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Evaluating Bridge Structures, Pavement Maintenance, Roadside Management, Deicing Salts, Transport of Hazardous Materials

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Laboratory Static Load Tests on Five Sunshine Skyway Bridge Girders

Morris W. Self, Department of Civil Engineering, University of Florida, Gainesville

The Sunshine Skyway that crosses the entrance to Tampa Bay south of St. Petersburg, Florida, on a 6.5-km (4-mile) bridge consists of two separate two-lane structures constructed primarily of prestressed concrete girders on reinforced concrete pile bents. The older of the two structures was completed in 1954 and was one of the earliest of its type in the U.S. The girders were post tensioned with steel bars grouted in steel conduits, but spalling and rusting were observed before the bridge was 20 years old. Florida Department of Transportation engineers investigated the structural integrity of the bridge and transported six girders to the University of Florida in Gainesville for laboratory tests. Five of these girders were tested under static loads to determine ultimate flexural capacities, and two were found to be extensively spalled with corroded prestressing bars. These two were the only ones to exhibit a premature fracture of the bars. The tests indicated that no serious loss of strength would be likely to occur before extensive spalling of the concrete revealed badly corroded or fractured bars. Girders can therefore be periodically inspected for removal and replacement before collapse becomes imminent.

The highway structure that crosses the entrance to Tampa Bay south of St. Petersburg, Florida, is known as the Sunshine Skyway. It consists of two separate two-lane bridges extending 6.5 km (4 miles) across the bay; about 5 km (3 miles) of it are of low-level prestressed concrete girders on reinforced concrete pile bents and about 1.5 km (1 mile) of high-level steel girders and trusses that carry traffic without interruption over the main ship channel to the Port of Tampa. The first, the northbound bridge, was completed in 1954, and the second, the southbound bridge, was completed in 1969.

Periodic maintenance inspections by the Florida Department of Transportation (DOT) have revealed extensive corrosion on prestressing and reinforcing steel in girders and pile bents of the older bridge. The problem has evidently been accelerating in recent years. A corrosion survey conducted in 1973 listed 81 girders in 54 of 349 spans in poor or critical condition from spalled concrete and exposed and severely corroded prestressing steel. Florida DOT engineers, concerned about the deterioration and probable reduction in strength of the bridge girders, initiated a program of in situ tests to evaluate the strength capacity of the bridge superstructure (1).

After consultations with engineering faculty at the University of Florida, a research program was contracted to load test bridge girders in the Department of Civil Engineering Laboratory. Six girders were removed from the older bridge and transported from Tampa Bay to Gainesville.

THE OLDER BRIDGE

Bridge Construction

One of the salient features of the older bridge is the post-tensioned girders with composite reinforced concrete deck slab, one of the earliest uses of this type of construction in the United States. The girders were cast and post tensioned in casting yards near the construction site. Each girder was post tensioned by three 25.4-mm (1-in) diameter, 1105-MPa (160 000-lbf/in^2) ultimate strength Macalloy steel bars. Two bars are essentially straight; the third is draped parabolically. The bars were post tensioned through 31.8-mm (1.25-in) conduits and grouted. The Macalloy steel was imported from England, but the concrete materials were local products.

The reinforced concrete piles and caps extend about 2.5 m (8 ft) above mean high water, and depths in the bay along most of the length of the bridge average about 5 m (16 ft). The pile bents are spaced at 14.6 m (48 ft), providing 14-m (46-ft) bridge spans.

Environmental Conditions

The bridge girders are usually 2.5 m (8 ft) to 3 m (10 ft) above the water surface when the bay is calm. The mean tidal range is 0.5 m (1.5 ft), but sustained winds may raise or lower the tidal range by as much as 1.25 m (4 ft). In rare hurricanes, the tidal stage may rise 2 m (6 ft) or more. The bay is relatively open to the Gulf of Mexico, with wide passes to the west and southwest on either side of Egmont Key. The generally shallow waters of the bay become severely choppy when wind velocities rise to 24 km/h (15 mph) and higher. The salinity of the bay waters around the bridge varies with rainfall and freshwater runoff, but the mean is 32 to 33 parts per thousand, compared with 37 parts per thousand in the open Gulf waters at this latitude.

Description of Corrosion

As a result of frequent exposure to chop and northeasterly winds, the eastern sides of the girders show appreciably more cracking, spalling, and corrosion than the western sides. Most of the deterioration occurs near the ends of the spans, where waves strike the pile bents and splash up against the lower girder flanges. The end blocks are reinforced with vertical stirrups and longitudinal tie-bars. The lower tie-bars extend into the tapered region where the girder section changes from rectangular to I-shape. Here the concrete cover is as little as 6 mm (0.25 in).

Saltwater penetrating this thin layer of concrete produces corrosion and expansion of the steel and spalling of the concrete and further exposure to corrosion. Eventually the prestressing conduits become exposed, and corrosion progresses down the length of the conduits and penetrates the prestressing bars. Concrete cover over the conduits varies from about 13 to 32 mm (0.5 to 1.5 in). In many girders several meters of conduit were disintegrated, and in a few girders the prestressing bars had reduced in section to the point where they fractured under the prestress force (these girders have been replaced). Figures 1 and 2 show typical spalling and corrosion of girders.

STRUCTURAL PROPERTIES OF BRIDGE GIRDERS

Geometric Properties

The bridge superstructure consists of six post-tensioned girders spaced on 1.83-m (6-ft) centers with a 178-mm (7-in) reinforced concrete deck 11.43 m (37.5 ft) wide. This provides for two lanes of traffic plus walkways on either side as shown in Figure 3. Composite behavior of the girders and the deck slab is provided by steel shear ties and concrete keys. The girders span between the pile bents as simple beams, and the concrete deck is provided with control joints at each bent so that flexural restraint at the supports is negligible. Precast transverse diaphragms are located at the one-third points of the span and are grouted in place and post tensioned with a single transverse 25.4-mm (1-in) Macalloy bar. The girder dimensions are shown in Figure 4.

Mechanical and Physical Properties

The properties of the girder material were determined by laboratory tests on samples from the girders that were tested in the laboratory. The compressive strengths of the deck and girder concrete were evaluated from core tests and were found to be 29.9 MPa (4336 lbf/in^2) and 43.72 MPa (6339 lbf/in^2) respectively.

The percentage composition of the Macalloy prestressing steel is approximately 97.4 iron, 0.6 carbon, 1.2 manganese, and 0.8 silicon. The Department of Materials Science tested corrosion and notch sensitivity of this steel (2).

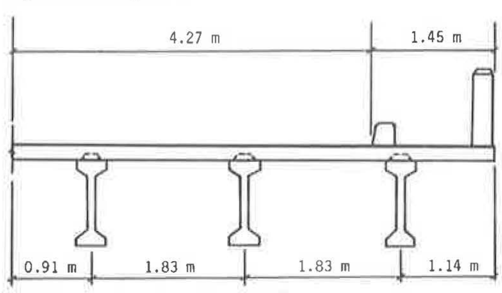
The results of tension tests indicated that the ultimate tensile strength in air without notching was about 1070 MPa (155 150 lbf/in^2). When the specimens were notched, the strength fell 50 percent. An additional reduction in strength occurred when notched specimens were immersed in a synthetic seawater solution while tested.

In summary, the Macalloy steel used in the Sunshine Skyway Bridge is a medium-carbon, silicon, manganese steel exhibiting moderate tensile properties with low impact strength, high notch sensitivity, and an apparent adverse reaction to a corrosive environment. A stress-strain diagram for the bars is shown in Figure 5.

Figure 1. Typical concrete spalling and steel corrosion in bridge girders.

Figure 2. Exposed and corroded prestressing bar in girder lower flange; lower bar is tie bar from end block.

Figure 3. Bridge cross section.



LABORATORY LOAD TESTS

We selected six composite bridge girders on the basis of their different stages of deterioration and removed them from the Sunshine Skyway for testing. In order not to disrupt traffic, we took the girders from one lane at two spans about 0.5 km (0.3 mile) apart. This provided two undamaged exterior girders, two undamaged interior girders and two severely corroded interior girders. The deck and diaphragms were sawed free, and the girders were removed from the bridge, loaded onto flatbed trucks, and transported to Gainesville.

Description of the Test Procedure

The test arrangement is shown in Figure 6. The girders were placed on support blocks with neoprene bearing pads on a span of 14 m (46 ft), measured between center of bearings. A loading frame constructed of rolled steel beams was placed over the girder and secured to existing floor anchors. The test load was applied by two hydraulic jacks placed between the load frame and girder. The load was metered by electric load cells placed between the jacks and steel plates bearing on the girder flange. Deflections were measured in relation to a taut wire stretched over supports located at each end of the girder.

A single concentrated load to the girders was applied and increased at a relatively constant rate; it was held static at intervals to inspect for cracks and to read deflections.

The original design load specified for the bridge could not be documented, but some evidence was found to indicate that it was designed for the standard AASHO H20-44. Because there is little difference between the distribution of bending moments and shears produced by the H truck loading and by a single concentrated load, the latter was used to facilitate testing.

The theoretical ultimate flexural and shear capacities of the girders were calculated in accordance with recommendations of the American Concrete Institute Standard 318-71. The resulting equations for flexural capacity (M_u) and shear capacity (V_u) are

$$M_u = A_{ps} f_{ps} d[1 - 0.59\,(A_{ps} f_{ps}/bdf'_c)] \quad (1)$$

$$V_u = 0.6\sqrt{f'_c}\; b_w d + V_d + [(V_i M_{cr})/(M_{max})] \quad (2)$$

where

A_{ps} = area of prestressed reinforcement,

Figure 4. Typical girder dimensions.

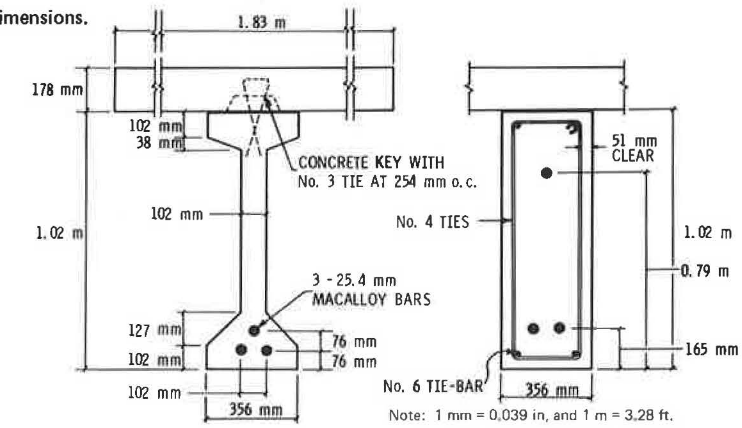


Figure 5. Tensile stress-strain test for Macalloy steel bar.

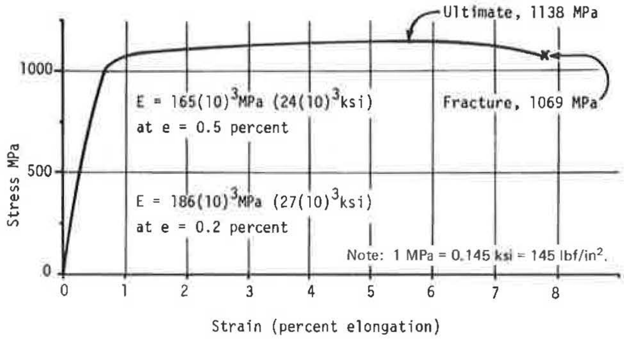


Figure 6. Alternate test load positions.

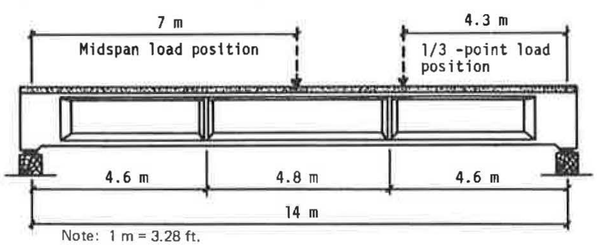


Figure 7. Theoretical ultimate flexural capacity of girders in terms of a single load P_u.

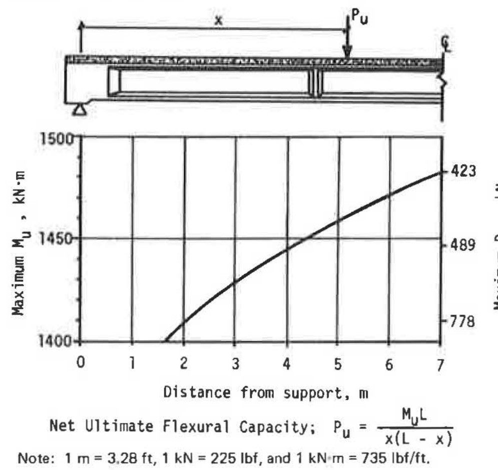
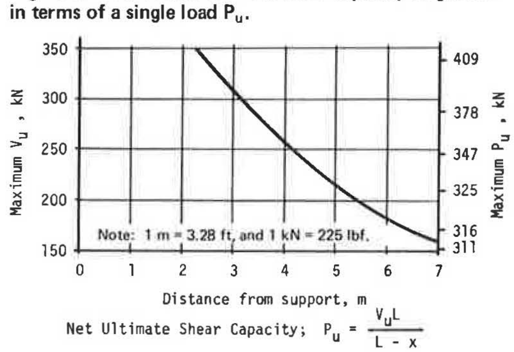


Figure 8. Theoretical ultimate shear capacity of girders in terms of a single load P_u.



b = effective flange width,
b_w = web width,
d = effective depth,
f'_c = compressive strength,
f_{ps} = stress in prestressed reinforcement,
M_{cr} = cracking moment,
M_{max} = maximum applied load moment,
V_d = dead load shear force, and
V_l = applied load shear force.

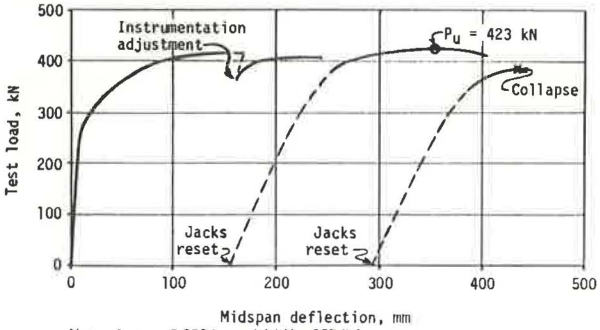
Net ultimate capacities for use in estimating maximum applied test loads were obtained by subtracting bending moments and shears produced by the girder dead weight. The results, plotted in Figures 7 and 8, show that a midspan load is the most critical for both flexure and shear and that shear is more critical than flexure. The maximum capacity for a midspan test load is 420.8 kN (94 600 lbf) for a flexural failure and 314 kN (70 600 lbf) for a shear (diagonal tension) failure.

Test of Girders 139-S1 and 139-S2

Both girders were loaded in the same manner, and the structural behavior under loading was essentially the same for both. The midspan test load was applied through a matched pair of hydraulic jacks spaced about 15 cm (5.9 in) on each side of the girder centerline. Heavy steel plates were used to provide lateral distribution of the load over the width of the slab. The test results are plotted in Figures 9 and 10.

The test ultimate capacity of both girders was 422.6 kN (95 000 lbf) compared to the predicted flexural strength of 420.8 kN (94 600 lbf). It is also interesting to note here that the test load developed an ultimate bending moment at midspan exactly equal to the ultimate moment that would be produced by a standard AASHTO

Figure 9. Test of girder 139-S1, midspan load.



Note: 1 mm = 0.039 in, and 1 kN = 225 lbf.

Figure 10. Test of girder 139-S2, midspan load.

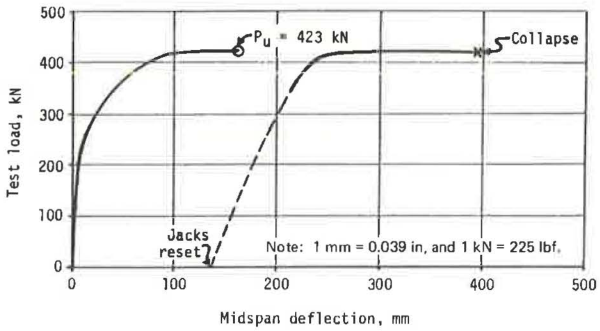
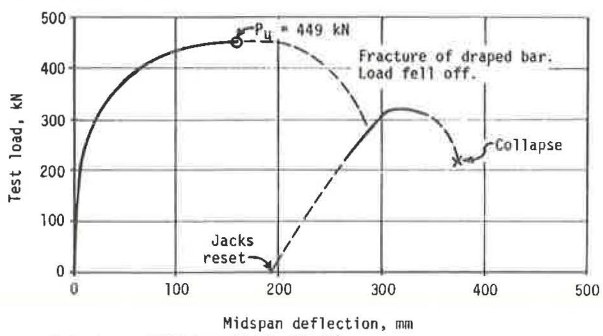


Figure 11. Test of girder 171-S3, one-third point load, showing one lower bar extensively corroded.



Note: 1 mm = 0.039 in, and 1 kN = 225 lbf.

HS20 truck load. Note that the predicted shear capacity was exceeded, even though the girder web was only 101.6 mm (4 in) and no web reinforcing was provided. This was the result of not considering the shear capacity of the flange, the redistribution of stress in the web after cracking, and dowel strength of the reinforcing in the flange when capacity was calculated.

Test of Girder 171-S3

This girder exhibited evidence of extreme corrosion of the prestressing conduits and bars at one end of the span. One side of the lower flange was spalled from the end block for a distance of about 2.2 m (7 ft), exposing badly rusted conduit and about a meter of rusted prestressing bar. The rusted surface of the bar was irregular, and the rust penetrated about 3 mm (0.12 in). The girder web was severely cracked at the level of the upper inclined prestressing bar. A single crack through the web followed the bar from the end block to the diaphragm block. After the test, when we removed the concrete to expose the steel, we found extensive rusting of the conduit and the bar.

We loaded the girder by placing a single concentrated load just outside of the diaphragm 4.3 m (14 ft) from the support. This produced the maximum bending moment in the damaged section of the girder and also produced high shear and diagonal tension in this region.

The test data are plotted in Figure 11. The first crack occurred under a load of 205 kN (46 000 lbf). The lower flange cracked vertically at the load point, and the crack progressed diagonally upward through the web as the test continued. As the load approached 445 kN (100 000 lbf), diagonal tension and flexure cracked the web extensively throughout the end region. At 449 kN (101 000 lbf), we heard the upper bar fracture, and the load quickly fell off to 294 kN (66 000 lbf) with a rapid increase in deflection. Later inspection revealed a brittle fracture of the upper bar at the load point and at a point on the bar where the surface had eroded to a depth of about 2 mm (0.08 in). After releasing the load, resetting the jacks, and reloading, a second bar failed at about 311 kN (70 000 lbf), and the girder collapsed. The second bar to fail was the one that had been initially exposed by spalling of concrete. The break occurred in the region of exposure near the end block.

A bond failure between the bar and the conduit permitted the bar to slide several millimeters as the beam "opened up" at the load point. This bar also failed by brittle fracture in a region of reduced cross section because of corrosion. The third bar was not corroded or rusted, and it failed ductily with extensive necking.

As in the previous tests, the maximum test load was essentially equivalent to the AASHTO HS20 truck loading for bending moment and shear in the region of the one-third point of the span (1 kN•m = 735 lbf/ft and 1 kN = 225 lbf).

Capacity	Test Load	AASHTO HS20 Load
M_u	1599 kN·m	1582 kN·m
V_u	347.4 kN	331.8 kN

Test of Girder 139-S3

Of the six girders removed from the Skyway bridge, 139-S3 was most extensively corroded. One side of the lower flange was spalled from the end block for a distance of about 3 m (10 ft), exposing badly rusted conduit, rusted prestressing bar, and rusted end block reinforcement. One prestressing bar had completely fractured as a result of extensive rusting. There was a 2-m (6.6-ft) long, barely visible crack running parallel to the lower prestressing bars on the side opposite the spalled side. After the load test, the steel was exposed and revealed extensive rusting of the conduits of both lower bars for an additional meter beyond the point exposed by the pretest spalling.

This girder was loaded in the same way as girder 171-S3, by placing a single concentrated load just outside of the diaphragm and 4.3 m (14 ft) from the support, and producing maximum stresses in the damaged section of the girder.

The test data are plotted in Figure 12. The first crack occurred at a load of 178 kN (40 000 lbf). The lower flange cracked vertically at the load point, and then the crack progressed diagonally upward through the web as the test continued exhibiting the typical diagonal tension crack. At a load of 320 kN (72 000 lbf), the cracks were so large we could see the prestressing bars in the flange and web. At 329 kN (74 000 lbf), the remaining lower prestressing bar fractured and the load

Figure 12. Test of girder 139-S3, one-third point load, showing one lower bar fractured before test.

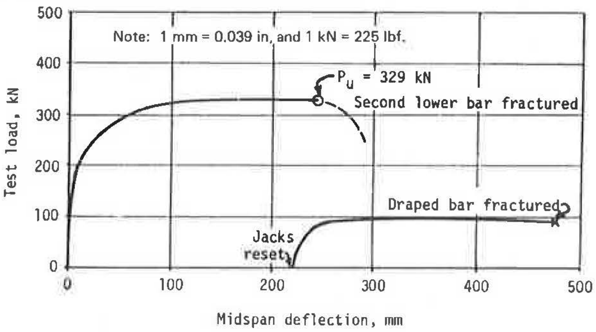


Figure 13. Test of girder 171-S2, one-third point load.

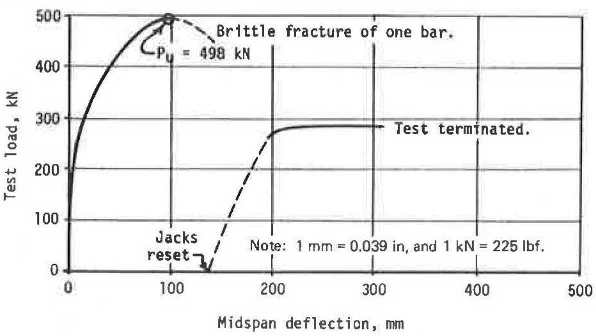
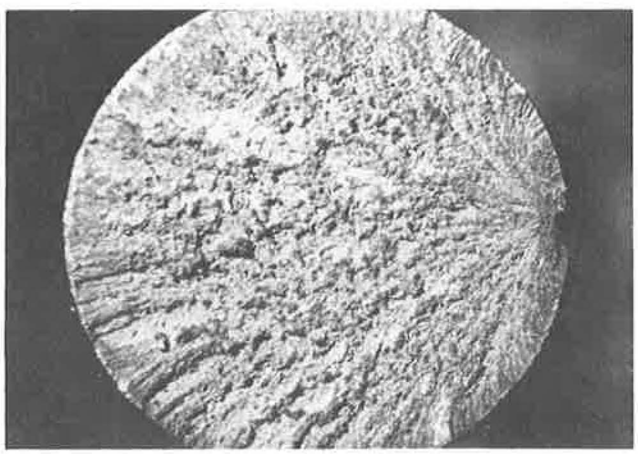


Figure 14. Typical brittle fracture of Macalloy bar.



