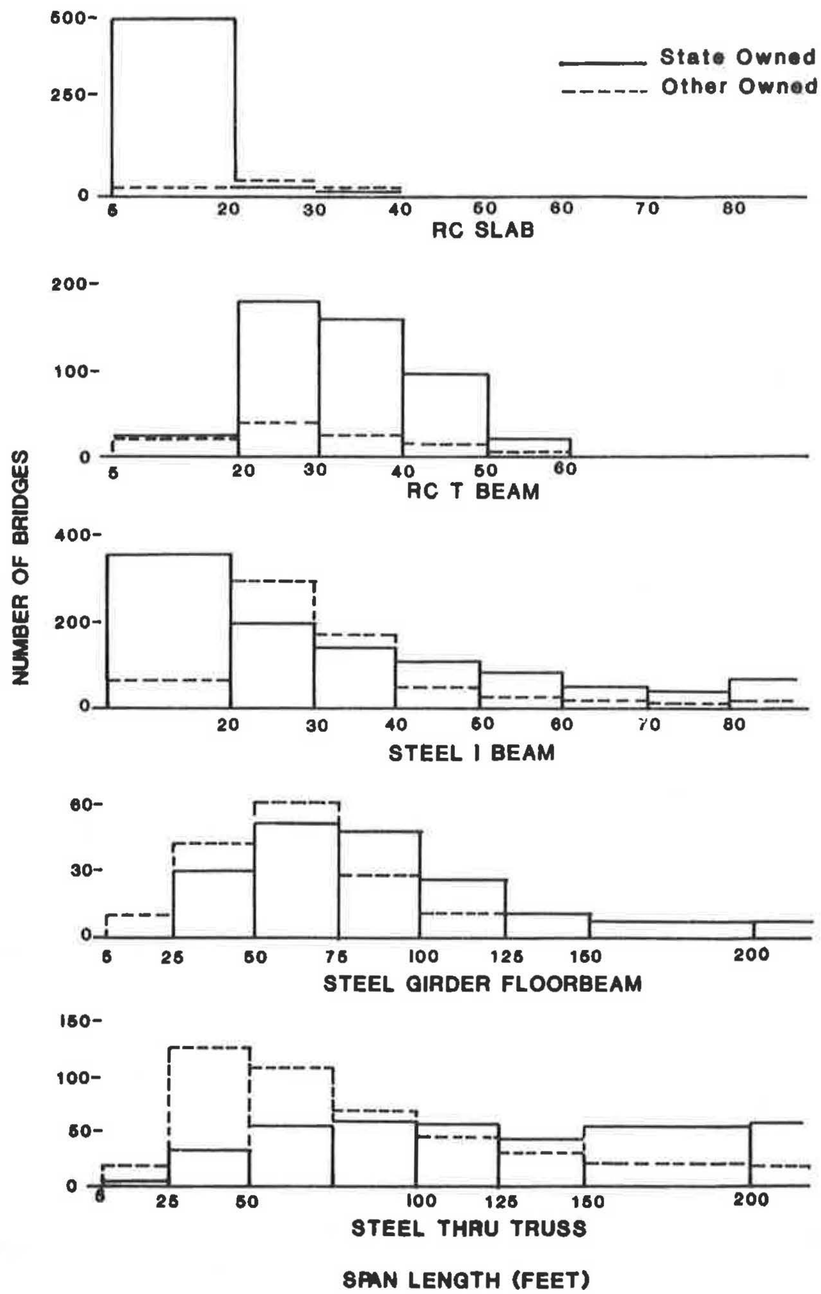


FIGURE 3 Span length distribution.

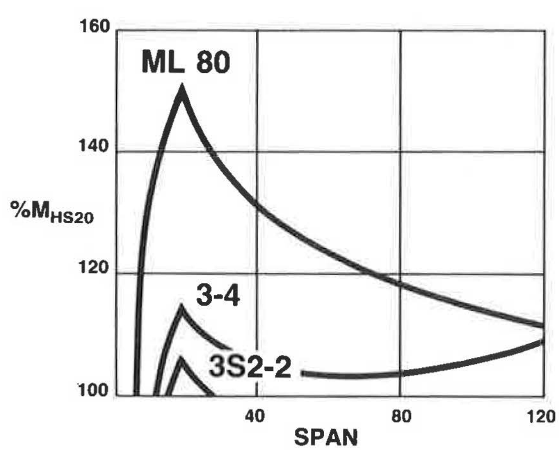


FIGURE 4 Effect of bridge loads.

bination vehicle postings were controlled by the HS20 loading in all cases except one borderline case where the Type 3-4 loading controlled. This vehicle is an Interstate loading and would probably not require posting.

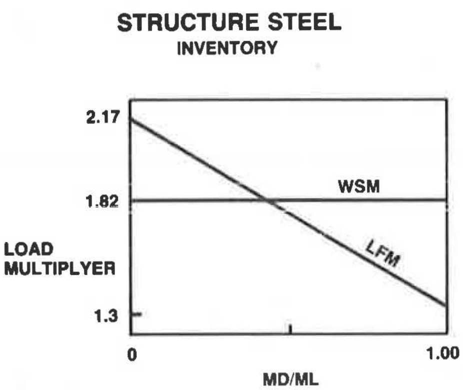
Span Length, Age, and Original Design Load

In all cases but one, the long span structures were designed for HS20-44. Most states have used this design load since 1944. It is plausible that the construction of ever-longer span structures was possible with a constantly advancing technology so that longer spans are of newer construction and design loads. This being the case, span length, age, and original design loads can be considered together.

In most cases the posting weight limit increases. This is partly a result of the effect described earlier and the fact that short spans, which are

	SHORT	MEDIUM	LONG
INVENTORY			
WSM	4	5	4
LFM	4	5	3
OPERATING			
WSM	3	2	1
LFM	3	1	1

FIGURE 5 Sample group profile.



	OPERATING
WSM	1.33
LFM	1.30

FIGURE 6 Load multiplier to first yield.

predominantly stressed by live load, will be posting prone, whereas long spans, which are predominantly stressed by dead load, will be, to a degree, relatively free of load posting (Figure 6).

POLICY DEVELOPMENT

Safe Load Posting Level Flowchart

The method developed to determine the safe load capacity will bridge the gap between current inventory and operating load levels of AASHTO. It will provide more than one stress level for evaluation. It will be flexible in the decision-making process, balancing risk and safety levels and taking structural redundancy, load variations, permit operations, load enforcement, and level of inspection into account.

This method of selecting safe load capacity should help raise the load posting level of deficient bridges and at the same time reduce the risk of bridge failure. The recommended method will be equally applicable to reinforced concrete, prestressed concrete, structural steel, and timber. Equivalent load factors that can be used for LFM analysis have been developed.

The 10 factors affecting the posting level previously identified are shown in Figure 7. The purpose of the safe load-stress level decision flowchart shown in Figure 8 is to quantify and document the input to solidify the logic for selecting the posting level to be used. A discussion of some of the various decision nodes in the flowchart follows.

Member Condition

Steel does not lose strength with age or deterioration because loss of section is used to discount corrosion of the metal, and because fatigue is not an issue here. Concrete actually gains strength with age, and even some heavily deteriorated concrete bridges have been shown to suffer no measurable loss of strength (3).

Based on these findings, the research team recommends that only members with material in critical condition, with an SI&A superstructure condition rating of 3 or below, should be rated no higher than inventory stress level (4).

Inspection Frequency

In order to minimize the need to post numerous bridges, AASHTO allows the rating agency to use load levels higher than inventory rating, for posting purposes. The research team recommends that posting levels greater than inventory rating should be allowed if the inspection frequency were reduced to once a year or less and if other pertinent factors do not prevail.

Level of Enforcement

Load levels higher than inventory rating for posting are acceptable only if the risk of overload is small. Enforcement of posting limits becomes more critical as the posting value decreases. Structures located on highways with permanent truck scales attain the highest level of confidence.

The level of enforcement was divided into three categories in an effort to quantify the enforcement factor. Enforcement Level 1, is assigned to structures on routes where truck load limits are vigorously enforced. Routes with a moderate level of enforcement are assigned to bridges with Enforcement Level 2. Other roads are assigned to Enforcement Level 3. Bridges with Enforcement Level 1 should be allowed higher posting levels, and bridges with Enforcement Level 3 should be posted at lower posting levels (Imbsen et al., 1983).

Incidence of Maximum Load

AASHTO (1,p.24) recognizes that the probability of having a series of closely spaced vehicles of maximum allowed weight becomes greater as the maximum allowed weight for each unit becomes less. Similarly, structures with a lower posting have a higher incidence of maximum load and, therefore, a lesser stress level should be imposed. On the other hand, structures with a higher posting have a lower incidence of maximum load due to a more distributed load spectrum and, therefore, are allowed a higher stress level.

Years to Replacement

Terminal rating is a procedure that allows a bridge scheduled for replacement to be posted at a higher level. The theory is that a condemned bridge may be allowed to deteriorate at an accelerated rate. The pitfall of using terminal rating is that the scheduling of replacement funds cannot be guaranteed.

