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Pasco-Kennewick Bridge

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COVER: The Pasco-Kennewick Bridge (Washington State), which won the Presidential Award for Design Excellence in 1985, exemplifies excellence in all aspects of bridge design.

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Bridge Drainage System Needs Criteria

by
Dah-Cheng Woo

Introduction

Are bridge deck drainage systems really needed on every bridge? Such systems are customarily provided, but the evidence suggests that they may not be useful as designed. For instance, it is not unusual to find that drainage grates are completely plugged and covered with weeds; yet, when it rains, the stormwater just runs off the bridge without causing any problems. A thorough reevaluation of bridge deck drainage systems is therefore in order.

A bridge drainage system consists of a bridge deck drainage system and bridge-end drainage provisions. A bridge deck drainage system consists of bridge deck gutter scuppers and bridge deck surface drainage provisions. As mid-deck scuppers are not desired on most bridges, vehicle hydroplaning danger must be checked. This article presents a methodology for determining where bridge deck gutter scuppers are or are not needed, and a procedure for testing where the danger of vehicle hydroplaning should be addressed. They are developed from existing knowledge, and an example of their application is illustrated. Also presented are general guidelines for determining the need for bridge-end provisions.

The designs of the gutter scuppers and the bridge-end drainage provisions, if they are needed, are not included in this article.

Needs of a Bridge Deck Drainage System

When rain falls on a bridge deck, it runs off the pavement surface as sheet flow, then into the gutter section to form gutter flow (figure 1). This gutter flow grows as it moves down the longitudinal slope of the bridge deck, and its spread increases along the way. Sheet flow poses a hydroplaning danger to the bridge's vehicle traffic; gutter flow interferes with vehicle traffic when it encroaches on the traveling lane. The objective of bridge deck drainage is to control sheet flow and gutter flow to eliminate their adverse effects on bridge traffic.

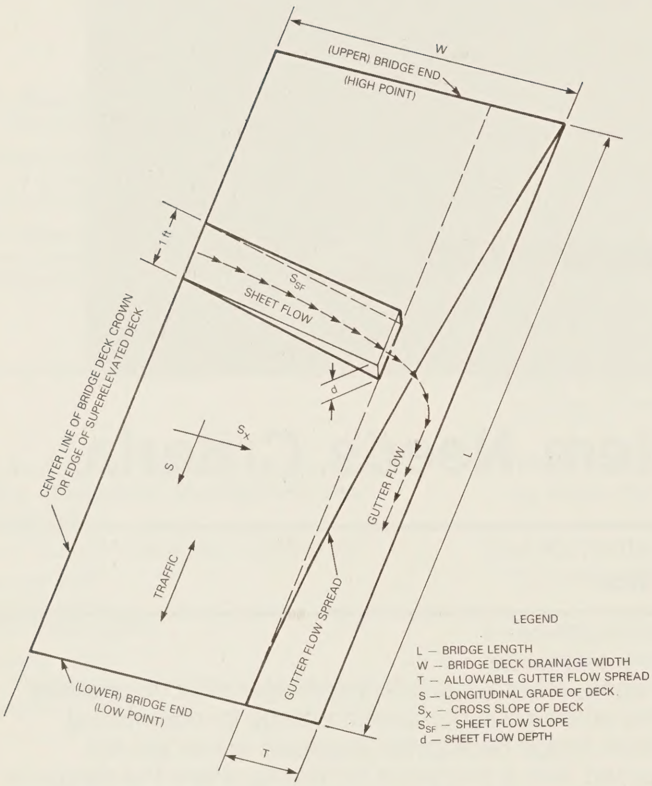


Figure 1.—Bridge deck flow diagram.

However, since a storm is a natural event, its magnitude and frequency are not subject to human control. Bridge deck drainage therefore can only be designed to control its effects to a certain manageable level. Because it is not possible to use the bridge deck drainage design to dictate the design storm, the design storm must first be determined.

The discussion of the needs includes the following items:

- Selection of the design storm.
- Gutter scupper requirements.
- Hydroplaning considerations.

Selection of design storm

The Rational Method approach of time of concentration is used in selecting the design storm. From the established policy of highway drainage design, the return period (or frequency) of the storm is first selected; then the time of concentration to the first scupper is calculated. The design storm can be determined by a trial-and-error procedure.

Time of concentration (t_c) can be computed by the following equations: (*1*)[1]

$$t_c = t_o + t_g \quad (1)$$

$$t_o = \frac{0.93\,(W^{0.6})\,(n^{0.6})}{(i^{0.4})\,(S_x^{0.3})} \quad (2)$$

$$t_g = \frac{L_o}{60\,V_a} \quad (3)$$

$$V_a = \left(\frac{1.12}{n}\right)\,(S^{0.5})\,(S_x^{0.67})\,(T_a^{0.67}) \quad (4)$$

$$T_a = 0.65\,T$$

Where:

t_c - time of concentration (min)
t_o - time of surface runoff (min)
t_g - time of gutter flow (min)
W - width of drainage area (ft)
n - Manning's roughness coefficient
i - rainfall intensity (in/hr)
S_x - cross slope of bridge deck surface
L_o - distance from high point to first scupper (ft)
V_a - average gutter flow velocity in a reach (for T from O to T) (ft/sec)
S - longitudinal slope of bridge deck surface
T_a - spread for average gutter flow velocity, V_a (ft)
T - design spread of gutter flow (ft)

[1] Italic numbers in parentheses identify references on page 36.

When applying these equations in checking gutter scupper requirements, the length of the design bridge L is used here to equal L_o in equation 3. The nomograph solutions of equations 2 and 4 are plotted in figures 2 and 3. (*1*)

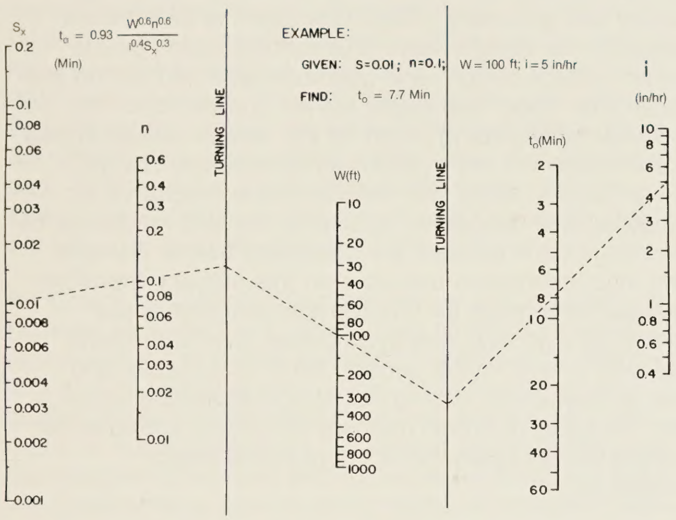


Figure 2.—Time of surface runoff t_o (min).

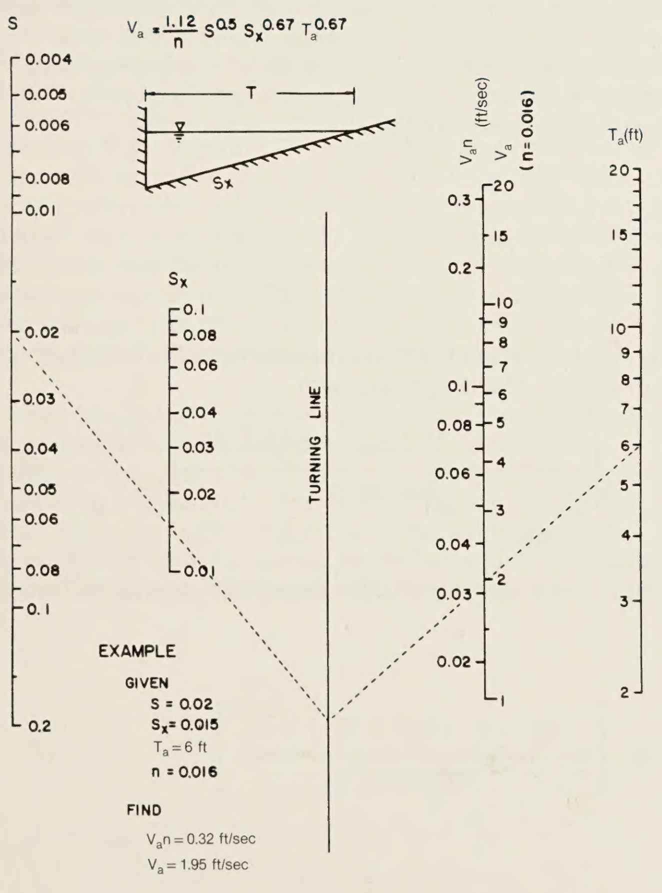


Figure 3.—Average gutter flow velocity in a reach of triangular channel V_a.

To determine the design storm, a trial value of rainfall intensity i must be assumed first, then the time of concentration t_c is computed. From the known local intensity-duration-frequency (I–D–F) curve, the rainfall intensity for the selected return period and the computed t_c can be obtained, then checked with the assumed trial value. If they are not reasonably close, a new trial value of i should be assumed; this procedure is repeated until the assumed and obtained rainfall intensities are almost the same.

There are other methods for determining the design storm. In some States, for example, the State highway agency specifies the general or regional storms for use in its highway drainage designs.

Gutter scupper requirements

To prevent gutter flow from interfering with vehicle traffic, it should be kept within the allowable or design gutter flow spread T at any place on a bridge deck for the design storm.

The quantity of the gutter flow increases gradually with distance as it picks up more sheet flow on the way down the gutter. Because of the disturbance caused by the passing vehicle traffic and the continuous interfacing of sheet and gutter flow, the mechanics of this flow are very complicated. For the purpose of the present discussion, the bridge length for reaching the allowable gutter flow spread can be computed by the following simplified equation: (*2*)

$$L = \frac{24400 \left(S_x^{1.67}\right) \left(S^{0.5}\right) \left(T^{2.67}\right)}{C \; n \; i \; W} \qquad (5)$$

Where:

L - bridge length (ft)
C - coefficient of imperviousness in Rational Formula
n - Manning's roughness coefficient representing bridge deck surface roughness
i - design storm intensity (in/hr)

For parking lots and pavements, C is usually taken as 0.9—this assumes that some of the storm runoff is trapped and stored on the pavement surface. For a typical deck pavement surface, n is usually taken as 0.016. A nomograph of this equation based on these assumptions is presented in figure 4. Using this equation for a planned bridge with the given design bridge deck slopes S and S_x and allowable gutter flow spread (T), the bridge length can easily be checked to see if gutter scuppers are needed for the design storm.

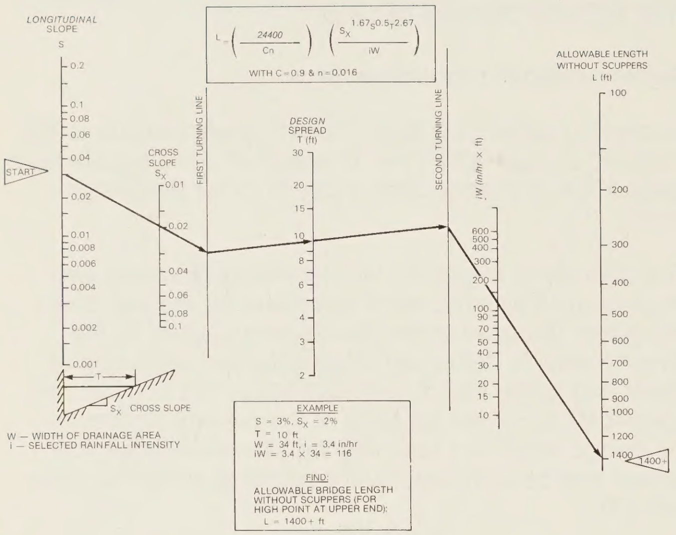


Figure 4.—Gutter scupper requirement nomograph.

Hydroplaning considerations

Hydroplaning is defined here as the complete loss of tire and pavement contact, thus the control of a vehicle. Technically, hydroplaning occurs when the wheel spindown rate reaches 10 percent. This process starts with a reduction in tire-pavement friction caused by water on the pavement. As water depth increases, the wheel can—at certain speeds—become completely locked.

Although the occurrence of accidents on wet pavement is a well-established problem, it was not investigated closely until a recent report by Harwood et al., which discussed wet-pavement exposure and accident frequencies for vehicle speed of 40 mi/h (64.4 km/h) or less. (*3*) Although pavement friction is greatly reduced on wet pavement for a vehicle traveling at 40 mi/h (64.4 km/h), hydroplaning could happen only when water depth is 1 in (25.4 mm) or more. (*4*) The hydroplaning consideration discussed here is limited to the maximum allowable speeds of 55 and 65 mi/h (88.5 and 104.6 km/h).

Hydroplaning involves many elements: Design storm characteristics (intensity and duration), vehicle characteristics (speed, load, and tire properties), and surface texture and geometry. Sheet flow depth is determined based on the design storm characteristics and on the known surface texture and geometry selected for the particular site. Sheet flow depth is then compared to the potential hydroplaning depth for the design vehicle speed to determine the site's potential hydroplaning danger. These factors, along with the corrective measures for mitigating this danger by increasing the skid resistance of the bridge deck surface, are discussed below. There is very little information available on this subject; therefore, the best knowledge for the average condition is presented as a general design guideline. Due to the facts that only a very limited general trend is available, and that an individual's driving habits and ability are uncertain, the effect of rainfall intensity on vehicle speed is discussed but not taken into account in this design procedure.