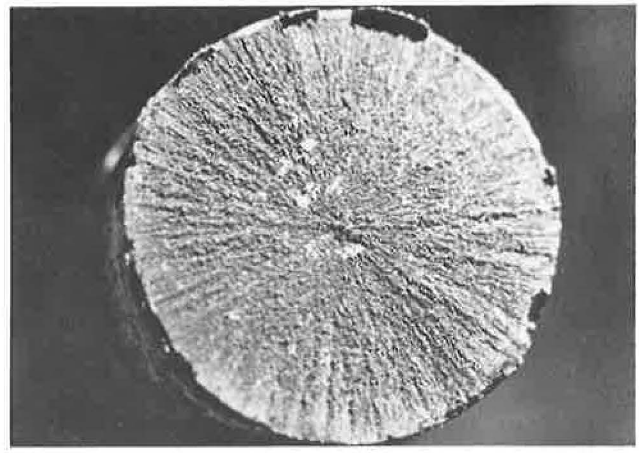
fell off. At this point the load was released, the jacks reset, and the test continued. At a load of 98 kN (22 000 lbf) the draped bar fractured.

As was expected, the draped bar, which showed no evidence of corrosion, failed ductily with extensive necking. The lower bar failed by brittle fracture in a region where it was pitted by corrosion.

The maximum test load is compared below (1 kN•m = 735 lbf/ft and 1 kN = 225 lbf) with the standard AASHO H20 and AASHTO HS20 truck loadings for ultimate bending moment at 4.3 m (14 ft) from the support.

Capacity	Test Load	AASHO H20 Load	AASHTO HS20 Load
M_u	1242 kN·m	1246 kN·m	1582 kN·m
V_u	263.8 kN	252.2 kN	331.8 kN

Figure 15. Typical ductile failure of Macalloy bar.



Although this girder did not develop the HS20 capacity that all the others tested did, it did essentially develop the H20 capacity, even though one prestressed bar was ineffective as a result of corrosion-induced fracture before the test.

Test of Girder 171-S2

This girder appeared to be in excellent condition. No spalling or cracking was present.

In order to show the percentage of strength loss from corrosion in previously tested girders 171-S3 and 139-S3, this girder was loaded in the same manner, by placing a single concentrated load just outside of the diaphragm and 4.3 m (14 ft) from the support.

The test data are plotted in Figure 13. The cracking load was 245 kN (55 000 lbf). This girder showed more uniform and evenly distributed cracking than previously tested girders, where damaged areas occurred near the load point. At a load of 498 kN (112 000 lbf), the load fell off. An autopsy following the test revealed that the draped bar failed by brittle fracture at the load position in the span. At this point the conduit and bar were located in the girder web with about 25 mm (1 in) of concrete cover. No spalling was noted, but the conduit was extensively rusted on one side, and the bar was slightly pitted.

The test was terminated before the two lower bars fractured, but later inspection revealed that they had undergone extensive ductile yielding.

The test load of 498 kN (112 000 lbf) is 10 percent higher than a concentrated load equivalent to an HS20 truck loading.

SUMMARY AND CONCLUSION

Of the five girders tested, only two were extensively damaged by the corrosion evidenced by spalled concrete and rusted steel. These same two girders were the only ones to exhibit a loss of strength when the failure load was less than the theoretical ultimate capacity.

Girder 171-S3 failed under a test load 8 percent below the theoretical capacity. A prestressed bar began fracturing at a point of corrosive pitting on the bar surface. The failure surface, magnified about five times, is shown in Figure 14, which clearly shows the propagation of the brittle fracture from the surface pit. The bars that were not pitted by corrosion failed in a ductile manner with necking and the associated loss of section area before breaking. Figure 15 shows the typical ductile

Table 1. Summary of test results.

Girder	Pre-Test Condition	Load Position	Predicted Ultimate Capacity (kN)	Test Collapse Load (kN)	Equivalent AASHTO Load	Apparent Loss of Strength (%)
139-S1	No apparent damage	Midspan	423	423	HS20	None
139-S2	No apparent damage	Midspan	423	423	HS20	None
171-S2	No apparent damage	⅓ point	489	498	HS20 plus	None
171-S3	Extensive corrosion	⅓ point	489	449	HS20	8
139-S3	Extensive corrosion; one bar fractured	⅓ point	311	329	HS20	33

Note: 1 kN = 224.8 lbf.

failure surface, when fracture propagates from the center of the section.

In girder 139-S3, one of the prestressed bars fractured before the girder was removed from the bridge. As expected, the girder suffered a 33 percent loss of strength as a result of the missing bar. Of the remaining two bars, one was corroded and failed by brittle fracture at near ultimate strength, and the other failed ductily with no evidence of corrosion.

Another brittle fracture occurred in girder 171-S2, although we saw no spalled or cracked concrete before the test. Here, again, the fracture was induced by corrosive pitting at the bar surface. However, the bar appeared to have reached the theoretical ultimate strength before failure.

The test results are tabulated in Table 1. The predicted ultimate capacity is the theoretical maximum concentrated load capacity for an uncorroded girder. The tabulated predicted capacity is based on flexural not shear strength, although the collapse mechanism was a combination or interaction of flexure and diagonal tension. When there was no obvious initial damage, the test capacity equaled or exceeded the predicted capacity. The equivalent AASHTO load includes consideration of load factors ($M_u = 1.5\ M_{d1} + 2.5\ M_{11}$), distribution factor (1.09), and impact factor (1.30), in accordance with the AASHTO specifications. The test load for girder 171-S2 was 10 percent above the equivalent HS20 ultimate load. All other test loads were essentially the same as the equivalent AASHTO loads.

These tests indicate that on the Skyway bridge there is no immediate danger of girder collapse from corrosion. Even the 33 percent strength loss of one prestressing bar does not reduce the girder strength below that required for the AASHO design load. Furthermore, the load-deflection curves show that the girders are versatile enough ductily to redistribute the load to adjacent girders should one completely collapse.

Periodic visual inspection will reveal extensively corroded girders for selective removal and replacement. This process will provide time during which additional studies can be undertaken to evaluate remedial measures.

It is of interest to note that the corrosion occurs primarily in the end regions of the girders, where they are exposed to salt spray from waves splashing against the piers. Very little corrosion has been observed in the midspan regions even though the concrete is apparently less than 25 mm (1 in).

ACKNOWLEDGMENT

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REFERENCES

1. T. L. Larsen and others. Structural Evaluation of Sunshine Skyway Superstructure. State of Florida Department of Transportation, Research Rept. 178, Nov. 1973.
2. R. W. Gould. Detection and Prevention of Re-Bar Failure in Concrete Structures. Department of Materials Science and Engineering, Univ. of Florida, Gainesville, Jan. 1975.

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Basic Evaluation of the Structural Adequacy of Existing Timber Bridges

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The properties of wood as a structural material in highway bridges are discussed. These properties give a timber bridge a very different load-carrying capability than that of a steel or a concrete bridge. Wood has the advantage of being able to sustain overloads for short time periods but is subject to decay and deterioration. By recognizing these characteristics, the engineer can determine the safe loading for the structure and can recommend procedures for prolonging its life. Guidelines are offered to assist the engineer in his work, and suggestions for presenting the information to the owner and recording it are made.

Wood is structurally very different from concrete or steel and therefore performs differently as the various components of a timber bridge. Some of the characteristics of wood that affect its structural behavior and set it apart from concrete and steel are discussed below.

WOOD CHARACTERISTICS

Wood is orthotropic in nature; that is, its strength properties differ in three directions, longitudinally, radially, and tangentially to the grain or axis of fiber orientation. Wood strength is greatest in the longitudinal, or parallel to grain direction, and weakest across the grain. The strength of timber connectors depends, then, on the direction in which the load is applied relative to the direction of the grain. Stresses for loads applied intermediately between parallel and perpendicular to grain directions are computed by the use of Hankison's formula (1).

Wood is a variable material. Each species of tree has basic structural properties unique to itself. Growth variations and manufacturing processes also produce characteristics in each individual board that control the strength properties. Sawed timbers are graded and assigned strength values according to the species and to the defects in each piece. Characteristics determined visually and used in assigning strength properties are slope of grain, knots and their locations, pitch, wane, density, checks or splits from uneven drying, and size variations—to name a few.

In the 1930s, glue-laminated structural timber came into use in this country. Selective placing of individual boards with proper gluing has resulted in a structural member of better strength properties than was previously available in solid-sawed timber. During the early years of production of glue-laminated material, manufacturing controls were rather loose, and not all of the properties of the finished product were known as they are today. In evaluating early bridges constructed with glue-laminated material, the investigator should examine and analyze this material in the light of current knowledge.

The strength of wood and its dimensions change with changes in equilibrium moisture content. As the moisture content falls below the fiber saturation point, wood shrinks in the tangential and radial directions but not in the longitudinal. In most domestic wood, moisture content will fall during service from first installation. This may be important to the behavior of the timber structure, particularly in relation to connections. Split rings and shear plates, for example, tend to cause wood members to split during drying, weakening connections in timber trusses.

Generally, strength and stiffness increase as moisture content falls. The moisture exposure conditions and the moisture content of wood members should be considered when evaluating the strength of a timber bridge. In recent years, bridge decks have been constructed of glue-laminated planks about 1.22 m (4 ft) wide. Tests by the Forest Products Laboratory in Madison, Wisconsin, indicate that this product is superior in strength to nail-laminated decks. These large panels also provide more protection to the deck-supporting members. The use of this type of deck will also influence the life expectancy of the structure.

The length of time during which a wood member is subjected to a load allows for an adjustment in the safe working stresses that may be used. The reason for this is wood's good energy-absorbing properties. The permissible stresses are higher for short load durations and lower for longer durations.

For impact loading, the permissible stresses may be doubled. For short time loads, such as wind and seismic forces, the stresses may be increased by 33 percent. For a 7-d duration stresses may be 25 percent greater, and for a 2-month duration stresses may be increased by 15 percent over those basically allowable. In contrast, for permanent loading, the stresses must be reduced to 90 percent of the allowable total. These variations are important considerations in analyzing timber bridges.

AASHTO (2) generally recognizes these variations in allowable stress. For example, impact forces need not be considered. Wind and seismic stresses are increased by one-third, similar to concrete and steel. The stresses for 7-d and 2-month duration loads are recognized by AASHTO for loadings other than traffic loads.

Before 1974, the exclusion of vehicle loads for making load duration adjustment implied that the increased stresses would be permitted for this loading. Some of these departures from recognized and published criteria for design of timber structures introduce a measure of conservatism in timber bridges that should be recognized when evaluating existing ones.

Probably the most serious drawback to timber structures is that they are subject to decay and attack by various organisms. Unprotected wood can be critically weakened by decay and insect infestation when temperature and moisture conditions are favorable. It is essential that these be properly protected against either by physical cover or preservative treatment.

EVALUATING TIMBER BRIDGES

In evaluating the load-carrying capability of a wholly or partially timber bridge, each bridge component and its connections must be investigated for strength and then the entire structure evaluated as a unit. This paper is primarily concerned with highway bridges, although the principles involved will generally apply to railroad bridges also.

The age of a bridge is important for reasons other than simply age, because the permissible stresses and grading rules for lumber have varied throughout the years. Design principles have also changed. As an example, compare the 1961 AASHO bridge specifications with current practice.

First, all of the tabular stresses and the grades of lumber are different. Some of the previously permitted stresses have been reduced, some increased. Horizontal shear is now 655 kPa (95 lbf/in^2); in 1961 it varied from 760 to 1000 kPa (110 to 145 lbf/in^2) for Douglas fir on the West Coast. Modulus of elasticity was previously set at 11 100 MPa (1.6 million lbf/in^2) for all grades of Douglas fir. Now these vary from 10 400 to 13 200 MPa (1.5 to 1.9 million lbf/in^2).

Glue-laminated timber was given only partial consideration for selective manufacture, whereas now the full values recommended by the industry are accepted. Shape factor has allowed current practice to provide for a reduction in permissible stress. This principle was known, at least in theory, as the form factor in 1954 (3).

In recent years, it has been found that, when a series of connectors such as bolts are placed in a row parallel with the applied stress, they do not develop each connector's full shear value (4). If three or more connectors are in a row, a reduction must be made in their combined strength. This could be a significant strength-reducing factor in trusses with a built-up tension cord or at-heel connections. The structure should be analyzed according to current design criteria and strength characteristics. Steel components and metal connectors in an older bridge should also be evaluated differently from when they were first installed.

Loading and traffic must be considered and compared with those when the structure was first erected if it is several years old. If it is decided that a load or speed limit should be posted, enforcement must be taken into account. Posting an unrealistic load limit is sure to cause violations. If the normal traffic that has been using the structure is heavier than the posted limit, questions are sure to be raised as to the validity of the limit. Some sort of survey of probable frequent or occasional heavy loads would be in order. Whether or not the bridge can carry single or multiple land loads will influence its load-carrying capability.

The engineer's judgment and experience will play an important role here because of the nature of timber. The engineer or person making the evaluation of a bridge must be well informed on the use of timber as a structural material and must understand its capabilities as well as its limitations. This person must also be able to recognize the species and grade characteristics of the wood components.

Accidental or unintended loads may significantly reduce the safe live load capacity of a structure or its components. A build-up of asphalt pavement or sand and gravel on the deck can seriously reduce the live load capacity of a bridge. For example, a 15.25-m (50-ft) single-lane bridge of glue-laminated stringers spaced at 1.83 m (6 ft) on centers with a laminated deck is designed for an AASHTO HS-15 loading. The application of 5 cm (2 in) of asphalt will increase the bending moment in the stringers by as much as 20 percent and will increase their reactions by about 17 percent. Therefore, it is important that all loads be considered in evaluating a timber structure and that proper maintenance be assured.

Load sharing may be considered to improve the load-carrying capability of individual members. For example, close-spaced floor joists supporting a stiff deck will assist each other by transferring the load through the deck; like members deflect in proportion to their stress. Therefore, if we control the deflection of two closely spaced members by a stiff deck, then one member cannot deflect much more than its neighbor under application of load.

Damage from abuse or decay is a frequent cause of failure and should be determined by careful inspection. Note any stress concentration defects, such as splits, indentations, or crushed wood in bending members, and determine their effects on the member. Decay, if not too extensive, may be controlled by application of preservatives or proper protection. If, for example, the ends of floor stringers are covered with dirt and debris at the abutments, decay may begin. Proper maintenance can control this. Some experimental work is currently being done on introducing preservatives into wood by fumigation (5). These studies may prove very beneficial in prolonging the life of wood structures.

A surface treatment with oil-borne creosote or pentachlorophenol is not as reliable as pressure treatment but will significantly reduce exposure to decay. Treatment of ends of members with an oil-borne preservative is particularly effective, because moisture migrates much more rapidly through the end grain than through the sides of wood members.

Well-sealed ends of members also control splitting, which both weakens the member and exposes more surface to organic attack. If decay has occurred, its extent must be known. A simple wood auger may reveal the depth of defective wood, or, if needed, there is a special tool that can drill an undisturbed core sample for further evaluation. Samples should be taken, if possible, where the member will not be weakened. The hole should be protected with preservatives and plugged after the sample has been taken.

When analyzing a timber bridge, it is helpful to obtain all the available records concerning its design, construction, maintenance, modification, and history of use. Unfortunately, this material is not always available for highway bridges, but, if the original plans and design notes can be found, the engineer's task is simplified. The design can then be compared with the materials used and actual dimensions and details of the built structure.

The key to sound performance, for all structures, is in the connections. This is particularly true for timber bridges. Many times these are not visible, and the engineer will experience some difficulty in discovering what was used. Split rings and shear plates are concealed between wood members. These significantly increase the shear strength of bolted connections. Sometimes a probe may be used to good advantage in locating these.

Guardrails for both traffic and pedestrians are usually a problem, particularly with older timber bridges. Current AASHTO standards require that traffic rails be capable of resisting a lateral force of 4.45 kN (10 000 lbf) applied 68 cm (27 in) above the roadway surface. When working with the permissible stresses specified by AASHTO, the engineer generally finds that wood rails do not conform. Judgment is needed to evaluate the adequacy of railings.

Occasionally, there may be a built-in condition that becomes a problem when the structure ages. A compression member, such as a strut in a truss or a column, may have been designed as an axially loaded member. Then, because of deformation or a connection partially failing or a later modification of the framing, the member suffers induced bending stresses. The combined bending plus axial load stresses may prove excessive even for relatively small eccentricities. Truss members must be analyzed for stress reversals from loading combinations.

Short compression members may be completely adequate normally, but, if tension occurs, connections can be inadequate. Most species of wood have higher allowable stresses for short columns than for tension. In Douglas fir this difference can be 30 percent or better in some grades. Secondary stresses in trusses should also be examined.

The discussion up to this point has dealt with bridge superstructure. Of equal importance, however, are the components of the substructure. Timber pilings, if completely covered by concrete, cannot economically be examined. The best solution here is to find out from available records what sorts of piles were used. If no settlement or other foundation defects are evident, it can reasonably be assumed that the pilings are performing adequately.

If piles are exposed above ground or to water, they can be evaluated for damage or deterioration. Cribbed timber piers or abutments, if present, should be examined with particular care. These are exposed to severe abuse both from decay and from ice or debris in the stream.

Failure of timber piers or abutments usually occurs from deterioration rather than traffic loading. Generally, potential failure is not as evident as it would be in the superstructure. If untreated timber has been used for this part of the structure, periodic inspection is advisable. Any detected movement is a reliable indication that remedial repairs are necessary.

A general evaluation of the total structure should be made. Bracing systems for lateral loads must be secure and functional. An appraisal of the weathering, deterioration, damage, modifications, and general maintenance will aid in determining the safe useful load that can be carried. The superstructure should be securely anchored to its supports, and the components of the supports should be positively fastened together. The anchorage system should be capable of resisting lateral forces.

Consider the effects of a member or a joint failing, and whether other members will take over or will collapse. As in other structures, this is an important economic consideration. A value should be placed on a secondary or backup system if such exists. Generally, temperature stresses need not be considered for timber bridges. The effects of any known traffic accidents must be analyzed. Some members may have been so severely overstressed that they are weakened.

The roadway surface should be examined for proper drainage and the effects of wear. A timber deck is not affected by the application of salts during winter, but it is subject to abrasion by tire chains and tire studs. An asphalt wearing surface over a timber deck virtually eliminates abrasion and, further, seals the deck from moisture. This also provides protection for the supporting structure under the deck.

Application of a wearing surface must be evaluated for its increased loading effects, as mentioned previously. End rotation for individual members, such as a series of simple-span beams or stringers, will adversely affect the deck at the end joints and may require remedial work. Because bridges constructed of arches or trusses can develop a permanent sag, the centerline should be measured and checked later on. Alignment of members, particularly trusses, should be observed. This can indicate loosening of connections or yielding of supports. When vehicles are passing over the bridge, observations should be made of the various components and connections for any indication of loose joints, sway, or uneven twist that is not normal for that type of structure.

PRESENTATION OF THE EVALUATION

The evaluation of the structural adequacy of a timber bridge should be logically presented and well documented. Recommendations given to the owner should relate to periodic or special inspections, maintenance, repairs, or possible replacement. A final net load limit for gross vehicle weight or a combination of axle loads should be indicated. Normal design criteria (6) should be followed in evaluating the entire bridge and its use.

A person writing a bridge evaluation should cover the following.

1. Determine the strength properties of the wood members and connections. At this time assign basic values to the various timber members based on the wood species and estimated grade. These may be adjusted for the conditions present, such as weathering or decay. If critical members are found to be unacceptable, the bridge evaluation can terminate at this point.
2. Assess stream conditions for a river crossing. Foundations and substructure conditions may limit the useful purpose of the structure; evidence that might indicate flood waters of greater volume than permitted by the stream profile could limit the bridge's usefulness.
3. Acquire necessary information on present or anticipated traffic loads. If heavy loads are anticipated and the bridge is limited to lighter loads by some key members, further studies may not be warranted at this time. Alternate routes might be considered.
4. Make a checklist or table in order to record the condition of the various components in the field. Later, the structural analysis can be used to complete this list, which can serve as a structural evaluation of each member as it affects the load capacity of the bridge. Sometimes a relatively economical repair or modification can be made to a few members or connections to upgrade the entire structure. Checklists of this sort could also be made for both the superstructure and the substructure. These would normally include an identification of each component and connection, a description of its condition, and rated load-carrying capacity as it affects the bridge. Any needed repairs could be indicated.
5. Examine and record critical defects or deficiencies that limit the useful load or life of the structure. This includes such things as sag, bracing condition, decay, damage, guardrail conditions, departure from original plans, modifications, and other items of a critical nature. Roadway surface smoothness, although not a structural item, is important for riding quality and for its effects on impact forces.

One excellent guide has been prepared (6), although some of the discussion in it should probably be expanded or modified to fit the particular situation being considered. The manual departs in several places from the AASHTO specifications, which are intended for new constructions and provide for future increased loading, deterioration, and other conditions of long continuing use regarding permissible stresses.

The evaluation of bridges as presented here and in the manual tends to eliminate some of the unknowns. This can then permit somewhat higher stresses. The manual suggests that stresses at the operating rating be 75 percent of the material yield point. This is considered the absolute maximum stress level permitted. Next, the inventory rating or working stress level is assigned as 55 percent of yield point. These values do not apply to timber because there is no yield point that can be used; each grade of a species is limited by its particular growth characteristics.

In the examples shown in the manual, a stress level for wood is set at 133 percent of the basic strength adjusted for the condition of the member. This is a good place to start but should be used with caution. A safety factor against ultimate collapse is probably a better criterion. If, for example, the bearing of a beam is highly stressed in compression perpendicular to grain, failure is not likely, but, if a truss member has a tension splice that is highly stressed, complete collapse could occur. The manual states that it is sufficient to consider only one unit of maximum weight in each lane for spans up to 61 m (200 ft). This might be better evaluated by knowing the expected traffic during the projected life of the structure. Usually, timber bridge spans are well below 61 m, so this may not be a consideration.

This presentation is quite general and obviously cannot be used as a detailed method for the structural evaluation of timber bridges. Hopefully, some new ideas and a different look at how a timber bridge can be analyzed will occur to the reader.

REFERENCES

1. U.S. Forest Products Laboratory. Wood Handbook. U.S.D.A. Agriculture Handbook, U.S. Government Printing Office, Revised, 1972.
2. Standard Specifications for Highway Bridges. AASHO. Washington, DC, 1973.
3. A. D. Freas and M. L. Selbo. U.S. Government Printing Office, Forest Products Laboratory, Technical Bull. No. 1069, Feb. 1954.
4. Timber Construction Manual. American Institute of Timber Construction, 2nd Ed., 1974.
5. R. D. Graham, M. E. Corden, and T. C. Scheffer. Controlling Biological Deterioration of Wood With Volatile Chemicals. Forest Research Laboratory, Oregon State Univ., Corvallis, 1975.
6. Manual for Maintenance Inspection of Bridges. AASHTO, Washington, DC, 1974.

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Tests on Treatments for Reflective Cracking

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Eighteen test sections with a thin overlay either of 31.8 mm (1.25 in) of asphalt concrete or of 12.7 mm (0.5 in) of asphalt concrete finishing course were built in 1971 and 1972 to determine to what extent they prevented reflective cracking. Of these 18, 5 treatments were found to significantly reduce reflective cracking—heater scarification plus Petroset, asphalt rubber membrane interlayer, fiberglass, heater scarification plus Reclamite, and 200/300 penetration asphalt. Other performance aspects, such as roughness, rutting, deflection, and asphalt properties are reported, and costs in terms of construction and actual maintenance are given. Each treatment's failure or success is reviewed and considered before determining the conclusions and recommendations.