Fatigue

Certain types of bridges need to be investigated for fracture-critical details, such as partial-length

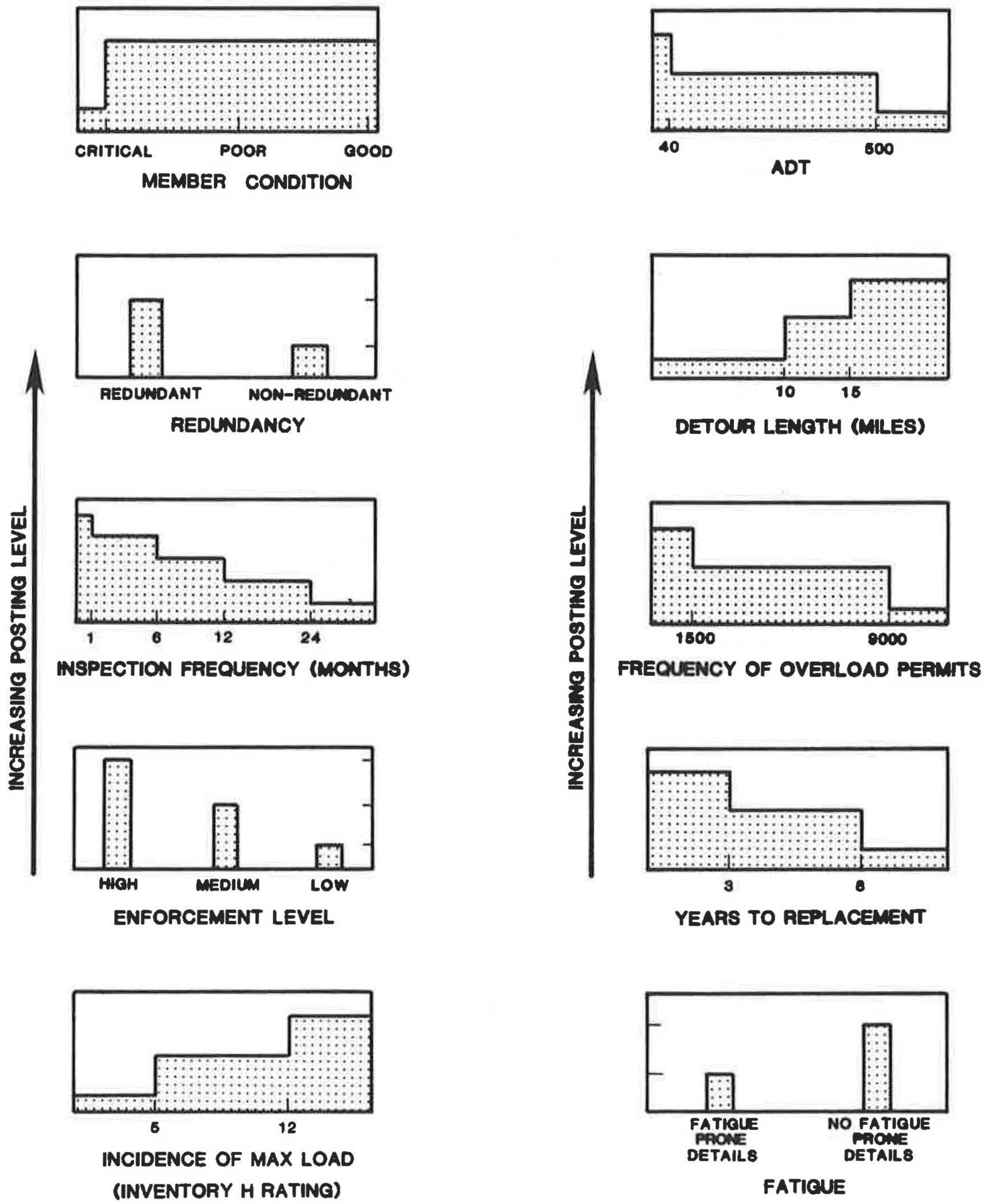


FIGURE 7 Factors affecting posting level.

cover plates. The detail must then be investigated to determine if it poses a fatigue problem. During the telephone survey, it was learned that one state limits its allowable strength to 129 percent of inventory. This was said to be done to reduce the risk of fatigue damage, which increases dramatically as the allowable stress is increased. This can be theoretically proven with a cumulative damage analysis. For example, a 10 percent reduction in the stress range will result in an increase of 10 to 30 percent in the fatigue life.

Detour Length

Of all the factors the research team considered in selecting a safe load posting level, the detour length around a posted structure is the only nonstructural consideration. It is recognized that the longer the detour, the higher the posting level required to offset the detour costs that will be incurred. It was found that most detour lengths are less than 6 mi. Breakpoints should be set high so that only exceptional cases are given special treatment. Breakpoints of 10 to 25 mi are chosen because 10 is near the arithmetic mean and 25 is at the 95th percentile.

Safe Load Posting Levels

The decision flowchart prescribes specific levels of posting that approximate even steps from inventory to operating rating. Actually the last step to

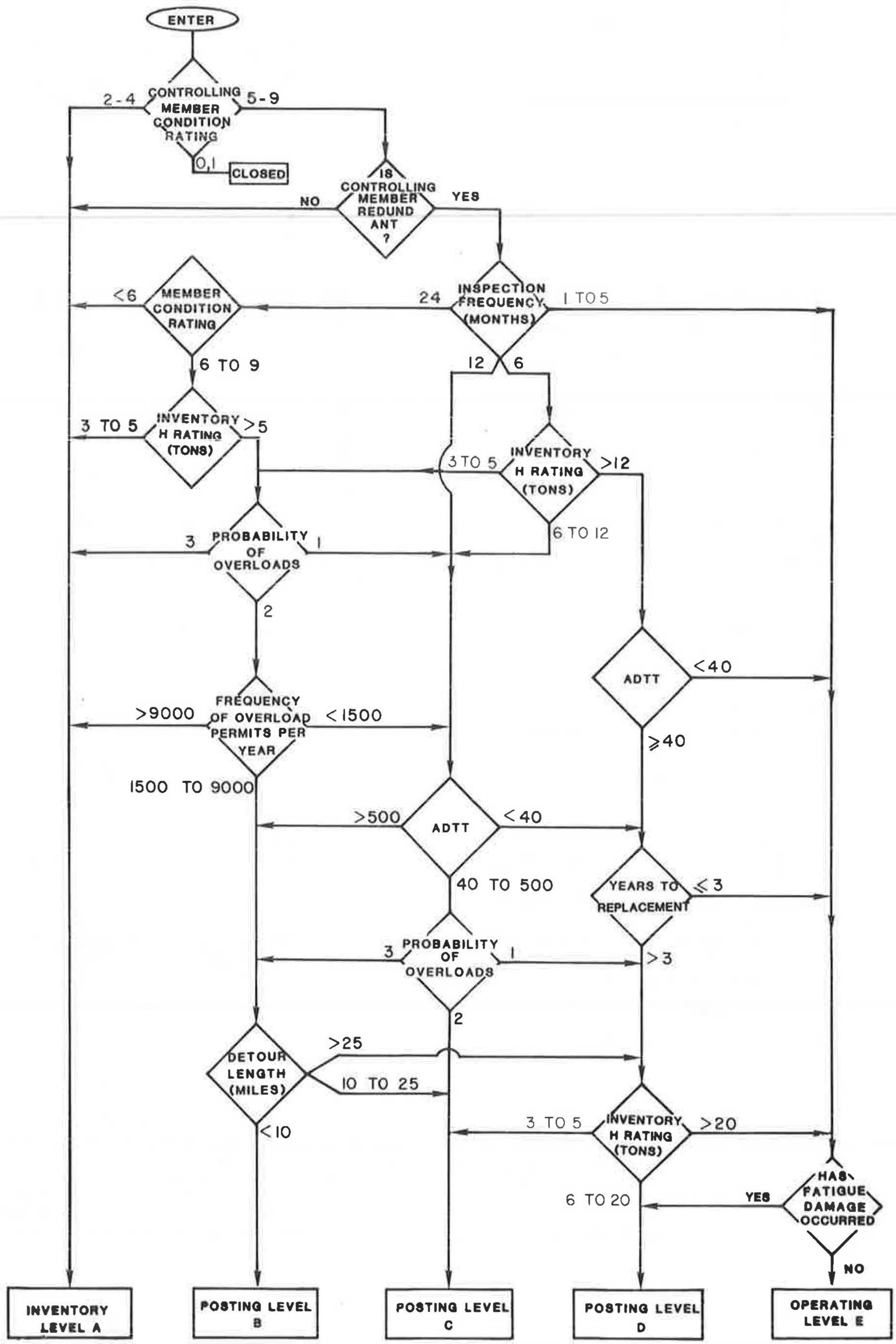


FIGURE 8 Safe load-decision flow chart for steel, timber, and concrete with plans.

operating rating varies, because operating as a percent of inventory differs for various materials. Figures 9 and 10 show the tabulation of the various posting levels. Figure 9 is for WSM and Figure 10 is for LFM.

POSTING LEVEL	CONC	STEEL	TIMBER
A	100	100	100
B	110	110	110
C	120	120	120
D	129	129	126
E	138	136	133

FIGURE 9 Safe load posting levels for working stress method.

POSTING LEVEL	SERVICEABILITY
A	$D + \frac{5}{3} L$
B	$D + \frac{4.5}{3} L$
C	$D + \frac{4}{3} L$
D	$D + \frac{3.5}{3} L$
E	$D + L$

FIGURE 10 Safe load posting levels for load factor method.

Safe Load Posting Level Decision Flowchart for Concrete Without Plans

A decision flowchart for reinforced concrete bridges where no plans exist is shown in Figure 11. The output of the flowchart is a posting value.

Signs of Distress

The 1978 AASHTO allows a bridge without record plans to be unposted when the structure has been carrying normal traffic for an appreciable length of time and shows no signs of distress. The researchers therefore recommended that bridges with an SI&A rating of 4 to 9 that show signs of distress should be posted for no more than 15 tons unless other circumstances allow for a greater capacity. Signs of distress are more fully defined in Figure 12.

Year of Design

The year in which a bridge was designed indicates the design load. Since 1944, most bridges have been designed with HS20 load. Before 1944, the majority of bridges were probably designed for a load somewhat less than HS20. Using this hypothesis, bridges built after 1944 showing no signs of deterioration should remain unposted. On the other hand, bridges built before 1944 showing signs of deterioration should be posted for a value of 15 tons unless other pertinent conditions prevail.