- *Determine sheet flow depth* — As shown in figure 1, after rain hits the bridge deck surface, water flows in a sheet across the surface to the edge of the gutter flow. Consider a 1-ft wide sheet flow path from the high point to the edge of the gutter flow spread; the water depth varies from 0 at the high point (center line of bridge deck crown or edge of super-elevated deck) to the sheet flow depth d at the edge of the gutter flow spread (W–T). The sheet flow follows a length of:

$$\frac{(W-T)}{S_x}\quad\sqrt{\left(S_x^2+S^2\right)}$$

and has a flow depth d (ft) computed from the Rational Formula and Manning's equation,

$$d = \left[\frac{C\,i\,n\,(W-T)\,\left(S_x^2+S^2\right)^{0.25}}{64900\;S_x}\right]^{0.6} \qquad (6)$$

For C = 0.9 and n = 0.016, equation 6 can be written as

$$d = \left[\frac{i\,(W-T)\,\left(S_x^2+S^2\right)^{0.25}}{4507000\;S_x}\right]^{0.6} \qquad (7)$$

From the given design conditions, the sheet flow depth (d) (ft) at (W–T) is obtained.

- *Check for vehicle speed and rainfall intensity* — The influence of rainfall on traffic flow was qualitatively studied in a previous work sponsored by the Federal Highway Administration (FHWA). (*5*) This study showed a general reduction of traffic speed in rain by comparing the actual observed traffic flow in dry weather to that under various rainfall intensities. In a more recent study, an empirical relationship among rainfall intensity, driver visibility, and vehicle speed was derived. (*6*) In both studies, limited field data were obtained. Although both studies show a reduction of average vehicle speed in rainfall conditions, there are drivers who always drive much faster than the average. Therefore, for a safe design, the design vehicle speed used here in checking hydroplaning danger does not allow for any reduction because of heavy rain.

- *Check for hydroplaning dangers* — Hydroplaning on a highway pavement is a complicated problem for which it is very difficult to collect field data to validate the laboratory and analytical results. In fact, only two meaningful studies have been conducted on hydroplaning on pavement. A Texas A&M study, sponsored by the FHWA, was the only one conducted with full-scale field tests on highway pavement; a German study relied on a detailed laboratory test of automobile tire performance on wet pavement. (*4*, *7*) However, each study has its own limited study conditions. From the test results of these two studies, empirical equations relating vehicle speed to tire pressure and tread depth, percentage of tire wheel spindown, pavement texture depth, and water depth were derived. Assuming average values of these variables and a bridge deck surface texture (TXD) of 0.038 in (1.0 mm), the rainfall intensities for potential hydroplaning danger are obtained from equation 7. Table 1 illustrates these values for vehicle speed of 55 mi/h (88.5 km/h) and table 2 for vehicle speed of 65 mi/h (104.6 km/h). By increasing the bridge deck surface texture to 0.076 in (1.93 mm), it is estimated that the hydroplaning rainfall intensity for a vehicle speed of 55 mi/h (88.5 km/h) can be raised by 500 percent over that shown in table 1; and for 65 mi/h (104.6 km/h), by 60 percent over that shown in table 2.

These tables show that the hydroplaning rainfall intensity decreases with increasing bridge width due to the increase in length of sheet flow on the bridge deck, or with increasing longitudinal slope of the bridge deck. On the other hand, increasing the cross slope will increase the hydroplaning rainfall intensity as the velocity of the sheet flow on the bridge deck increases, thus reducing its depth.

Table 1.—Hydroplaning rainfall intensity, i (in/hr)[1]
For: V = 55 mi/h (hydroplaning sheet flow depth d = 0.08 in)[2]
n = 0.016
C = 0.9
TXD = 0.038 in

S	S_x	(W – T) 24	36	48	58
0.01	0.01	3.7	2.5	1.9	1.5
	0.02	5.9	4.0	3.0	2.5
	0.04	8.7	5.8	4.4	3.6
	0.06	10.8	7.2	5.4	4.5
	0.08	12.5	8.3	6.2	5.1
0.02	0.01	3.0	2.0	1.5	1.2
	0.02	5.3	3.5	2.6	2.2
	0.04	8.4	5.6	4.2	3.5
	0.06	10.6	7.1	5.3	4.4
	0.08	12.3	8.2	6.2	5.1
0.04	0.01	2.2	1.5	1.1	0.9
	0.02	4.2	2.8	2.1	1.7
	0.04	7.5	5.0	3.7	3.1
	0.06	9.9	6.6	5.0	4.1
	0.08	11.8	7.9	5.9	4.9
0.06	0.01	1.8	1.2	0.9	0.7
	0.02	3.5	2.4	1.8	1.5
	0.04	6.6	4.4	3.3	2.7
	0.06	9.1	6.1	4.6	3.8
	0.08	11.2	7.5	5.6	4.6
0.08	0.01	1.6	1.0	0.8	0.7
	0.02	3.1	2.1	1.5	1.3
	0.04	5.9	4.0	3.0	2.5
	0.06	8.4	5.6	4.2	3.5
	0.08	10.5	7.0	5.3	4.4

[1]Computed from equation 7
[2]Determined from ref (4)

Table 2.—Hydroplaning rainfall intensity, i (in/hr)[1]
For: V = 65 mi/h (hydroplaning sheet flow depth d = 0.047 in)[2]
n = 0.016
C = 0.9
TXD = 0.038 in

S	S_x	(W – T) 24	36	48	58
0.01	0.01	1.5	1.0	0.8	0.6
	0.02	2.4	1.6	1.2	1.0
	0.04	3.5	2.4	1.8	1.5
	0.06	4.4	2.9	2.2	1.8
	0.08	5.0	3.4	2.5	2.1
0.02	0.01	1.2	0.8	0.6	0.5
	0.02	2.1	1.4	1.1	0.9
	0.04	3.4	2.3	1.7	1.4
	0.06	4.3	2.9	2.1	1.8
	0.08	5.0	3.3	2.5	2.1
0.04	0.01	0.9	0.6	0.4	0.4
	0.02	1.7	1.1	0.8	0.7
	0.04	3.0	2.0	1.5	1.2
	0.06	4.0	2.7	2.0	1.7
	0.08	4.8	3.2	2.4	2.0
0.06	0.01	0.7	0.5	0.4	0.3
	0.02	1.4	1.0	0.7	0.6
	0.04	2.7	1.8	1.3	1.1
	0.06	3.7	2.5	1.8	1.5
	0.08	4.5	3.0	2.3	1.9
0.08	0.01	0.6	0.4	0.3	0.3
	0.02	1.2	0.8	0.6	0.5
	0.04	2.4	1.6	1.2	1.0
	0.06	3.4	2.3	1.7	1.4
	0.08	4.3	2.8	2.1	1.8

[1]Computed from equation 7
[2]Determined from ref *(7)*

If the design storm at a given bridge site is larger than those listed in tables 1 and 2, then hydroplaning could occur, and corrective measures must be taken. Since, for structural and safety reasons, drainage inlets are not desired in the mid-deck area, increasing the deck surface roughness (or texture depth TXD) is recommended to mitigate this hydroplaning danger.

- *Mitigation of hydroplaning danger* — Corrective measures for reducing hydroplaning dangers on a bridge deck surface include thin overlay for friction (TOFF), milling, grooving (longitudinal or transverse), and others. (*8*, *9*, *10*) The degree to which these measures should be applied at a particular site depends upon the seriousness of the individual problem. Since the hydraulic roughness characteristics of these measures are not known, it is recommended that these measures not be applied to the gutter section in order to avoid any adverse effect on gutter flow characteristics.

Given the limited nature and data of the studies cited and the inexact initials of field conditions, the above information on hydroplaning should be considered only as a general guideline. More research with field data is needed to develop reliable and accurate design methods.

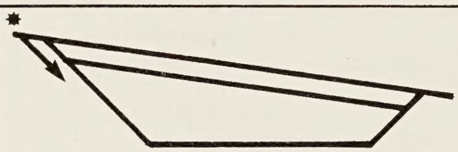
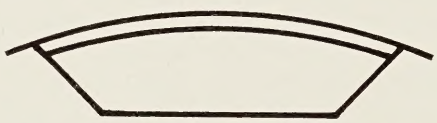
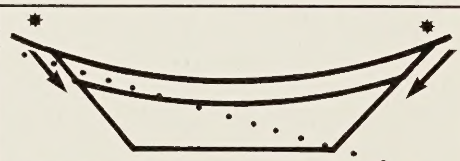
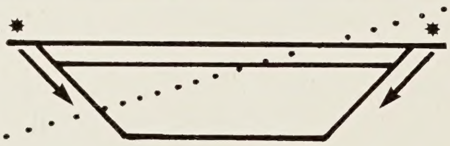
Example: (See figure 1 for definitions.)

The following example illustrates the use of the above-described procedure for determining bridge deck drainage requirements. This procedure comprises derivation of design storm, determination of needs of bridge deck gutter scuppers, and checking of hydroplaning danger on the bridge deck under the proposed bridge design conditions.

Given: The following are proposed bridge design conditions.

S	=	3%
S_x	=	2%
W	=	34 ft (10.4 m)
T	=	10 ft (3.0 m)
L	=	1,000 ft (305 m)
TXD	=	0.038 in (1.0 mm)
n	=	0.016
V	=	55 mi/h and 65 mi/h (88.5 and 104.6 km/h)

Problem: To determine bridge deck drainage requirements.

CASE	GRADE AT BRIDGE	RURAL PAVING (NO CURBS ON APPROACH PAVEMENT)	URBAN PAVING (CURBS ON APPROACH PAVEMENT)
A	 ONE AND HIGHER THAN THE OTHER	USE CATCH BASINS AT LOW END ONLY	USE CATCH BASINS AT BOTH SIDES
B	 BOTH ENDS LOWER THAN CENTER	USE CATCH BASINS AT BOTH ENDS	USE CATCH BASINS AT BOTH ENDS
C	 BOTH ENDS HIGHER THAN CENTER	USE NO CATCH BASINS. USE DRAINS ON BRIDGE DECK	USE CATCH BASINS AT BOTH ENDS AND DRAINS ON BRIDGE DECK
D	 FLAT GRADE	USE CATCH BASINS AT BOTH ENDS ONLY WHEN NO DRAINS ARE USED ON BRIDGE DECK	USE CATCH BASINS AT BOTH ENDS

POOR PRACTICE

* Note: Positive bridge-end drainage is required.
Reference: NCHRP No. 67, Bridge Drainage Systems, p. 27. *(11)*

Figure 5.—Minnesota's criteria for bridge-end drainage.

Solution:

(a) To select the design storm.

From the established policy of highway drainage design, a return period of 25 years is selected for the storm in this example.

1. Let $i = 3$ in/hr (76.2 mm/h)
   From figure 2, for
   $W = 34$ ft, (104 m)
   $n = 0.016$, and
   $S_x = 0.02$,
   it is obtained that
   $t_o = 5.2$ min
   From figure 3, for
   $S = 0.03$ and
   $T_a = (10)(0.65) = 6.5$ min,
   it is obtained that
   $V_a = 3.1$ ft/sec (0.95 m/s)

   Then, for $L = 1{,}000$ ft, t_g and t_c can be computed

   $$t_g = \frac{1000}{(60)(3.1)} = 5.4 \text{ min}$$

   and $t_c = t_o + t_g = 5.2 + 5.4 = 10.6$ min. From the local I–D–F curve (figure 6), for duration of 10.6 min, $i = 4$ in/hr is obtained. Since 4 in/hr is larger than the assumed 3 in/hr, try $i = 3.5$ in/hr.

2. Let $i = 3.5$ in/hr and repeat the above procedure. An $i = 3.4$ in/hr is obtained from t_c and the I–D–F curve. Since 3.4 in/hr is smaller than the assumed 3.5 in/hr, try $i = 3.4$ in/hr.

3. After the third trial, the assumed i and the one obtained from t_c and the I–D–F curve are almost the same. The design storm of 3.4 in/hr with a return period of 25 years is determined for this sample bridge site.

(b) To check for gutter scupper requirements.

1. From the design storm i and W, $i\ W = (3.4)(34) = 115.6$ ($i\ W = 895.0$)

2. Using the given conditions of S, S_x, and T, and the computed value of i W, the bridge length (L) without scuppers is found to be 1,400 ft (figure 2). Since this is longer than the given bridge length of 1,000 ft, this bridge does not require a gutter scupper.

(c) To check for hydroplaning danger.

1. From the given W and T,
   $W - T = 34 - 10 = 24$ ft ($W - T = 10.4 - 3.0 = 7.4$ m)

2. For $V = 55$ mi/h (88.9 km/h)

   From table 1: For $W - T = 24$ ft (7.4 m)
   For $S = 0.02$ and $S_x = 0.02$, $i = 5.3$ in/hr (134.6 mm/h)
   $S = 0.04$ and $S_x = 0.02$, $i = 4.2$ in/hr (106.7 mm/h)
   Then, for $S = 0.03$ and $S_x = 0.02$, i is computed as

   $$i = \frac{5.3 + 4.2}{2} = 4.75 \text{ in/hr}$$

   $$(i = \frac{134.6 + 106.7}{2} = 120.6 \text{ mm/h})$$

   Since 4.75 in/hr is greater than the design storm of 3.4 i/hr, there is no hydroplaning danger on the bridge deck for a maximum vehicle speed of 55 mi/h.

3. For $V = 65$ mi/h (104.6 km/h)

   From table 2: For $W - T = 24$ ft (7.3 m)
   For $S = 0.02$ and $S_x = 0.02$, $i = 2.1$ in/hr (53.34 mm/h)
   $S = 0.04$ and $S_x = 0.02$, $i = 1.7$ in/hr (43.18 mm/h)
   Then, for $S = 0.03$ and $S_x = 0.02$, i is computed as

   $$i = \frac{2.1 + 1.7}{2} = 1.9 \text{ in/hr}$$

   $$(i = \frac{53.34 + 43.18}{2} = 48.3 \text{ mm/h})$$

   Since 1.9 in/hr is less than the design storm of 3.4 in/hr, there is hydroplaning danger on the bridge deck for a maximum vehicle speed of 65 mi/h.