The primary object of any pavement design is not simply to provide a roadway of safe and desirable ride performance but also to extend these characteristics over a maximum useful life with minimum maintenance. However, because of the highly complex nature of flexible pavement structures, cracking, rutting, and other surface failures do occur and are influenced by environmental, traffic, and original design factors. Restoration to extend the useful life of deteriorating roadways typically involves the application of a thin asphalt overlay on the old pavement.

Historically, however, application of these thin overlays, which are generally of 10.2 cm (4 in) or less, results in a new and complex problem known as "reflective cracking" that is defined as the migration of a subsurface cracking pattern into and through the overlay. Once the overlay fractures, general erosion occurs and severely affects performance and requires further costly maintenance.

In an attempt to better understand the mechanism of reflective cracking and to pursue the development of new methods and materials to prevent it, a case study was conducted by the Arizona Department of Transportation, in conjunction with the federal National Experimental and Evaluation Program (NEEP) project on reducing reflective cracking in bituminous overlays (1). The NEEP project objective was to improve and develop materials, methods, and technologies to prevent or greatly minimize the occurrence of reflective cracks in overlays placed over previously cracked bituminous pavements.

This paper describes the Arizona test program, a case study of 18 selected roadway test sections, each evaluated by a carefully chosen set of parameters, materials, and application methods. The following is a summary of the test criteria and our results and recommendations (2).

Our preliminary activities involved extensively researching material and treatment, selecting and evaluating test site conditions, and finding an effective means for data accumulation and reduction. Eighteen individual roadway test sections were chosen to accommodate the range of desired test parameters. Beside each test section was a control section that served as a normalizing base for comparative measurement. This allowed engineers to observe and accumulate qualitative results from each test section, contrast them, and predict the influence of individual parameters. From these results, recommendations were made based on the effectiveness of crack prevention, cost, and other factors.

TEST PROGRAM

The test program was conducted on a 14.4-km (9-mile) section of highway (Minnetonka-East) near Winslow, Arizona, on I-40. Winslow is considered a high desert region, at an elevation of 1524 m (5000 ft), and has less than 20.3 cm (8 in) of rainfall annually. Temperature variations range from -18°C (0°F) during the winter to 38°C (100°F) during the summer. Minnetonka-East provided moderate to heavy average daily traffic (10 000 ADT), a fairly severe climate, and a history of extreme cracking problems. This section of highway was eligible for overlay during 1967 and was selected for use in the NEEP test program in 1970, the year the program was initiated.

Preparatory to designing the test, the nature and degree of distress were extensively evaluated. This involved core sampling, structural support testing, visual surveys, rut depth measurements, Benkelman beam tests, and traffic surveys. Rutting and Benkelman beam conditions are given below (1 mm = 0.039 in).

Test	Maximum	Minimum	Average
Rutting, mm	38.1	0.0	14.3
Benkelman beam, mm	1.9	0.05	0.9

Survey of the road conditions revealed extensive cracking, including block (flexural) and shrinkage (thermal) cracks. Spalling and rutting were also noted. The photographs in Figure 1 show the highway condition in 1969.

Federal participation was limited to an overlay thickness of 31.8 mm (1.25 in) of asphalt concrete (AC) and 12.7 mm (0.5 in) of asphalt concrete finishing course (ACFC) (3). Design engineers considered this thickness inadequate for the structural support necessary for long-term performance. However, as will be seen from the test conclusions, rather significant and impressive results were obtained with this relatively thin overlay.

The 18 test sections were unique in design, treatment, and materials used. The following table briefly describes each individual treatment by test section number (1 cm = 0.39 in).

Test Section Number	Description
1	Asphalt rubber plus precoated chips
2	Heater scarification plus Petroset
3	Asphalt rubber membrane interlayer placed over AC and under ACFC
4	Asphalt rubber membrane interlayer placed over AC and under ACFC
5	Asbestos fortified AC mix
6	No ACFC, 5 cm AC
7	Los Angeles Basin 120/150 penetration asphalt
8	Los Angeles Basin 40/50 penetration asphalt
9	Four corners 120/150 penetration asphalt
10	Los Angeles Basin 200/300 penetration asphalt
11	Emulsion-treated base in place of AC
12	Petromat placed under overlay
13	Fiberglass placed under overlay
14	Petroset flush of overlay before ACFC placed
15	Petroset placed in cracks
16	Reclamite placed in cracks

17	Reclamite flush with old AC
18A, B, C	Heater scarification of old AC plus Reclamite flush and varying AC overlay thickness
Controls	Conventional (standard) overlay

Although various test sections were opened to traffic as they were completed, construction was finished in June 1972 and the sections exposed to unrestricted traffic. It should be noted that in the 3½ years after the 1972 completion of overlaying, the highway has been subjected to loads equivalent to the first 9 years of original service. That is, 1975 ADT was 10 600 [i.e., the number of equivalent 80-kN (18 000 lbf) loads was 159 213] as compared to the 1958 ADT of 3342 (i.e., the number of 80-kN loads was 39 486).

Climatic variations were rather severe during the test period and rainfall was above average. Also, the test region has a freezing index of 700, which is quite high.

Figure 1. Typical roadway cracking on Minnetonka-East, February 1969.

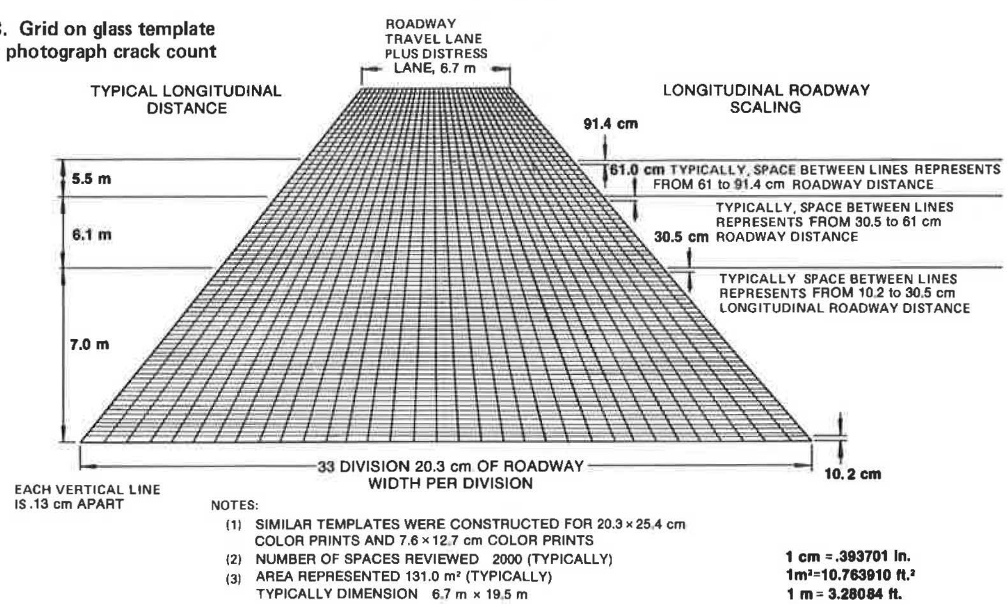
ANALYSIS

The Minnetonka project was designed to select the materials and treatments that significantly reduce reflective cracking. We therefore used a special photographic technique and an optical grid to accurately determine the extent and type of cracking before and after overlay. We used 35-mm color film to photograph 7.6-m (25-ft) highway panel sections from a mobile camera platform (Figure 2). Only the most severely cracked areas in the travel and distress lanes for each test section were photographed.

In addition, the highway was divided into 152.4-m

Figure 2. Mobile photography van.

Figure 3. Grid on glass template used for photograph crack count survey.



(500-ft) lengths, and one 7.6-m (25-ft) panel per 152.4-m length was randomly selected for photographing. Eighty-eight locations were photographed in March 1971.

The initial check of each print's quality led to the development of a glass template (Figure 3) to aid in our analysis. This template, designed to compensate for the distortion resulting from photographing at an oblique angle, divided each photo into several thousand parts.

We scanned each part line by line and coded each crack onto a computer form to indicate exact location. Coded forms were keypunched and processed by a special computer program that counted cracked and uncracked areas and computed the percentage of area cracked. The program also put each grid line into proper perspective by comparing the photos with the actual field cracking. We found that distortion did occur with distance, however, so we calculated the distance between each grid line up to the point where clarity was lost. Generally this point corresponded to a distance between grid lines of 0.9 m (3 ft) or more. As a result, 0.9 m was incorporated into the program as the logical end point after which no further grid lines would be counted for cracking.

The above procedure, although initially somewhat cumbersome because the cracked area was so large, proved efficient for measuring the magnitude of the original cracked area. It was also possible to differentiate to some degree between fatigue, or flexural cracking, and shrinkage cracking. Each photo location was photographed throughout 1975. The five photographs in Figure 4 provide the typical cracking history given below.

Photograph	Date	Cracking Before Overlay (%)	Cracking After Overlay (%)	Reflective Cracking After Overlay (%)
A	3/25/71	23.1	–	–
B	3/24/72	23.7	–	–
C	2/6/73	–	2.9	12.2
D	2/26/74	–	3.6	15.2
E	3/13/75	–	8.7	36.7

RESULTS

The percentage rankings presented in the table below are a true representation of cracking after overlay (1 mm = 0.039 in).

Test Section Number	Treatment	Percentage of Reflective Cracking Appearing by 1975
	31.8 mm AC overlay and 12.7 mm ACFC overlay and	
2	Heater scarification with Petroset	3
3 and 4	Asphalt rubber under ACFC	4
13	Fiberglass	5
18A	Heater scarification with Reclamite	6
10	200/300 penetration	8
12	Petromat	12
15	Petroset in cracks	12
5	Asbestos	13
7	120/150 penetration Los Angeles Basin	14
11	Emulsion-treated AC	14
17	Reclamite flush	15
14	Petroset flush	16
	Control sections	17
9	120/150 penetration four corners	18
16	Reclamite in cracks	20
8	40/50 penetration Los Angeles Basin	20
1	Rubberized asphalt seal coat	19
6	50.8 mm AC, no ACFC	64

The percentage of area cracked after overlay was divided by the percentage of area cracked before overlay. This section ranking, one of the most important parts of this study, clearly reveals those five treatments that were capable of significantly reducing reflective cracking when used in conjunction with an ACFC or other suitable open-textured surface. These percentages are particularly significant when a very thin overlay is used.

Generally, ridability is one of the key design criteria for both new pavements and rehabilitated old ones. Mays-Meter testing was performed before and after overlay treatment as shown below.

Test Section Number	Treatment	Percentage of Original Roughness
10	200/300 penetration	21
12	Petromat	26
13	Fiberglass	43
17	Reclamite flush	45
7	120/150 penetration Los Angeles Basin	48
9	120/150 penetration four corners	50
15	Petroset in cracks	50
–	Control section	57
14	Petroset flush	59
2	Heater scarification with Petroset	61
5	Asbestos	62
16	Reclamite in cracks	65
3	ACFC over rubberized seal coat	85
8	40/50 penetration	85
4	ACFC over rubberized seal coat	91
6	No ACFC	91
11	Emulsion-treated base	99
1	Rubberized seal coat	107

It was found that those sections constructed without ACFC (sections 1 and 6) or blade laid (section 11) gave the poorest performance. Test sections with ACFC over a chip seal (3 and 4) or with asphalt of a higher viscosity (8) performed slightly better. And, test sections with asphalt of lower viscosity (7, 9, and 10) or matting (12 and 13) performed the best.

We also found that basic asphalt properties influenced the reduction of reflective cracking more than any other property. The 400 kPa•s (4 million poise) at 25°C (77°F) viscosity [equivalent penetration about 45, absolute unaged viscosity of 0.3 kPa•s (3000 poise) at 60°C (140°F)] was critical to crack initiation. That is, the longer an asphalt can maintain a viscosity below 400 kPa•s, the less likely it is that reflective cracks will appear. Actual crack formation is triggered and intensified by cold temperatures. So once the asphalt reaches the 400-kPa•s level, it becomes highly susceptible to cracking. All system designs, then, should use asphalt of the lowest possible viscosity allowed by strength requirements and should use it in such a way that aging is retarded as much as possible.

CONCLUSION

This report, the culmination of over 4 years of careful planning, construction, and objective data analysis, provided much meaningful information that should be of value to federal, state, and local agencies restoring existing roadways and constructing new ones.

Our recommendations refer to overlays, in particular thin overlays of 10.2 cm (4 in) or less placed over badly cracked, rutted, or otherwise distorted bituminous pavements. Overlaying can also improve skid resistance and ridability. One should bear in mind, however, that no one treatment is a cure-all for bad roadway conditions. The following recommended crack-preventing treatments should rather be integrated into a total overlay design and carefully tailored to the nature of the distress.

Five treatments found to have significantly reduced reflective cracking are

Figure 4. Typical history of cracking on control section.

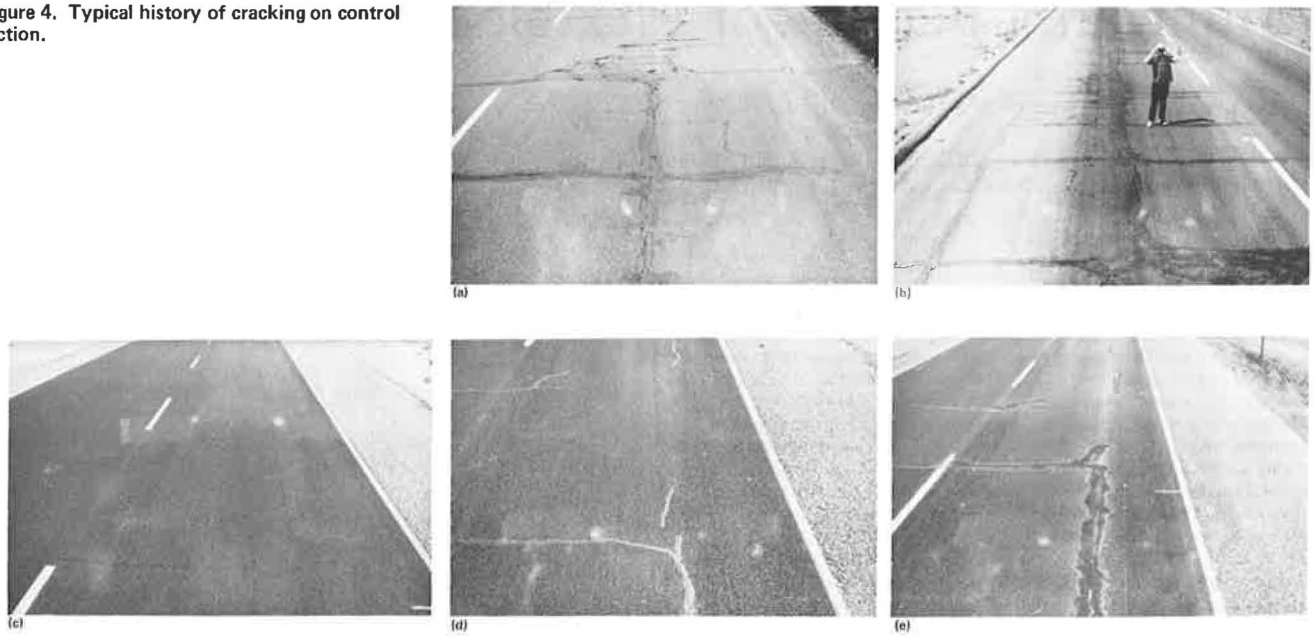


Table 1. Initial versus long-term costs of cracking treatments.

Treatment	Percentage of Reflective Cracking in 3 Years	Initial Cost per m²	Cumulative 3-Year Maintenance Cost per m²	Total Cost
50.8 mm AC, no ACFC	64	1.89	1.11	3.00
31.8 mm AC plus 12.7 mm open graded ACFC	17	1.87	0.75	2.62
31.8 mm AC plus 12.7 mm open graded ACFC plus treatment 200/300 penetration asphalt	8	1.87	0.14	2.00
Heater scarification plus Reclamite	6	2.28	0.10	2.38
Fiberglass	5	2.93	0.07	3.00
Asphalt rubber under ACFC	4	2.77	0.05	2.82
Heater scarification plus Petroset	3	2.28	0.05	2.33

Note: 1 mm = 0.039 in and 1 m² = 1.196 yd².

1. Heater scarification with Petroset,
2. Asphalt rubber membrane seal coat under ACFC,
3. Fiberglass membrane,
4. Heater scarification with Reclamite, and
5. 200/300 penetration asphalt.

As can be seen from the tables, some crack-preventing treatments compare quite favorably in price with cumulative maintenance cost figures. Application considerations are

1. One or more of the above treatments in combination should be used for all thin overlays of 10.2 cm (4 in) or less;
2. Heater scarification should always be to a depth of at least 19.1 mm (0.75 in);
3. AC asphalt of the lowest possible viscosity and the slowest aging characteristics should be used;
4. Applications using an asphalt rubber membrane seal coat under the AC or ACFC should be used with chips to provide direct transfer of vertical loads;
5. Fiberglass membrane material, although somewhat cumbersome to use during construction, could possibly be utilized during maintenance as a pre-overlay treatment on selected small areas;
6. Existing roadways being considered for overlay should be carefully investigated for possible stripping tendencies. Should stripping appear likely, efforts should be made either to give no structural value to the existing AC or to reconstruct the existing surface; and
7. Open-textured surfaces should be placed on top of densely graded overlays. This provides not only good skid resistance but improved appearance by hiding narrow reflective cracks.

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REFERENCES

1. Reducing Reflection Cracking in Bituminous Overlays. National Experimental and Evaluation Program, FHWA Circular Memorandum No. CMPB-16-70, May 12, 1970.
2. G. Way. Prevention of Reflective Cracking in Arizona, Minnetonka-East—A Case Study. Arizona Department of Transportation, Rept. No. 11, May 1976.
3. Additional Stage Construction on Pavement Structures Constructed or Authorized to be Constructed Prior to October 1963—1968 Interstate Cost Estimate. FHWA Circular Memorandum No. IM21-1-67, May 9, 1967.

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Milling and Planing of Flexible Pavement

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In 1975, a 38-mm (1.5-in) layer of asphalt concrete was removed from 12 km (7.5 miles) of four-lane divided pavement using three different methods: hot milling, cold milling, and hot planing. The site, NY-5 between Albany and Schenectady, is a major thoroughfare with an average annual daily traffic volume of 28 000 to 50 000 vehicles. Its curb and expensive color-contrasted median made removing and replacing the wearing course more economical than raising curbs and manholes and reconstructing the median. Air and noise pollution were monitored, and neither exceeded industrial or construction limits. Tests on the asphalt before and after removal showed virtually no effect on its properties, although the three machines had very different operating characteristics. Their effective removal widths ranged from 1.5 to 3.7 m (5 to 12 ft), the depths from 10 to the full 38 mm (0.4 to 1.5 in) in one pass, and the forward speeds from 3.1 to 12.2 m/min (10 to 40 ft/min). The net result was an effective removal rate—full depth per 10-h day—of 1505 to 5936 m^2 (1800 to 7100 yd^2). All three machines provided efficient means of removing old asphalt, but several factors must be considered before selecting any of the processes for a given location.

For years scarifying was the only method used to remove a specific thickness of flexible pavement in New York State. The result was irregularities in the surface that required a major truing-and-leveling course, which increased costs. In addition, scarification was used only on projects of less than 20 900 m^2 (25 000 yd^2); anything larger required a full-scale reconstruction program.

In the fall of 1974, the New York State Department of Transportation examined the possibility of removing flexible pavement by other means and then overlaying to the same grade. Three general techniques—planing with heat, milling with heat, and milling without heat—were found. Each is unique and has advantages and disadvantages. We decided, therefore, to compare them on a single project.

Early in 1975, personnel of the Engineering Research and Development Bureau and design engineers from Region 1 (the Albany area) agreed that 12 km (7.5 miles) of NY-5 (Central Avenue) between Albany and Schenectady could be used. This highway could no longer be overlaid because its drainage inlets were already well below the proper grade. Another 38 mm (1.5 in) of overlay would have made them deep enough to create a driving hazard. Curb heights would have been reduced to only a couple of inches, and the color-contrasted median would have to have been rebuilt.

Each of the three techniques was evaluated on the basis of (a) precision of cutting depth, (b) production rate, (c) air and noise pollution, and (d) depth of heat penetration. Skid tests were performed on the original pavement, on the planed or milled surfaces, and on the new overlay. The pavement was sampled both before and after each process and the samples physically analyzed for changes in penetration and viscosity. The possibility of reusing material was explored, and cores were extracted to examine the bond between the overlay and the milled or planed surface.

TEST SITE AND EQUIPMENT

The test site is a four-lane divided highway with a 3.7-m (12-ft) median. The average annual daily traffic (AADT) volume is 27 260 vehicles, although for the highway's busiest section, where there are several large shopping centers, AADT is 49 200. The 12-km (7.6-mile) section to be rehabilitated was divided into three areas of about 64 750 m^2 (77 460 yd^2); each process removed 38 mm (1.5 in) of existing pavement thickness (Figure 1).

Heater-Planer

The heater planer (Figure 2A), furnished by Jim Jackson of Little Rock, Arkansas, heats, planes, and cuts the existing surface; blades the cuttings into a windrow; and loads them into a truck. The remaining surface appears smooth but is abrasive in texture. The operating width of the machine is 2.4 m (8 ft) and the wheelbase 5.5 m (18 ft), with tandem rear driving wheels. It can cut 9.5 mm (0.4 in) deep in one pass, flush to all curbs, inlets, manholes, or other obstructions within the paved areas. Open flames fueled by liquid propane provide the heat. Two virtually identical heater-planers were used interchangeably throughout the test section.

Hot Miller

The hot miller (Figure 2B), furnished by the Wirtgen Corporation of Ridgefield, New Jersey, mills the flexible pavement surface after heating it with both open-flame and infrared burners fed by propane gas. It can mill 51 mm (2 in) deep and 3.6 m (12 ft) wide in one pass. The resulting surface has a waffled texture with striations no more than 9.5 mm (0.4 in) deep. This unit is not self-contained and requires a grader, an autoloader, and occasionally a front-end loader to remove the milled material.

Cold Miller

The cold miller (Figure 2C), furnished by the G. J. Payne Company of Carson, California, mills a flexible pavement surface to a depth of 51 mm (2 in) without heat. It has a 1.5-m (5-ft) wide drum for milling and blades for windrowing the milled material. The milled surface has a waffled texture with striations no more than 9.5 mm (0.4 in) deep. It is not self-contained and requires a payloader to remove loose material. Two millers, differing only in their dust control (water spray) systems, were used.

At the end of each work day, a full two-lane width of surfacing had to be removed to prevent drop between lanes in the same direction. Each operation was followed by a vacuum sweeper before the lane was opened to traffic. The planed or milled surface had to be ±6.35 mm (±0.25 in) of the design grade, have a 21-mm/m (0.25-in/ft) cross-slope, and could not be torn, gouged, shoved, broken or excessively grooved.

SAMPLING PROCEDURES

Pollution

Air and noise pollution were monitored before and during

Figure 1. Planing and milling locations (not to scale).