Rating

Analysis Method

The telephone survey showed that most states use WSM or a combination of WSM and LFM. Concurrently, many reasons were offered against LFM. From the comparative analysis of rating methods, the research team found that WSM and LFM yield nearly the same results, with LFM requiring notably more effort. Based on this observation, the policy should state that two levels of analysis be employed:

1. Analysis Level 1 would use WSM as the primary tool for bridge rating.
2. Analysis Level 2 would prescribe a second, more detailed analysis using revised rating criteria and the use of LFM, if deemed pertinent. The use of LFM should be reserved for instances when its use will be advantageous. These cases exist when a reinforced concrete span uses Grade 60 reinforcing steel or when a steel I-beam span uses compact sections and has laterally unsupported compression flanges.

Special Considerations

In cases where Level 2 analysis is performed, other factors in the analysis may be considered:

1. Use a three-dimensional computer analysis;
2. Use a more refined live-load distribution factor;
3. Reduce the impact factor in situations in which vehicle speeds can be effectively controlled;
4. Use the actual number of lanes that a structure carries, instead of design-lane loads;
5. Construct curbs to reduce the number of lanes or to place wheel loads in more favorable locations;
6. Erect signs or traffic lights to limit a bridge to one truck at a time;
7. Evaluate materials through sampling and testing; and
8. Use load testing to evaluate capacity.

Posting Limits and Rating Vehicles

The ML-80 truck, which has a gross vehicle weight (GVW) of 36.64 tons, was found to control single-vehicle posting in a significant number of the cases. For this reason it was decided that the ML-80 should be included as a rating truck and that the posting limit for single vehicles should be increased to 36 tons. Any state weighing tolerances will not be included here because the posting of bridges for a weight limit higher than that allowed by law would confuse the public.

Combination vehicles are allowed to have a GVW of up to 40 tons. As a result of the comparative analysis, it was found that the 36-ton HS20 truck controlled combination postings in all cases but one. The Type 3-4 truck controlled one posting. However, this vehicle is restricted to the Interstate system and to designated primaries only. In addition, the Type 3-S4 was also considered but rejected because it did not control posting.

It was recommended that bridges should be rated for the H vehicle and posted to a maximum of 19 tons, rated for the ML-80 vehicle and posted to a maximum of 36 tons, and rated for the HS20 vehicle and posted to a maximum of 35 tons. The exception is for the HS vehicle with a required posting greater than 35 tons when posting is required for single vehicles. In this

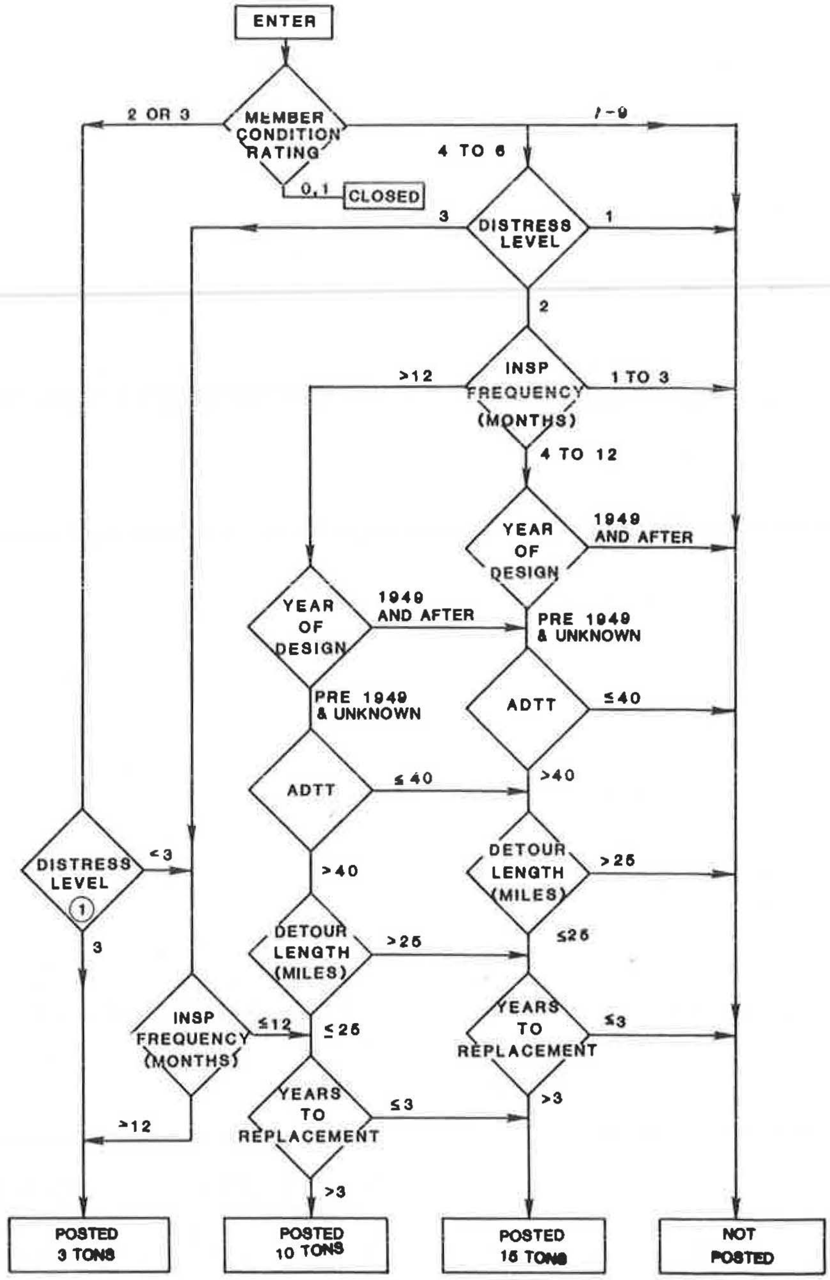


FIGURE 11 Safe load posting value decision flow chart for reinforced and prestressed concrete without record plans.

DISTRESS LEVEL PC	RC	DESCRIPTION
1	1	NO L.L. DISTRESS
1	2	HAIRLINE CRACKS
2	3	WORKING CRACKS
3	3	CRACKS PLUS VISIBLE L.L. DEFLECTION

FIGURE 12 Distress level.

case, the structure should be posted at the rated value for single vehicles and 40 tons for combination vehicles.

The Posting Process

General

Bridge management is a major activity of which the posting process is simply a part. The posting process begins with a field inspection. From the data collected, a load rating analysis of the structure can be performed for use in completing the FHWA structure inventory and appraisal data base. A load posting analysis--the focus of this study--can then be used to determine the maximum allowed load limit for posting. Finally, permit load rating analyses can be performed periodically to review overload permit applications.

Load Rating Analysis

The posting process, developed by the research team and shown in Figure 13, begins with the collection of data from the field inspection and with the load-rating analysis. It must then be determined if posting is required at the inventory rating. If not, the bridge remains in service unrestricted and is placed on a 2-year inspection cycle.

Load Posting Analysis Level 1

If posting is required at inventory rating, the decision flowchart can then be used to determine a safe load posting level. With this, the load posting analysis can be performed to determine if posting is still required. If not, the bridge is unposted, and the structure is scheduled for inspection every 12 months or less.

Load Posting Analysis Level 2

If posting is required at the safe load posting level, alternatives to posting are examined. These alternatives include an evaluation of posting criteria, such as wheel load distribution factor, impact factor, and the use of load factor method of analysis. The use of a more detailed analysis, such as three-dimensional computer modeling may be justified. If the evaluation reveals that criteria have changed, a second load posting analysis can be performed. The extent of this reevaluation depends on the posting value resulting from the Level 1 posting analysis and the minimum desirable posting criteria.

Operation of Posted Structure

The posting agency must now operate a posted structure. This includes notifying the public of the posted structure and detour routes. Examples of those affected include school and emergency services, the trucking industry (both local and long distance), and in some cases, local commerce. Other duties in operating a posted structure include maintaining signs, inspecting the structure every 12 months or less, enforcing posted load limits, and issuing overload permits.

Emergency Posting

Emergency posting as a routine part of the posting process was considered and rejected based on the conversations during the district field visits. Conditions rarely warrant temporary posting while emergency repairs are made. In addition, rapid reversals in the posted load limit before, during, and after emergency repairs only tend to undermine the user's confidence in the reality of bridge postings.

Most cases of emergency posting result from traffic accidents. This can be from an over-height vehicle striking an overpass or from river traffic striking piling. Emergency posting rarely results from the findings of a routine inspection. It was decided that if an emergency posting is truly an emergency, three options exist. The bridge should be (a) posted for 10 tons; (b) posted for no trucks, one-lane traffic, or both; or (c) closed to all traffic.