4. Doubling the bridge deck surface texture (TXD) from 0.038 in to 0.076 in will increase the hydroplaning rainfall intensity by 60 percent

   $i = 1.9(1 + 0.6) = 3.04$ in/hr

   But this rainfall intensity is still less than the design storm of 3.4 in/hr. To eliminate the potential hydroplaning danger in this example, the bridge deck surface texture must be further increased to 0.1 in, or approximately 3 times the original 0.038 in.

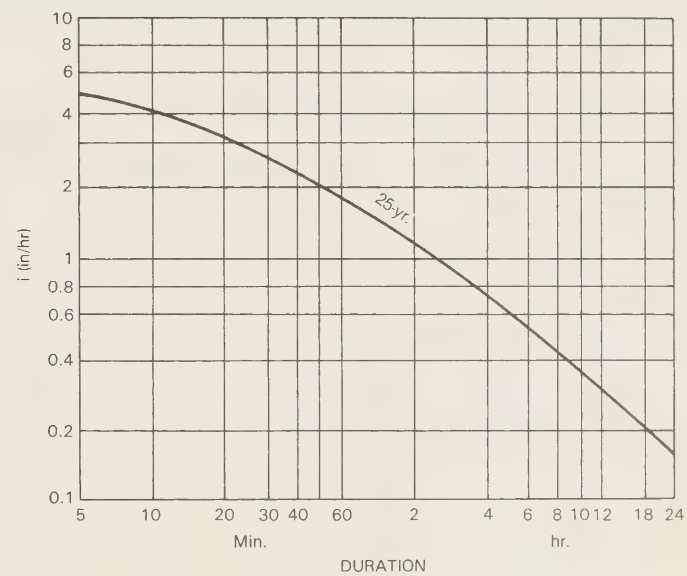


Figure 6.—Local I–D–F curve.

Needs of Bridge-end Drainage Provisions

Regardless of whether a bridge deck drainage system is required, bridge-end drainage must always be provided. Such drainage serves two purposes: (1) it prevents stormwater from upslope roadways from running onto the bridge deck, and (2) it intercepts stormwater from the bridge deck, preventing it from running onto the roadway.

The design of the bridge-end drainage is normally done by bridge and hydraulic engineers. However, guardrail posts are sometimes placed in front of the bridge-end drain thereby interfering with the stormwater flow into the bridge-end inlet. General guidance of this design is provided in reference 2. Figure 5 shows Minnesota's criteria for the bridge-end drainage requirement. (*11*)

Conclusion

The procedure outlined in this article for determining bridge deck drainage requirements tends to be rather conservative; this is because of the extremely limited data available on certain of the elements. Further research will be needed to refine this procedure. Topics for further study include quantifying the effect of rain on vehicle speed, improving hydroplaning estimation equations with field data, and accurately determining Manning's "n" value for various roughened surfaces.

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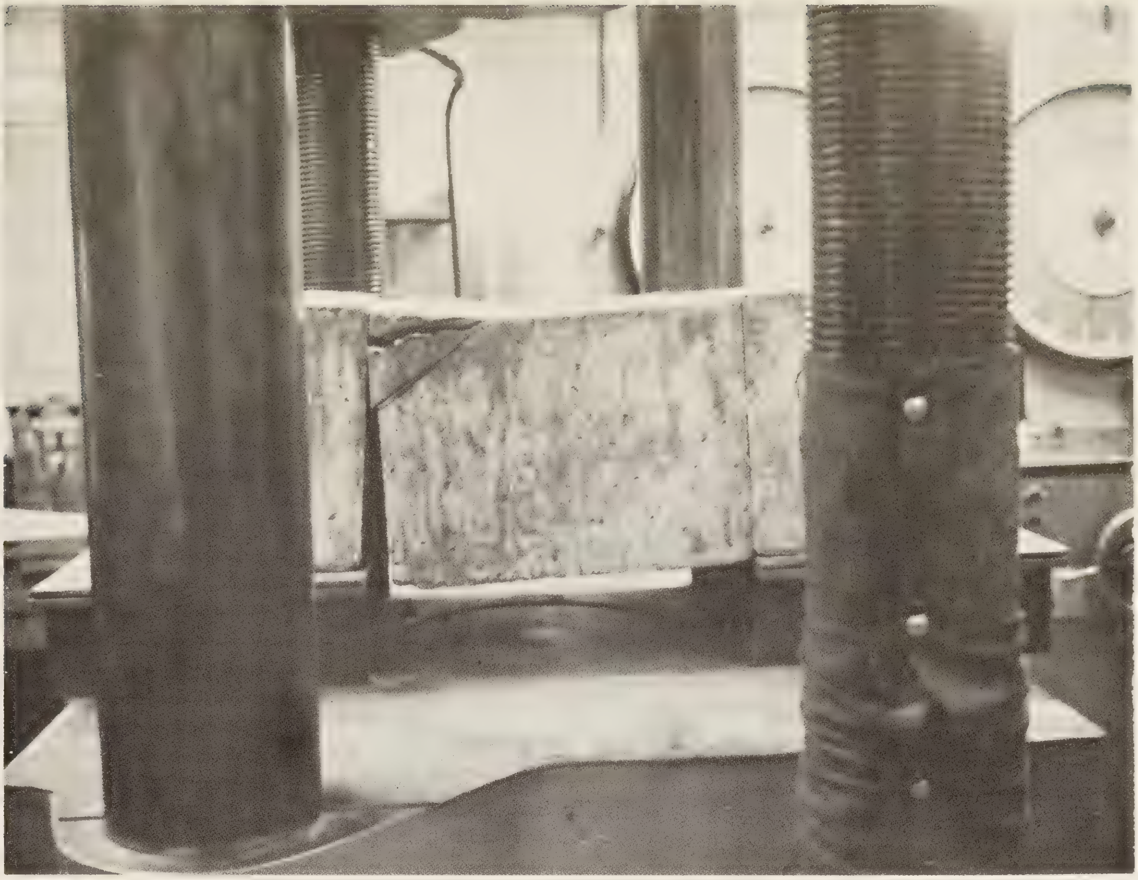

Evaluation of Stainless-Steel Pipes for Use as Dowel Bars

by

Kevin N. Black, Roger M. Larson, Loren R. Staunton

Introduction

Dowel bars are an important load transfer mechanism that reduce the damage to highway pavements caused by pumping and subsequent faulting at the slab-joint interface. Damage to the pavement when these devices corrode has been a significant problem in the past. In an effort to find a method for improving pavement performance and reducing repair costs, a study was performed to evaluate the effectiveness of using stainless-steel pipes as dowel bars. Based on long-term (40 years) field performance studies, stainless-steel (type 316) was selected because of its proven ability to resist corrosion. Pipe was chosen over a solid stainless-steel bar for cost considerations. The data from the initial tests produced useful results on load-carrying capacity of concrete-filled and hollow-dowel pipe.

Load Transfer Theory and the Dowel

The dowel reduces damage to pavements through its ability to limit the loading on one end of a slab by transferring up to 50 percent of the load to the adjacent slab. This is defined mathematically using structure theory as applied by Westergaard to relate load and deflection. (*1*)[1] For a concrete slab supported by the sub-

[1] Italic numbers in parentheses identify references on page 43.

grade, deflection is inversely proportional to the pavement thickness as expressed below:

$$Z \propto \frac{P}{d^{3/2}}$$

where

Z = deflection
P = load (wheel load)
d = pavement thickness

This expression can be converted to an equation and then be used to determine equivalent thickness between doweled and undoweled pavement. For example, if the deflection of the doweled slab is Z_1 and the deflection of the undoweled slab is Z_2, equating the deflection, $Z_1 = Z_2$, would yield the following:

(Assuming that the dowels transfer one-half the load across the joint $P_1 = \frac{1}{2} P_2$):

$$\frac{P_1}{d_1^{3/2}} = \frac{P_2}{d_2^{3/2}}$$

solving for the pavement thickness, d_2 yields:

$$\frac{P_1}{2d_1^{3/2}} = \frac{P_2}{d_2^{3/2}}$$

$$d_2^{3/2} = 2d_1^{3/2}$$

$$d_2 = 2^{2/3} d_1$$

$$d_2 = 1.556\ d_1$$

where

P_1 = P/2
P_2 = full load, P
d_1 = thickness of slab with dowel
d_2 = thickness of slab without dowel

If this expression is evaluated using a representative doweled pavement thickness of 9 in, (229 mm) then the undoweled pavement would have a thickness of more than 14 in (356 mm). A 9-in (229 mm) doweled pavement (with 100 percent joint efficiency) is structurally equivalent (based on joint deflections) to a 14-in (356 mm) thick undoweled pavement assuming no load transfer.

The Test Specimens

In performing this test, several dowel designs were evaluated. This study tested five specimens with the following configurations:

(1) 1.25-in (31.75 mm) diameter solid stainless-steel (Nitronic 33™) dowel bar.

(2) 1.66-in (42.16 mm) outside diameter stainless-steel pipe with 0.065-in (1.65 mm) wall thickness, hollow.

(3) 1.66-in (42.16 mm) outside diameter stainless-steel pipe with 0.065-in (1.65 mm) wall thickness, concrete-filled.

(4) 1.66-in (42.16 mm) outside diameter stainless-steel pipe with 0.109-in (2.77 mm) wall thickness, hollow.

(5) 1.66-in (42.16 mm) outside diameter stainless-steel pipe with 0.109-in (2.77 mm) wall thickness, concrete-filled.

The concrete-filled pipes were filled with a general A3 (Virginia Department of Transportation) paving class mix and cured for a minimum of 30 days before the concrete slabs were cast around them. The unfilled pipes were plugged on the ends with cork stoppers. This prevented the pipes from being filled during the placement of the concrete for the test slabs.

The test slabs were constructed with the same mix used to fill the pipes and were cured for a minimum of 30 days in accordance with ASTM C873–80. Average concrete compressive strength for all five slabs was approximately 6,700 psi (46.2 MPa) as determined using cylinders as specified in compression test method, ASTM C39–83b. No admixtures were used to alter the properties of the concrete. The slump tests yielded values of 4 in (101.6 mm). Table 1 lists the properties of the concrete mixture.

Table 1.—Properties of concrete mixture

Class of concrete	Design minimum laboratory compressive strength at 28 days psi	Aggregate size number	Nominal maximum aggregate size	Minimum grade aggregate	Cement content lb/cu yd minimum	Maximum water lb water per lb cement	Consistency inches slump	Air content percent
A3 General Use	3000	57	1 in	A	588	0.49	1 – 5	6 ± 2
A3 Paving	3000	57	1 in	A	564	0.49	0 – 3	6 ± 2

SI conversions:
1,000 psi = 6.89 MPa
1 lb/cu yd = 0.593 kg/m³
1 in = 25.4 mm

Testing

Several factors affect the ability of the dowel to transmit loading: Diameter, alignment, grouting, looseness, spacing, number, corrosion, joint spacing, slab thickness, and dowel embedment distance. In these preliminary tests, no effort was made to evaluate all these variables, since the primary purpose was to evaluate how pipes would compare to solid bars. Previous testing had shown that larger-diameter dowels reduced the stresses in the pavement slabs. (*2*) This experiment was designed to verify these earlier results while evaluating the potential for larger-diameter, hollow and concrete-filled pipes to be used as dowels.

As a control for the test, a solid stainless-steel dowel (1.25-in (31.75 mm) diameter) was used. Testing consisted of manually loading and unloading (automatic cycling controls were not available) a pavement test specimen to 24 kips (106.7 kN) for 100 cycles or until failure, using a Baldwin Universal Testing Machine as shown in figure 1. If failure did not occur on or before the hundredth cycle, the slab was then loaded until it did fail. The damage was photographically recorded so that a determination of the failure mechanism could be made as illustrated in figure 2. The dowels were then removed to allow examination so a determination of pipe failure or concrete failure could be made. A comparison of the pipes' response to loading can be seen in figure 3.

Results

The results of the testing are summarized in tables 2 through 4 and have been plotted on the graph in figure 4. Three different responses can be seen, although one is less apparent since only one point was plotted before failure. These responses can be summarized as follows:

- As the moment of inertia of the dowel increases, deflection decreases.
- The 1.66-in (42.16 mm) O.D. concrete-filled pipes deflected less than the 1 1/4-in-(31.75 mm) diameter solid bar tested.
- The hollow stainless-steel pipes provided no effective load transfer (failed immediately under initial loading as shown in figure 4 as points at the origin).

The concrete-filled pipes deflected the least. These had a larger moment of inertia and a larger bearing area than the 1 ¼-in- (31.75 mm) diameter solid steel bar. The deflection over the full 100 test cycles did not change significantly although it was larger for the 0.065-in (1.65 mm) wall-thickness specimen. Because neither dowel failed during the load cycling, each dowel was then loaded to failure. The 0.109-in (2.77 mm) wall-thickness specimen failed at 39 kips (173.5 kN) and the 0.065-in

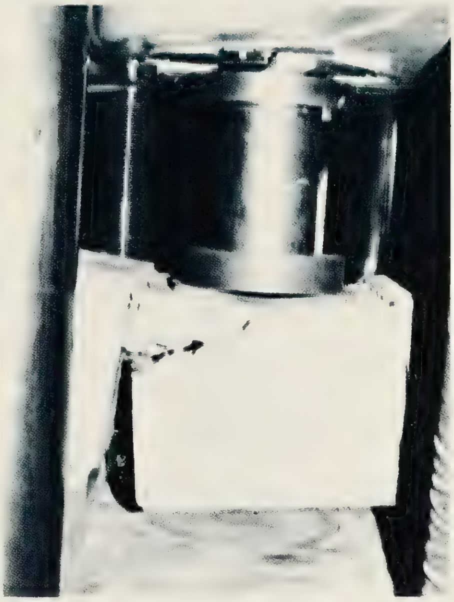

Figure 2.—Failure of model pavement slab.