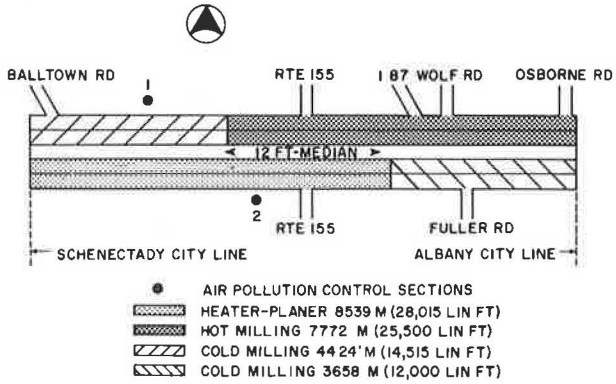
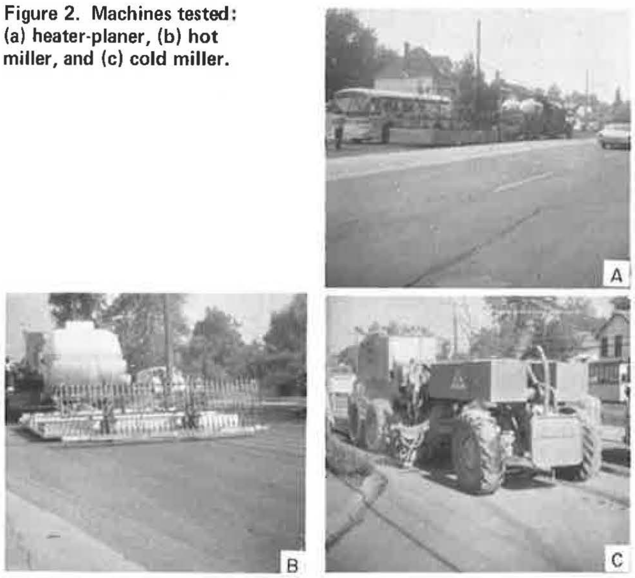


Figure 2. Machines tested: (a) heater-planer, (b) hot miller, and (c) cold miller.



milling and planing. Readings taken before the work began established normal levels for the area.

For air pollution, both particulate and hydrocarbon emissions were monitored. The particulate sampler was mounted on a hand cart and pulled alongside the machines. Filters from the sampler were taken to the laboratory, where particulates were determined in micrograms per cubic meter. Two types of hydrocarbon samples were used, one for readouts at the job site and the other for analysis at the laboratory.

The first was an infrared hydrocarbon analyzer that worked on the nondispensive infrared principle: Energy from an infrared source is absorbed by hydrocarbons. Direct readouts were recorded at the burner units, in front of the machine, and 4.6 m (15 ft) away. The second hydrocarbon collector was a glass cylinder containing activated charcoal, connected in series with a limiting orifice and pump (Figure 3). After collecting the sample, the cylinder was capped and returned to the laboratory, where the filters were analyzed and the results recorded in micrograms per liter.

Each machine's noise level was measured at five points—two approaching the noise meter, one at it, and two past it (Figure 4). The meter was positioned 1.4 m (4.5 ft) above the ground and recorded noise for an accumulated time of only 2 min in any hour in decibels on the A scale [dB(A)], the frequency response closest to that of the human ear (1, p. 2). In addition, a 20-min tape recorded for each machine from 45.7 m (150 ft) before the meter to 15.2 m (50 ft) past it and provided permanent records.

Heat Penetration

Temperatures were measured in the pavement below the machines that used heat. Thermocouples were inserted in the pavement and connected to an automatic recorder that produced a reading every 3 s.

PHYSICAL ANALYSES AND DISCUSSION

The pavement was sampled before and after planing and milling to determine if any oxidation had occurred. The samples were chemically analyzed in the laboratory to check penetration at 25°C (77°F) in accordance with AASHTO T 49 and ASTM D 5 and viscosity at 60 and 132°C (140 and 275°F) in accordance with AASHTO T 202 and AASHTO T 201, respectively.

Skid tests using the state's skid trailer were run before and after each process. In addition, production rate was monitored to establish daily productivity of each machine in square meters.

Air Pollution

Particulates

The normal particulate count along the route was done by New York State Department of Environmental Conservation personnel, who sampled air twice before work was in progress. Each sampling lasted 4 continuous hours on 4 consecutive days and showed particulate concentrations of 85 and 318 $\mu g/m^3$. Several samples, taken beside each device, gave recorded concentrations 5 to 10 times greater near the heating devices and several thousand times greater near the cold miller (Table 1). However, while these concentrations were extremely high within 1.5 m (5 ft) of these devices, they dropped markedly with increasing distance.

Because the particulates are relatively heavy, they do not remain suspended for any significant time. Thus, at 7.6 m (25 ft) from the machines, concentrations were well within any industrial limits. Also, the wide range of measured values for each machine was explained by prevailing winds and gusts, either natural or from passing vehicles.

In sum, large concentrations of solid particulates were found near the devices—particularly the cold miller—but they did not disperse to an objectionable degree.

Hydrocarbons

These were measured by two methods, charcoal sampling tubes and an infrared analyzer. Table 1 summarizes the hydrocarbon emissions, recorded by the first method, which were similar for all three machines. Results for the cold millers, however, are questionable, because asphalt particles may have entered the tubes and increased the readings. Substantiating this was the fact that no hydrocarbons were detected by the infrared analyzer. Table 2 shows that, although high hydrocarbon concentrations were found in the flame areas of the heating devices 4.6 m (15 ft) from the machines, concentrations were significantly lower; no hydrocarbons were detected from the cold miller at any distance. The intense heat is thought to have vaporized hydrocarbons at the road surface directly under the burners, but they would quickly revert to their liquid or solid states as soon as temperatures fell to ambient levels a meter or so away.

All machines were below the maximum allowable industrial limit of 133 000 $\mu g/m^3$ at any distance. The results show that hot planing creates least hydrocarbons and hot milling the most. The infrared analyzer, on the other hand, showed that neither produced any significant concentration and that cold-milling produced no detectable hydrocarbons.

Noise Pollution

While New York State has not established standard noise level specifications, tentative limits have been developed and are used here (2, p. 15). The highest recorded noise level, 94 dB(A), was produced at the meter by the hot miller's vacuum sweeper, a piece of equipment already in general use; its autoloader was next, at 92 dB(A). All other equipment ranged from 70 to 89 dB(A). In the tentative New York noise pollution specification, the maximum allowable noise level at 7.6 m (25 ft) would be 94 dB(A) for an accumulated time of 6 min during any 1 h. Thus, Table 3 shows that no milling or planing machine or any related equipment exceeded the proposed requirement.

Figure 3. Hydrocarbon collector with charcoal filters.

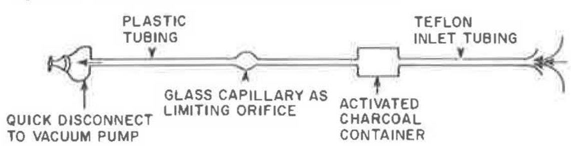
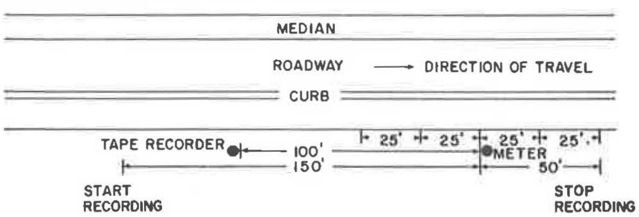


Figure 4. Noise pollution measured incrementally as planer and millers approached and passed meter and continuously by a tape recorder; meter and recorder set up 7.6 m (25 ft) from curb.



Heat Penetration

Little prior information existed on depth of heat penetration or any damage to existing pavement resulting from the use of these machines. This was investigated by using an automatic temperature recorder. Figure 5 shows general heat penetration with depth, and Table 4 gives general temperatures of the existing and removed surfaces. The heater-planer can be seen to have generated tremendous open-flame heat, creating surface temperatures over 176°C (350°F) that penetrated the pavement to a depth of only 6.35 to 9.5 mm (0.25 to 0.40 in).

This is the key to shaving or planing the pavement, because when a depth of 12.7 mm (0.5 in) was tried, the pavement began to shear in large chunks. Since the temperature at this depth was about 66°C (150°F), it would appear that below 76°C (170°F) the pavement is too cool to be planed or shaved. This also implies that the generally used minimum compactive temperature of 76°C is also the beginning temperature for effective removal by planing or shaving.

Figure 5 shows penetration under the hot miller to be similar to that under the planer, but, because of its milling action, the miller removed pavement regardless of temperature. While the hot miller produced surface temperatures of only 132°C (270°F), it penetrated the pavement as well as the planer did. The infrared heater and the miller's slower operating speed obviously led to greater penetration. The hot miller removed the full 38.1 mm (1.5 in) in one pass, so it was actually cold milling (based on 76°C) below the 12.7-mm depth.

Table 4 shows the heat retention of the removed material and indicates a large variation in temperatures of material being loaded into the trucks. Although the hot miller's heat penetrated the pavement more effectively, blending of cooler material from below 9.5 mm yielded cooler material—±32°C (90°F). The heater-planer loaded the heated material without blending, and temperatures were about 76°C (170°F).

Physical Analysis

Pavement samples taken before and after each process were analyzed in the laboratory for penetration and vis-

Table 1. Air pollution measurements.

Process	Particulates ($\mu g/m^3$)[a]	Charcoal Sampler for Hydrocarbons ($\mu g/L$)[a]	Process	Particulates ($\mu g/m^3$)[a]	Charcoal Sampler for Hydrocarbons ($\mu g/L$)[a]
Control	85 to 320	0.07	Cold miller 303[b]	23 560 to 152 000	0.20 to 0.48
Heater-planer	1 280 to 2 430	0.04 to 0.48	Cold miller 304[b]	60 780 to 450 000	0.22 to 0.46
Hot miller	1 950 to 2 950	0.10 to 0.90			

[a] Readings were taken within 1.5 m (5 ft) of each machine.
[b] Both millers, differing only in spray-bar capacity, were used in both cold-milling test areas.

Table 2. Hydrocarbon sampling by infrared analyzer.

Process	Probe Location	Approximate Concentrations ($\mu g/m^3$)[a] Number	Mean	Standard Deviation
Heater-planer	In flame area, unprotected	13	24 672	2 195
	In flame area, wrapped in asbestos	21	19 152	38 770
	About 4.6 m from machine	58	1 796	1 796
Hot miller	Attached to front of machine, open-flame burner on slightly	116	17 822	20 150
	Attached to front of machine, infrared burner on full	23	57 257	39 900
	1.5 m ahead of machine, infrared burner off	24	0	0
	1.5 m ahead of machine, infrared burner on	115	11 172	8 113

[a] Background readings before milling or planing for control purposes showed no detectable concentrations of gaseous hydrocarbons; maximum industrial standard allowable is 200 ppm.

cosity, with a view to recycling and the attendant need to determine material changes. Two test series, each including four samples, were performed for each of the three reconstruction processes. Table 5 gives the means for each test. No significant differences appeared at the 95 percent confidence level, using both the F- and the t-tests. Thus, the material was neither damaged nor oxidized by the burners or millers.

Skid Resistance

All surfaces were tested before and after each operation to determine any changes in skid resistance (Table 6). Only one value is given for the original surface, but it represents the entire length. All processes resulted in skid numbers much greater than 30, a value generally identifying unacceptable pavement.

Unfortunately, the milled and planed surfaces did be-

Table 3. Noise measurements.

Process	Noise Levels [dB(A)[a]] 15 m Before Meter	7.6 m Before Meter	At Meter	7.6 m Beyond Meter	15 m Beyond Meter
Heater-planer	70	76	82	76	80
Hot miller	78	80	81	80	85
Cold miller	78	83	86	87	86

Note: 1 m = 3.3 ft.

[a]Taken at 2-min maximum times, 7.6 m from the curb. The proposed maximum at that distance is 94 dB(A) for an accumulated time of 6 min during any 1 h.

Figure 5. Penetration of heat from pavement surface.

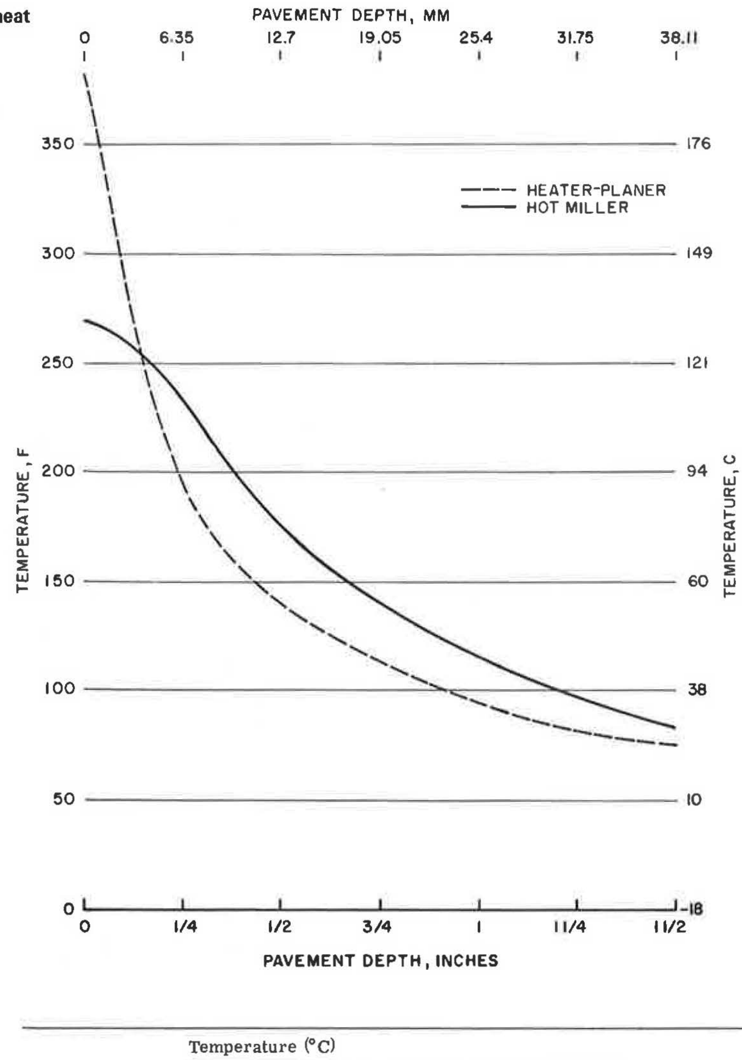


Table 4. Overall average temperatures.

Process	Temperature (°C) Surface After Burners but Before Planing or Milling	In Windrow	After Loading in Truck	Remaining Surface After Planing or Milling
Heater-planer	176[a]	76	76	49
Hot miller	132	54	32	32

Note: 1°C = (1°F - 32)/1.8.

[a]Maximum possible reading on thermometer.

come smoother with time. Although additional skid tests could not be scheduled, a similar condition was observed when transverse grooves were cut for drainage on a rural four-lane, divided flexible pavement having an AADT of 6000 vehicles. These grooves began closing in a few weeks and within 8 weeks were fully closed and smoothed out. Thus, it is doubtful that the milled or planed surfaces would be acceptable for any length of time, and resurfacing should follow milling or planing within 2 weeks.

PRODUCTION RATE AND GENERAL COSTS

After pavement removal, production rates were calculated for each of the three operations, in terms of square meters of pavement removed per 10-h day. It should be noted that hot milling was a one-machine operation, while hot planing and cold milling both involved two machines. Also, not all were always in use, so that working day figures in Table 7 are not necessarily half the machine day figures.

To remove the entire 38.1 mm (1.5 in) of material from the 7.3-m (24-ft) wide roadway in one direction, the hot miller required two passes, the cold miller five passes, and the heater-planer twelve passes. On the other hand, the hot miller progressed at an average speed of 3 m/min (10 ft/min), the cold miller at 4 m/min (13 ft/min), and the heater-planer at 12.2 m/min (40 ft/min), resulting in respective production rates of 5956, 1884, and 1500 m^2/machine day (7124, 2254, and 1794 yd^2/machine day).

Although the hot miller had the highest production rate, it required the most supplementary equipment. Directly behind it was a power grader to windrow the milled material, followed by an autoloader to place the

Table 5. Physical properties.

		Penetration at 25°C, 0.1 mm (100 g/5 s)		Viscosity at 60°C (Pa·s)		Viscosity at 135°C (m^2/s)	
Process	Time	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
Heater-planer	Before	41.0	8.9	783.7	2302	7.38	1.496
	After	41.0	10.7	1062.1	3781	8.43	0.616
Hot miller	Before	48.6	10.7	853.1	2209	8.64	0.982
	After	50.0	17.4	848.4	5734	7.70	1.354
Cold miller	Before	33.0	3.8	1331.3	3265	8.73	0.782
	After	32.0	6.3	1533.0	7036	9.06	1.653

Note: 1 g = 0.035 oz., 1 Pa·s = 0.021 lbf·s/ft², 1°C = (1°F - 32)/1.8, and 1 m²/s = 0.039 ft²/h.

Table 6. Skid-resistance measurements.

	Original Pavement			After Process			New Pavement Overlay		
Process	Number	Mean	Standard Deviation	Number	Mean	Standard Deviation	Number	Mean	Standard Deviation
Heater-planer	26	35.4	3.4	40	51.9	4.5	6	45.3	3.8
Hot miller				15	49.5	4.9	16	46.8	1.5
Cold miller				4	54.8	3.8	21	46.9	2.9

Table 7. Production rates.

Process	Avg. Machine Speed (m/min)	Cutting Width per Pass (m)	Avg. Cutting Depth per Pass (mm)	Total Working Days	Total Machine Days	Avg. Material Removed Daily (m^2)	Material Removed per Machine Day (m^2)	Total Material Removed 38 mm (m^2)
Heater-planer	12.2	2.4	9.5	23.3	43.9	2862	1500	65 831
Hot miller	3.0	3.7	38.1	10.1	10.1	5956	5956	60 153
Cold miller	4.0	1.5	38.1	20.5	33.9	2903	1884	63 883

Note: 1 m = 3.3 ft, and 1 mm = 0.039 in.

Table 8. Typical gradations.

	Sieve Size (mm)				Sieve Size (μm)				
Process	25.4	12.7	6.35	3.17	841	420	177	74	Pan
Hot miller	100.0	92.7	60.5	11.7	2.6	1.4	0.6	0.3	0.0
Cold miller	97.2	82.9	61.0	33.5	8.4	4.0	1.4	0.3	0.0

Note: 1 mm = 0.039 in, and 1 μm = 0.039 mil.

Table 9. Pavement overlay thickness.

	Core Sample of Overlay Thickness (mm)								
Process	1	2	3	4	5	6	7	Mean	Standard Deviation
Heater-planer	41.28	47.63	28.58	44.45	38.10	45.45	53.98	42.63	7.96
Hot miller	53.98	38.10	38.10	44.45	53.98	34.93	31.75	42.18	8.93
Cold miller	53.98	44.45	50.80	34.93	38.10	50.80	38.10	44.45	7.56

Note: 1 mm = 0.039 in.

material in trucks. Occasionally, a front-end loader removed material missed by the autoloader. Also, because of its size, this machine had limited maneuverability, which meant more hand removal around utility boxes, drainage inlets, and traffic-counter loops. Cold milling needed only a front-end loader to remove material and could mill in close quarters leaving very little for hand removal. The heater-planer was self-contained and required very little hand removal. These are important considerations in determining which machine to use in a particular situation. The hot miller is by far the most efficient, but its size may not be suitable for narrow city streets, and the equipment train must be considered at intersections where traffic cannot be effectively diverted for a long time.

Material Reuse

Both milling processes removed material in pieces ranging from 25.4-mm to 75-μm (1-in to no. 200) sieve. Table 8 summarizes gradations for both the cold- and the hot-milling processes. Such gradations could not be run on hot-planed material, which, on cooling, fused into large conglomerates. Before cooling, however, material from hot planing looked like well-graded 1A top and hot enough—79°C (175°F) or more—to transport to other locations to be placed, leveled, and somewhat compacted into a solid, firm surface course.

In one location, a parking lot of about 7432 m^2 (8888 yd^2) for sanitary waste trucks, this material was placed 101 mm to 152 mm (4 to 6 in) thick and leveled by a bulldozer. Although the surface was not rolled, it held together firmly and supported large loads of truck traffic.

A second location was a small trailer park, where the material was dumped and leveled by a small farm tractor with a plow. A third location was an automobile junkyard that used material in a muddy area from the heater-planer and from cold and hot milling. The milled material was placed in the same way as crushed stone in a muddy zone, and formed a strong, firm base. Although all the milling and planing material was used immediately, the milled material could have been stockpiled for later use.

Overlay Thickness

Seven cores were drilled at random from each milling or planing location after the new wearing course was placed. We examined these microscopically for thickness because of the difficulty of distinguishing between the new pavement and the older surface. The mean thickness for the hot miller and heater-planer (Table 9) was 42.4 mm (1.67 in) and for the cold miller 44.5 mm (1.75 in). The standard deviation for the processes was about 7.6 mm (0.3 in). Based on these cores, all processes removed more than the required 38.1 mm (1.5 in), but the mean depth stayed within a 6.4-mm (0.25-in) tolerance. In addition, as the need for a microscope shows, the bond between the overlay and the milled or planed surface was extremely tight.

SUMMARY AND CONCLUSIONS

The alternative to these milling and planing operations was full reconstruction—a costly, lengthy choice that would have inconvenienced motorists, nearby residents, and businesses. According to design estimates, completion would have taken about a year and a half and cost \$2 to \$2.5 million. This project, on the other hand, took only 47 d from the first milling and planing operations to completion of the new overlay. It cost less than \$900 000. Thus, from the points of view of both cost and convenience, this operation was successful. Our specific conclusions follow.

1. None of the operations exceeded any tentative or established air or noise pollution standards. All machines would be acceptable in residential areas.
2. The effective pavement softening temperature—76°C (170°F)—did not penetrate beyond the 9.5 mm (0.4 in) by the heater-planer, or beyond 12.7 mm (0.5 in) by the hot miller, which cold milled below that depth.
3. A great deal of heat and propane were lost or expended by the heater-planer's open flame. The hot miller used about half as much propane.
4. None of the operations oxidized or in any other way damaged the removed material; nor did they have any adverse effect on the remaining pavement.
5. The hot miller was the only one of the three machines capable of removing the entire 38.1 mm (1.5 in) of material for an entire lane width in one pass. The heater-planer had a maximum cut 9.5 mm (0.4 in) deep and 2.4 m (8 ft) wide, while the cold miller cut the full depth only 1.5 m (5 ft) wide.
6. The hot miller was less versatile than the other operations in terms of removing material from around obstructions and requiring more hand removal.
7. The heater-planer contained its own material pickup belt that required only haul trucks to follow it and consequently had the shortest lane closure behind it.
8. From randomly drilled cores, hot milling and hot planing were found to achieve a mean depth of 42.4 mm (1.67 in), and cold milling 44.5 mm (1.75 in). The design called for removal of 38.1 mm (1.5 in) by all three operations.
9. Planed or milled material can be reused. Hot-planed material can be transported and replaced within 40 min of removal to form a firm, fairly smooth riding surface. Hot- and cold-milled material served as an excellent base and may lend itself to recycling for resurfacing.
10. These processes have proved excellent where pavement must be removed in busy areas and cause minimum disruption while maintaining proper slope and grade.

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REFERENCES

1. Noise From Construction Equipment and Operations, Building Equipment, and Home Appliances. Office of Noise Abatement and Control, U.S. Environmental Protection Agency, Rept. No. NTID300.1, Dec. 31, 1971.
2. Construction Noise Survey. Bureau of Noise Control, New York State Department of Environmental Conservation, Rept. No. NC-P2(4/74), April 1974.