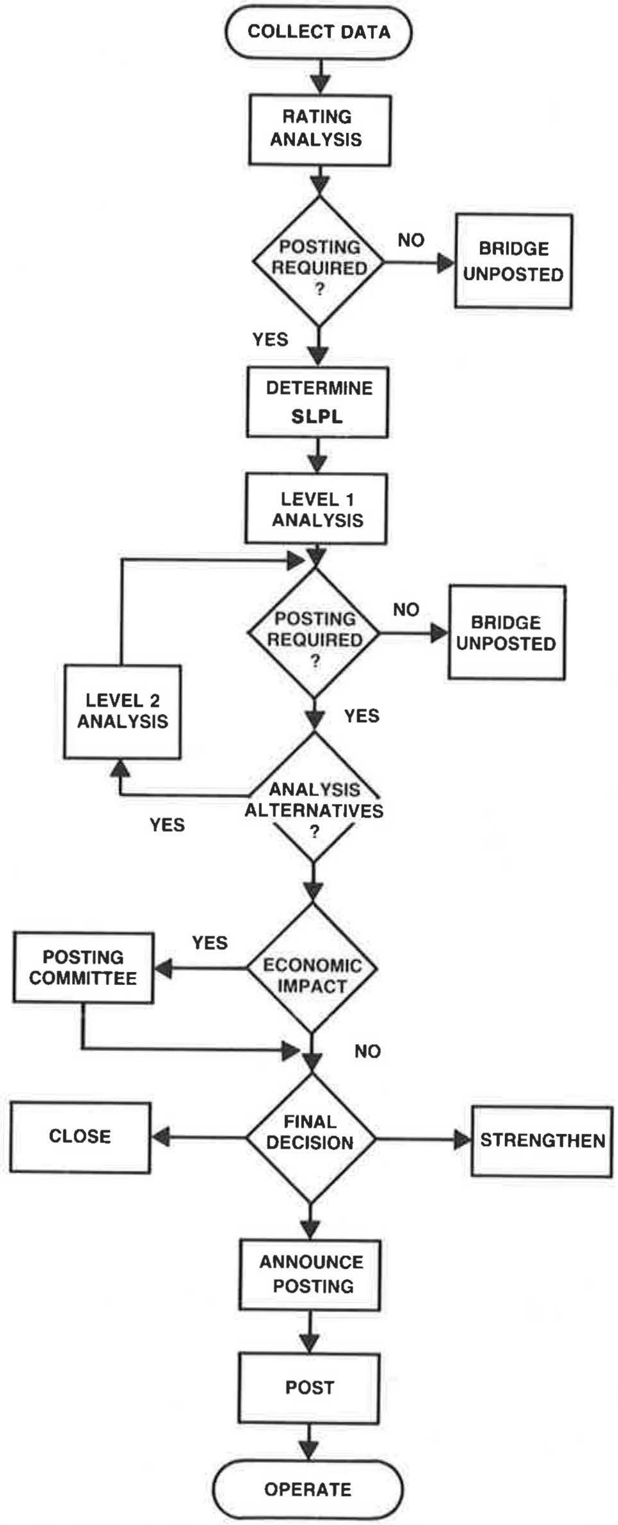


FIGURE 13 Posting determination process.

Time Limit

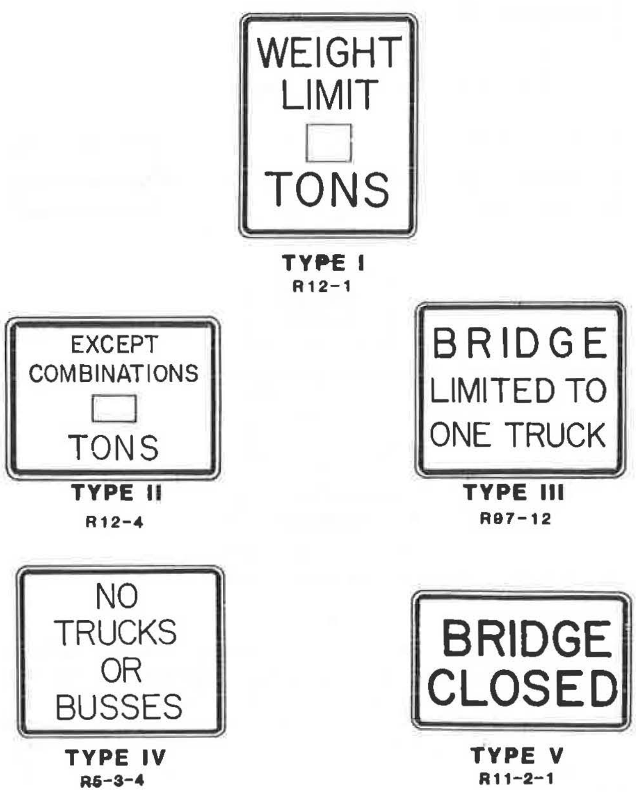
A time limit needs to be imposed on the posting process, so that a posting revision will not be held up in committees. The time limit from inspection to erection of the sign should be specified as a function of the percent reduction in the posting. This time limit should not exceed one month for a proposed posting reduction of less than 10 percent. A time limit of one week should not be exceeded for a posting revision of 10 to 50 percent. Beyond 50 percent, the time limit should be set at 24 hours. For example, if an unposted bridge is found to require a 12-ton posting, for an H-truck loading, a time limit of one week would be imposed, because the reduction in posting would be 8/20 = 40 percent.

Signs

During the district field visits, various sign types were discussed. These discussions aided the research team in selecting signs for the posting policy (Figure 14).

Type I: Weight Limit XX Tons

This sign specifies a single weight limit. As it is currently used, this sign restricts combination vehicles that generally accommodate more load because of the greater number and spacing of axles. This sign is best suited for structures with low weight limits where the difference between the two limits is small and the probability of there being a combination vehicle of low GVW is less. Otherwise, the Type I sign should be used in combination with Types II, III, or IV signs.



SOURCE: PENNDOT, OFFICIAL TRAFFIC-CONTROL DEVICES, PUB 68.

FIGURE 14 Posting signs.

The sign axle weight limit XXX lbs was also considered. This type of posting is more accurate than assuming that "each axle load maintains a constant relationship to the total load" (1,p.21). However, dual postings for axle loads and GVW will be more confusing. This sign also lacks a legal definition in the sponsoring state. For these reasons, the sign was rejected.

Type II: Except Combinations XX Tons

A bridge can be posted for both single and multiple vehicles when Type II and Type I signs are used, as they currently are in all districts. When the HS20 rating is equal to or greater than 36 tons and the Type I sign is required, this sign should call for a maximum limit of 40 tons to allow passage of all combinations.

Many states now use silhouette signs for multiple postings. Much information can be quickly grasped from these signs, which also lend themselves to the posting of double bottom combination vehicles. However, concern was expressed during the district field visits about the use of silhouette signs. These signs require a legal definition. Confusion by truck drivers was also anticipated concerning whether to count the number of trailing units or the number of axles. Word message signs are legally defined and understood by the user. Trends in other states and pressure for a uniform sign practice may eventually call for change in this policy.

Type III: Bridge Limited to One Truck

When special conditions warrant, the use of this sign along with the Type I sign will increase posting load limits.

Type IV: No Trucks or Buses

This sign is to be used with the Type I sign when the posted load limit is 3 tons. This sign will preclude the use of the bridge by trucks and buses and allow local officials to enforce weight limits without actually weighing them.

The sign Passenger Cars and Pick Ups Only was also considered for this application. This sign was ultimately rejected because it lists only two of many vehicles that are allowed to use the structure.

Type V: Bridge Closed

This sign is to be used when the structure must be closed.

CONCLUSIONS

The policy assessment was based on a small sample of bridges. The results of the analysis are tentative. However, the following conclusions are offered.

1. The proposed posting policy has the advantage over the existing policy of providing an objective decision-making tool for arriving at a desirable level of posting. The proposed policy does not eliminate engineering judgment but does provide a logical pattern for it.
2. The proposed posting policy will provide a uniform procedure for the selection of an optimum stress level with the use of the safe load posting-level decision flowcharts.

3. The proposed posting policy provides flexibility by allowing the engineer to perform a Level 2 analysis or inspect bridges at increased frequency.
4. The proposed posting policy protects the state's capital investment in its bridges. This is mainly a result of the protection from overloads provided by posting policy.
5. The proposed posting policy protects the public safety for the same reason as the aforementioned.

ACKNOWLEDGMENTS

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Prescription for Steel Girder Bridge Rehabilitation

RAYMOND E. DAVIS, RAMIN RASHEDI, and KHOSROW KHOSRAVIFARD

ABSTRACT

Increases in design or permit live loads, coupled with material deterioration, currently require rehabilitation or replacement of many bridges. Current AASHTO live-load distribution criteria, originally developed for fully loaded structures, result in ultraconservatism when applied to overload vehicles occupying only one or two lanes. Indiscriminate use of these criteria may suggest needless rehabilitation. Application, even if time-consuming, of currently available, sophisticated, computerized analyses that treat such partial-width loadings accurately may effect significant economies by demonstrating structural adequacy. Plans for overlayment on a deteriorated concrete deck on steel girders at a California site and upgrading for a new live-load permit vehicle, based on AASHTO distribution criteria, will require auxiliary steel flanges, introduction of composite behavior and web posttensioning to carry increased dead and live loads. A grillage analysis with the CURVBRG computer program suggests questionable need for posttensioning. Further analyses with FINPLA and STRUDL finite-element programs demonstrate a method for assessing web stresses at the posttensioning brackets.

In 1975 the California Department of Transportation (Caltrans) upgraded its specifications for bridge loadings with the objective of producing initial designs commensurate with later ratings for overload (1). The revisions introduced a modular design vehicle called the Permit- or P-series vehicle, significantly longer and heavier than H-series vehicles used previously. Unlike H-series trucks, P-series vehicles are used singly or in conjunction with an HS20 vehicle in an adjacent lane, and load factors are significantly lower than H-series factors.

California Department of Transportation, 1120 N St., Sacramento, Calif. 95844.

As a result of the new, heavier loadings, a program was initiated in which existing bridges on the state highway extra legal load (SHELL) system are screened to determine structural adequacy. For steel girder bridges, this screening utilizes the CURVBRG program, written by Mondkar and Powell at the University of California, Berkeley (2), and implemented into the Caltrans operating system by the Structural Research Unit in 1974 (3). Bridges that are proven to be structurally inadequate in the screening process must be either replaced or rehabilitated, the latter frequently by applying posttensioning forces to webs or flanges, by introducing composite behavior, or both.

Mancarti has discussed (4) current plans for major rehabilitation of this nature at the Yuba Pass overhead and separation. This work involves a number of phases including: (a) replacement of bearings; (b) addition of transverse and longitudinal stiffeners; (c) addition of auxiliary bottom flanges; (d) removal and replacement of existing curbs and railings; (e) scalping a thin, upper layer of the deck, placement of shear connectors to introduce composite action, and subsequent thickening of the deck; and (f) addition of reinforced steel brackets and posttensioning tendons to the sides of the girder webs. AASHTO-specified live-load distribution was used in determining requirements for strengthening.

Mancarti noted that posttensioning forces would be significantly larger than any applied previously in this manner in California, resulting in a request that the Structural Analysis Unit investigate distributions of prestress forces into webs in the vicinity of the brackets. The study is detailed in this paper.