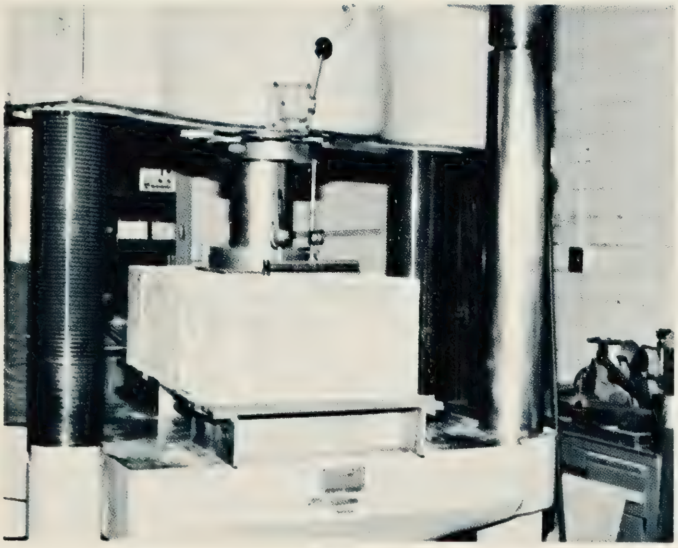

Figure 1.—Specimen loaded for testing on Baldwin Universal Testing Machine.

Figure 3.—Dowels after all testing was completed.

Table 2.—Solid dowel bar deflection data

Cycle Number	Deflection (in) Load	Unload	Difference
0	1.389	1.389	0
10	1.389	1.372	.017
20	1.387	1.365	.022
30	1.387	1.361	.026
40	1.387	1.358	.029
50	1.386	1.356	.030
60	1.385	1.354	.031
70	1.386	1.353	.033
80	1.386	1.351	.035
90	1.385	1.349	.036
100	1.386	1.349	.037

Maximum load at failure: 34 kips

SI conversions:

1 in = 25.4 mm
1 kip = 4.448 kN

Table 3.—Concrete filled 0.109-in (2.77 mm) wall-thickness dowel pipe deflection data

Cycle Number	Deflection (in) Load	Unload	Difference
0	1.235	1.235	0
10	1.230	1.218	.012
20	1.229	1.217	.012
30	1.228	1.216	.012
40	1.228	1.216	.012
50	1.228	1.215	.013
60	1.227	1.215	.012
70	1.227	1.215	.012
80	1.227	1.215	.012
90	1.227	1.214	.013
100	1.227	1.214	.013

Maximum load at failure: 38.8 kips

SI conversions:

1 in = 25.4 mm
1 kip = 4.448 kN

Table 4.—Concrete filled 0.065-in (1.65 mm) wall-thickness dowel pipe deflection data

Cycle Number	Deflection (in) Load	Unload	Difference
0	1.191	1.147	0.044
10	1.105	1.086	0.019
20	1.075	1.056	0.019
30	1.059	1.041	0.018
40	1.041	1.022	0.019
50	1.029	1.010	0.019
60	1.023	1.004	0.019
70	1.016	0.997	0.019
80	1.010	0.990	0.020
90	1.004	0.984	0.020
100	1.000	0.980	0.020

Maximum load at failure: 32.7 kips

SI conversions:

1 in = 25.4 mm
1 kip = 4.448 kN

(1.65 mm) specimen failed at 33 kips (146.8 kN).

The solid bar served as the control for the experiment, allowing comparison of its deflection history with the others. Its performance suggests partial failure at onset of testing with increasing deflection as cycling progressed. Incremental increases in deflection decreased over the test period, being smaller in the latter cycles than at the initial loading. Because total failure did not occur at the conclusion of the 100-cycle test, the control specimen was loaded to failure at 34 kips (151.2 kN).

The hollow pipes failed immediately on the first cycle. These results are plotted in figure 4 and are visible at the origin. The 0.109-in (2.77 mm) walled pipe failed at 22 kips (97.9 kN) and the 0.065-in (1.65 mm) pipe failed at 18 kips (80.1 kN).

Discussion of Results

This experiment was designed to examine the load resistance of hollow stainless-steel pipes, concrete-filled stainless-steel pipes, and a stainless-steel bar to determine how these different structures compare as load transfer mechanisms. The results represent laboratory test values and must be related to actual highway conditions in order to be meaningful. Loading in these tests was 24 kips (106.7 kN) applied to two dowels, resulting in an effective weight carried by each dowel of 12 kips (53.4 kN). By comparison, legal limits on highways are about 20 kips (89.0 kN) for a single axle or approximately 10 kips (44.5 kN) per wheel, which is less severe than the laboratory loading. Past research showed that a wheel loading is primarily carried by the two or three dowels (spaced at 12 in (304.8 mm)) directly under the wheel, or more if there are dual wheels. (*2*) The test specimens and the loading method reported here were devised not to simulate a truck loading on a pavement slab, but to provide a quick relative comparison of performance.

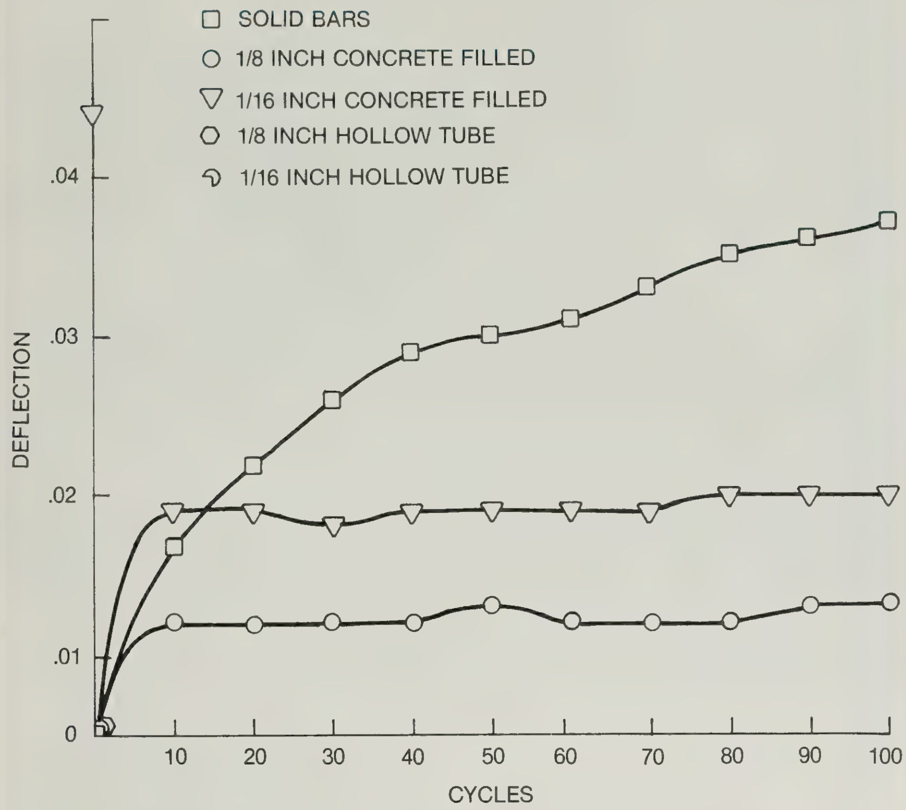

Figure 4.—Comparison of deflection versus loading of dowels.

In relating the laboratory results to field conditions, the hollow pipes would not be expected to perform well, since they failed immediately at a total loading of 22 kips (97.9 kN) (11 kips (48.9 kN)/dowel) and 18 kips (80.1 kN) (9 kips/ (40.0 kN) dowel) for the 0.109 (2.77 mm) and 0.065-in (1.65 mm) wall-thickness pipes, respectively. The failure of these pipes was immediate, without completing one full testing cycle. This finding indicates that a hollow stainless-steel pipe of this size might not be adequate for heavily loaded pavements or possibly even for a single heavily overloaded truck.

In contrast to the relatively poor performance of the hollow pipes is the greatly improved response of concrete-filled pipes. The filled pipes did not exhibit any appreciable deflection. They performed better than the solid bar. This could logically result from:

- The concrete-filled dowels having a greater moment of inertia than the other specimens.
- The bearing area of these pipes being larger than the solid-steel dowel, thus reducing the concrete stress at any one location.

The solid bar exhibited a typical dowel response. (*2*) Continuous load-unload cycling resulted in a cumulative increase in deflection as testing progressed, although incremental deflection decreased. Since there was no permanent deformation of the bar (figure 3, center), it seems likely that the cumulative deflections are a result of the crushing of the concrete surrounding the bar. This cumulative deflection was not evident for the concrete-filled pipes, which have a larger bearing area. This reinforces the fact that bearing area has an important impact on deflection.

Independent evaluations of larger-diameter dowels and pipes by researchers at the University of Illinois found similar results. (*3*)[2] They concluded that larger-diameter dowels have a significant effect on reducing pavement stress. This supports the findings of Cashell noted earlier and the need for further testing to correlate the different variables affecting dowel performance. (*2*)

Economic Considerations

The deflection theory presented earlier can be used to support financial savings that could be realized by using dowels to reduce concrete thickness by up to 35 percent (9 in (228.6 mm) versus 14 in (355.6 mm)). Although the load transfer device reduces concrete costs, it increases the steel costs associated with the dowels—and this first cost depends on joint spacing, dowel size and spacing, type of steel/coating, and method of placement. If dowels are placed in baskets on the grade, the cost of the concrete in place can also be indirectly increased. This cost differential was addressed by the Portland Cement Association, which found the cost of dowels to be equivalent to 1 (25.4 mm) or 2 in (50.8 mm) of concrete.(*4*) Thus, the savings are still significant for using dowels. These results also hold true for the stainless-steel pipes which

[2] Synder et al., "Rehabilitation of Concrete Pavements," FHWA–RD–88–071, Volume I – Repair Rehabilitation Techniques. Not yet published.

have comparable costs with the carbon-steel dowels as can be seen in tables 5 and 6.

Further, cost savings are realized using stainless-steel dowels when the full pavement life cycle is considered. The ability of stainless steel to resist corrosion eliminates the initial cost difference favoring carbon steel. Evidence of this was provided in an earlier study conducted by William Van Breemen of the New Jersey State Highway Department.(*5*) This study revealed that pavement failure at the joints in concrete pavement resulted from rusting of the steel dowels. Van Breemen's experimentation with treatments to reduce this problem included Monel (stainless) steel, galvinization, oil, grease, and lead paints. Seven years after the study began, samples removed from the pavement had all deteriorated substantially except for the Monel samples which appeared in near-perfect condition.

Table 5.—Cost comparisons of different materials used in fabricating dowels

Metal	Diameter (in)	Length (in)	Spacing (in)	Lb/ft width	$/ft width
Hot rolled (control)	1 1/4	18[2]	@ 12	6.26 @ $ .40[3]	$2.50
Hot rolled	1 1/4	12	@ 12	4.17 @ $ .40	$1.67
S.S.[1] pipe w/concrete (0.065-in wall)	1 1/4 Nom.	12	@ 12	1.11 @ $3.20[4]	$3.55
S.S pipe w/concrete (0.065-in wall)	1 1/4 Nom.	12	@ 18	0.74 @ $3.20	$2.37
S.S pipe w/concrete (0.109-in wall)	1 1/4 Nom.	12	@ 12	1.81 @ $3.20	$5.79
S.S. pipe w/concrete (0.109-in wall)	1 1/4 Nom.	12	@ 18	1.20 @ $3.20	$3.84

SI conversions:

1 in = 25.4 mm
1 lb/ft = 1.49 kg/m

[1]S.S. = stainless steel

[2]18-in (457.2 mm) length is commonly used. It is a conservative value to allow for errors in sawing the joint plane over the dowel. Structurally, only a 12-in (304.8) length is needed (Ref 2).

[3]Structural steel in place is estimated at $.40 per pound. Coatings to prevent corrosion could raise this cost.

[4]Stainless steel was estimated for this example at eight times the cost of hot-rolled steel.

Conclusion

The purpose of this experiment was to study the effects of stainless-steel pipe (concrete-filled and hollow) having an outside diameter of 1.66 in (42.16 mm) with a solid-steel bar having an outside diameter of 1.25 in (31.75 mm) and similar to those presently used as dowels in Portland cement concrete pavement. These tests suggest that larger bearing areas have a positive effect on load transfer performance. The experiment also indicates that the moment of inertia of the dowel (concrete-filled or solid) has a notable effect on the deflection. Preliminary tests demonstrate that 0.109-in (2.77 mm) wall thickness, type 316 stainless-steel pipe filled with concrete would be cost-effective and superior to 1.25-in- (31.75 mm) diameter solid-steel bars when used in jointed concrete pavement with short (maximum of 15 ft (4.6 m)) joint spacings. This design would assure satisfactory long-term (about 40 years) dowel performance. The use of a 12-in (304.8 mm) length dowel would require a method to verify accurate placement in order to be acceptable under field conditions.