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Abridgment

Roadside Management in North Carolina

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North Carolina's climate favors the establishment and rapid growth of vegetation and is complemented by a topography that ranges from flatlands and swamps to rugged mountains and valleys interspersed with streams, rivers, and waterways. Across this terrain, North Carolina maintains the largest state highway system, approximately 120 968 km (75 021 miles), in this country. Maintaining the roadside of this vast highway network at an acceptable level for minimum costs is a formidable challenge. The potential for degrading our natural environment through inadequate, improper, or untimely roadside maintenance is great.

Recognizing this, the North Carolina Division of Highways adheres to a philosophy of roadside maintenance that attempts to maintain a highway facility in as near its original condition as age, normal deterioration, and changing traffic conditions will permit. We also improve those roadside areas where time and nature will assist in the enhancement of the facility.

In recent years, significant design changes have greatly improved the state's ability to maintain our roadsides.

1. Generally, flatter slopes are maintained near the travelway and the cut slopes of drainage channels. This will ultimately facilitate slide removal and maintenance of storm drainage systems.
2. Drainage berms across and down the backs of cut slopes have increased their stability and thus reduced maintenance.
3. Low-growing shrubs and plants at bridge ends reduce the effort required to keep these areas stable and presentable.
4. Detailed erosion control contract specifications and construction standards that include permanent and life-of-contract measures increase the stability of slopes, minimize obstruction of drainage structures caused by erosion, and consequently reduce related maintenance.
5. Ditches inside and outside the typical section are receiving appropriate treatment, such as paved ditches, jute mesh, and fiberglass roving, to minimize erosion. The combination of more stable slopes and ditches will greatly reduce or eliminate the need for back sloping and ditch pulling.
6. By leaving silt detention basins, ditch checks, and silt fences in place after a project is completed, the new project will stabilize itself in time and reduce or eliminate the possibility of damage caused to areas outside the rights-of-way. In the recent past, it was not uncommon for maintenance crews to be required to clean out drainage structures and remove eroded materials from adjacent property almost immediately after acceptance of a new project.

In spite of these and other similar design modifications, anticipated maintenance costs in North Carolina are increasing at an annual rate considerably in excess of anticipated revenue increases. These increases are projected on the basis of no additional maintenance personnel and in spite of a recent 14 percent reduction in our equipment fleet. The professional staff of the Division of Highways is not at this time advocating or anticipating any reduction in the level of services in the foreseeable future. However, it is obvious that the projected increases in the cost of maintenance operations and declining growth rates in revenues cannot continue indefinitely. North Carolina must take every measure possible to offset the spiraling cost of maintenance by better managing maintenance resources and continuing to incorporate into construction projects those features that will result in reduced future maintenance costs.

North Carolina is progressing steadily in the development of a maintenance management system that has reached the stage where planned work quantities and cost of an annual maintenance program by line item activity on both a county and a statewide basis can be reasonably projected. Work is proceeding toward developing the means and methods that will permit objective evaluation of the effectiveness of our efforts and will properly rank line item maintenance activities.

Roadside maintenance is a critical part of the maintenance management process, as indicated by the cost of the following activities in North Carolina.

Activity	Annual Cost ($000 000)
Maintenance of unpaved shoulders	9.7
Routine mowing	5.5
Manual and machine clearing of the roadway	3.5

Activity	Annual Cost ($000 000)
Ditch protection, seeding and mulching, top dressing, plant and plant maintenance, and control of vegetative growth by chemical application	1.6
Rest area and welcome centers	1.2
Litter pick-up, picnic tables, and litter cans	1.2
Repair of on-site and off-site damage caused by erosion	0.5
Approximate total	23.5

This total represents approximately 28 percent of the total estimated expenditure for fiscal year 1975 to 1976 (including resurfacing by state forces).

The maintenance of unpaved shoulders is included as a roadside or landscape item because we consider a rutted, unstable, low shoulder a displeasing sight as well as a safety hazard.

Unpaved shoulders are secondary to resurfacing in maintenance cost. It is significant that North Carolina expends more money on the former than on patching of paved travelways. Unpaved shoulder work function includes the cutting of high shoulder build-up, building up low shoulders, blading the cross slope to a uniform section, and pulling side ditches, including disposal of surplus materials. This is expensive principally because old facilities have substandard pavement widths and shoulders. The ideal solution would be for all projects to be designed with a full depth, full width paved shoulder with the slope of the base course extending to the ditch line or shoulder point, or both. However, from an economic, design, and construction program standpoint, this is not practical for all projects.

In design and construction, a paved shoulder with a width of 1.2 m (4 ft) and with a pavement structure less than that of the travelway is considered the minimum. Past experience indicates that the problems of low and high shoulders are confined to a distance of not more than 0.6 m (2 ft) from the edge of the travelway. After spot paving some 0.6-m wide shoulders on high traffic volume facilities, we discovered that, if an additional full depth pavement width 0.3 to 0.6 m (1 to 2 ft) from the edge of the travelway were given, most of our low shoulder problems would be solved. Furthermore, we felt that this narrower, full depth design would eliminate the need to continue using present color contrast and rumble effect on 1.2-m paved shoulders and would also eliminate replacement problems during future resurfacing operations. This is under study by our design section.

Assigning maintenance activities their relative priorities is a difficult task, as evidenced by experiences during the 1975 mowing season, when we instituted a new mowing procedure. We delayed the first mowing until seed heads were developed and reduced the mowed area, principally along the primary routes. Excessive delays, however, coupled with an extremely favorable early growing season, put the system literally "waist high" in grass and weeds. Public reaction was extreme.

In response, the professional staff made two basic mistakes: drastically raising the level of service over too short a time frame and advertising what we were going to do. However, the end result proved more beneficial than harmful. The public accepted the reduction of mowed areas and 15-45 cm (6-18 in) cut height. Mowing and machine clearing operations are now limited to within 12 m (40 ft) of the travelway except in areas of critical sight distance and where development dictates otherwise.

Maintenance forces have also begun herbicide treatment in areas around and underneath guardrails. This program has been very successful in eliminating unsightly growth and the need for hand labor. However, much care must be exercised in application. Environmental considerations could jeopardize the use of herbicides in the future. This could be offset if designers would place guardrails on pavement, base, or sterilized areas.

It is not an uncommon practice for states to employ meandering mowing patterns, which reduce the mowed areas and thus costs. North Carolina's experience, in most cases, has indicated that varying from straight line patterns adversely affects the cost of mowing. The deadheading time required to cut out the curved areas offsets the advantages gained by reducing the mowed area. This is substantiated by the fact that while mowed areas are reduced, equipment and personnel have remained constant.

A review of our equipment and the personnel required to maintain the previously mentioned horizontal and vertical limits that has just been completed indicates the need for one regular mower and one operator for each 161 to 242 adjusted kilometers (100 to 150 miles) of roadway (adjusted kilometers being computed on the basis of six times Interstate and other multilane divided road kilometers, plus two times the undivided road kilometers, plus one times the secondary road kilometers). The range of 161 to 242 adjusted kilometers takes into account varying growing seasons and roadway typical sections from the coastal areas to the mountains. Contour mowers and operators are assigned to the field divisions on the basis of one mower per 564 to 806 adjusted kilometers (350 to 500 miles).

Looking to the future, the present trend for new construction seems to indicate that the great increases in mowing areas of the past 20 years will not be a problem. Future plans seem headed in the direction of modifying existing facilities rather than building new ones. However, without any increase in area, continuing inflationary increased mowing costs are anticipated unless some substantial changes are made.

Some states have explored the possibility of contracting their mowing out to competitive bidders to lower overall cost. The multiple duties of mower operators (truck drivers, for instance, during the off season) and the possible poor response time of a contractor make this approach of doubtful benefit for North Carolina.

During the months after we reduced mowed areas, we received many requests to harvest the hay on the right-of-way. However, these requests were rejected based on questionable legal authority to allow it, on traffic control and safety problems, plus the fact that such harvesting would be contrary to our original plan of regenerating these areas and beautifying the roadside.

In the maintenance unit, it is believed that the ultimate salvation in mowing rests on the development of an economical low-growing grass. Research being done jointly in Kentucky by the New Jersey Department of Transportation and Cornell University is much needed. The North Carolina public would probably accept an uncut grass height of 20.5 to 25.64 cm (8 to 10 in), and, if such a low-growing grass were established on the primary system, at present prices, we could realize a direct maintenance cost reduction of approximately $50 million over a 20-year period.

We are presently maintaining 5 welcome centers and 44 rest areas at a cost of approximately $1.2 million annually. Two additional welcome centers are currently under construction, and future plans include 10 additional rest area sites.

Maintenance of these facilities includes the buildings, grounds, drives and parking areas, utilities, and the furnishing of supplies. They remain open to the public 24 h a day, 7 d a week. Uniformed custodians are on duty at welcome centers continuously, and at rest areas

16 h a day, 7 d a week. Personnel needed requires five custodians at each welcome center and three custodians per pair of rest areas. These personnel are in an annual salary range of $7000 to $8700.

The duties of custodians include making minor repairs to buildings, general upkeep of the grounds, contact with the public, and the sampling and testing of sewage treatment effluent and water supplies. Regular maintenance forces are responsible for pavement maintenance and some mowing operations.

Over the last 4 years, attempts have been made to reduce staff. However, public demand coupled with excessive vandalism necessitates continued staffing at the current level. The cost of maintenance of rest areas and welcome centers is more or less fixed. There is very little that improved management can do to reduce the cost of this operation. Present trends indicate that the future will require considerably more service in this area. Current expenditures for this maintenance activity could well double over the next few years.

It is difficult to believe that state departments of transportation will continue to set aside these valuable land areas or continue to use them in a way that will require additional expenditures of public funds. Some believe lease agreements could render the same service to the traveling public and also serve as a source of revenue rather than expenditure. In North Carolina, however, there is some question as to whether state highway rights-of-way may be leased, because of the wording of the documents by which the original rights-of-way were acquired.

The maintenance of a relatively litter-free 120 968-km (75 000-mile) system is a formidable and expensive task that consists basically of

1. Routinely scheduled litter pickup by state maintenance forces,
2. Utilization of approximately 250 litter pitch-in cans located on the primary system,
3. Cooperative effort with some county governments permitting the utilization of dumpsters on highway rights-of-way,
4. Enforcement of a general statute prohibiting the littering of public roads, and
5. Cooperative efforts between the Division of Highways and local civic organizations.

State litter crews, generally scheduled to pick up before each mowing cycle, average two laborers using picks and bags and a small dump truck. This may be considered an obsolete means of litter control, but using prison inmates at a cost of $1 a day influences our management. By agreement with the North Carolina Department of Correction, approximately 1000 inmates are furnished daily to the Division of Highways for hand work. During the last reporting year, approximately 5700 inmate days and 10 500 free labor man days were utilized in litter pickup.

The maintenance unit has made numerous studies of the use of litter pickup machines with no success. The major problems appear to be (a) a very high percentage of grass pickup (50 to 90 percent), (b) failure to pick up flattened cans, and (c) frequent breakdowns.

In June of 1972, the North Carolina Beer Wholesalers and U.S. Brewers Association contributed $38 000 toward the purchase of 250 pitch-in cans to be installed across the state. The Division of Highways prepared the sites and installed the containers in our most troublesome litter areas. Observation indicates this has been a very successful program.

In 1973, the North Carolina General Assembly enacted a bill that permits municipalities and counties to place garbage containers on the state highway system rights-of-way. Placement is subject to the approval of the Board of Transportation and is not allowed on fully controlled access facilities. The Division of Highways prepares approved sites on a reimbursable basis, and the municipality, or county, is totally responsible for the maintenance of the containers. Under this program, 150 dumpster-type containers are located throughout 11 counties. This has contributed to a considerable reduction in household garbage being placed on highway rights-of-way in these areas at virtually no expense to the Division of Highways.

In 1971 the North Carolina General Assembly amended an existing statute to make littering of highway rights-of-way a misdemeanor punishable by a fine of not less than $10 or more than $200. After the amendment, the Division of Highways placed appropriate signs throughout the state. In calendar year 1975, 712 convictions were obtained out of 808 arrests; 346 written warnings were issued. The number of annual arrests and convictions has been very constant.

In 1971, before the development of our maintenance data-collection system and based on random sampling techniques, cost of litter pickup was estimated at $2.4 million. During fiscal year 1975 to 1976, the documented cost was $1.2 million. Although costs may not have fallen by 50 percent, it is reasonable to assume that costs are substantially lower for maintaining the same or a slightly better level of service.

In 1973, the North Carolina General Assembly enacted the Sedimentation Control Act of North Carolina. This act established a sedimentation commission, which was given authority to establish rules and regulations governing all land-disturbing activities on one or more contiguous acres. These provisions apply to current highway construction and maintenance activities by state forces. In addition, the regulations require that all previously disturbed land areas that contribute to accelerated erosion and sedimentation be corrected.

This statute, together with subsequent rules and regulations, has had a profound effect on the management of roadside maintenance activities. As examples, all supervisory maintenance personnel must receive training in the latest acceptable methods and techniques in erosion and sedimentation control as may be required in such routine activities as maintenance of unpaved shoulders, blading unpaved roads, pulling ditches, and flattening fill slopes. The methods and techniques include construction of silt detention basins and silt check dams, ditch treatment and brush barriers, and seed-bed preparation and seeding. In order to implement these measures, scheduling of activities was re-examined to provide timely erosion control measures.

In compliance with the act, a 1974 statewide survey of each road was made to determine the extent of existing uncovered land areas on which there was accelerated erosion, and those areas that may be contributing to off-site sedimentation damage. This survey resulted in the development of a corrective program estimated at $5 million. This program required flattening of cut slopes, ditch treatments (fiberglass roving, jute mesh, rip-rap, and so on), construction of silt detention basins, seed-bed preparation and seeding and mulching—with all of this work being carried out by state forces.

The Division of Highways proposed completion of this program within a 5-year span, if funds are available. In the absence of any additional funds appropriated specifically for this purpose, approximately 29 percent of this work has been completed. This program has been planned to completion, although obviously the time must be extended beyond the originally estimated 5 years.

Maintenance of North Carolina's 120 968-km system is carried out by fewer than 7000 permanent employees, who include district administrative and supervisory personnel. The only major maintenance activity contracted out is plant-mixed asphalt resurfacing. With their rather limited resources, maintenance forces are accomplishing a reasonably productive, efficient operation on this vast highway system.

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Abridgment

Wildlife Considerations in Managing Highway Rights-of-Way

Douglas L. Smith, Environmental Design and Control Division, Federal Highway Administration

Over the years there has been an increasing public awareness and concern for our natural environment and a demand that we manage our public lands to ensure their protection and wise utilization.

Highways are often attacked as destroyers of wildlife habitat and detrimental to wildlife populations. Although this is sometimes true, highways often provide a wildlife habitat better than that before the highway was built. In many agricultural states, highways often provide the only habitat for many miles, because the surrounding land is under cultivation.

Few people are aware of the great challenge and responsibility state highway departments have in managing highway rights-of-way. There are over 4.9 million km (3.1 million miles) of rural U.S. highways (1). The Interstate highway system alone accounts for more than 67 500 km (42 000 miles) (1) and each kilometer (0.62 mile) of Interstate utilizes up to 12 hm^2 (30 acres) (2). It has been estimated that the soil and planted portions of highway, railroad, and utility rights-of-way embrace some 20 million hm^2 (50 million acres) of the contiguous United States (3).

Highway rights-of-way are unique compared to other intensively managed land areas, such as parks, forest, and wildlife refuges, most of which are in large single blocks. Highways, however, are long, narrow ribbons. This configuration has both advantages and disadvantages for wildlife management.

The major disadvantages are primarily problems associated with managing these large tracts of land as one unit. For example, to manage a 260-hm^2 (640-acre) wildlife refuge, one would seldom have to travel more than 3 km (2 miles) to reach any one spot within the area. To manage the same amount of highway, an engineer would have to travel at least 21 km (13 miles) to get from one end to the other.

The extensiveness of the highway system is also an advantage. Highways traverse areas of very diverse land use, such as intense agriculture, industry, and city. Because of this, there is increased potential for these areas to support wildlife populations. Highways preserve habitat in urban areas and are often the only large green spaces around. In agricultural areas, highways provide habitat diversity to land devoted primarily to monocultures.

In addition to preserving habitat, highway rights-of-way often create a boundary or transition, called an "edge," between plant communities. Edges increase plant species diversity and often create habitat conditions that were missing before the highway was constructed.

All animals require food, shelter, and water to survive. Most plant communities supply these requirements to at least a few species. Often, however, the number of species or the total number of animals that can be supported by a particular community is low because one or more of these requirements is limited. Highway rights-of-way, in creating an edge, often increase the ability of the area to support a larger and more varied wildlife community; they often supply a requirement for a species that had previously been limited or missing.

In addition to providing habitat for species indigenous to an area, highways may also be responsible for a species extending its range. The kangaroo rat has apparently increased its range northward across the Columbia and Snake rivers via highway and railroad bridges (4). In Illinois, the meadow vale is extending its range southward in response to the state's reduced mowing practices along the Interstate highway system.

Vegetation is not the only aspect of highway rights-of-way that can benefit wildlife. Many species, especially birds, have taken advantage of the various structures associated with highways. Cliff and barn swallows build nests under bridges and in culverts (5, 6). The cave swallow was once considered threatened, because it was thought to be restricted to nesting in sinkholes and caves. Recently, however, they were also discovered nesting in culverts (6).

These few examples demonstrate the great potential highway rights-of-way have for preserving or enhancing wildlife habitat. In order to fully realize this potential, however, highways need to be properly managed. In most cases, this will not require any significant increase in effort or expense on the part of the highway agencies. In some instances, proper wildlife management may result in an overall reduction in highway maintenance expense and effort.

The question now arises of how we can optimize the wildlife potential of highway rights-of-way while providing a safe and pleasant driving experience for the motorist.

The presence of birds and small mammals in rights-of-way is not a significant safety hazard. Collisions, however, with large animals such as deer cause extensive property damage and even human fatalities. Proper wildlife management includes managing against unwanted species. When a highway passes through an area with a high potential for collisions between large animals and motor vehicles, management must focus on reducing collisions.

At the present time, the only practical method for keeping deer and other large animals off the highway is the use of 2.4-m (8-ft) fencing. In many of the western states, deer migrate from summer to winter ranges. Highways often cut through their migration routes, creating a safety hazard. In these situations, if deer-proof fencing is to be used, overpasses or underpasses or both should also be provided to allow the deer to cross safely. In areas that support large deer populations, care should be taken to avoid planting species that may attract these animals to the highway.

The Federal Highway Administration, in cooperation with several state highway agencies and other state and federal agencies, is sponsoring, through the Federally Coordinated Program of Highway Research and Development (FCP), several studies investigating methods for reducing collisions between large animals and motor vehicles. There have been many recommendations for optimizing wildlife habitat, and several are just as appropriate for managing against unwanted species or for any other management objectives.

One recommendation is to develop a close working relationship with state wildlife agencies, which most state highway agencies have already done. Often, however, the agencies only work together on the initial planning and construction of a highway. Coordination and cooperation should extend to the operation and maintenance of the highway.

The maintenance engineer and the wildlife biologist should work closely with one another. The biologist can help the engineer realize wildlife values along rights-of-way, while the biologist will gain a better appreciation of the problems associated with highway operation and maintenance.

Another recommendation is to develop a management plan as early as possible and to implement it during design, construction, and maintenance. The development of a management plan for existing highways is as important as plans for new construction.

A rights-of-way management plan must establish and address many different goals, of which wildlife management along the highway is only one. The wildlife management portion of a rights-of-way plan must be integrated with other goals or objectives such as safety, aesthetics, and highway compatibility with surrounding land use.

When developing a wildlife management plan, the highway engineer must also consider such things as the surrounding ecosystem, the species present, and the plant community within the right-of-way and its effect on the system. In addition, consideration must be given to how the highway facility impacts the system, and how potentially hazardous large animals are. After addressing these basic questions, a decision can then be made concerning how the right-of-way can be managed to benefit specific species or certain types of wildlife.

These recommendations are steps that should be incorporated into the general highway planning and operational program of a state highway agency. The following recommendations are more specific ways of enhancing habitats of birds and small mammals and can be easily applied to current highway programs.

Rescheduling and reduction of rights-of-way mowing are probably the easiest solutions to implement and cost the least. The area immediately adjacent to the highway needs to be cut for safety, but this mowing can be reduced in most situations to once a year. Outside of this safety area, there is little need even for yearly mowing.

The reduction of mowing not only provides better habitat for certain wildlife species, but it can also reduce maintenance costs and be aesthetically more pleasing than a heavily mowed section of highway. The reduction in mowing will also encourage wildflowers to colonize the right-of-way, thus providing an attractive and pleasant driving experience.

In areas where mowing is necessary, it should be scheduled to avoid the nesting season for ground-nesting birds, which usually occurs between early spring and mid-July (7). Studies have shown that numerous bird species such as ducks and ring-necked pheasants will move into the right-of-way soon after mowing has stopped (8).

If it is necessary to mow the entire right-of-way, one should try to allow woody vegetation to establish itself around the right-of-way fence. This is especially valuable in agricultural areas where the fence rows, even though not very wide, provide excellent habitat for small animals.

Wildlife habitat can also be enhanced by planting species that will provide food and shelter. In developing a landscaping plan, try to select plant species that will both meet the landscape objectives and benefit wildlife. Many shrub and tree species beneficial to wildlife are also good landscape plants.

In addition to proper selection of plant species, thought must be given to the planting design. Shrub species used for wildlife benefit are often most effective when planted in groups. This type of arrangement makes them an effective cover and a good food source.

In selecting plants to be used along highway rights-of-way, it is advisable to have the help of a landscape architect, a botanist, or a plant ecologist, as well as a wildlife biologist. It is impossible to give a list of plants that will suit all situations. This is where close coordination between the highway agency and the state wildlife agency can pay off.

Another recommendation concerns the development of borrow pits, byproducts of highway construction, that have long been considered a necessary evil. Recently, however, some of their potential value has been recognized. The state of Nebraska has developed many of the their borrow pits along the Platte River into recreation sites. Borrow pits can also be of great benefit for wildlife. Given adequate soil and other environmental conditions, they can be turned into excellent aquatic and wetland habitats that can support a wide variety of fish and wildlife. The final contouring of the borrow pit to maximize its value to wildlife should add little or no cost to a highway construction project.

These examples are only a few of the many ways in which the wildlife value of highway rights-of-way can be enhanced without adding to highway construction or maintenance costs; in many instances costs can even be reduced.

Proper rights-of-way management, including management for wildlife, requires a carefully prepared management plan developed on an area-by-area basis. What is right for one section of highway may be completely wrong for another. Consideration of wildlife in managing highway rights-of-way requires these basics: an awareness of and an appreciation for the roadside ecosystem, a set of management goals or objectives, and a good working relationship between the highway engineer and the wildlife biologist.