DESCRIPTION OF PROTOTYPE

Figure 1 shows a general plan elevation view of the left structure. The first (simple) span is already composite, Spans 2 through 4 are not. Plans call for partially remedying this situation in the suspended sections of Spans 2 and 4 by removing portions of the deck down to the upper steel flanges and affixing shear connectors to produce composite action. An enlarged elevation showing posttensioning details is shown in Figure 2. Figure 3 shows the posttensioning

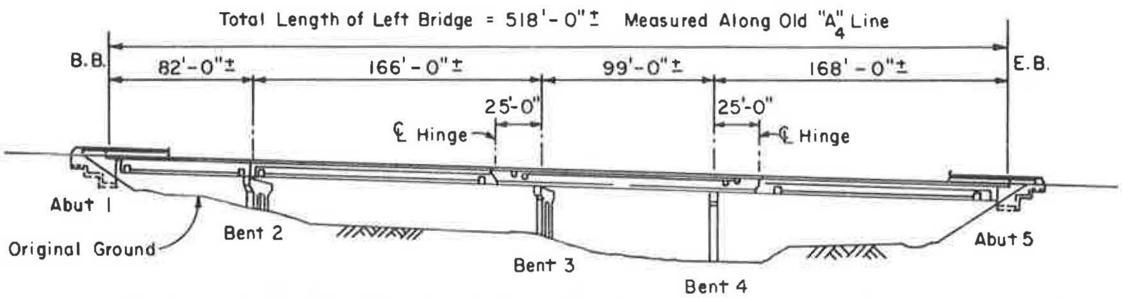


FIGURE 1 Elevation—Left bridge.

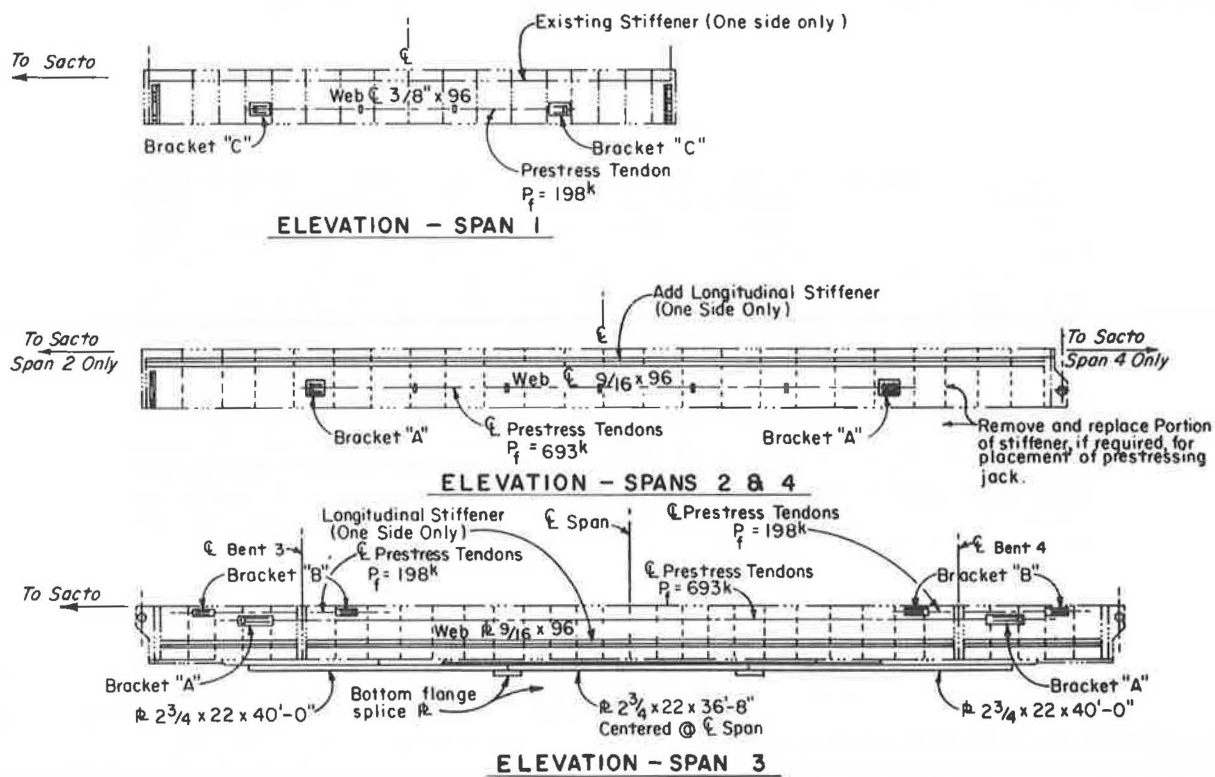


FIGURE 2 Web Posttensioning details.

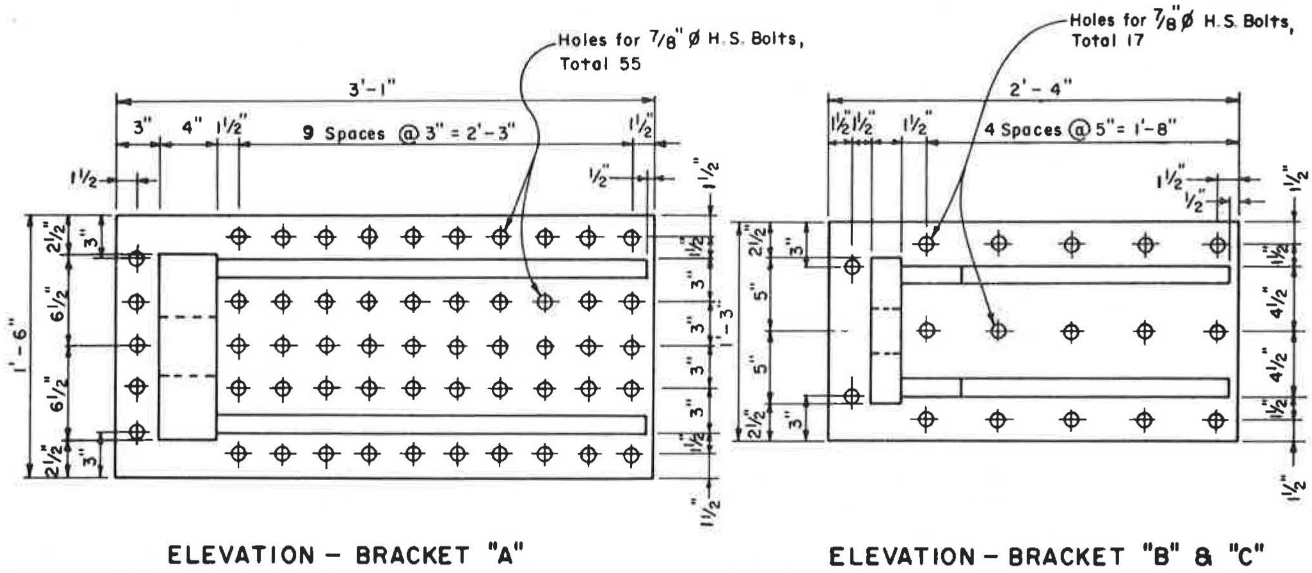


FIGURE 3 Posttensioning brackets.

brackets. The section between hinges, including Span 3, is controlled by negative moment and will remain noncomposite.

PROPOSED METHOD OF ANALYSIS

Available Programs

The current repertoire of programs available to the Structural Analysis Unit includes no single code, with the possible exception of STRUDL, capable of treating the entire problem. However, the following codes contain facilities that can be used to significant advantage in the analysis.

CURVBRG

This program provides a powerful tool for treatment of steel-plate girder bridges with concrete slabs. It is one of the few available programs for curved girder analysis that treats warping torsion. The program is not restricted to curved girders but may treat straight ones or those with angle points in plan.

This code treats articulation (hinging) of girders and uses a generator that greatly facilitates application of live loads to the structure. Output comprises envelopes of stresses, moments, shears, displacements, and so on. Construction stages may be specified so that scaling factors can be applied to physical properties of component materials that differ from stage to stage.

Load trains of wheel reactions at proper spacings may be applied and stepped along any user-specified lines of nodes. Various load cases can be superposed as load combinations with simultaneous application of multiplying factors that may include impact factors, load factors, and so on.

Use of a very simple grillage model minimizes the expense of program usage, even for large and complex structures; however, formulating input can be tedious, and determination of representative construction stages can be complex, in a problem such as this one. The analysis does not treat loads acting in horizontal planes, and, therefore, the important posttensioning forces. Nonetheless, CURVBRG will serve a useful function in the analysis.

CURVBRG's compilers have established favorable comparisons between experimental and program results from a number of curved girder model tests. In addition, field tests of six lightly instrumented plate girder structures by Caltrans' Office of Structures Maintenance, a literature search, and subsequent CURVBRG analyses of tested structures by the Structural Analysis Unit have yielded some excellent correlations between computed and measured strains (details to be published).

FINPLA

FINPLA (5) uses a finite-element analysis to treat prismatic folded-plate structures with eccentric plate and beam elements. The program was extensively tested by the senior author in 1968 on a steel box girder bridge over the Sacramento River at Bryte Bend (6,7) where excellent correlations were established between program-predicted strains and those measured by 1,200 strain-gauge circuits attached to steel elements or embedded in the composite, concrete deck slab. In these tests, the program's power to analyze steel box girder behavior was amply demonstrated, and the influences of longitudinal and transverse stiffening elements, diaphragms, frame elements, and so on, were considered. These capabilities strongly suggest use of FINPLA in this study, especially because it can readily treat forces in horizontal planes such as web posttensioning.

As far as is known, the code was not designed for analysis of open-web type structures and has not previously been used as such. It cannot treat the structure's articulation directly and there is no live-load generator, therefore, no convenient method of ascertaining critical live-load positioning.