Table 6.—Comparison of dowel systems for a 10-in (254.0[1] mm) pavement

Length (in)	Diameter (in)	Mat'l	Spacing (in)	On a basis of per ft of pavement width Shear area (in^2)	Brg. area (in^2)	Moment I (in^4)	Weight (lb)
18	1 1/4	Steel	12	1.23	>7.50	0.120	6.26 (Control)
12	1 1/4	Steel	12	1.23	7.50	0.120	4.17
12	1 1/4 (Nom) Pipe .065 wall 1.66 OD	Stain- less	12	0.328	9.96	0.104	1.11
12	1 1/4 (Nom) Pipe .109 wall 1.66 OD	Stain- less	12	0.534	9.96	0.161	1.81
		Two above pipe with concrete filling (144 lb/cf)					
12	1 1/4 (Nom) Pipe .065 wall 1.66 OD	Stain- less + concrete	12	2.166	9.96	0.373	1.11 (St.) 1.76 (Conc.) 2.87 Total
12	1 1/4 (Nom) Pipe .109 wall 1.66 OD	Stain- less + concrete	12	2.166	9.96	0.373	1.81 (St.) 1.76 (Conc.) 3.57 Total
		Above pipe spaced at 18-in C to C (Values are 2/3 of above 12-in spacing values)					
12	1 1/4 (Nom) Pipe .065 wall 1.66 OD	Stain- less + concrete	18	1.44	6.64[1]	0.249	1.91 Total (0.74 stainless)
12	1 1/4 (Nom) Pipe .109 wall 1.66 OD	Stain- less + concrete	18	1.44	6.64[1]	0.249	2.38 Total (1.20 stainless)

[1] 11 percent deficient compared to 1 1/4-in dowels which have 7.50 in^2 of bearing area (Brg. Area).

SI conversions:

1 in = 25.4 mm
1 in^2 = 645.16 mm^2
1 lb/cf = 16.02 kg/m^3
1 lb = 0.454 kg

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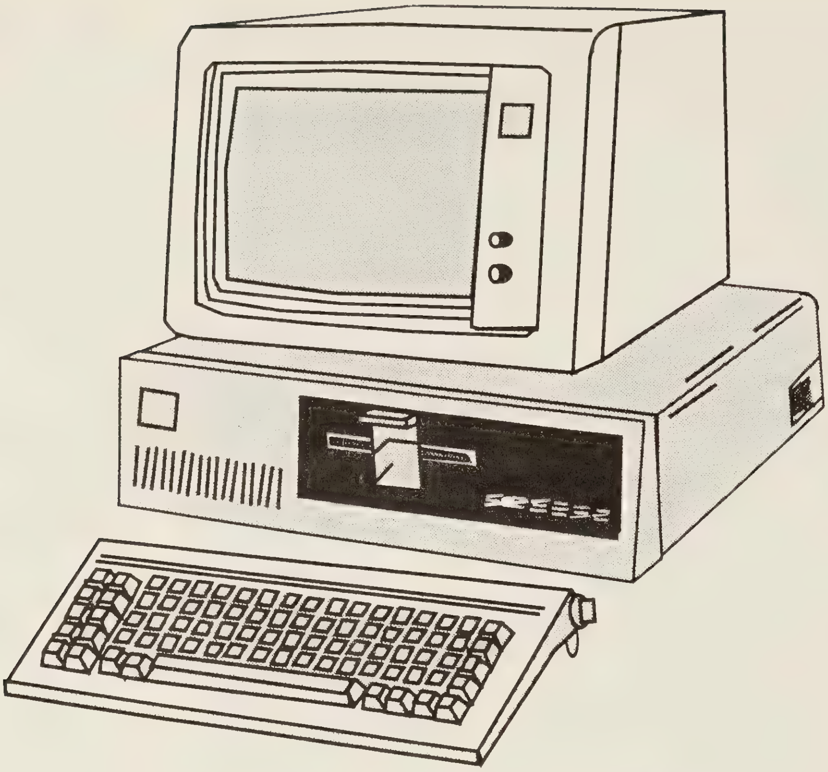


The McTrans Center

by
Antoinette D. Wilbur

Background

In May 1986, the Federal Highway Administration (FHWA) awarded a competitively bid contract to the University of Florida Transportation Research Center in Gainesville to establish the Center for Microcomputers in Transportation (McTrans Center). The objective of the McTrans Center is to "facilitate the exchange of information on uses of the microcomputer and associated software among transportation professionals."

The McTrans Center replaces three other microcomputer support centers that had been operated for several years by the Transportation Systems Center in Cambridge, Massachusetts. These were the Safety and Traffic Engineering Applications for Microcomputers (STEAM), Microcomputers in Transportation Planning (MTP), and Microcomputer Applications in Highway Projects (MAHP) centers. These three centers provided support and distribution of primarily safety, traffic engineering, and urban and statewide planning microcomputer software. A major goal of the new McTrans Center has been to expand into *all* areas of highway engineering and planning to address the needs of a broader segment of the transportation user community.

Among the many services provided by the McTrans Center are the publication of a quarterly newsletter, operation of a telephone hotline, and distribution of—and user support for—microcomputer software and documentation.

Newsletter and Hotline

When McTrans opened for business on July 14, 1986, it inherited a mailing list of approximately 6,000 names from the three earlier support centers. Since then, the number of McTrans "members" has almost doubled; this primarily reflects the expanded audience served by the new Center. The McTrans newsletter provides the chief means of communicating with the membership and keeping it up-to-date on new Center products and services.

The McTrans newsletter contains articles of general interest to microcomputer users as well as technical articles and helpful hints about specific highway engineering programs. Included with the newsletter is the McTrans Product Catalog which contains brief descriptions of and ordering information for software offered by McTrans. Quarterly catalog supplements announce new products; a complete updated catalog is distributed to all members every year.

The McTrans telephone hotline provides quick and easy access to the Center for information about it and the software it distributes. Most questions are answered immediately; if necessary, the McTrans staff will research the question and return the call. The telephone hotline number is (904) 392–0378.

Software Distribution

When McTrans first opened, it distributed only 20 microcomputer programs, mostly in the traffic engineering and urban planning applications areas. The number of programs and the breadth of applications has since increased dramatically. McTrans currently distributes over 150 different products, including programs for culvert analysis, construction and project management, pavement design and management, and many other aspects of highway planning, design, and construction.

Examples of the wide range of programs available from McTrans include:

- The Highway Capacity Software (HCS)—this is a faithful implementation of the procedures in the 1985 *Highway Capacity Manual.*
- The Culvert Analysis Software (HY-8)—this automates the methods described in the FHWA report "Hydraulic Design of Highway Culverts."
- The Pavement Management System (PMS)—this program is used in determining the condition of flexible pavements and in helping the user formulate decisions on the type of reconstruction or rehabilitation required.

A new product available from McTrans is GTRAF, which provides both animated and static graphics to help users interpret the results of the NETSIM microscopic traffic simulation model. GTRAF and most of the other programs available from McTrans run on the IBM PC and compatible microcomputers.

In its first year, McTrans distributed only public domain software which developers contributed to the Center so that it could be shared with others. Public domain programs are contributed to McTrans both by the Federal Highway Administration, and by individuals and State and local transportation agencies across the country. McTrans prices these programs to cover the cost of reproduction and to help offset Center operating costs.

For member convenience in accessing a wider selection of highway engineering programs, the McTrans Center also lists proprietary software in its product catalog. Depending on the developer's preference, McTrans either distributes the program directly or refers the potential customer to the appropriate source. In either case, the McTrans Center collects a commission on the sale.

Another significant area of McTrans expansion is the listing of foreign-developed highway engineering software. Through an arrangement with MVA Systematica in Great Britain, McTrans lists MVA's software in its product catalog. MVA Systematica is the developer of many well-known transportation planning packages; it also distributes traffic engineering software developed by the Transport and Road Research Laboratory in the United Kingdom. Customers in the United States can order these products through McTrans in U.S. dollars, and the Center forwards the order to MVA for processing.

Foreign customers wishing to purchase software distributed by McTrans also benefit from this arrangement. Foreign orders for McTrans software are sent directly to MVA Systematica using the McTrans Overseas Order Form. Not only does this speed up the processing of foreign orders, but, since MVA accepts payment in the customer's own currency, also eliminates the inconvenience of purchasing directly from McTrans in U.S. dollars.

McTrans Levels of Support

Besides serving as a clearinghouse for distributing microcomputer software, McTrans also provides seven separate levels of support (LOS) to users for the programs it distributes. When a package is contributed to the McTrans Center, its LOS is determined jointly by McTrans and the donor. Factors such as the anticipated number of users, the complexity of the software, the amount of testing it has undergone, and the contributor's willingness to share in the support are all considered in determining the appropriate LOS. A surcharge is added to the software's price to fund any support to be provided by McTrans.

LOS 1: Full Technical and Maintenance Support. At this level, the McTrans Center provides technical assistance to software users and maintains the program code. The software is distributed with a registration card which users return to the McTrans Center. Registered users receive program "notes," as needed, advising them of potential problems and the corrective actions to be taken. When "bugs" are discovered in the software, McTrans corrects the problem and provides free updates of the disks and documentation. If enhancements are made to the software to improve its operation, the McTrans Center will distribute a new release to registered users.

The registration period lasts for a year; users must then reregister the software to continue to receive LOS 1 support. The first LOS 1 program at McTrans was the HCS; other programs will be designated as LOS 1 in the near future.

LOS 2: Technical and Upgrade Support. As with LOS 1 software, purchasers of LOS 2 software are registered for a year, and the McTrans Center provides them with technical assistance in program use. The Center also maintains records of bugs and suggested enhancements and forwards this information to the software developer. The developer, in turn, is responsible for providing the needed software maintenance and for furnishing updates and corrections to McTrans. The McTrans Center then distributes free updates to registered software users.

LOS 3: Limited Technical Support. Purchasers of LOS 3 software are *not* registered and receive only limited technical support from McTrans. Any bugs in the software are reported to the developer; however, the developer is not obligated to provide corrections to the code. Although these programs do not receive the support and maintenance provided at higher levels, LOS 3 software is generally well tested and reliable.

LOS 4: User Supported Freeware/Shareware. LOS 4 software is copyrighted by the developer and distributed by McTrans as a user service. Purchasers of shareware are often invited to send the developer a voluntary contribution if they find the software useful. McTrans encourages the shareware concept, but provides no support for this software; any support offered or provided is entirely between the developer and the user.

LOS 5: Totally Unsupported. LOS 5 users are "on their own." The software is useful and, based on past experience, fully operational. However, neither McTrans nor the developer provide any technical support or maintenance for LOS 5 programs.

LOS 6 and LOS 7: The last two levels of support apply to proprietary software. Software designated as LOS 6 is proprietary software that McTrans supports through "first line" technical assistance when users need help. The developer corrects any bugs that are discovered; McTrans distributes free updates for some specified period of time after purchase. At LOS 7, all technical support and maintenance are provided by the developer.

McTrans Staffing

The McTrans Center is operated by an extensive staff whose goal is to provide the best possible service to the highway transportation user community. As director of the Center, Dr. Charles E. Wallace is responsible for its overall management and operation. A full-time manager, Mr. William M. Sampson, handles McTrans' day-to-day operation. Other McTrans staff includes a data control specialist who is responsible for managing the membership data base and for processing and tracking software orders. A storekeeper fills the orders and packages the disks and documentation for mailing; a full-time accountant handles all expenditures and revenues. An information specialist is responsible for all publications issued by the Center, including the McTrans Newsletter and other brochures, notices, etc. A part-time computer programmer, an administrative secretary, and a clerk-typist complete the McTrans Center staff.

In addition to the regular staff, several University of Florida graduate students provide assistance with the software maintenance activities and research answers to users' technical questions. Faculty members in the University's Department of Civil Engineering (and other departments as needed) are another valuable resource available to McTrans for consultation and advice.

Funding Support

The contract for the McTrans Center was awarded with the understanding that FHWA would provide funding support for the Center's establishment, but that McTrans would gradually become completely user-supported. The timetable for this transition to self-sufficiency was 2 years.

FHWA provided initial funding for the McTrans Center's first year of operation. During this period, microcomputer software and documentation were distributed at a nominal charge to cover the cost of disk reproduction, printing, shipping, and handling. McTrans' pricing algorithm resulted in a charge of $2 per disk. Documentation was priced at about 6 cents per page, with a minimum charge of $5. The newsletter and access to the telephone hotline were free.

Self-Sufficiency — Year 3 and Beyond

At the end of May 1988, FHWA's funding support ended,[1] and the McTrans Center became completely user-supported. To raise additional revenues, prices for software and documentation were increased. The minimum price for unsupported software increased to $10 per disk with a surcharge added for each higher level of support. The price for certain documentation also increased slightly.

Several alternative revenue sources were investigated to support the McTrans Center when Federal funding ended. A membership or subscription fee was considered; this suggestion was not implemented, however, since it was felt that even a nominal fee would cause a large erosion in membership, particularly among public employees. Also, providing telephone assistance to paid members would be difficult to administer. More importantly, the opportunity to provide "low tech" assistance to novice users would be largely eliminated if a membership fee were required.

The continued financial success of the McTrans Center is expected to come from an expanded product base and by increasing the number of members on the mailing list. New products are being introduced, including videotapes on software use, training seminars, and so on. To facilitate customer purchases, McTrans now accepts charge cards and blanket purchase orders. This allows users to place orders as needed by telephone, rather than mailing in a check or issuing a one-time purchase order for payment. The McTrans Center will also be providing LOS 1 maintenance and support for more programs in the future.

If you have a useful software package—either public domain or proprietary—that you would like to distribute through McTrans, contact the Center staff to discuss distribution arrangements. For more information about the McTrans Center or to add your name to the Center's mailing list, contact:

The McTrans Center
University of Florida
512 Weil Hall
Gainesville, FL 32611
Hotline phone number: (904) 392–0378

Antoinette D. Wilbur is a transportation specialist in the Systems and Software Support Branch in FHWA's Office of Traffic Operations. She is an expert in offline signal timing optimization models and has served as the FHWA contracting officer's technical representative for the McTrans Center since it opened in 1986. Before joining the Office of Traffic Operations, Ms. Wilbur worked in the FHWA Office of Safety and Traffic Operations R&D.

[1]Although no longer under contract to FHWA, McTrans continues to serve as a distribution center for FHWA-developed software.