REFERENCES

1. Highway Statistics—1974. U.S. Department of Transportation, Government Printing Office, 1974.
2. The Fifth Annual Report of the Council on Environmental Quality. Government Printing Office, 1974.
3. F. E. Egler and S. R. Foote. The Plight of the "Right-of-Way" Domain—Victim of Vandalism. Fu-

ture Median Services, Mt. Kisco, NY, 1975.
4. H. E. Broadbooks. Northward Extension of the Kangaroo Rat in Washington. Journal of Mammalogy, Vol. 39, No. 2, 1958, pp. 229-301.
5. J. T. Emlen, Jr. Territory, Nest Building, and Pair Formation in the Cliff Swallow. The Auk, Vol. 71, No. 1, 1954, pp. 16-35.
6. R. F. Martin. Synotopic Culvert Nesting of Cave and Barn Swallows in Texas. The Auk, Vol. 91, No. 4, 1974, pp. 776-782.
7. C. Trautman, R. Dahlgren, and J. Sebbert. Pheasant Nesting. South Dakota Conservation Digest, Vol. 26, No. 1, 1959, pp. 18-21.
8. R. B. Oetting and F. Cassel, Waterfowl Nesting on Interstate Highway Right-of-Way in North Dakota. Journal of Wildlife Management, Vol. 35, No. 4, 1971, pp. 774-781.

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Abridgment

South Dakota's Harvesting of Crops in Highway Rights-of-Way

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South Dakota has 40 470 hm^2 (100 000 acres) of roadside rights-of-way in vegetation, approximately 5.1 hm^2 per km (20 acres per mile) of Interstate highway road ditch, and 2.5 hm^2 per km (10 acres per mile) of primary and secondary highway road ditch. Since the 1940s we have allowed ditch mowing by the abutting owner or other interested party for no charge, and crop removal was satisfactory during the early years.

In the early 1960s, after Interstate roadside vegetation developed, we tried letting out mowing contracts in 8.1-km (5-mile) sections. This was not successful because feed was abundant elsewhere and contract administration was somewhat of a nightmare. We continued this policy through the 1960s and early 1970s and allowed abutting owners or others to mow on a first come, first served basis. No permit was required, and the unit foreman usually gave oral permission. The system worked satisfactorily with a minimum of complaints from the public and problems for the highway division.

Then came 1973 and dry weather. Pastures began drying up, which considerably heightened competition for mowing highway rights-of-way. The highway division attempted to administer an equitable system by requiring written permits to mow Interstate rights-of-way, but used oral permission for primary and secondary systems. Still, complaints came in from agricultural people about their not being able to obtain a mowing permit. Then the game people began encouraging nonmowing of road ditches or at least restricting permit dates until after the pheasant hatch.

Complaints to state legislators from rural land owners about alleged discrimination in issuance of permits brought others into the problem in 1973 and 1974. The result was a 1975 statute authorizing the Department of Transportation to establish roadside mowing regulations, after which the department held 1975 spring hearings to obtain public testimony concerning private mowing regulations.

Considerable testimony was taken, mostly concerning proposed mowing dates between June 15 and September 1. The agriculture people sought earlier and later dates, while the game people wanted only a short period in late July and early August.

The game people in South Dakota are convinced that highway rights-of-way should be a wildlife nursery and sanctuary.

In April 1976 the Department of Transportation board adopted the following rules and regulations in which an owner is the person or persons entitled to the possession of real property abutting a state trunk or Interstate highway; abutting means any land that adjoins the state trunk or Interstate highway system; rights-of-way include state trunk and Interstate highway systems being maintained by the Division of Highways; and division refers to the South Dakota Department of Transportation, Division of Highways.

1. Mowing permits—by whom issued. No person shall mow and remove any grass from the rights-of-way unless such persons shall first have been issued a permit by the district engineer or his authorized representative.
2. Form of permit, application, fee. The Office of Maintenance of the division shall prepare the application for the permit as to form and content, and there shall be no fee for the permit.
3. Reservation of right to issue permits. The division reserves the right not to issue permits for mowing on any or all portions of the rights-of-way.
4. Application for permit by nonlandowner. If a nonlandowner makes application for a permit, such application must be accompanied by a waiver signed by the landowner.
5. Commencement of mowing. No mowing of rights-of-way may commence west of the Missouri River prior to June 15, and no mowing may commence east of the Missouri River prior to July 10, and all mowing must be completed by September 1 of each year.
6. Mowing of newly constructed right-of-way. Mowing of newly constructed sections of highway will not be allowed for a period of 3 years or until the grass has become permanently established.
7. Liability insurance. Any person mowing within the rights-of-way must carry liability insurance in the minimum amount of \$50 000 property damage and \$100 000 in personal liability.
8. Area of rights-of-way that may be mowed. The area of the highway rights-of-way that may be mowed will be limited to the following: (a) mowing up to the edge of roadway shoulder will be allowed; (b) mowing the median of divided highways is prohibited; and (c) mowing of the areas inside interchanges will be allowed provided

access to the area is made by other than the main highway.

9. Manner of mowing. All mowing must be done in a workmanlike manner and the area left in a neat condition upon completion of work.

10. When hay must be removed. All hay must be removed from the rights-of-way within 30 d after being processed; any hay not removed within the time limits or in the manner prescribed by this section may be removed by the division.

11. Access to work area. Methods of obtaining access to work area of highway right-of-way are (a) access to work area on Interstate and controlled access highways is limited to using gates provided in the right-of-way fence, and if no gate exists one may be installed by the permittee and becomes the property of the state; (b) under no condition will it be permissible to enter or leave the work area through use of the main highway; and (c) the division will not be responsible for providing access roads outside the right-of-way line.

12. Parking of haying equipment. When haying equipment is not in use it must be parked near the right-of-way line.

13. Liability of permittee. The following shall constitute the instances of liability of the permittee: (a) the permittee shall be held responsible for any damage to fences, signs, landscape planting, or other highway features resulting from his or her mowing and haying operations; (b) the permittee shall hold the division, its officers, or employees harmless from any claims or actions brought by any person against the division, its officers, or employees as a result of the negligence of the committee or his or her agents or employees.

Mowing in violation of these regulations is a misdemeanor, which, upon conviction, carries a maximum penalty of a $500 fine or one year in jail or both.

Because of an extremely dry spring in 1976 the Department of Transportation did temporarily amend the starting date for mowing east of the Missouri River to June 15. In recent years private mowing complaints were minimal, in part because of the decision to issue permits only to the abutting landowner.

The maintenance foreman and superintendents were kept busy issuing permits, keeping track of permitted sections, and observing mowing operations for violations. Some violations were observed, and enforcement is a definite problem. There is a decided reluctance to file violation warrants with law enforcement people.

Removal of the harvested crops has always been a problem, and, while these regulations and the permit appear to have helped this problem, it has by no means been eliminated.

Some of the benefits from our private mowing policy follow. Less mowing is necessary by our own maintenance forces. Much litter is removed from the rights-of-way, and this helps to keep drainage areas cleaned out. Public relations with those who live along the highway are also better, because they are not charged for the hay they get in our ditches.

In many cases noxious weeds are cut down before they go to seed, and sight distances are improved, which reduces safety hazards for the traveling public. Many people feel that the ditches look much better when they are mowed out.

Some of the disadvantages and problems include erosion started by spinning equipment wheels on the inslopes and backslopes. There is also a constant problem with getting the private harvesters to remove their hay from the rights-of-way within the specified time limit. Furthermore, some operators do a very poor job of mowing and leave an unsightly mess.

Maintenance personnel are harassed when spraying noxious weeds in the rights-of-way by people who want to mow the ditches. The ecology and conservation people do not want any mowing done in the rights-of-way and the bee keepers in some areas criticize mowing the flowered vegetation their bees feed on.

These are a few of the problems and advantages South Dakota has encountered in its rights-of-way mowing policies.

Publication of this paper sponsored by Committee on Roadside Maintenance.

Abridgment

Approaches to Roadside Management

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The states in the Northwest maintain an acceptable roadside with an increasing inventory of work with fewer dollars and a smaller work force. The Washington State Highway Department, after identifying the problem many times, is developing a process that includes establishment of roadside management plans.

Roadside management is not a "buzz" word, but an accurate term identifying a team effort in roadside design, construction, and continuing maintenance. It is the process by which roadside development and maintenance are planned and accomplished in harmony with each other. Long-range goals are identified, and all activities are given priorities according to their importance in relation to the long-range goals or immediate needs or both, and their interactions.

The difference between this concept and normal procedure is interaction. In the past, each roadside activity was evaluated on its own merits and not always in relation to its impact on other activities.

Many times design and construction of the roadside create built-in maintenance problems. Construction people follow plans and specifications, and the maintenance crews wait in the wings until the contractor has finished. As soon as the contractor leaves the project, the maintenance crews take over, usually with the attitude that construction people did not mitigate the problem created by the design.

The concept of roadside management involves a team effort by engineers, landscape architects, and members of other disciplines who identify long-range goals and incorporate them into the design, construction, and maintenance of the roadside. Each of the disciplines recognizes the team goal and subsequently makes the necessary trade-offs that are an inherent part of any team effort. It is important that they recognize that the concept of roadside management applies not only to roadside improvements, but also to those roadsides that have never had an improvement project.

By using aerial photos illustrating existing field conditions as base sheets, all roadside management plans must provide at least the following information: goals and objectives; key plant materials; views to be preserved or screened; vegetation to be protected, supplemented, controlled, eradicated, or selectively thinned; general horticultural requirements (e.g. fertilizing, pest control); irrigation needs (including programing of irrigation controllers); mowing limits and frequency; roadside drainage (design and maintenance); priority of each activity; manpower requirements (annually and monthly and skill levels); equipment needs; and estimate of costs.

A roadside management plan is of great benefit as a communications tool connecting top management with the employee on the job. Too often we overlook the need to provide the employee with a communications device to interpret what is desired. The plan can also be used as a basis for the supervisor in preparing his next year's program needs that will be passed up through channels to be incorporated in the overall annual maintenance program. Once the plan is approved, the supervisor has a document to follow. At the same time, performance in the field can be evaluated by higher management.

The plan also illustrates what will not be done and acts as a basis for lower expenditures on equipment, materials, work hours, improved product quality and as a common reference for all levels of decisions.

We believe that the management approach to roadside design, construction, and maintenance is the responsible approach to making our highways aesthetically pleasing elements of the environment through which they pass. After the development of a roadside management plan, continuing maintenance should fall below previous expenditure levels, and benefits should increase.

A roadside management plan will ensure that a right-of-way will serve its highest and best use, whether this be habitat for upland game birds or landscape planting throughout a city. The goals and objectives of roadside design, construction, and maintenance can be accomplished at the lowest organizational level, if it is accomplished as a part of the plan.

Publication of this paper sponsored by Committee on Roadside Maintenance.

Abridgment

Economics of Roadside Mowing

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Expenditures for highway maintenance in the United Kingdom are being severely cut at the present time. The policy of the central government is to reduce 1975-76 expenditures by 20 to 25 percent by 1980.

As a result of this policy, the highway authorities have had to examine their maintenance standards, even though many engineers maintain that current standards are already inadequate and for some functions have not achieved the recommended target standards laid down as national criteria in the Marshall Report (1). Some of these standards for greenery cutting follow.

THE MARSHALL REPORT

The object of grass, tree, and hedge cutting is to prevent obstruction of sight lines at bends and traffic signs, to inhibit the growth of injurious and other weeds, to maintain a tidy appearance, and, in the case of trees adjoining roads, to prevent them from becoming a danger to road users.

Suggested Standards for Grass Cutting

Rural Roads

On the first 2 m (6 ft) of verges and on central reserves of motorways and trunk roads, grass should be kept below 15 cm (6 in) and elsewhere on the roadside below 30 cm (12 in).

For other roads, the minimum suggested is one cut width of one pass of the mower per year plus additional cuts as necessary to maintain visibility at bends. On more important roads and on roads with well-used footways, more frequent and wider cuts (including up to the full width every second year) may be considered necessary. Steep banks starting from the edge of the carriageway should be cut more frequently to avoid reducing its effective width or obstructing pedestrians.

Urban Roads

On motorways and trunk roads in urban areas, all grass should be kept down to 7.5 cm (3 in). On other roads, however, for highway purposes the same standards as for rural roads should apply.

Suggested Standards for Hedge Trimming

Where it is the responsibility of the highway authority, hedge trimming once a year should be sufficient on rural roads; it is needed more frequently in urban areas. Where there is a special requirement, for example to preserve visibility at bends or across central reserves, cutting should be done as required. (In the United Kingdom, highway boundary hedges are normally the responsibility of the adjoining landowner.)

Chemical Sprays

It may be necessary to use chemical sprays to eliminate weeds and control growth around posts carrying signs, along guardrails, on the edges of curbs, and on footways. They may also be used to control the growth of grass on the strip adjoining the edge of the carriageway and on central reservations. Their use should be the minimum compatible with the required results.

Trees Adjoining Roads

All trees adjoining roads, whether owned by the highway authority or not, should be periodically inspected for potentially dangerous conditions. In the case of trees owned by the highway authority, any necessary corrective measures should be taken as soon as is reasonably possible. In the case of privately owned trees, the owner or occupier of the land should be warned of any danger and requested to take the necessary action.

REPORT ON ROADSIDE MAINTENANCE, APRIL 1974

In April 1974, the Lincolnshire County Council reviewed its attitude toward roadside maintenance and adopted the following policy.

The close cutting of highway verges ensures a dense, weed-free surface able to be scavenged and maintained in a clean and tidy condition for use as required by pedestrians and equestrians.

In urban areas, high standards of verge maintenance are expected by the public, and additional features such as shrubs, tree plantings, and other landscaping are much appreciated.

In rural areas, lower standards of maintenance have been introduced, largely because of limited highway maintenance monies and the interest of naturalists and conservationists. In addition, many verges of rural roads have, by virtue of their apparent neglect, been subject to abuses, including indiscriminate use by vehicles and tipping of rubbish and excavated materials from adjoining ditches, or are so overgrown that they are frequently unusable by pedestrians and equestrians.

In urban areas, locations agreed by the county surveyor (in practice his representative, the divisional surveyor) shall be maintained in a reasonable, weed-free, close-cut condition, free from irregularities. The verges of rural main roads shall be coarse-cut to full width as required during the growing season, subject to the need to protect flora on certain verges. On other roads, one swath width shall be coarse-cut as required during the growing season.

At junctions and corners the verge shall be cut to full width. The verges shall be cut overall not earlier than the end of the flowering season.

TRUNK ROAD POLICY—GRASS CUTTING AND HEDGEROW TREATMENT

In July 1975 the Department of the Environment (now Department of Transport) issued a technical memorandum describing revised and lower standards of roadside maintenance for trunk roads and motorways. Some of these standards are as follows.

1. In future, grass cutting on land forming part of trunk roads and motorways is to cease as a general practice, and grass is to be cut only in certain restricted places and circumstances. These are:
   a. It may exceptionally be necessary to cut a swath alongside fields in areas where stubble-burning takes place on land adjoining the highway, and where there may be a particular danger of fire spreading to the highway. Farmers must, however, be pressed to fulfil their obligation of ensuring that there is no fire risk before they burn, e.g. by ploughing a strip round the fields.
   b. At sites where long grass or weeds would reduce the minimum stopping sight distances set out in "Layout of Roads in Rural Areas" or cause danger, for example, by obscuring junctions with other roads, gaps in central reservations, or road signs. Grass alongside motorway hard shoulders may exceptionally require cutting to preserve sight lines when traffic is actually using the hard shoulders during motorway works.
   c. Treatment by chemical means may be required on central reservations to prevent growth falling on the carriageway and causing "kerb shyness" in the fast lane.
   d. Where long grass or weeds would interfere with substantial pedestrian traffic and particularly with the use of the verge by schoolchildren, or cause danger by concealing from drivers the presence of pedestrians on the verge and particularly of children who might run out onto the carriageway.
   e. In built-up areas where some cutting may be necessary for the preservation of amenity. The frequency of such cutting must be agreed with the Regional Controller (Roads and Transportation).
   f. At sites listed as being of outstanding botanical interest, where a special system of management is required.

   In the new regime newly sown grass will not need treatment beyond that set out in the Specification for Road and Bridge Works.
2. In the course of time it may become necessary to deal with scrub growth where this cannot be tolerated. This problem will not, however, arise for several years. In places noxious weeds (as defined in the Weeds Act of 1959) may establish themselves, and require to be eliminated if they give rise to complaints by seeding onto neighbouring land. This is not seen as a major problem, and if it should arise it can be met by selective cutting or spraying.
3. The use of chemical sprays should be planned in co-operation with the Department's Horticultural Adviser and should be agreed by the Regional Controller (R & T) before being put into operation.
4. One consequence of the new regime may be a tendency for grass and weeds to establish themselves in gravel drains. Special attention would be needed to deal with this, but it is not anticipated that it will amount to more than some intensification of present activity.
5. Hedgerows bordering the trunk road or motorway, but not owned by the Highway Authority, are not to be cut except on repayment unless there is an Agreement to maintain them. Boundary hedges owned by the Highway Authority should be cut only so as to comply with the legal obligation to prevent nuisance caused by growth overhanging neighbouring property, and where necessary for road safety or the visibility of signs. Hedges on central reservations may sometimes need to be cut, so as to avert encroachment on the outside lane of the carriageway.

Implementation of this policy involved considerable effort because the areas described in 1b, 1d, 1e, and 1f can only be identified "on the ground."

Areas where measurements were required were at bends, private entrances (houses, farms, industries), road junctions, urban areas (amenity cutting), and verges used by pedestrians. In addition, the survey team also recorded accident blackspots with particular reference to vertical alignment. The average survey time by a two-man team was 16 to 20 km (10 to 13 miles) per day, but this was very dependent on environment.

REPORT ON ROADSIDE MAINTENANCE, APRIL 1976

The Lincolnshire County Council is the highway authority for 8392 km (5203 miles) of county roads, and views on the application of the revised and lower trunk road standards to the county road network were solicited from elected representatives throughout the county. Typical comments recorded were "present standards on county roads should be maintained"; "non-cutting will give rise to problems with tipping"; "farmers could cut more but there are difficulties with high verges and drainage grips"; "one swath adjoining road is essential for pedestrians"; "visibility at bends must be ensured"; "weeds and coarse grass will swamp wild flowers"; "brush and scrub would cause damage to the carriageway and would hinder maintenance of ditches and hedges"; and "weeds would spread to adjoining agricultural land."

These comments, together with a report on the revised trunk road grass cutting policy, were reported to the County Council transportation committee when they met in April 1976. The members were asked whether they wished to revise the policy for county roads in the interests of economy.

The revised policy that was recommended and adopted goes some way in the direction of the trunk roads policy and will enable some economies to be made in rural areas.

The present policy of close cutting is to be continued on all county roads in urban areas.

On main roads in rural areas the revised policy is a one-swath width adjacent to the carriageway on both sides of the road cut regularly, and the full width of the roadside verge cut regularly at the inside of bends together with splays at junctions and entrances.

Full width cutting will be carried out at the end of every third growing season to prevent the establishment of scrub and to deter other nuisances.

On minor roads in rural areas the assistance of the adjoining landowner should be enlisted, and if necessary severe obstructions should be modified to encourage adjoining landowners to cut the grass. Where the adjoining landowner declines to cut the grass, then those verges will be cut overall every third year to prevent the establishment of scrub.

The future expenditure on roadside maintenance will be carefully monitored to confirm the significance of any savings in real terms in order that the policy may be reconsidered and amended as necessary.

REVISED POLICIES OF ROADSIDE MAINTENANCE

The summer of 1976 was the driest ever recorded throughout England, and consequently grass growth was at an all-time minimum.

It would therefore be inappropriate to draw any comparisons between the roadside maintenance costs of 1976 and other years. However, the long-standing dry grass on trunk road verges presented a considerable fire risk. There were many instances of roadside fires, some of which spread to adjoining property. All presented a traffic hazard by reducing visibility.

REFERENCES

1. Report of the Committee on Highway Maintenance (Marshall Report). Her Majesty's Stationery Office, London, 1970.
2. Report of the Committee on Highway Maintenance (Marshall Report). Her Majesty's Stationery Office, London, 1970, Appendix 1: Proposed Initial Standards of Maintenance.

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Economic Analysis of the Environmental Impact of Highway Deicing Salts

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This paper reports on an analysis of the cost of damages that result from sodium chloride used to melt snow and ice on highways. An extensive literature search and several surveys were made to determine the types and extent of damages that have occurred. The major cost sectors examined were water supplies and health, vegetation, highway structures, vehicles, and utilities. A conservative cost estimate was developed for each sector. The total annual national cost of salt-related damage approaches $3 billion, about 15 times the annual national cost of the salt and its application. The highest direct costs result from damage to vehicles, but the most serious damage appears to be the pollution of water supplies and the attendant degradation of health. It is difficult to assign costs to this, and therefore the estimate may substantially understate actual indirect costs to society. These findings indicate that some areas should, on the basis of local conditions, reduce the amount of salt used.

Extensive use of the salts sodium chloride (NaCl) and calcium chloride ($CaCl_2$) for removing snow began during the early 1960s. Before that time, highway maintenance departments depended primarily on abrasives, such as sand and cinders, combined with plowing, to clear snow and ice from highways; salt was generally added to the abrasives to prevent freezing. However, maintenance departments gradually began to appreciate salt's accelerated melting effect.

Through experimentation, maintenance engineers learned that direct application of salt before, during, and after a snowstorm greatly facilitated their snow removal operations, in terms of both time and money. Since that discovery, the use of salt for snow and ice removal has grown rapidly, in some cases by as much as 900 percent in the past 15 years (1). This extensive use of salt, however, has now been associated with a significant amount of damage.

There is no question that salt is an excellent tool for removing snow. There also is no question that, in terms of time and budget constraints for snow removal operations alone, salt used in large quantities with plowing is essential.

Highway departments trying to create the safest driving conditions believe that providing most bare pavement in least time is best achieved by extensive use of salt—given budget constraints. However, some highway departments, in their eagerness to perform well and meet these goals, have inadvertently used salt inefficiently. The education and understanding fostered by the Salt Institute, the Environmental Protection Agency (EPA) (2, 3), the Massachusetts Special Commission on Salt Contamination (4), and many others have corrected some of this misuse. The result has been a more effective use of salt with no essential reduction in level of usage.

Total salt use for snow and ice removal in this country now stands at approximately 9 million Mg (10 million tons) each year. Many highway departments apply as much as 37 Mg/lane·km (25 tons/lane·mile) in one season. On a four-lane highway, this amounts to 56 kg of salt in the runoff for every meter of the highway (38 lb/ft). As we build more highways, we can expect salt use to increase, if we continue with our present policy (and if no alternative method for snow and ice removal is found).

The concept of June travel in January, better known as the "bare pavement policy," has led highway departments and the public to the situation in which we currently find ourselves. And over the past 10 years, and especially the past 4 to 5 years, the number of reports of damage to water supplies, vegetation, and the very vehicles and highways that originally served as the only focus of attention has grown.

The reports of salt-related damage are extensive and well documented. There have been several excellent works summarizing these reports (2, 3, 4, 5, 6) and many excellent in-depth studies on certain areas of damage. On careful examination of all these studies and the other literature, one can only conclude that this very serious problem requires careful assessment as do the current guidelines for snow-removal procedures.

It is in this context that the idea for the EPA study was formulated. Previous studies, as thorough as they were, did not assess the entire problem of damage in terms that included economic analysis. Logically, except in extreme cases, no amount of damage claims without the necessary economics could allow a rational salting policy to be formulated. Without such an economic analysis, no alternative to salt, whether it be more snow left on the highways or a more expensive replacement for salt, could ever be economically justified.

Consequently, although this analysis of damage from road salt was undertaken with a view to quantifying the problem, the lack of cost data in some instances prohibited it.