STRUDL

STRUDL provides a finite element capability and undoubtedly is capable of treating the whole program but would be somewhat cumbersome in the total analysis of a structure and loadings as complex as these. More details of the program's use are provided.

General Plan of Analytical Procedure

The ultimate plan of approach involved using all three programs according to the following steps.

1. Make initial stress analysis using CURVBRG. To assess complete stress fields, begin from inception of bridge construction in 1961, considering all pertinent construction stages.
2. Because CURVBRG is a relatively inexpensive program to run, examine all potentially critical live-load conditions, including HS-series trucks, HS-series lane loadings, and P-series trucks in whatever combinations design specifications require. The factoring facility in the section on load combination permits convenient consideration of unfactored and factored (ultimate load) cases.
3. Compare critical load stresses with allowable stresses to confirm need for posttensioning.
4. Use CURVBRG to determine which type of live loading produces critical envelope stresses. Because program output comprises envelopes of stress maxima without preserving vehicle positions producing such maxima, make a guess pertinent to two or three potentially critical positions, and rerun CURVBRG with these input as single loadings to determine vehicle types and positions producing stresses comparable in magnitudes to envelope stresses. Record output displacements of each girder at hinges for dead and critical live loads for the longitudinal section from Bent 2 to the first hinge point. In the analysis of the longitudinal section between hinges, including Span 3, use output girder shears from CURVBRG to determine reactions on the cantilevered girders for subsequent input to FINPLA in lieu of displacements.
5. Establish input for FINPLA stiffness matrices using output displacements from CURVBRG for Span 2 up to the first hinge as boundary conditions at hinges. Note that separate stiffness matrices are required for dead and live loadings. For the section between hinges, include hinge reactions in the load vector.
6. Input dead and critical live loadings to FINPLA in conjunction with appropriate stiffness matrices.
7. Substructure Spans 1, 2, and 3 to FINPLA blocks containing posttensioning brackets and bounded by transverse sections spaced at 48 in. for the small brackets, and 52 in. for the large brackets.
8. Determine global X- and Z-displacement fields at FINPLA sections comprising substructure boundaries over web depths and across substructure (block) widths.
9. Use the POLYFIT program to establish polynomial regression functions for displacement fields in the FINPLA global X- and Z-directions at each of four boundaries for each substructured mesh.
10. Subdivide the two substructures into finite-element meshes with internal element dimensions comparable in size to bolts securing posttensioning brackets to the webs, using STRUDL CSTG constant-strain triangles and isoparametric IPLQ quadrilaterals with two (x and y) degrees of freedom at each node.
11. Subject substructure meshes to three, plane stress finite-element analyses with STRUDL:
    a. First, designate nodes at four boundaries as rigid supports, assume the posttensioning force to be distributed equally to each bolt, apply as uniformly distributed pressures on faces of small elements (1-in. squares) adjacent to holes.
    b. Second, apply no posttensioning forces but allow boundary nodes to displace as determined by FINPLA in Steps 8 and 9, because of dead, live, and impact loadings and posttensioning forces. Use POLYFIT regression functions established through a relatively small number of FINPLA nodes distributed over FINPLA section (web) depths and FINPLA block widths to assess displacements at all nodes comprising substructure mesh boundaries.
    c. Third, combine principal stress fields from a and b to assess total stress fields resulting from application of posttensioning forces on a mesh with boundaries free to move in accordance with restraints imposed by the surrounding web.

DETAILS OF ANALYTICAL PROCEDURES

CURVBRG Analysis

Despite CURVBRG's adaptability, analyses for various construction stages can be complex. The program permits use of successive construction stages to simulate, for example, erection of steel girders, pouring a concrete deck slab, placement of barrier curbs, application of live loads, and so on. Elastic properties (moduli, Poisson's ratios) and densities of each component material are input separately. In general, augmentation of concrete slab density is required because internal computation of selfweight (dead load) of deck elements will be based on thicknesses and effective widths input for girder section properties, and these dimensions may vary significantly from actual ones. Density augmentation may also be used to account for dead loads of steel stiffeners, cross-frames, wind bracing, and so on, which would not otherwise be included in dead-load computations for the steel girders.

Contributions to structural behavior of various component materials during different construction stages may be assessed by internal application of scaling factors used to reduce moduli. Typical input for material properties may be those given in Table 1. Note the requirement for unit consistency.

TABLE 1 Typical Input for Material Properties in Deck Construction

Material	Elastic Modulus (ksi)	Poisson's Ratio	Density (k/in.^{3})
Steel	29,000	0.285	0.0002836
Concrete (composite)	3,000[a]	0.180	0.0000868[b]
Concrete (noncomposite)	1	0.05	0.0000868[b]

[a]More accurate figures may be obtained from American Concrete Institute formulas.
[b]Concrete densities will usually be augmented as noted previously, and different material types will be required for exterior and interior girders because of differing augmented densities.

Because of stress relaxation in the concrete deck as a result of creep, it is customary to reduce the elastic modulus of deck concrete, usually by a factor of 4, for long-term loads such as barrier curbs, overlays, wearing surfaces, and so on.

The program accounts for effects of transverse slab strips on load distribution and production of force coupling in the slab, diaphragms, wind bracing, and so on; therefore, it will usually be necessary to specify two separate types of concrete for a noncomposite deck slab that contributes to transverse, but not to longitudinal, strength. Table 2 may be cited as typical input for the aforementioned materials in a simple case.

TABLE 2 Typical Input for Materials for Deck Construction

Material	Construction Stage	Activity	Scaling Factor
Steel	1	Erect bare steel	1
	2	Pour wet concrete deck slab	1
	3	Place barrier curbs	1
	4	Apply line loads	1
Deck concrete	1	–[a]	0
	2	–[a]	0
	3	–[a]	0.25
	4	–[a]	1

[a] Activities 1-4 for deck concrete same as those for steel.

With the Table 2 factors in mind, behavior of the prototype with the following construction stages should be considered.

- Stage 1: Erect bare steel (1961). The steel is given a scaling factor of 1, the 9 5/8-in. concrete deck, 0.
- Stage 2: Pour 9 5/8-in. deck slab (1961). Scaling factors are the same as in Stage 1 because concrete is still noncomposite in all spans for this load.
- Stage 3: Place existing barrier curbs (1961). Steel scaling factor is 1, deck concrete is 0.250 for long-term load in Span 1 and 0 in noncomposite Spans 2, 3, and 4.
- Stage 4: Place auxiliary steel flanges in Span 3, shear connectors in Spans 2 and 4 (1985). The dead loads of these flanges are carried by the steel girders only. Concrete has a scaling factor of 0. The scaling factor for existing steel is 1, for new steel, 0, and a separate material must be specified. For all subsequent stages, the auxiliary flange steel will be characterized by a scaling factor of 1; however, the new and existing steel flanges will be working at different stress levels, the existing flanges exhibiting stresses caused by loads of original components, the new flanges, those of components removed or added in subsequent stages.
- Stage 5: Remove 2 in. of deck and existing barrier curbs. Span 1 is now acting compositely with a 7 5/8-in. slab, and loadings are really short-term, and concrete scaling factors are somewhat questionable. In Spans 2, 3, and 4, the concrete deck is noncomposite, the scaling factor for concrete, 0.
- Stage 6: Add 6 3/8 in. of concrete deck in Span 1. In Span 1, the existing 7 5/8-in. deck is acting compositely to support dead load of the additional 6 3/8 in. of wet, strengthless concrete. Loading is long term and a scaling factor of 0.250 should logically be used for the 7 5/8-in. layer and 0 for the 6 3/8-in. layer. There are now two layers of concrete in the deck acting at different stress levels, the upper essentially unstressed and the lower stressed as a result of dead load of preexisting components plus the new layer. Actually, as a result of shrinkage, the upper layer will in time develop tensile stresses in itself, and the lower steel flanges will develop compressive stresses in the existing concrete layer and upper steel flanges.
- Stage 7: Add 6 3/8 in. of wet concrete to Spans 2, 3, and 4. Neither the 7 5/8-in. nor 6 3/8-in. layers of concrete acts compositely at this time; concrete scaling factor is 0, steel factor is 1, and the dead load of concrete is carried only by the steel girders.
- Stage 8: Place new barrier curbs. This loading is long term. All spans now have a 14-in. deck but Span 3 remains noncomposite with a scaling factor of 0. Concrete in Spans 1, 2, and 4 is composite with a scaling factor of 0.250.
- Stage 9: Apply live loads. These are short-term loadings. Deck concrete in Spans 1, 2, and 4 has a scaling factor of 1, and in noncomposite Span 3 a scaling factor of 0.