Lead-Pigmented Paints—Their Impact on Bridge Maintenance Strategies and Costs

by
John W. Peart

Introduction

The removal, containment, recovery, and disposal of lead-pigmented paints is fast becoming one of he most critical cost items in steel bridge maintenance. The article will examine the problems associated with the removal of lead-based paint and assess impacts on maintenance strategies and cost.

Background

For the decades up to and including the 1970's, paints containing lead were the predominant coating used to protect steel highway bridges against corrosion. Earlier formulations contained such lead compounds as lead sulfate or lead acetate. More recently, the oil/alkyd systems with heavy pigment loadings of "red lead" or lead silico-chromate have been used, e.g., AASHTO M72 and M229. (*1*)[1]

As late as 1983, a Highway Planning and Research study surveying State highway agencies found that more than one-half of the States still specified oil/alkyd systems containing lead or chromate pigments. Sixteen States used these systems exclusively. (*2*)

[1]Italic numbers in parentheses identify references on page 51.

Lead pigments in paints have been a recognized hazard for many years. In the 1960's, the Consumer Product Safety Commission restricted the use of lead-based paints for consumer products to 0.06 percent lead by weight. Although restrictions on the content of paints for industrial use have not been proposed, it's utilization in these formulations has decreased drastically because of health concerns and the availability of other material. Unfortunately, this has not been the case for highway structures and pavement markings.

This *laissez-faire*-induced peacefulness was broken in 1978 with the maintenance blasting and coating of an approach to the Tobin-Mystic Bridge in Boston. The resultant assessment of potential environmental damage initiated extensive sampling and testing to define total particulate and lead concentrations in the air and on the surrounding ground. The result was the requirement that all abrasive blasting and painting be accomplished within a containment enclosure and that the particulate matter be removed from the air by wet scrubbers prior to exhausting. These restrictions resulted in an exponential increase in cost over what was originally estimated.

The problem of containment of the lead-pigmented paint is exacerbated by the fact that open-abrasive blasting is the most cost-effective and common way both to remove the old paint as well as to prepare the steel surface for the new coating. The majority of bridges built before 1970 were erected with the mill scale intact and then painted. Many new replacement coating systems require that the mill scale be removed before painting. Hand- and power-tool cleaning is less effective than abrasive blasting in providing the required cleanliness level for these materials.

Many government jurisdictions are implementing tighter controls on abrasive blasting either through stricter interpretation of present codes or the formulation of new regulations. Examples include Wisconsin; Minnesota; Seattle, Washington; St. Louis, Missouri; Denver (city and county); and California's south coast air quality management district. (*3*) Dry abrasive blasting has been outlawed by some municipalities.

Allegheny County, Pennsylvania, with 1,470 bridges in its jurisdiction, has recently enacted stringent regulations and controls on abrasive blasting. (*3*) These regulations have resulted from citizen lawsuits and the fear of liability litigations because of silicosis.

Figure 1.—Containment structures utilized on the Tobin-Mystic Bridge.

Figure 2.—Dust being generated by abrasive blasting.

Silicosis is caused by the ingestion and retention of free silica contained in respirable sand particles. Unfortunately, most public works contractors use sands containing silica because of their availability, price, and suitability for surface cleaning.

Personal injury liability litigations related to silicosis are proliferating in may States. (*4,5*) Liberalized interpretations of liability laws have expanded the litigation to the injured person's employer, the company and executives who hire the contractor, abrasive suppliers, and safety and blasting equipment manufacturers and distributors.

The recent public attention given to sandblasting and silicosis can only focus additional attention on the removal of lead-based paint from bridges. In the late 1970's, California, Missouri, and several other States proposed a National Cooperative Highway Research Program (NCHRP) project to investigate the problems associated with the removal of lead-based paint. The Transportation Research Board coordinated the project and the results are documented in their December 1983 report No. 265, "Removal of Lead-Based Paints." (*6*)

The objectives of the study were twofold:

- Assessment of regulations pertinent to the removal of lead-based paints and the degree of risk posed by the painted structure.

- Evaluation of available technology for the removal, containment, and recovery of the residue.

The major conclusions of the 1983 report were:

- For the majority of bridges, lead-paint debris does not pose a significant threat, but for certain urban bridges, it is necessary to carefully monitor and control the wastes.

- There is no proven cost-effective technology for the removal and containment of lead-based paint. Further equipment development is recommended.

Although the report does not totally reflect today's situation, it remains a definitive publication on regulations, containment methods, and the economics associated with the removal of lead-based paint.

Partial or complete containment of lead-based paint residues and their controlled disposal is a reality in many States today: California, Wisconsin, Iowa, Oregon, Virginia, North Carolina, New Jersey, and Pennsylvania.

State implementation of requirements for the containment of lead-based paint blast debris is the result of hazardous waste regulations enforcement. The Resource Conservation and Recovery Act of 1976 states that any industrial waste containing more than five parts per million of leachable lead must be treated as hazardous waste. In addition to this universal requirement, some States have a total maximum permissible total lead concentration requirement for nonhazardous industrial waste.

Analysis of abrasive debris from the removal of lead-based bridge paint varies significantly because of variations in procedures of the specified analytical method. Even with this nonprecision in results, much of the debris contains sufficient concentrations of lead to be classified as hazardous waste.

Figure 3.—Containment structures utilized on Severins Bridge, Cologne, West Germany.

Much abrasive residue is generated (8 to 10 lbs/ft^2 surface) during the removal of accumulated paint, corrosion, and mill scale from bridges. If the residue is classified as hazardous waste, disposal costs become prohibitive. While transportation and disposal costs vary from State to State, $300 to $600 per ton is a realistic estimate.

North Carolina and Virginia, aware of these disposal costs, compared the costs of alternative removal methods to those of abrasive blasting. (*7*) They found the most cost-effective procedure was to remove lead-based paint by using power tools, followed by abrasive blasting to attain the required substrate cleanliness and profile. Although it is less productive to use power tools than abrasive blasting, the dual system is economically more feasible because of the limited amount of hazardous waste generated by the power-tool cleaning; this process enables the leachable lead level from the second cleaning step to be sufficiently low that it is not classified as hazardous waste. Adding to the positive economics of this approach is that smaller amounts of debris generated by the use of power tools can be contained by an unsupported plastic film or tarpaulin. Debris generated by blasting requires a steel-reinforced containment structure.

The cost of removal, containment, and disposal of lead-based paint through abrasive blasting usually depends upon bridge location, accessibility, configuration, length of spans, local regulations, and monitoring requirements. A "ball-park" estimate is provided below for a structure of moderate complexity of configuration and of factors affecting cost. No cost factor is included for sampling and analysis of the environment during the cleaning operation.

	Cost/ft^2
Abrasive Blasting SSPC–SP–10	$3.50 - 4.50
Containment/ recovery	1.50 - 3.50
Transportation and disposal	2.00 - 3.00
Total	$7.00 - 11.00

The estimate was derived using rationalized cost elements from references 6 and 7 and available cost data.

Realizing the potential impact of these costs on State implementation of their bridge maintenance programs, the Federal Highway Administration (FHWA) designated the removal of lead-containing bridge paint as a High Priority National Program Area. The immediate result of this was a symposium/ workshop organized on the removal of lead-based paint, held February 29–March 1, 1988, and March 2–3, 1988, respectively.

Symposium

The Lead Paint Removal Symposium, sponsored by the Steel Structures Painting Council was attended by 349 people gathered together to voice concern over the mounting crisis of lead paint removal. Symposium information covered three basic objectives:

- To identify regulatory issues.
- To define associated management, engineering, and production problems.
- To review the strategies and methods being utilized and to assess their effectiveness.

Selected excerpts of the information presented are discussed below.

Bridge maintenance requirements and the impact of regulated waste disposal

Past budgetary constraints have deferred the maintenance of failing bridge corrosion protection systems. This reduction in the priority of painting has resulted in critical steel loss—particularly in areas of high corrosion rates (e.g., expansion dams on truss bridges).

The Pennsylvania Department of Transportation (PennDOT) bridge engineer defined and quantified the problems being experienced by Pennsylvania. PennDOT maintains 15,000 bridges, 5,500 of which are steel. A 1980 survey indicated that 3,000 of these bridges were fast approaching a critical condition because of the lack of painting. The economic impact of this problem is exacerbated by the inflated cost of rehabilitation and painting. In 1984, it cost PennDOT $35 million to rehabilitate the Liberty Bridge. This bridge originally was built in 1929 for $2 million.

Maintenance painting costs are further increased by the costs of containment, storage, and disposal of paint residues. For example, Pennsylvania currently has 5,500 tons of nonhazardous waste stored at an island maintenance site, and 10,000 additional tons is expected to be generated this summer by maintenance painting operations. Historically, this material would be disposed of in landfills. However, for more than a year, efforts to obtain a disposal permit have been unsuccessful.

Contract definition and contractor qualifications

Qualified painting contractors are increasingly reluctant to bid on public work jobs. From a contractor's viewpoint, the primary reasons for this reluctance are poorly written, nondefinitive specifications; and the resulting poor interpretation and evaluation of proposals and contractor capabilities based on these documents.

Traditionally, specification language makes the contractor responsible for identifying, interpreting, and implementing all Federal, State, and local laws. This is a difficult task because of the constant state of change from both a regulatory and interpretative viewpoint. Information on clarification of the regulations is difficult to obtain because of the bureaucratic process. Contractors state that this results in inherent danger of litigation and a potential increase of unanticipated costs. Additionally, it is difficult for the owner to determine if all regulatory criteria are being met by contractors' proposals.

The following example was cited. Last year, in proposals solicited for the repainting of a bridge spanning a river and a park, the following cost variations were noted:

- Estimated blasting and coating costs ranged from $1.87/sq ft to $3.88/sq ft.
- Estimated nonhazardous waste disposal costs ranged from $50 to $250/ton.
- Estimated hazardous waste disposal costs ranged from $70 to $650/ton.

Three of the bidders could not obtain performance bonds; the remaining two proposers could not provide an acceptable containment plan. The project, therefore, was cancelled.

Hazardous waste identification criteria

Data were presented documenting the difficulties associated with the precision and reproducibility of the results obtained with the Environmental Protection Agency's (EPA's) Toxicity Characterization Test Method 1310. Large variations in results were obtained—both between analyses and between laboratories—resulting in reclassification of paint waste from nonhazardous to hazardous.

This method is being replaced by Test Method 1311, which adds 38 organic compounds to the list against which the waste is to be evaluated. People who have compared the two methods note that the change of the acid used to adjust the pH of the leachete has increased the lead sensitivity. This results in the reclassification of many lead-containing bridge cleaning residua from nonhazardous to hazardous. The new method is the current regulation, it has not been finalized and is subject to change; its use in determining waste classification is, thus, a risky proposition.

Lead Paint Removal Workshop

The FHWA-sponsored Lead Paint Removal Workshop immediately followed the symposium. The large turnout for the workshop demonstrated the criticality of the problem and the broad awareness of it; attendees represented 31 different States, the District of Columbia, Puerto Rico, and two foreign countries.

Breakout groups were convened to define the problem elements, and identify research solutions for each of the following areas: (1) equipment and containment, (2) regulations and compliance, and (3) maintenance and management strategies.

Task group reports and consensus recommendations will be reviewed and analyzed to develop a design for FHWA contract study to develop cost-effective removal, containment, and recovery methods. This research program will be initiated in fiscal year 1989.

Conclusion

The problems associated with the removal and replacement of bridge paints containing lead are both current and critical. A dialogue and a cooperative approach to the problems should be established among the concerned parties. This will require close coordination between a State's transportation department, its environmental agency, and the proposed contractors. Each job is unique and will require an environmental impact assessment and determination of the degree of containment and motoring required. The methods used to control and dispose of the waste generated must also be well defined before contracting.

If this cooperative approach is not initiated and improved methods are not developed through research, the removal and replacement of lead-containing paint on existing bridges will be a costly venture.

REFERENCES

(*1*) American Association of State Highway and Transportation Officials, Part I, Specifications: M72–4, "Red Lead Ready-Mixed Paint" and M229–80, "Basic Lead Silico Chromate Ready-Mixed Primer."

(*2*) D.S. Leyland, "A Survey of Coating and Systems Specified by the State Highway Departments," Technical Paper 83–10, *Maine Department of Transportation*, Bangor, ME, July 1983.

(*3*) Wendy B. Smith, Esquire, and Anthony J. Sadar, Allegheny County Health Department, Pittsburgh, Pennsylvania. "The Development of a Sandblasting Regulation in Allegheny County, Pennsylvania." Proceedings of the Steel Structures Painting Council, 5th Technical Symposium, New Orleans, LA, January 21–22, 1987.

(*4*) Craig R. Nelson, Hulsen, Nelson, and Wanek, "Living with Litigation on Abrasive Cleaning." Proceedings of Steel Structures Painting Council, 5th Technical Symposium, New Orleans, LA, January 21–22, 1987.

(*5*) Richard Robidoiux, "Case History on Exposure to Unconfined Silica-Sandblasting." Proceedings of the Steel Structures Painting Council, 5th Technical Symposium, New Orleans, LA, January 21–22, 1987.

(*6*) Transportation Research Board Report 265, "Removal of Lead-Based Paints," December 1983.

(*7*) William M. Medford, North Carolina DOT, Thomas V. Neal, Jr., Virginia DOT, Lloyd M.Smith, S.G. Pinney and Associates, Inc. Proceedings of the Steel Structures Painting Council, 5th Technical Symposium, New Orleans, LA, January 21–22, 1987.

John W. Peart is a research chemist in the Materials Division, Office of Engineering and Highway Operations Research and Development, Federal Highway Administration. He is involved with research dealing with cost-effective environmentally acceptable coatings and corrosion control methods for steel highway structures. Before joining FHWA, Mr. Peart spent 12 years as R&D manager of the National Shipbuilding Research Program in the area of surface preparation and coatings.