I was impressed not only with the extensive damage reports and the costs of that damage but with the extensive amount of damage in one sector. Rusted vehicles and bridges, although costly, can be replaced. Damaged vegetation can be ignored by those who do not incur the costs. But pollution of our water supplies is a serious matter. This is not to say that all road salting leads to water pollution, but it has in many instances.

The medical implications of salt in drinking water must not be taken lightly. Because of many unknowns, the economic analysis of the damage to water does not demonstrate how potentially serious the problem may be. Each of us must become aware of the facts on road salt damage in our environment. It is to be hoped that in this way a rational solution will be found and serious damage to our environment be prevented.

The fact that salt damage has reached this magnitude should come as no surprise to anyone who understands the way our current policy evolved. Highway maintenance departments' primary goal is to provide maximum highway safety and convenience. Although current practices may be near the optimum in terms of department goals (and highway budget constraints), in many locations these practices are far from the optimum in terms of the whole environment, man-made and natural. This has occurred primarily because those who determine present maintenance policies are largely unaffected by the adverse environmental conditions. Legal liability in some instances may have forced highway departments to attend only to accident prevention, and there have been no outside forces to regulate a department's activities.

Determining the precise levels or combinations of winter maintenance policies that maximize social well-being within certain constraints is the crux of the problem. Theoretically, at least, this should be accomplished by assigning prices to the various social and environmental values and then choosing those that maximize total net value. However, it is not possible to assign exact costs to all items, because their values are often subjective. This is especially difficult in cases where irreversible, permanent damage to health and vegetation occurs.

The concept of "public pressure for bare pavement" may actually have evolved simply because the public was unaware of environmental damage and thought that more bare pavement resulted only in a small increase in the maintenance budget. This attitude seems to be rapidly changing as the public becomes more aware of the need for a sound environment, and citizens of many cities and towns are working with state and local departments of public works to develop reduced salting and better water. The public information programs that have increased public awareness seem to have paved the way for policy changes to be favorably received. There is every reason to believe that a rational salting policy would be welcomed by every community.

METHOD

During the course of the EPA study, a complete review of the literature on snow and ice removal, salt use, and salt damage was made, and the most relevant documents—over 450 in number—were obtained. Each one of these documents was carefully screened for validity and relevance; over 300 were retained in a bibliography at the end of the EPA report.

The second portion of the research involved mailing several hundred questionnaires and letters to universities, public works departments, public health departments, and water suppliers. There were over 100 respondents who indicated that they had incurred damage or knew of damage in their areas or who provided us with documents or contacts pertaining to salt-related damage. Follow-up was done in many cases to clarify responses or obtain further information.

Nearly 200 personal contacts from the literature and surveys were made by either letter or phone. Almost all of these people provided information on salt-related damage. But, as in the case of the literature and the surveys, "hard" data on damage costs were few.

The study was essentially restricted to readily available data, either as publications or as accessible records. Unfortunately, most of the research on the potential damages of deicing salts has been in the nature of case studies. This orientation has led to interesting but often irrelevant findings useless for decision making at the local or state level.

Deicing salts are foreign substances that change characteristics of natural or man-made resources directly or indirectly in a way that reduces usefulness. These reductions can be related, at least at a conceptual level, to dollar costs that describe the damages in terms of a common denominator. These costs (estimated or imputed) are an expression of the value of the "lost usefulness" of the respective resource, or the cost of restoring that resource to its full usefulness.

The main problem of applying this framework to actually estimating costs is the nature of the relations between damages and salt application. These damages in the environment result from complex processes.

Generally, numerous factors other than salt must be taken into consideration in analyzing salt use and any suspected damages from it, but present knowledge of the interaction among factors is limited. In addition, the processes also depend on such phenomena as precipitation patterns and on the amount of salt applied. All of this produces considerable uncertainty in the known relations between damages and salt use, which impedes the development of a microanalytical model that translates relations into functional and quantifiable expressions.

Consequently, the EPA cost analysis was based on a general model expressing the expected (or average) annual cost in a particular damage category as the product of the expected magnitude of total damages (which in turn is the product of the probability of occurrence times the damage per occurrence) and the cost per unit of damage.

This general cost model provided the basis for examining the literature and other materials accumulated during the study. Considerable effort was given to adapting the available information to fill the data needs. However, in most cases, because the estimation procedures used to quantify parameters of the cost function were too broad, costs were estimated on an ad hoc basis, for example by a weighted extrapolation of detailed cost data for a particular state or region.

The analysis works best with damages to automobiles. Multivariate analysis is used to determine the incidence of salt-related damage; motor vehicle registrations provide reliable data on the total population at risk; and automobile prices can be used to determine the unit cost of salt-related damage. It is interesting that this method yielded the highest cost estimate of all categories examined.

RESULTS

Damage to natural components of the environment from road salt has been mentioned frequently in the literature. There is an abundance of information on how damage occurs and numerous reports of specific damage cases and, in some instances, what the cost incurred was. Nevertheless, the proponents of salt use have continually denied the significance of such findings, dismissing them with such comments as "the value of a tree does not compare to the value of human life." Damage to vegetation, although extensive and costly, is not the major component of damage; contamination of water supplies and and the impact on human life are.

In general, salt damage to natural resources is usually irreversible, too costly or difficult to reverse, or remedied only by the passage of time. Irreversibility means a significant risk, because the true meaning of the damage may only emerge in the future, when it is too late.

Assessing damage to natural resources from road salt has always been difficult. Not all the processes by which damage occurs are known, nor is the exact relation between salt use and damage. The effects of salt in nature are also often cumulative and therefore require lengthy studies for complete understanding. Finally, because irreversibility is so poorly understood, frequent disagreement over the extent of damage makes it difficult to assign costs.

Our cost analysis of damage to natural resources was conservative. The figures developed provide a lower bound, but actual costs may far exceed these numbers.

Man-made goods are also substantially damaged by road salt. Vehicles passing over the salt solution on the highway are sprayed with a highly concentrated solution, and salt deposits on metal surfaces accelerate corrosion. Salt splash from vehicles and direct runoff can coat highway structures and those nearby and make them more vulnerable to corrosion. Salt seepage through pavement will eventually damage the roadway. Runoff and percolation will also allow the salt solution to eventually attack underground wires and pipes.

Damage to Drinking Water Supplies and Health

The contamination of water supplies is possibly the most serious damage that results from the use of road salt. Salt percolating down through the soil can enter groundwater supplies. It can also enter surface supplies as direct runoff from highways. Processing water to remove salt is an extremely expensive and complicated task rarely done. Although not always possible, the safest means of preventing the salt from reaching water supplies is to catch the highway runoff and direct it to a high-flow stream or river that eventually reaches the ocean without entering another water supply. Although new highways may provide for runoff, the cost must be considered. This has not been incorporated into most existing roadways, and to do so now would be either very costly or impossible.

The details of salt contamination in various parts of the country have been extensively recorded. Some of the first cases of serious salt infiltration were caused by improper storage facilities, but most salt pollution is now caused by runoff from streets and highways. The following examples should help to illustrate the extent of the problem.

Before 1940, all Massachusetts data indicated chloride content of public water supplies was less than 10 mg/L (implying a sodium content less than 3 to 6 mg/L). Recent data (1976) indicate that at least 117 communities (over 260 suppliers) have one or more of their public water supplies containing sodium above 20 mg/L. Two towns experienced such severe contamination of public wells that new wells had to be dug at a cost of approximately $150 000 for each town.

These data do not include private water supplies, for which few data exist. After several residents detected bad tasting, corrosive water, the town of Goshen, Massachusetts, discovered that many private wells were above 250 mg/L sodium and the school well was 390 mg/L. Residents and the school were forced to purchase bottled water.

Water supplies in Connecticut have been showing similar salt intrusion. Recently a few supplies appear to have leveled off, possibly in response to the one-third cut in the state's use of salt, a measure initiated because of salt infiltration.

Since 1964 the state of New Hampshire's budget has included funds for replacing wells contaminated by salt. Originally set at $100 000 a year, the budget was increased to $200 000 in 1974, when 50 wells were replaced.

These damage figures and the replacement costs for New Hampshire—along with the salting intensities of all the snowbelt states and the relative importance of well supplies in other states as compared to New Hampshire—imply that the direct costs of replacement alone for the nation are close to $50 million annually. These costs cover the replacement only of seriously polluted wells, those with over 250 mg/L chloride (162 mg/L sodium), a guideline set by the Public Health Service in 1962.

Measuring the cost of health degradation from elevated sodium levels in drinking water is virtually impossible. For years now medical research has established that intake of sodium chloride is a critical factor in many conditions such as hypertension, cardiovascular diseases, renal and liver diseases, and metabolic disorders (7). Intake of salt also endangers many pregnant women.

Recent research has further strengthened the link between salt and hypertension and between salt and the other diseases and problems mentioned above (8, 9, 10, 11, 12, 13). Freis cites "epidemiological studies in unacculturated peoples showing that the prevalence of hypertension is inversely correlated with the degree of salt intake . . . " (13) and concludes that

On the basis of present knowledge, it would seem wise for individuals with a family history of essential hypertension to accustom themselves to a truly salt-free diet (less than 1 gm of salt or 15 mEq of sodium per day) and to prevent their children from acquiring the habit of eating salted foods.

The American Heart Association, backed by many leading medical researchers and physicians, has recommended a limit of 20 mg/L sodium in drinking water for patients whose diets are restricted to less than 1 g of sodium per day (7).

According to recent estimates, approximately 23 million Americans are suffering from hypertension and should restrict their sodium intake (14). This group, together with others who should restrict their sodium intake to 20 mg/L, represent 20 to 25 percent of the population; some researchers claim the percentage is as high as 40 percent (9). Unfortunately, many of those who should restrict their salt intake are not aware that their lives are at risk. For one person, water containing more than 20 mg/L sodium may not be a significant danger, while such water is potentially very harmful to people who are or should be on low sodium diets (approximately 4 to 5 percent).

In addition, education of the public will undoubtedly result in greater awareness of hypertension, and more people will restrict their salt intake. Complete reversal of the trends that increase sodium presence might take years once action is taken, and endanger many more people than just those currently on salt-restricted diets.

Several years ago, the state of Connecticut adopted a 20 mg/L standard, and Massachusetts is now in the process of doing so. A Massachusetts advisory committee of 16 medical authorities has overwhelmingly—one dissented because he felt the evidence inadequate—recommended this standard (15).

An indication of the cost of salt-contaminated drinking water in terms of effects can be obtained by estimating the expenditures required to remove the hazard. Total costs can be roughly obtained by assuming that all hypertensive people would purchase bottled drinking water once their water supplies exceeded a 20 mg/L sodium concentration. For the normal adult, an average daily drinking water consumption of 2.2 L has been estimated, and 3.8 L (1 gallon) of bottled water sells for about 50¢, so the average person would spend approximately \$106 ($2.2/3.8 \times 365$ d $\times$ 50¢) a year.

The number of people on low sodium diets exposed to drinking water above 20 mg/L sodium attributable to road salt is extremely difficult to estimate. Massachusetts' experience indicates that approximately 27 percent of the water supplies (not necessarily the population) are affected by high sodium concentrations from road salting. As a broad estimate, roughly 25 percent of the population under conditions similar to those in Massachusetts are affected, and 4 percent have been estimated to be on low sodium diets.

By using salting intensity as a weight to make other states in the snowbelt comparable to Massachusetts, we estimate a total cost for the nation of \$105 million. Massachusetts relies more heavily on groundwater than the nation as a whole (23 percent versus 21 percent), so the estimate should be somewhat lower, \$96 million for bottled water to people on low sodium diets.

In summary, the annual direct and indirect costs of water supply contamination may add up to almost \$150 million nationwide. This figure is meant to convey an impression of the magnitude of the damage, not to describe actual costs. Note that we have not included any costs to industry of special processed water requirements.

Damage to Vegetation

Experiments and empirical studies have clearly demonstrated that many trees used for roadside planting in the snowbelt are sensitive to increased sodium concentrations in the soil and that there is a direct link between the deterioration or death of roadside vegetation and salt application.

Salt directly interferes with the chemical processes by which plants absorb nutrition and affects the osmotic balance, thus inhibiting the water intake of plants. It may replace vital nutrients.

In addition to these direct effects, sodium in the soil may also result in a rapid deterioration of the soil itself. Westing (16) has said that "when sodium comes to occupy more than about 15 percent of the total cation exchange capacity of the soil, soil structure begins to deteriorate ... permeability and water-holding capacity decrease markedly." The soil becomes low in nutrients, and little, if any, vegetation can grow in it. In some cases this in turn has led to severe erosion and the eventual clogging of drain sewers. Continued application of road salt has been shown to have a cumulative effect on the soil.

Although drainage conditions are important in determining how far from the edge of the highway vegetation is affected, most damage occurs within 9 to 12 m (30 to 50 ft). Other factors such as drought, low soil fertility, low soil permeability, pollution from vehicle exhaust, and mechanical injury to roots also contribute to the damage.

However, comprehensive studies have shown that salt is often the prime factor leading to death of vegetation (17, 18, 19). These studies were based on soil samples and analyses of sodium and chloride contents in leaf and twig samples. One study clearly demonstrated the effect of salt by comparing tree damage on salted and unsalted roads in the same towns (20).

The usual sequence of events in salt damage in plants is increasing sodium and chloride concentration in plant tissue, reduced growth, falling leaves, dropping twigs, dying limbs, and death. Clinton E. Carlson found in controlled experiments that "once foliar symptoms of salt were noticed, it was not possible to prevent further damage—the trees always died even though they were taken off the salt solution and given only pure water."

Heavy salt use and the resulting damage to vegetation can lead not only to personal property damage and possibly crop damage but also to the creation of unsightly highways, reduced property values, and failed highway beautification programs. There are also real costs involved in terms of increased highway maintenance for removal and replanting.

Although the botanical and chemical evidence is sufficient to document widespread deterioration of roadside vegetation in areas characterized by the heavy use of deicing salts, the empirical support is somewhat meager. Apart from studies of specific stretches of highway and vegetation, the data base relating to deicing salts and vegetation damage on a macroscale is limited to reports of specific instances.

Rich (20) reports that in 1957 the New Hampshire Highway Department removed 13 997 dead trees along 6000 km (3700 miles) of highway. The estimated cost of removal was \$1 million, or more than \$70 per tree. According to other reports, Winchester, Massachusetts, which has applied as much as 31 Mg/km (55 tons/mile), has lost an average of 56 trees a year since 1963 (6). Similarly, Newton, Massachusetts, which also applied amounts of salt far above the average for towns and cities in the state, is reported to have lost about 500 trees a year between 1965 and 1970.

The problem with evaluating these reports is that they tend to ascribe all tree deaths to salt application. Because there are no national statistics on the number of trees dying each year, it is impossible statistically to establish the net effects of deicing salts in terms of damages to roadside trees. Similarly, data on the risk—roadside trees possibly exposed to deicing salt runoff—that would be useful in applying microanalytical findings to a macrolevel framework are unavailable. As a result, accurate national damage estimates simply cannot be generated.

Direct costs are maintenance and removal in the case of death; indirect costs involve losses to individual property owners resulting from the death (or deterioration) of a fully grown shade tree that may or may not be replaced by a small young tree. Evaluating indirect costs is of course the most difficult task. Fortunately, a number of studies have been made of the monetary value of shade trees.

The International Shade Tree Conference (21) base the monetary value of a shade or ornamental tree on three basic factors, the size, kind, and condition of the tree, and in August 1973 they adopted $1.55/cm² ($10.00/in²) of trunk cross section as the value of a perfect shade tree specimen. This figure would add $10 milmion in indirect costs to direct costs [using a fairly conservative average of 25.4-cm (10-in) diameter for the 13 997 trees removed].

The measures established by the International Shade Tree Conference may be somewhat arbitrary and, more importantly, may apply primarily to urban trees, but they provide an indication of the potential magnitude of the problem. If only 6 percent of all tree deaths in the New Hampshire example were attributed to deicing salts, the total annual direct and indirect costs would be $660 000, an amount comparable to the cost computed for the water contamination damages. Extrapolating from this figure to a national level by using salting intensity by state produces a total annual figure of about $45 million. This is a representative cost figure. If dollar amounts could be determined for damage to privately owned vegetation and roadside vegetation other than trees, the total damage costs would be far higher.

Damage to Highways and Highway Structures

A thorough search of the literature and a survey of all snowbelt state highway departments and approximately 100 large city highway departments have disclosed that there has been extensive salt-related damage to bridge decks. By far the most devastating damage is the general deterioration of the West Side Highway in New York City.

On December 15, 1973, the northbound roadway between Little West 12th Street and Gansevoort Street collapsed. The city transportation administration contracted consulting engineers to conduct a complete examination (22). The analysis, conducted from July 9 to November 14, 1974, comprised four substantive volumes, the last dated May 30, 1975. The following is taken from the most recent report (22, p. 21).

> The deterioration of the West Side Highway has been a continuous problem. As early as the mid-fifties, public officials had anticipated its early demise. The use of salt to remove ice, combined with heavy traffic, has caused disintegration of large sections of the roadway.

"The deterioration," they continue, "is a direct result of water and waterborne salt leaking through the expansion joints and the concrete deck." They conclude, not surprisingly, that in part the "Use of salt for deicing should be minimized. Other methods and materials for maintaining traffic during ice and snow conditions should be considered." The report also concludes that, although restoration of the highway is feasible, the cost of the work, estimated at $58 to $88 million, is almost prohibitive.

Although the cost of the repairs to the West Side Highway may not be representative of typical bridge damage, there are other examples of costly salt-induced deterioration. It was recently reported that four Washington, D.C., bridges had become dangerous to traffic because salt had caused extensive corrosion of the reinforcing steel. The cost of the repairs is $11.7 million (23).

The actual cost outlay by state highway departments for the repair of bridge decks was estimated to be $40 million in 1971; total outlays, including repairs that would halt the deterioration in quality, were estimated between $80 and $120 million.

Evidence from individual states, particularly West Virginia, provides a check on these figures. The cost of maintaining West Virginia's 6000 bridges in good condition was recently said to be approximately $12 million, or about $2000 per bridge. Assuming that bridges are distributed throughout the state in the same proportion as the population, that 100 percent of the bridges located in severe deterioration regions require periodic maintenance and repair of salt-induced damages, and that only 20 percent of the bridges in moderate regions require such maintenance, we conclude that nearly 100 000 bridges are adversely affected.

If the West Virginia estimates are representative of other severe deterioration regions, a yearly cost of $200 million for the nation's bridges is estimated. Similar procedures estimate the cost of special construction techniques to prevent rapid deterioration of new bridges at about $10 million annually. Direct costs of salt damage to bridge decks will be in the range of $200 to $250 million annually. In addition, the relevant direct costs should also include necessary repairs of structural damages. Although these damages are insufficiently documented for the nation as a whole, specific instances such as the West Side Highway can be cited to indicate the potential magnitude of the problem.

The direct cost estimates include only (a) expenditures by highway agencies for special design features on new bridges and (b) repair of existing structures to counter the adverse effects of road salting. Full social costs would include delays to motorists during repair, repair costs for damages to ball joints and front end alignment from travel on uneven bridge surfaces, and the cost of accidents caused by rough bridge deck surfaces. Some of these, although potentially important, would be exceedingly difficult to measure accurately.

On the basis of a recent discussion of vehicle behavior at sites of traffic obstructions (24), the cost of lost commuter time during bridge repair has been estimated. The conservative figure comes close to $250 million annually. In summary, the total annual costs of bridge deck damages related to salt use can be said to exceed $500 million.

Corrosion of Motor Vehicles

It is likely that people have directly observed vehicle corrosion more than any other form of salt-related damage. The link between the application of salt on highways and the corrosion of automobiles is well documented. Previous studies have concluded that road salting causes a doubling of the normal corrosion rate (25). Although it accepted these figures, the Environmental Protection Agency report has distinguished four major cost categories:

1. Costs of protective measures both by manufacturers and by owners,
2. Costs of repairs required to maintain the ability of the automobile to function at the same level as without salt-induced corrosion,
3. Losses in economic value of the automobile as a result of salt-induced corrosion, and
4. Costs of accidents attributable to automobile malfunctioning associated with salt-induced corrosion.

The third category is the most amenable to analysis, because we can attribute depreciation rates to the influence of salt. A regression model for depreciation rates was built on an economic model of used automobile prices and data on used automobile prices (30 on the average) for three makes of automobile in each of 44 metropolitan regions. The interim models included temperature, humidity and rainfall, proximity to ocean, sanding intensity, air pollution (SO_2 levels), income per capita, and vehicles per capita, all of which were found to be unimportant and were therefore removed from the analysis. The final model predicted depreciation rates on the basis of state salt use, city salt use, snowfall, and kilometers driven.

The cost of incremental depreciation from salt was estimated by evaluating the stocks of automobiles in various environments and multiplying by the incremental depreciation attributable to salt use. On this basis, the total annual national cost of automobile depreciation caused by road salt was $1.4 billion. Extrapolating from the above model and from known estimates of costs of damage and maintenance for trucks and buses, an additional cost of $690 million was estimated. Thus, the total annual cost to owners of motor vehicles is in excess of $2 billion.

Other Damages

Damages attributed to the use of deicing salts have been noted in areas other than those discussed in the preceding sections, but available evidence, of course, is limited to a few reports. A brief review of this evidence suggests, for example, that on a national scale the potential effects of deicing salts on underground cables and electric utility lines may be substantial.

One of the best-documented instances of salt-related damage to underground power transmission lines is the case of Consolidated Edison's (ConEd's) facilities in New York City. This company maintains the largest system of underground electric facilities in the world. The winters of 1972 to 1973 and 1973 to 1974 contrasted strikingly in terms of salt applications and resulting damages to this underground cable system. After extensive analysis of the damage, ConEd, in an in-house memo in 1974, stated that "Altogether, it is safe to estimate that the salt spread on the streets of New York City resulted in additional expenditures by ConEd in excess of $5 million during the winter of 1973 to 1974."

No estimate of the cost to consumers as a result of power outages has been made, but data suggest that several hundred power outages during the severe winter months can be attributed to salt use. The costs of such extensive power losses are very significant in terms of inconvenience, lost production time, and lost personal time.

It is likely that the costs incurred by ConEd are far higher than those incurred by any other municipal electric company. Many other instances of such damage that have not been so well documented and analyzed will probably be found to be salt related. The analysis will ideally pave the way for other large utility suppliers (and users) to thoroughly document and investigate their own reports of salt-related damage.

BENEFITS OF ROAD SALTING

Salt is beneficial insofar as it increases safety and saves time. The relations between salt use and these two factors are complex, especially with regard to safety. However, it is appropriate to report briefly on the work that has been done in these areas.

How much alternative winter maintenance policies affect highway safety cannot be established by directly comparing accident rates and maintenance policies—unless driver behavior is included. Although one would expect considerable research to have been directed toward understanding situations that involve the risk of injury and death, surprisingly little is actually known. Human behavior under conditions of financial uncertainty has a rich theoretical and applied literature, but such models are largely inappropriate for the analysis of accident risk, and attempts to model human behavior in situations involving the risk of life or limb have not been very successful. The existing evidence for a connection between safety and alternative winter maintenance policies is both meager and inconsistent. Deicing salts are often assumed to improve driving conditions. There is no question that salt can increase friction between the tire and road. Courts, in assigning liability for single-vehicle winter accidents have on occasion found highway department officials negligent for not applying enough salt to provide an acceptable level of safety to motorists. Also, the assumed causal relations between deicing salts and highway safety are a major rationale offered by highway department officials for the twenty-fold increase in the annual use of deicing salts since 1950.