Output CURVBRG stresses for the critically loaded girders in Spans 1, 2, and 3 are plotted in Figures 4, 5, and 6. Except in Span 1, these stresses are produced in exterior girders under influence of ec-

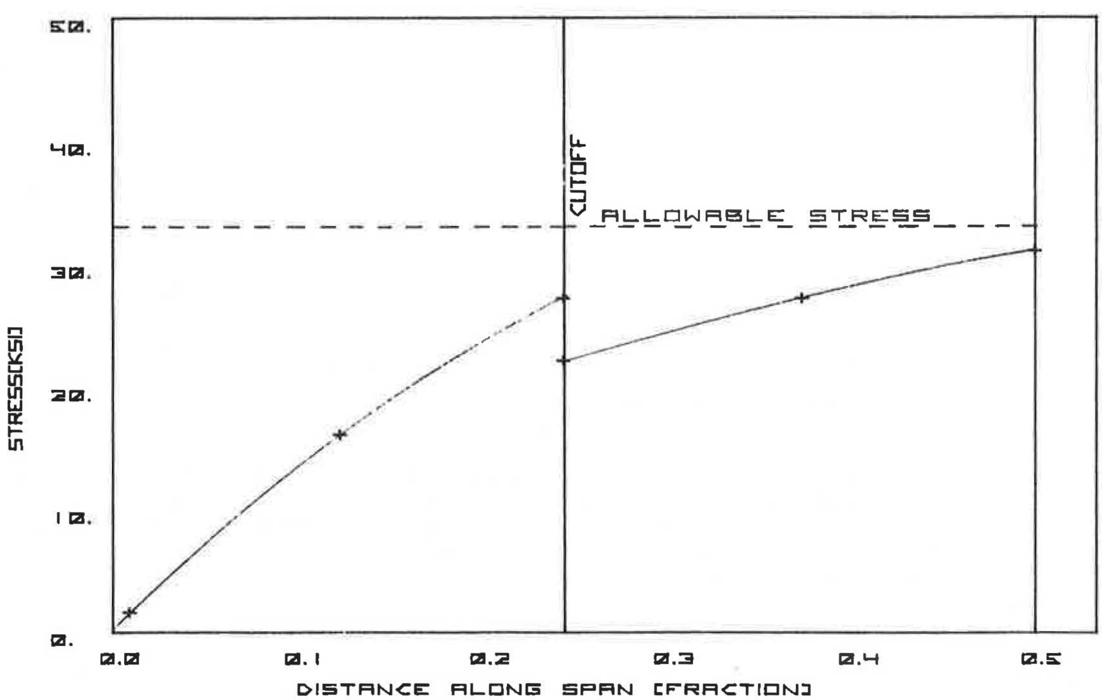


FIGURE 4 CURVBRG stresses Yuba Pass Overhead—Span 1.

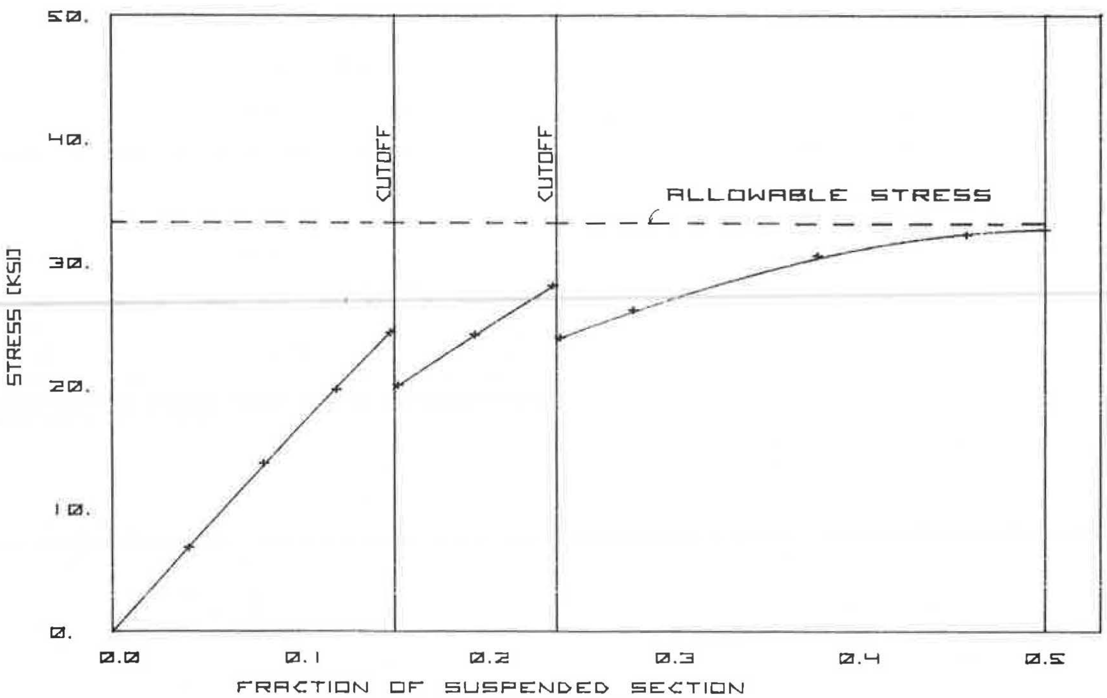


FIGURE 5 CURVBRG stresses Yuba Pass Overhead—Span 2 (suspended section).

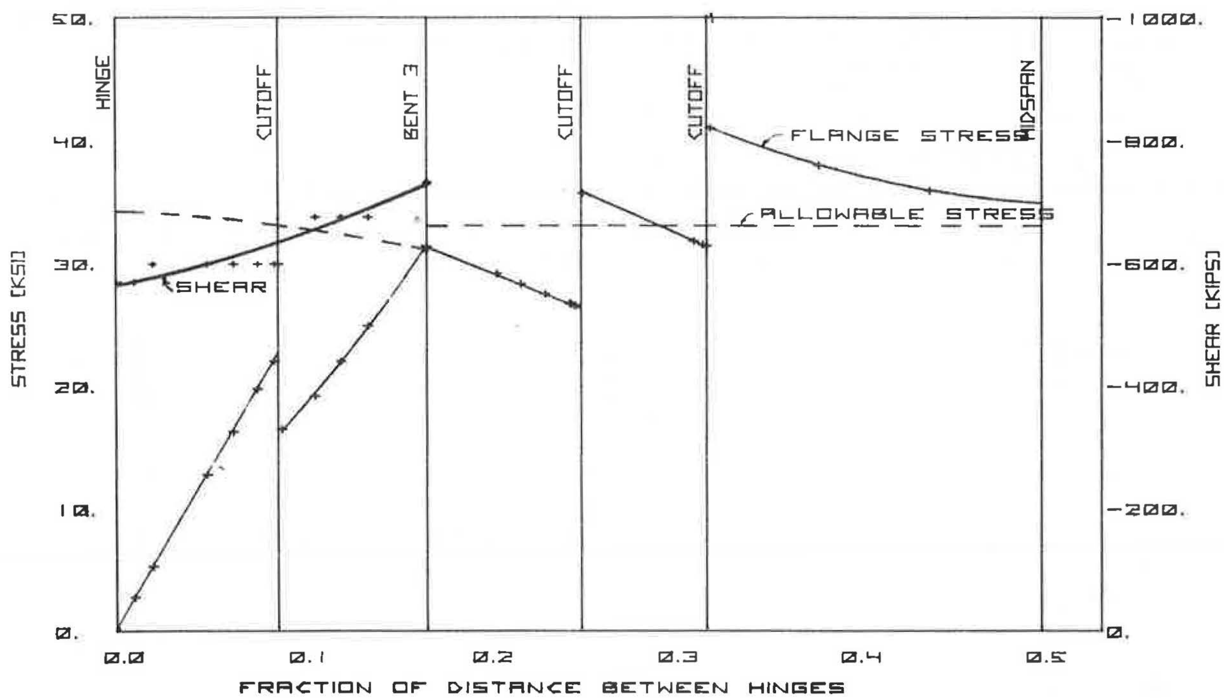


FIGURE 6 CURVBRG stresses Yuba Pass Overhead—Span 3 and cantilevers.

centrically placed P-series trucks, with H-series trucks in the adjacent lane, under which conditions total stresses from factored loads would be less than the 33-ksi yield stress in Spans 1 and 2 and at Bent 3. Stresses exceeding yield will occur at top flange cutoff points in Span 3.

FINPLA Analysis

The second phase of analysis utilized the finite element program, FINPLA, the primary use of which was to determine local effects of dead and live loads and general effects of posttensioning forces in the form of differential displacements at boundaries of blocks containing posttensioning brackets.

Loadings were of three types:

1. Dead loads of plate elements were expressed as regular surface loads (P/Plate area), which are calculated easily as products of plate thicknesses and material densities.
2. Loads of barrier curbs, longitudinal stiffeners, and other loads distributed longitudinally were

treated as distributed line loads, in kips per inch, acting at appropriate nodal joints between designated sections.

3. Loads of transverse stiffeners, wheel reactions, and so on, were treated as concentrated nodal joint loads, which, in the program, are entered as distributed line loads with the same beginning and end sections.

STRUDL Analysis

The total posttensioning force was divided equally among the bolts and applied as uniform pressures acting on faces of elements adjacent to the holes.

STRUDL output plots for a large bracket in Span 3 are shown in Figures 7 and 8, the former for major principal (maximum tensile) and the latter for minor principal (maximum compressive) stresses due to unfactored prestress forces and factored dead, live, and impact loads, respectively. Plots of the large bracket for Span 3 demonstrate that localized stresses significantly in excess of 33-ksi yield stresses are calculated in the vicinity of the bolts, some as large as 44 ksi. Major tensile stresses are concentrated along the column of bolts nearest the bracket edge in a direction opposed to that of the posttensioning force, while major compressive stresses are concentrated on the opposite row of bolts.

Several factors should be noted in connection with these plotted stresses:

1. No attempt has been made to include effects of nonlinear material behavior. Except for the influence of strain hardening, steel cannot be expected to sustain stresses much in excess of yield stress; therefore, the yield stress contour might be expected to spread over a broader area than indicated by the plots.

2. Whereas the designer specified the highest level of friction bearing between brackets and webs, included in this analysis was the tacit assumption that the bearing was frictionless and that the posttensioning force was distributed equally to the bolts, with no beneficial effects from clamping. If clamping had been assumed to be 100 percent efficient in the analysis, it would have been reasonable to spread the prestress force over the entire bracket depth, almost certainly eliminating the large concentrations of stress, but not necessarily eliminating widespread yielding along bracket boundaries.

3. Plots included here are for the most critical combinations of loadings, with live loading included, and are, therefore, not necessarily the most critical without live loading in all cases. Therefore, the girder analyzed in all cases has been that with maximum tensile stresses due to dead, live, and impact loadings. Tensile stresses on the un-prestressed end of the bracket should be the largest that can be expected in any girder, but compressive stresses at the other end of the bracket would be less than those if live loads were not included.

Potential Design Revisions in Span 3

CURVBRG results indicate need for posttensioning only in Span 3, where calculated posttensioning force required to bring CURVBRG calculated overstress within the yield stress of the web steel is 398 kips, significantly less than the 693-kip value indicated by the AASHTO load distribution specifications.