Recent Research Reports You Should Know About

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

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

National Technical Information Service
5285 Port Royal Road
Springfield, Virginia 22161

Requests for items available from the RD&T Report center should be addressed to

Federal Highway Administration
RD&T Report Center, HRD–11
6300 Georgetown Pike
McLean, Virginia 22101–2296
Telephone: (703) 285–2144

Systemwide Pavement Deterioration Analysis, Report No. FHWA/RD–87/070

by Pavements Division

The EAROMAR 2 (Economic Analysis of Roadway Occupancy for Maintenance and Rehabilitation) computer program was updated to make network level cost estimates. The new EAROMAR SW version has a data base consisting of information from the Highway Performance Monitoring System (HPMS), FHWA truck weight data distributions, and nationwide environmental factors. The new pavement damage models, which have been incorporated into EAROMAR SW, come from the latest work by the World Bank to update its Highway Design and Maintenance Standards Model (HDM) and the FHWA cost allocation study. Sensitivity results and regional estimates for the Interstate system demonstrate the systems' capabilities.

This report may only be purchased from the NTIS: (PB No. 87–182143/AS, Price code: A08).

Roadside Safety Library Information Retrieval System (LIRS), Vol. I, Users Manual, Report No. FHWA/RD–87/082, and Vol. II, Programmers and Operators Manual, Report No. FHWA/RD–87/083

by Safety Design Division

The Roadside Safety Library (RSL) maintained by the Federal Highway Administration is a collection of computer programs which simulate vehicle handling, vehicular crash with roadside objects including safety appurtenances, and post-collision behavior of occupants, along with a collection of a large number of crash test films which serve to supplement and to validate the Simulation Programs. The crash test films, with a test query system to retrieve pertinent information from these films, comprise the Testing Data Base.

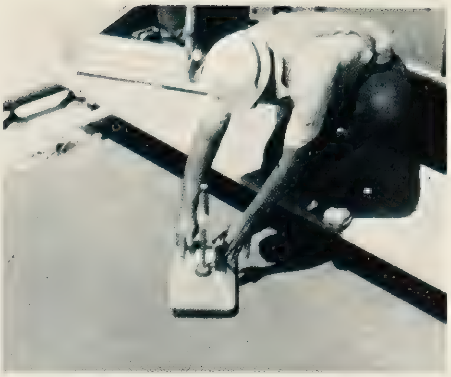
LIRS (Library Information Retrieval System) is an interactive program which guides a user to access available information on any of the computer programs existing in the RSL.

Volume I of this two-volume report is the users manual and gives a comprehensive outline of the system and how it works. Volume II, a programmers and operators manual, presents a concise description of the LIRS.

These reports may only be purchased from the NTIS: (Vol. I, PB No. 87–191110/AS, Price code: A03; Vol. II, PB No. 87–191128/AS, Price code: A04).







Safety Cost-Effectiveness of Incremental Changes in Cross Section Design—Informational Guide, Report No. FHWA/RD–87/094

by Safety Design Division

This guide presents information for estimating the costs and safety benefits which might be expected due to various improvements on specific sections of rural, two-lane roads. Such improvements covered in this guide include lane widening, shoulder widening, shoulder surfacing, sideslope flattening, and roadside improvements. This guide will be useful to those involved with the design of 3R-type projects, particularly for improvement projects which will be constructed on existing vertical and horizontal alignment and within the existing right-of-way.

The accident relationships with roadway geometrics and cost data contained in this guide resulted from research conducted for the Federal Highway Administration (FHWA). Research report FHWA/RD–87/008 entitled "Safety Effects of Cross Section Design for Two-Lane Roads, Volume I, Final Report" contains the major results and conclusions of the study. Research report number FHWA/RD–87/009 subtitled "Volume II, Appendixes" contains details on the data base and the data analysis.

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

Relationship of Consolidation to Performance of Concrete Pavements," Report No. FHWA/RD–87/095

by Materials Division

This report summarizes research on the relationship of degree of consolidation to critical performance properties of portland cement concrete (PCC) pavements and on the suitability of nuclear density guages for monitoring consolidation of such pavements. It will be of interest to materials and construction engineers concerned with PCC construction.

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

Optimization of Left-Turn Phase Sequence in Signalized Networks—MAXBAND 86, Vol. I, Summary Report, Report No. FHWA/RD–87/109; Vol. II, Users Manual, Report No. FHWA/RD–87/110; and Vol. III, Program Manual, Report No. FHWA/RD–87/111

by Traffic Systems Division

These reports describe the MAXBAND 86 program which was developed to provide the capability of optimizing the sequence of left turns and through movements at multiphase signalized intersections in coordinated signal networks. Previous and current research has shown that significant reductions in vehicular delay can be obtained by using this capability.

These reports may only be purchased from the NTIS: (PB Nos. 87–229357/AS, Price code: A06; 87–229365/AS, Price code: A14; 87–229373/AS, Price code: A14)

The computer program may be obtained by writing Dr. Stephen Cohen, Traffic Systems Division, (HSR–10), Federal Highway Administration, 6300 Georgetown Pike, McLean, Virginia 22101-2296.

Handbook of Guidelines for the Lightning Protection of Electronic Traffic Control Equipment, Report No. FHWA/RD–86/073

by Traffic Systems Division

This handbook is a synthesis of practice and application guidelines for the protection of electronic traffic control equipment from damage and/or disrupted operation due to lightning induced effects. It is intended to provide traffic engineers and other concerned persons with a basic understanding of this problem and possible approaches to its solution.

This report may only be purchased from the NTIS: (PB No. 86–169620/AS, Price code: A06).

Improved Methods to Eliminate Reflection Cracking, Report No. FHWA/RD–86/075

by Pavements Division

Existing elastic and viscoelastic fracture mechanics theories have been reviewed and modified to develop a theory capable of predicting the formation of reflective cracking in flexible overlays over rigid pavements due to thermal forces. The developed cracking model is based on fundamental material properties (creep compliance, indirect tensile strength, fracture toughness) and does not use an empirical distress function. Models also have been developed to predict the formation of reflective cracking due to load-associated fatigue. To this extent a finite element analysis model was developed to compute the critical tensile strain in the asphalt overlay over the existing joint or crack. This strain has been related to the allowable strain through laboratory fatigue tests. The developed models have been validated with limited field data. A complete verification was not possible because of a lack of field projects having the required input data.

This report may only be purchased from the NTIS: (PB No. 86–180080/AS, Price code: A09).

Improved Methods for Patching on High-Volume Roads, Report No. FHWA/RD–86/076

by Pavements Division

This report presents data that identify and categorize by pavement type, the kinds of distress found on high-volume roads requiring patching. Methods for making rapid repairs to the distressed areas are determined and described. Pavement types considered for which repair methods are provided include bituminous concrete, portland cement concrete (both slab design and continuously reinforced), and composite.

A search of both published and unpublished literature was conducted to determine the processes which are most suitable for the repair of high-volume roads. Procedures, materials, and equipment are evaluated and discussed together with traffic control, maintenance management, maintenance operations, training, and economics. Visits made to locations in nine States include over 30 sites where pavement repairs of various types were being made. Agencies included State Highway or State Transportation Departments, cities, and toll authorities. Detailed field reports of the findings are included.

This report may only be purchased from the NTIS: (PB No. 86–180072/AS, Price code: A09).

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

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

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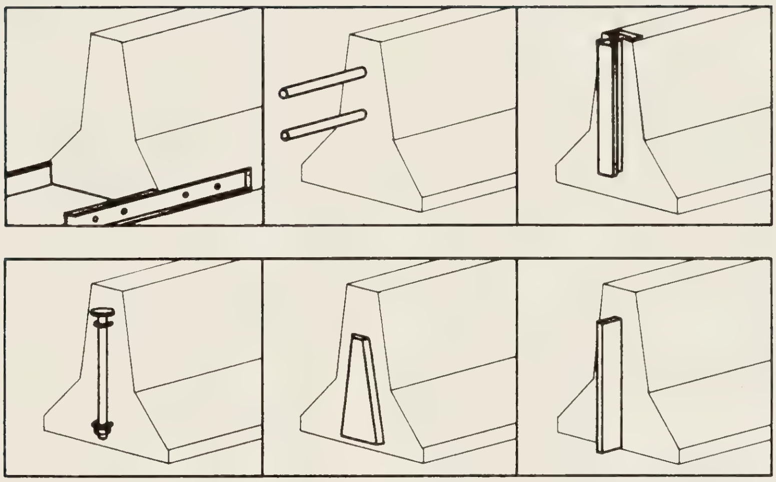


Portable Concrete Barrier Connectors, Publication No. FHWA–TS–88–006

by Office of Implementation

In the mid to late 70's, the portable concrete barrier (PCB) came into general use as a longitudinal construction zone barrier. At first, the PCB segments were simply butted together. It soon became evident that a method was needed to connect the barrier segments. This need resulted in the development of a wide variety of different types of connectors.

In general, the weakest points in a PCB installation are the connectors. Therefore, the overall strength of the PCB is controlled by the strength properties of its connections. A survey of different types of connections reveals that there is a significant variation in their respective structural capacities.

This report presents the state of the art in PCB connector technology. The report includes a description of the types of PCB connectors, a determination of connector strength, crash test results, and the application and maintenance problems. The recommendations concerning additional research and crash testing are currently under consideration by the appropriate offices within the FHWA.

Limited copies of this report are available from the RD&T Report Center. The publication also may be purchased from the NTIS: (PB No. 88–218854/AS, Price code: A08).

Retrofit Railings for Through-Truss Bridges Publication No. FHWA–TS–88–017

by Office of Implementation

Through-truss bridges that have remained in service are required to handle traffic that existed when they were built. Many of these bridges were designed with low-strength pedestrian railngs or with rails and posts which can snag colliding vehicles. This report provides information on use of two retrofit railing system for trough-truss bridges. The first system is the Lower Service Retrofit Railing (LSRR) which was designed to contain and redirect a 4,500-lb automobile colliding at 60 mi/h and 15 degrees. The LSRR is intended for use on through-truss bridges having the following characteristics:

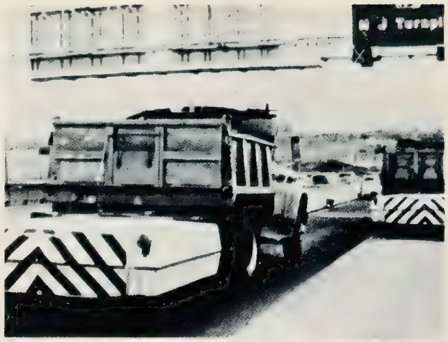
- One-lane traveled way.
- 20-ft wide, 2-lane traveled way.
- Automobile traffic only.
- Posted speed limit of 35 mi/h or less when carrying truck and bus traffic.

The second system is the High Performance Railing (HPR) which was designed to contain and redirect a 20,000-lb school bus colliding at 55 mi/h and a 15 degree impact angle without damaging truss members behind the railing.

The HPR is intended for use on trusses having a significant number of vehicles weighing 20,000 lb or less.

Retrofit bridge rails can make it possible to leave many of these bridges in service until replacement is economically feasible.

The report may only be purchased from the NTIS: (PB No. 88–168737, Price code: A03).



Truck-Mounted Attenuators, Publication No. FHWA–TS-88–018

by Office of Implementation

This report contains an overview of the availability, cost, deployment methods, and performance of various truck-mounted attenuator systems. The report provides practical guidance on where and how to use them in both stationary and moving operations.

This material will be of special interest to highway maintenance engineers and safety specialists responsible for the safety of maintenance operations.

The publication may only be purchased from the NTIS: (PB No. 88–168711, Price code: A03).

Seismic Design and Retrofit Manual for Highway Bridges Report No. FHWA–IP–87–6

by Office of Implementation

This manual is a guide to the seismic design and retrofit of highway bridges. It presents the basic principles of seismology, dynamics, and design as they relate to highway bridge structures. The manual highlights the importance of simplicity, symmetry, and integrity of seismic design concepts for bridges, as well as giving examples of acceptable structural design and designs to be avoided.

Methods for calculating design forces and displacements are developed and analyzed, and relevant computer software is discussed.

The manual should be useful to both beginners and experts in seismic design.

Limited copies of this manual are available from the RD&T Report Center. The manual also may be purchased from the NTIS: (PB No. 88–169503, Price code: A14).

Environmental Flow Charts, Report No. FHWA–IP–87–9

by Office of Implementation

This Implementation Package contains an annotated flow chart of each major environmental subject area for which the Federal Highway Administration (FHWA) has a responsibility. The specific concerns have evolved from focused laws, regulations, and executive orders; such as, the Clean Air Act, the Endangered Species Act, 36 CFR 800 (Protection of Historic and Cultural Properties), the Wetlands Executive Order, etc. Comprehensive environmental concerns are derived from the language of the National Environmental Policy Act (NEPA) and Section 109(h) of Title 23. Flow charts for these issues and processes are not included in this package.

This document provides an abbreviated desk reference which should be useful in identifying and tracking the myriad of environmental processes applicable to highway project development. It also will be useful when project development activities require coordination with other public and private agencies and entities.

The document is published in looseleaf format to facilitate changes and revisions as the various requirements change. The individual pages can be joined together to produce a fold-out chart for each subject. Users are free to modify the charts in any way they feel is useful and appropriate. The charts do not constitute a rule, regulation, or standard.

Limited copies are available from the Federal Highway Administration, Office of Environmental Policy, (HEV–20), 400 7th Street, SW Washington, DC, Telephone: (202) 366–9173.

Equipment Management Symposium—Proceedings Synopsis 1986 Report No. FHWA–TS–87–227

by Office of Implementation

This report summarizes the individual technical papers and presentations made at the Maintenance Equipment Management Symposium held in Indianapolis, Indiana, November 18–20, 1986. The papers and presentations relate the experiences of various States and county equipment managers with implementing automated Equipment Management Systems (EMS). The symposium was hosted by the Indiana Department of Highways and was sponsored by the Federal Highway Administration, Office of Implementation.

Reports were made by four States who contracted with FHWA to test implement an automated EMS as part of an FHWA sponsored pooled fund research study. Presentations were delivered by 13 States and two local highway agencies. Information also was presented by the firm, M^2 LTD, about both a Maintenance Management System (MMS) and Equipment Maintenance Systems Manual which was developed for local highway agencies under a Rural Technical Assistance Program project funded by FHWA.

This report may only be purchased from the NTIS: (PB No. 88–149703/-AS, Price code: A03).

Evaluation of Equipment for Measuring Voids Under Pavements, Report No. FHWA–TS–87–229

by Office of Implementation

This report documents the results of an evaluation conducted on equipment which can be used to detect voids under portland cement concrete pavements. Included in the evaluation were a proof roller, Benkelmen Beam, Dynaflect, falling weight deflectometer, ground penetrating radar devices, and transient dynamic response equipment. The evaluation was based on information compiled from a literature search and from previous field tests conducted by several States.

The report should be of interest to those individuals involved with pavement evaluation procedures and equipment.

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

New Research in Progress

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

NCP Category A—Highway Safety

NCP Program A.1: Traffic Control for Safety

Title: Highway Simulator (HYSIM) Maintenance and Support. (NCP No. 3A1C0122)
Objective: Provide nonpersonal support services to:

- Maintain HYSIM hardware and software.
- Modify hardware and/or software to meet requirements of specific experiments.
- Assist in testing subjects in HYSIM studies.
- Calibrate and operate simulator during experimentation.
- Assist in data reduction.
- Incorporate general purpose hardware/software into the HYSIM for enhanced research capability.

Performing Organization: ENSCO, Inc., Springfield, VA 22151
Expected Completion Date: March 1989
Estimated Cost: $185,000 (FHWA Administrative Contract)

NCP Program A.4: Special Highway Users

Title: Hazardous Materials Movement. (NCP No. 4A4E3012)
Objective: Develop a decision-making framework using existing data (or easily obtainable data) to:

- Identify the risks associated with hazardous materials highway transportation.
- Match those risks with specific areas of concern to develop risk rankings.
- Develop a method to identify options to mitigate risks and select the most cost-effective option with maximum risk reductions potential.
- Develop a model to evaluate the potential for catastrophic events.
- Develop and utilize a master list of options which will reduce the identified risk.
- Conduct field tests to evaluate and refine the information and decision-making process developed in previous tasks.

Performing Organization: Virginia Polytechnic Institute, Blacksburg, VA 24061
Funding Agency: Pennsylvania Department of Transportation
Expected Completion Date: May 1990
Estimated Cost: $250,000 (HPR)

NCP Program A.5: Design

Title: Vehicle Impact Tests of Concrete Median Barriers with Concrete Glare Screens. (NCP No. 4A5B1172)
Objective: Design and crash test a retrofit glare screen slip-formed on top of an existing safety shape concrete median barrier. A full-height, slip-formed, safety shape concrete median barrier tall enough to serve as a glare screen will also be tested and developed.
Performing Organization: California Department of Transportation, Sacramento, CA 95807
Expected Completion Date: December 1989
Estimated Cost: $175,000 (HP&R)

NCP Category B—Traffic Operations

NCP Program B.1: Traffic Management Systems

Title: Data Base Storage and Loop Detector Validity. (NCP No. 4B1A2052)
Objective: Study and determine the uses and most cost-effective storage for the freeway data collected from the surveillance and control system on Seattle area freeways. Develop algorithms for determining the validity of the data.
Performing Organization: University of Washington, Seattle, WA 98195
Funding Agency: Washington State Department of Transportation
Expected Completion Date: July 1989
Estimated Cost: $70,200 (HP&R)

NCP Program B.1: Traffic Analysis and Operational Design Aids

Title: Traffic Management through Public/Private Partnerships. (NCP No. 4B1C0062)
Objective: Develop a manual that will allow jurisdictions to match specific urban settings and problem areas with appropriate traffic management strategies in which public and private agencies can jointly participate.
Performing Organization: COMSIS Corporation, Wheaton, MD 20902
Funding Agency: Pennsylvania Department of Transportation
Expected Completion Date: September 1989
Estimated Cost: $100,000 (HP&R)

NCP Category C—Pavements

NCP Program C.1: Evaluation of Rigid Pavements

Title: Pavement Design Parameters (for Kentucky conditions). NCP No. 4C1B1252)
Objective: Determine the significance and sensitivity of the answers for the 14 newly applied variables incorporated into the 1986 AASHTO Guide for the Design of Pavement Structures. Some factors are well defined, but others are averages that need to be adapted to the locality in which they will be used. Provide specific values needed for Kentucky conditions for both flexible and rigid pavements.
Performing Organization: Kentucky Transportation Research Program, Lexington, KY 40506
Funding Agency: Kentucky Transportation Cabinet
Expected Completion Date: April 1991
Estimated Cost: $159,500 (HP&R)

Title: Preventive Maintenance and Rehabilitation Techniques to Mitigate the Effects of Corrosion-Related Deterioration in Continuously Reinforced Concrete Pavement (CRCP). (NCP No. 4C1C3042)
Objective: Determine which rehabilitation and maintenance method for CRCP are most effective in extending the life of the pavement when black-steel reinforcement shows marked signs of early distress. Perform tests on cathodic protection; longitudinal edge drains; corrosion inhibitors; rubblizing CRCP and AC overlays; 1 1/2-in overlay; 3-in and 5-in AC overlay with and without impervious membranes; full-depth patching; bonded concrete overlays; and effectiveness of epoxy-coated rebars and control sections. These sections are planned for I-43 Walworth County; U.S. Highway 53, Barron County; I-90/94 Sauk and Columbia Counties; and U.S. Highway 41, Brown County, Wisconsin.
Performing Organization: Wisconsin, Department of Transportation, Madison, WI 53707
Expected Completion Date: March 1994
Estimated Cost: $149,600 (HP&R)

NCP Program C.2: Evaluation of Flexible Pavements

Title: Field and Laboratory Investigations of Polishing of Aggregates and Skid Resistance of Flexible Pavements. (NCP No. 4C2A3493)
Objective: Develop mathematical models relating wet accident rate and skid number based on accident data and MU meter measurements. Measure macrotexture in the field and correlate with variables of the asphalt mix. Correlate aggregate polish value and macrotexture to pavement skid resistance. Determine if changes in polish value specifications are needed based on the earlier portions of the study.
Performing Organization: University of Puerto Rico, Mayaguez, PR 00709
Funding Agency: Puerto Rico Department of Transportation and Public Works
Expected Completion Date: April 1989
Estimated Cost: $106,500 (HP&R)

Title: Asphalt Concrete Overlay Design Implementation. (NCP No. 4C2C1092)
Objective: Implement the "EVERPAVE" overlay design procedure that was developed for the Washington State Department of Transportation under Study No. 479. Monitor test sites, and make refinements to the overlay design procedure. Develop a training package and conduct training classes.
Performing Organization: Washington State Transportation Center, Seattle, WA 98195
Funding Agency: Washington State Department of Transportation
Expected Completion Date: May 1990
Estimated Cost: $125,000 (HP&R)

Title: Performance of Overlaid Portland Cement Concrete Pavement in Indiana. (NCP No. 4C2C1102)

Objective: Develop an approximate estimate of the performance of overlaid portland cement concrete pavement in Indiana. Survey a total of 72 1-mile test sections in three environmental zones with three levels of truck traffic and having either one or two overlays of asphalt concrete. Collect and analyze data on deflection, roughness, and distress information over a 5-year period.
Performing Organization: Indiana Department of Highways, Indianapolis, IN 46204
Expected Completion Date: December 1994
Estimated Cost: $154,900 (HP&R)

NCP Program C.3: Field and Laboratory Testing

Title: Instrumentation and Evaluation of Prestressed Concrete Pavement. (NCP No. 4C3D0052)

Objective: Develop performance data from an actual prestressed pavement to be constructed along a section of U.S. 220 (SR 220), Section 1301, in Blair County, Pennsylvania. Develop and implement a detailed plan for instrumenting, monitoring, and evaluating the prestressed pavements; determine the suitability of the procedure used to design the pavement; develop recommendations for improved design and/or construction of prestressed concrete pavement to fit conditions in Pennsylvania.
Performing Organization: Construction Technology Laboratories, Skokie, IL 60077
Funding Agency: Pennsylvania Department of Transportation
Expected Completion Date: October 1989
Estimated Cost: $150,000 (HP&R)

NCP Category C—Pavements

NCP Program C.4: Management Strategies

Title: Digital Data Acquisition and Archiving Systems. (NCP No. 4C4C3112)

Objective: Conduct a feasibility study for a digital photologging system; design and develop a digital data acquisition system; develop procedures for digitizing 35mm film. Design and develop an archiving and retrieval system; develop and demonstrate new applications with the digital photologging system. Provide recommendations for future development of the digital photologging system.
Performing Organization: Ohio State University, Columbus, OH 43210
Funding Agency: Ohio Department of Transportation
Expected Completion Date: March 1990
Estimated Cost: $202,000 (HP&R)

NCP Program C.4: Management Strategies

Title: Connecticut Long-Term Pavement Performance Study. (NCP No. 4C4C3152)

Objective: Validate and refine existing performance curves and develop performance prediction models.
Performing Organization: Connecticut Department of Transportation, Wethersfield, CT 06109
Expected Completion Date: May 1993
Estimated Cost: $343,000 (HP&R)

NCP Category D—Structures

NCP Program D.1: Design

Title: Plastic Hinge Details for Bridge Column Bases. (NCP No. 4D1A2132)

Objective: Investigate the formation of plastic hinges in oversized bridge columns. Identify problems in suggested details and make recommendations for seismic detailing of plastic hinges in oversized columns. Conduct a literature search; perform tests of very small-scale models to guide design of 1/6 scale models and conduct tests of 1/6 scale models. Develop an analytic model based on the test results.
Performing Organization: Washington State University, Pullman, WA 99164
Funding Agency: Washington State Department of Transportation
Expected Completion Date: June 1990
Estimated Cost: $191,000 (HP&R)

Title: Detecting Incipient Failures in Bridges. (NCP No. 4D2B1082)

Objective: Develop a bridge monitoring system to be used to detect incipient failures in bridges. The system will be based on the use of diagnostic dynamic testing and a spectrum analyzer and an expert system computer program.
Performing Organization: Duke University, Durham, NC 27706
Funding Agency: Pennsylvania Department of Transportation
Expected Completion Date: August 1990
Estimated Cost: $387,000 (HP&R)

(The above information was incorrectly stated in the December 1987 issue of Public Roads magazine.)

☆U.S. Government Printing Office: 1988—202–880/80004

Title: Fatigue Testing of Prestressed Beams. (NCP No. 4D1B1092)
Objective: Perform inservice testing of a three-span prestressed concrete skewed bridge for live load distribution and actual live load stresses. Laboratory test two to three beams removed from the bridge for static and fatigue characteristics and estimate remaining life of the beams. Develop load rating methods for existing prestressed concrete I-Beam and similar bridges.
Performing Organization: Lehigh University, Bethlehem, PA 18015
Funding Agency: Pennsylvania Department of Transportation
Expected Completion Date: September 1990
Estimated Cost: $200,000 (HP&R)

Title: Test Procedures for Drive Posts. (NCP No. 4D1B3152)
Objective: Investigate the failure modes of typical single drive posts. Based on the findings, develop specifications and laboratory testing procedures which account for actual forces being experienced by drive posts in the field.
Performing Organization: CTL Engineering, Inc. Columbus, OH 43204
Funding Agency: Ohio Department of Transportation
Expected Completion Date: March 1989
Estimated Cost: $75,000 (HP&R)

Title: Evaluation of Corrugated Metal and Reinforced Concrete Pipe Used as Drainage Structures Under State Highways. (NCP No. 4D1D4032)
Objective: Evaluate cross-road culvert pipes on the State highway system for material condition and functional adequacy. Develop the expected life of different types of culvert pipes based on this evaluation. Inspect and analyze in-place culvert pipe for deterioration. Factors to be studied include: Age, material type, contact soils, water effluent, and watershed environment.
Performing Organization: Missouri Department of Transportation, Jefferson City, MO 65102
Expected Completion Date: April 1990
Estimated Cost: $600,000 (HP&R)

NCP Category E—Materials and Operations

NCP Program E.5: Maintenance Effectiveness

Title: Study of Rest Area Truck Parking. (NCP No. 4E5D2832)
Objective: Identify the factors which influence the demand for truck parking at rest areas. Develop empirical mathematical models which will predict the demand for truck parking at rest areas. Develop feasible alternatives of accommodating truck parking demands, and recommend policies and programs to implement the results.
Performing Organization: Byrd, Tallamy, MacDonald, and Lewis, Falls Church, VA 22042
Funding Agency: Ohio Department of Transportation
Expected Completion Date: April 1989
Estimated Cost: $136,450 (HP&R)

NCP Program E.9: Technology Transfer for Materials and Operations

Title: Design and Develop a Prototype Automated Highway Raised Pavement Marker Installation Machine. (NCP No. 4E9E0173)
Objective: Develop and test a prototype mechanical machine that will automatically install raised pavement markers using asphalt adhesives and completely remove the maintenance worker from any exposure to the flow of traffic.
Performing Organization: California Department of Transportation, Sacramento, CA 94274
Expected Completion Date: March 1990
Estimated Cost: $75,000 (HP&R)

U.S. Department
of Transportation

**Federal Highway
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