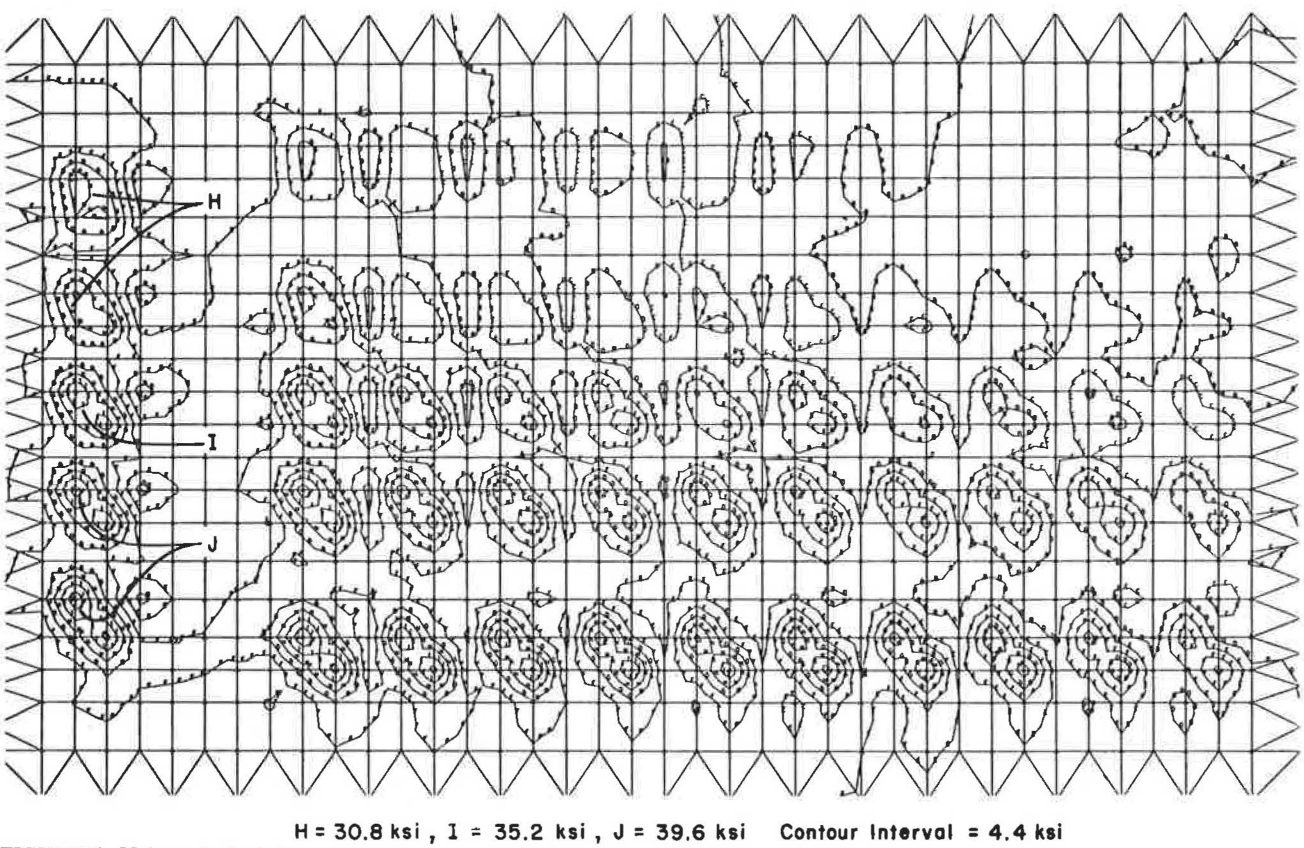
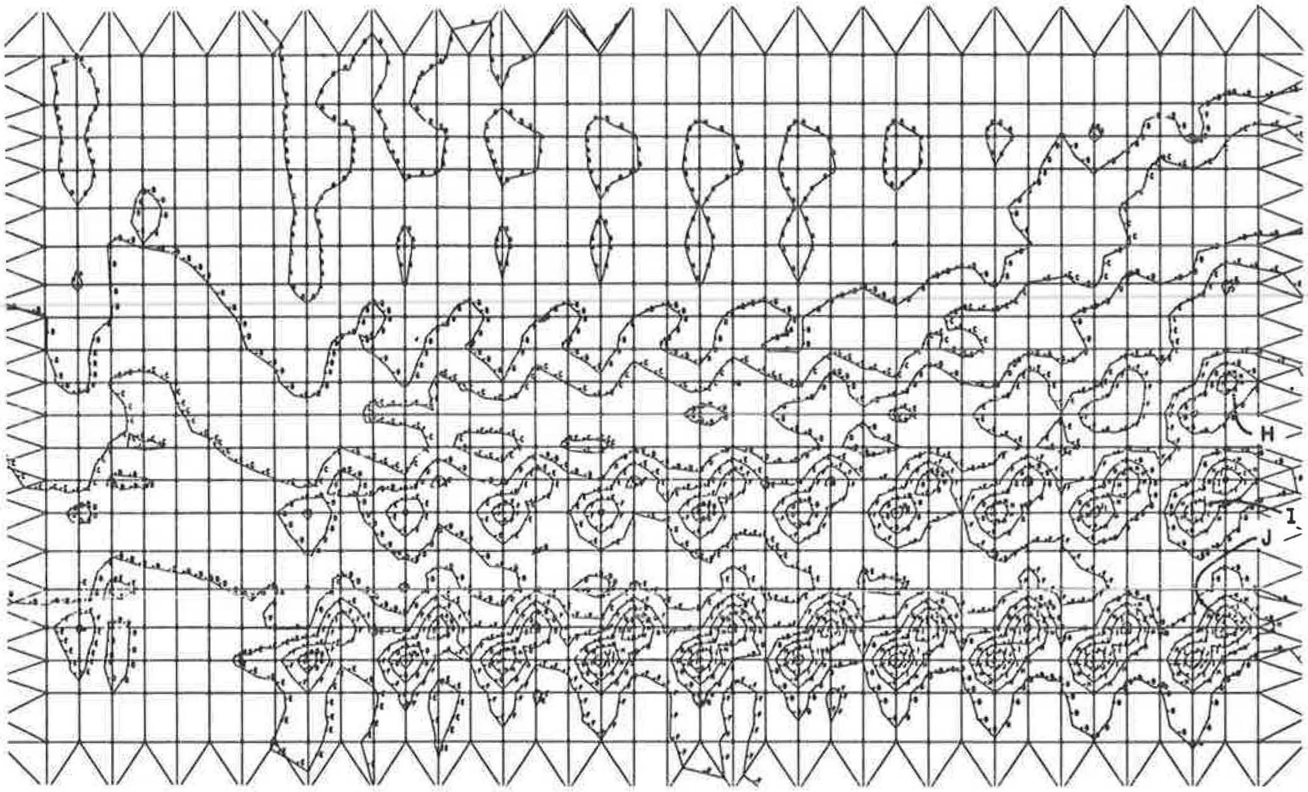


FIGURE 7 Major principal stress.



H =-23.8 ksi , I =-27.2 ksi , J =-30.6 ksi Contour Interval = 3.4 ksi

FIGURE 8 Minor principal stress.

SUMMARY AND CONCLUSIONS

Theoretical analyses (8) were made of the Yuba Pass overhead and separation, an existing plate girder and concrete deck structure, to assess need for posttensioning girder webs as recommended by the Caltrans' Office of Structures and Design, in conjunction with other rehabilitation for recent Caltrans live-load revisions and to remedy a deteriorated deck. Three main programs were used in the analyses, two of which (CURVBRG and FINPLA) emanated from the Caltrans research project Analysis, Design and Behavior of Highway Bridges (9).

CURVBRG, a grillage analysis developed for curved (or straight) plate girder bridges was used to

1. Assess maximum stresses resulting from complex construction phases from initial construction through repair phases exclusive of posttensioning;
2. Determine design vehicle types and locations for production of maximum stresses due to live load; and
3. Compute boundary (hinge) displacements or reactions, as required, for input to the second program, FINPLA.

FINPLA, a program previously used in analyses of cellular, folded plate structures, was used to produce relatively coarse finite-element analyses of the structure for dead, posttensioning, and critical live loadings to assess displacement fields at boundaries of small, substructured areas surrounding posttensioning brackets.

Substructured bracket areas were finally subdivided into finite-element meshes and analyzed by a third program, STRUDL, in three phases to assess principal local web stresses resulting from

1. Application of posttensioning forces with zero boundary displacements;
2. FINPLA output displacements at mesh boundaries due to dead, live, impact, and posttensioning forces; and
3. Combined posttensioning forces and boundary displacements.

CURVBRG analyses indicated subyield web stresses in Spans 1, 2, and 4, and at Bent 3 for factored load conditions without posttensioning. Stresses somewhat in excess of yield were computed at flange cutoff points in Span 3; however, the required posttensioning force was about one-half of that indicated by the use of AASHTO live-load distribution specifications in Caltrans' Office of Structures and Design.

The STRUDL finite-element analyses indicated localized stresses as high as 44 ksi (yield stress, 33 ksi) in the vicinities of bolts affixing brackets to webs.

RECOMMENDATIONS FOR IMPLEMENTATION

The authors recommended that all Caltrans designs for new structures or rehabilitation of existing structures that employ steel girders be reviewed with CURVBRG, with due consideration for all phases of construction to determine stresses, especially those resulting from partial-width live loads (e.g., Permit-series or combined P- and H-series loadings), for which AASHTO load distribution criteria may be highly conservative. Significant savings may be realized in new designs, and expensive rehabilitation of existing structures may be avoided by use of the program.

Use of CURVBRG in an analysis with complex construction procedures is illustrated and methods used to determine principal local web stresses are demonstrated. However, before future, extensive analyses are performed, the establishment of guidelines pertinent to stresses that will be considered excessive is recommended.

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