Three studies that contain information supporting the contention that deicing salts reduce highway accident rates are all flawed by serious design and analytic errors and omissions that make the results meaningless.

The key issue is really driver behavior. Some researchers have reported that salting may create a false sense of security in many drivers (26). This statement remains unproved, but other researchers have noted that improvements in safety can bring increases in speed that cancel out the impact of the improvements on accident rates (27, 28).

Under hazardous winter conditions, drivers do slow down, but probably only enough to make their perceived risk of injury the same as under normal conditions. Whether perceived risk is the same as actual risk is unknown, although an Ottawa consulting firm found that accidents on icy roads generally involve property damage, while accidents on bare (dry or wet) pavement in winter are more likely to involve personal injury (29).

Substantial research into the relations between salt and safety is a necessity. Salt used properly does increase friction, but this is not the only factor that determines accident rates. Continuity of conditions along a roadway, speed, and, above all, driver behavior must be considered. Such research will be extremely difficult, because determining what constitutes taking a risk under varying driving conditions is a difficult (if not impossible) task. Until such research is performed, we cannot assume that salt and safety are synonymous.

In fact, it is quite possible that the level of salt is not a fixed factor in determining safety. If the amount of salt were reduced and alternative measures were not taken to retain the same level of bare pavement, then presumably the public could be forewarned satisfactorily so that greater care and fewer trips could be taken during snowstorms. If highway speed were reduced, then the level of safety would remain the same. The only advantage of heavy salt use then would be time savings. This is certainly a very important factor; time savings must be maximized whenever possible.

Anyone who has driven under snow and ice conditions knows that progress is slowed, especially during rush hour. There have been scattered estimates of the cost of lost time but no major effort to assess the true value of lost time. The costs may be high, but a certain amount of care must be taken in developing the figures.

For example, it is false to suggest that a 1-h delay for all people in a city will result in a loss of one-eighth of the economic activity for that day. There may be losses to workers paid on an hourly basis, to industries that must shut down from staff shortages, and in rare instances to food stores because of spoilage. However, delay has little effect on the income or productivity (in the long run) of salaried workers, and there is probably little if any loss in terms of shopping expenditures, because shoppers will simply defer their errands to a later time.

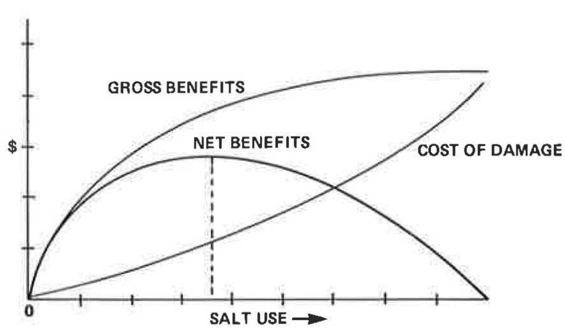
Better planning for the possibility of hazardous snowstorms would probably help reduce business losses. Nevertheless, the question of actual costs of lost time from snowstorms is still a problem open to research.

A recent study sponsored by the Salt Institute (30) attempted to establish the economic benefits lost if salt were eliminated. (No one, including the EPA, has suggested that salt be eliminated, just that it be reduced in environmentally sensitive areas.) Unfortunately the estimates, based on the assumption that 10 percent of the workers would be absent and all the rest would be 2 h late throughout all the snowbelt states for 20 days appear inflated. In addition, the researchers double-counted benefits by including both loss in wages and loss in value of goods produced.

A major point is that we should certainly expect the benefits to be substantially greater than the costs. Units have not been assigned to the axes in Figure 1 because little is known about the exact relation between level of salt use and actual costs and benefits. Nevertheless, it can be shown that the gross benefits and cost of damage curves are of the shape shown in this figure. The exact shape for a region or locality varies according to local conditions.

Net benefits are maximized at a level of salt use for which the gross benefits and cost of damage curves have identical slopes. Explained in another way, salt use is optimized at a level for which the marginal benefits equal the marginal costs. The gross benefits could be many times the costs at this point, and if the benefits were not significantly greater than the costs, one might suspect that salt is overused. There is no simple way to determine the best level of salt use, and the only way an agency could begin to do so would be to experiment with level and frequency of application. This is exactly what many agencies have done and are currently doing.

Figure 1. Costs, gross benefits, and net benefits of salt as a function of salt use level.



CONCLUSIONS

The costs of actual salt damage to water supplies and health, vegetation, vehicles, bridges, and utilities are immense. Annual damage costs, at a very lower bound, approach \$3 billion. This hidden cost is almost 15 times the annual national budget for the purchase and application of road salt and about 6 times the entire annual national budget for snow and ice removal.

Furthermore, heavy salt use in many instances upsets the natural ecological balance and results in damages that cannot be assigned a dollar value. This is one of the many reasons why the above amounts must be considered as lower bounds. The potentially most serious of all these damages are the irreversible ones, such as the risk of increased hypertension that results from the heightened levels of sodium in water supplies. As much as 5 percent of the population drinking water contaminated by road salt may be adversely affected.

The implications are clear. The costs of damage to bridge decks and vehicles are high but reversible; the damage to health may not be reversible. We can no longer afford to ignore the fact that we are depositing large quantities of salt into the water on which we are dependent every moment of our lives.

The most advanced medical research indicates that water with more than 20 mg/L of sodium is unhealthy and detrimental to a substantial portion of the population. The American Heart Association supports this fact. Disregard for the quality of drinking water in this and any instance is extreme negligence, and we must face the issue squarely. Road salt may be only one of the many serious pollutants in our environment, but that is no excuse to allow the present situation to continue. In order to avoid further damage and high costs, salt use for winter maintenance must be reduced.

It is public information programs that have given carefully designed reduced salting policies public acceptance. The most notable case is the state of Connecticut, where state salt use was reduced by 33 percent because of rising sodium content in water supplies. This reduction was apparently made with little change in level of service or accidents and with a resulting cost savings. Also, a 50 percent reduction in salt use was made in one area of Madison, Wisconsin, during the winter of 1974 to 1975.

A public opinion survey (with an 84 percent response rate) was used to determine the public's reaction to the cutback (31).

> Survey results indicate that Madison residents strongly support the limited salt program. More than 90 percent of the respondents believed that the program is a worthwhile experiment and should be continued, while 85 percent supported a reduced salt program on a city wide basis.

There is every reason to believe that the residents of individual cities and towns or other states would accept reduced salting if salt-related damages were made known to them.

RECOMMENDATIONS

In order to maintain the public's right to clean water, the level of salting in many areas should be reduced according to local conditions such as the effect of salt-laden runoff on water supplies, the level of public demand for bare pavement, and the size of the winter maintenance budget. Greater emphasis should be placed on nonchemical methods of snow and ice control (such as increased plowing and sanding).

Setting the level of bare pavement is a burden that need not be the sole responsibility of highway maintenance departments, who should seek advice from all interested groups and the public at large. Through public affairs programs, the public should be made aware of the trade-offs and alternatives. A city or town may want to form a special committee or hold a voter referendum to ensure the best solution. Changes in winter road policies should have public support and should be publicly announced before they are effected.

There should be a greater emphasis on training drivers in the skills of snow and ice driving and less emphasis on the concept of guaranteed June travel in January. Moreover, an operating policy to encourage motorists to stay off the roads during and immediately after storms would facilitate snow and ice removal.

Snowbelt states should test public and private water supplies and provide funds for replacing wells (as has been done in New Hampshire). State legislation should be passed allowing individuals to sue for damages when water supplies show an abnormal or hazardous increase in sodium content and placing the burden of proof on the highway departments that the cause was not road salt. If road salt is found to be the source of contamination, then corrective action, such as the installation of drainage systems or a reduction in salt use, must be taken to restore the damaged water supplies. Until an acceptable sodium level is reached, it would appear proper for the local governing body or the highway maintenance department responsible for the damage to provide bottled water to those using the contaminated supply.

Reasonable levels should be established on the basis of the sodium standard in the Safe Drinking Water Act and on the basis of natural background levels of sodium in the water supplies. Finally, the states should consider instituting a requirement that all salt users file an environmental assessment.

Although these measures may seem burdensome, they are necessary in order to ensure that we maintain our high quality of water and that large costs are not incurred as a result of winter highway policies.

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REFERENCES

1. M. M. Cohn and R. R. Fleming. Managing Snow Removal and Ice Control Programs. American Public Works Association, Chicago, Special Rept. No. 42, 1974.
2. R. Field and others. Environmental Impact of Highway Deicing. Environmental Protection Agency, Water Quality Research, Edison, NJ, 1971.
3. R. Field and others. Water Pollution and Associated Effects From Street Salting. Paper presented at the 45th Annual Conference of the Water Pollution Control Federation, Atlanta, GA, 1972.
4. A. D. Little, Inc. Salt, Safety and Water Supply: Toward an Improved Public Policy for Using Road Salt. Policy study prepared for the Special Commission on Salt Contamination of Water Supplies, Massachusetts General Court, 1972.
5. F. S. Adams. Highway Deicing Salts Are Potential Environmental Contaminants. Farm Economics, Pennsylvania State Univ., University Park, 1973.
6. H. McConnelly and others. Deicing Salts and the Environment. Habitat School of Environment for the Massachusetts Audubon Society, Belmont, 1972.
7. G. E. Garrison and O. L. Ader. Sodium in Drinking Water: Pitfall in Maintenance of Low Sodium Diet. Archives of Environmental Health, No. 13, 1966, pp. 551-553.
8. L. K. Dahl. Salt and Hypertension. American Journal of Clinical Nutrition, No. 25, 1972, pp. 235-244.
9. L. Page. Epidemiologic Evidence on the Etiology of Human Hypertension and Its Possible Prevention. Newton-Wellesley Hospital and Tufts Univ. School of Medicine, Boston, Mimeo, n.d.
10. L. B. Page, A. Demon, and R. C. Moellering, Jr. Antecedents of Cardiovascular Disease in Six Solomon Islands Societies. Circulation, No. 49, 1974, pp. 1132-1146.
11. Pointing the Finger at Salt. Newton-Wellesley Quarterly, Vol. 4, No. 1, 1973, p. 11.
12. More Proof of Salt-Hypertension Link. Journal of the American Medical Association, No. 237, 1977, pp. 1305-1307.
13. E. D. Freis. Salt, Volume and the Prevention of Hypertension. Circulation, No. 53, 1976, pp. 589-595.
14. Conquering the Quiet Killer. Time, Jan. 13, 1975, pp. 60-64.
15. Memorandum to Commissioner Jonathan E. Fielding from the Ad Hoc Advisory Committee on Recommended Sodium Level for Safe Drinking Water in Massachusetts, April 14, 1977.
16. A. H. Westing. Plants and Salt in the Roadside Environment. Phytopathology, No. 59, 1969, pp. 1174-1181.
17. R. E. Hanes and others. The Effects of Deicing Salts on Plant Biota and Soil—Experimental Phase. NCHRP, Rept. 170, 1976.
18. E. G. Button and D. E. Peaslee. The Effect of Rock Salt Upon Roadside Sugar Maples in Connecticut. HRB, Highway Research Record 161, 1967, pp. 121-131.
19. A. E. Rich. Effect of Deicing Chemicals on Woody Plants. In Symposium: Pollutants in the Roadside Environment (E. D. Carpenter, ed.), Univ. of Connecticut, Storrs, 1968.
20. A. E. Rich. Some Effects of Deicing Chemicals on Roadside Trees. HRB, Highway Research Record 425, 1973, pp. 14-16.
21. F. W. Holmes. A Partial Bibliography on Salt Injury to the Environment, Especially to Trees. Shade Tree Laboratories, Univ. of Massachusetts, Amherst, 1973.
22. Hardesty and Hanover. Report on the Inspection and Analysis of the West Side Highway (Henry Hudson Parkway) Battery to 42nd Street. Transportation Authority, New York City. Capital Project No. HW-19, 1975.

23. D. B. Feaver. Four Bridges to be Restored. Washington Post, Sept. 15, 1975.
24. Traffic Operation at Sites of Temporary Obstruction. Organization for Economic Cooperation and Development, Paris, 1973.
25. Vehicle Corrosion Caused by Deicing Salts. American Public Works Association, Chicago, Special Rept. No. 34, 1970.
26. E. S. Arvai. The Effect of Salt on the Number of Winter Accidents. Highway Safety Research Institute, Univ. of Michigan, HIT Laboratory Rept., 1971.
27. R. C. Anderson. The Economics of Highway Safety. Traffic Quarterly, Vol. 29, No. 1, Jan. 1975, pp. 99-111.
28. S. Peltzman. The Effects of Automobile Regulation on Safety. Journal of Political Economy, Vol. 83, No. 4, Aug. 1975, pp. 677-725.
29. R. D. Terry, Jr. Road Salt, Drinking Water and Safety: Improving Public Policy and Practices. Ballinger Publishing Co., Cambridge, MA, 1974.
30. R. Brenner and J. Moshman. Benefits and Costs in the Use of Salt to Deice Highways. Institute for Safety Analysis, Washington, DC, Nov. 1976.
31. H. L. Harrison. Overview Report: City-University Road Salt Study. Univ. of Wisconsin, Madison, June 1975.

Discussion

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This discussion is concerned only with the adequacy of the assumptions and the technical discrepancies in the vehicle corrosion portion of the original EPA document (32) from which Murray's paper was derived.

Murray assigns two-thirds of the $3 billion in annual damage, reported due to deicing chlorides, to vehicle corrosion damage. He represents this annual damage cost as the "very lower bound" of the total costs attributable to deicing chlorides, but it is important to recognize that his approach overstates the vehicle corrosion costs attributable to deicing salts.

The Ackerman approach (32, pp. 75-76) appears to be a reasonable method for determining differences in the depreciation rate. Whether the Anderson-Murray approach and the resulting regression equation are a valid distributor of the additional depreciation, developed as Ackerman does, is the principal point in question.

Before writing this discussion, a copy of the original data (33) from which the regression equation was developed was obtained. These data did not include mean January temperatures but otherwise corresponded with those described in the reports (32). Mean January temperatures were obtained (34, 35), and the data were then subjected to regression analysis by using BMD-02R program—stepwise regression—Rev 6/11/74. (This analysis was written in U.S. customary units.) The results of that regression analysis are as follows: depreciation rate equals 15.9418 plus 0.0469 state salt plus 0.0258 snow plus 0.1531 miles plus 0.0204 city salt. The multiple R for this equation is R = 0.9033, and the coefficient of determination is 0.816. Estimated standard error for state salt is 0.0156, for snow is 0.0068, for miles is 0.153, and for city salt is 0.0059. Other results follow.

Snow Versus Collinearity Correlation	
State salt	R = 0.64
City salt	R = 0.59

Depreciation Rate Versus Individual Correlation	
Snow	R = 0.81
State salt	R = 0.75
City salt	R = 0.75

Importance of four independent variables in contributing to the multiple R of R = 0.9033 follows (1 mile = 1.6 km).

Variable	R	R^2	Increase in R^2
Snow	0.812	0.659	0.659
City salt	0.877	0.770	0.110
State salt	0.898	0.807	0.037
Miles	0.903	0.816	0.009

The most obvious comment about these results is that if the assumptions on which the equation is based are valid, the authors had somewhat better data than they portrayed. According to these results state salt costs are increased by $167 million and city salt costs by $36 million.

The multiple R (0.79) is probably in error. In any case a multiple correlation coefficient of 0.79 does not imply that 79 percent of the variation in the dependent variable has been explained by the four independent variables used in their equation. It explains 62 percent, or 0.79^2. Note that in our cross check of the Murray-Anderson (regression analysis) data, a multiple correlation coefficient R = 0.90 was obtained.

In the regression analysis "the unit of observation is the city" (32, p. 77). Application of city regression estimators (depreciation rates versus city salt) to the state as a whole may be untenable, and the authors' justification for it is not obvious.

Regardless, the amounts used (32, Table 8, pp. 85-86) may well overstate the damages. For instance, in California, over 90 percent of the population is concentrated where deicing salts are not used. The 0.567 Mg/lane•km (1 ton/lane•mile) annual state salt and city salt figures seem a bit excessive.

The EPA document discusses 44 cities, 41 of which were incorporated in the regression analyses. According to the original paper, which was retained essentially intact in the EPA report, the 2 cities "Houston and New Orleans were deleted because of high rates of decay attributable to humidity and proximity to the ocean." Rainfall was substituted for humidity "because humidity rates proved difficult to obtain" (32, p. 81).

Which humidity data did the authors use to justify the deletion of these two cities? High depreciation rates in the absence of deicing salts are not a valid basis for their deletion. And if humidity is the basis for deletion, on which criteria were Tampa, Florida, and Charleston, South Carolina, retained? Both are in close proximity to the ocean and have high humidity rates, Tampa somewhat higher than either Houston or New Orleans, Charleston somewhat lower (35).

In citing others (36, 37, 38) and referencing Chance (39, 40) in the original study, the authors developed a skeletal technical perspective against which the validity of the regression equation assumptions could be evaluated. The technical data indicate that deicing salt and air pollution contribute about equally to corrosion damage and that a humid, air-polluted environment can produce four times the corrosion damage sustained in a dry, non-air-polluted environment. Note that even in a dry, non-air-polluted environment with no deicing salts there is still corrosion damage (36). The proxy variables, mean ambient January temperature, rainfall, and average annual sulfur dioxide concentration selected to represent the natural environment and air pollution did not contribute to the equation, but the technical data indicate that they should have. The variables selected, then, are probably inadequate.

Until this discrepancy is reconciled, the credibility of the regression equation and the costs derived from it are at best questionable. A more detailed study is required before costs can be ascertained by this approach.

The amount of corrosion sustained by a vehicle is not only a function of the corrosiveness of the environment in which the vehicle is operated, but also the amount of corrosion resistance built into the vehicle.

What the EPA report does not take into account is that, if in the process of being modified a product becomes less able to withstand the environment, is it valid to charge the losses resulting from this change to the environment and more specifically to one particular aspect of the environment? Furthermore, nowhere in the study do the authors deal with the long-term effects of the reduction in sheet steel thickness that accompanied the adoption of unibody construction by the American automobile industry. This construction was adopted on a wide scale in 1955. At that time the automotive industry's anticorrosion capability was low, and its relatively high anticorrosion capability achieved only recently. Whether the premature corrosion perforation of 1955 through early 1970 models is due largely to reduced metal thickness combined with inadequate anticorrosion technology is arguable (41).

No substantial basis for the $690 million in annual truck damage was developed. The arbitrary assignment of $30/truck based on the opinions of truck fleet managers is at best qualitative.

In summary, the approach the authors used has a great deal of potential. The correlations between salt and depreciation cannot be lightly dismissed. But the technical work cited indicates that correlations between air pollution and corrosion in the same range are also to be expected. Furthermore, no study to date has adequately dealt with the amount of corrosiveness attributable to the unpolluted base environment. The four to one corrosion ratio between a dry, non-air-polluted environment and a humid, air-polluted one indicates that the base environment does have a significant effect. The adoption of unibody construction has, to our knowledge, only recently been considered in this frame of reference and then only as a logical argument that does not assign costs. In our opinion, the costs developed in this section are excessive.

Although we have been extremely critical of the regression equation and the dollar costs developed therefrom, this was more in response to the unqualified use of the $2 billion figure in the conclusions than the method of approach. It is our belief that those responsible for the allocation of research efforts and funding, as well as others, rarely have the time or the desire to read qualifications buried in the text. As a result, unqualified, unchallenged conclusions, such as those appearing in the EPA-sponsored report, result in a distortion of effort. If, in fact, the EPA or some other group desires to determine and allocate the costs of vehicle corrosion, we would strongly recommend that consideration be given to the method developed by the authors, providing that the points stated in the discussion are adequately dealt with.

REFERENCES

32. D. M. Murray and U. F. W. Ernst. An Economic Analysis of the Environmental Impact of Highway Deicing. Environmental Protection Agency, EPA-600/2-76-105, May 1976.
33. BMD—Biomedical Computer Programs. Univ. of California Press, Los Angeles, Jan. 1, 1973.
34. The World Almanac and Book of Facts. Newspaper Enterprise Association, New York, 1975.
35. Climatic Atlas of the United States. Environmental Science Services Administration, Department of Commerce, June 1968.
36. H. J. Fromm. Corrosion of Auto-Body Steel and the Effects of Inhibited Deicing Salts. HRB, Highway Research Record 227, 1968, pp. 1-47.
37. Motor Vehicle Corrosion and Influence of Deicing Chemicals. Organization for Economic Cooperation and Development, Paris, 1969.
38. Vehicle Corrosion Caused by Deicing Salts. American Public Works Association, Chicago, Special Rept. No. 34, 1970.
39. R. L. Chance. Corrosion, Deicing Salts, and the Environment. General Motors Research Laboratory, Detroit, 1974.
40. Deicing Salts, Their Use and Effects. National Association of Corrosion Engineers, Houston, Publication No. 3N175, 1975.
41. M. C. Belangie. Vehicle Corrosion in Perspective. TRB, Transportation Research Record 651, 1977, pp. 48-52.

Robert Brenner, Institute for Safety Analysis

My colleague Jack Moshman and I recently published a paper on the benefits and costs in the use of salt to deice highways. In it, we examined the principal findings of the work by Murray in the original EPA study. We then developed our own model of the problem.

Murray deals only with the costs of the adverse effects of salt; he does not seek to estimate the economic value of the benefits associated with rapid deicing of highways. Clearly, the costs of adverse effects should be compared with the economic benefits, and community decision making on road salting should relate to the net difference.

I question the $2.91 billion estimate as the annual economic cost to snowbelt states resulting from the use of road salt. By changing some of the underlying assumptions and using different constants, we come up with something around a third of this estimate. But even if the higher figure is used, the benefits still far exceed the costs by a factor of six. In short, regardless of which assumptions are taken in numerical exercises such as Murray's or ours, the benefits of rapidly deicing roads are far greater than the costs. The complete comparative picture is unmistakable.

The fact that Murray assigns dollar values only to the adverse effects of road salting and a zero dollar value to the benefits is a transparent weakness. More disturbing than this, however, is the unclear safety logic, as expressed in this statement on safety that appeared in the presentation draft. "The use of salt for winter maintenance generally results in better traction on the highways, but because of a number of confounding factors, especially driver behavior, the link between salt and safety has not been proved." Improved traction, whether produced by better quality tires or by regrooving the pavement or by resurfacing sections with low coefficients of friction, means safer travel. Improved traction produced by salt means safer travel. Thus, the statement that salt improves traction but its link with safety is unproved is obviously self-contradictory.

He continues, saying that "While several studies have reported that salt reduces accidents the methods of data collection and analysis have been found to be mathematically unsound.

During the 30 years I have been associated with highway safety, the field has been crippled by statements to the effect that some obviously important safety (accident-preventing or injury-reducing) measure has not been proved to the satisfaction of some statistical purist. One can readily substitute "good brakes" or "properly trained drivers" for "salt": "While several studies have reported that good brakes reduce accidents, the methods of data collection and analysis have been found to be mathematically unsound." The absence of statistical proof of the link between good brakes and safety would hardly convince any rational person to venture onto a high-speed freeway in a vehicle whose brakes do not work.

Safety is the absence of hazards; hazard is the absence of safety. The hazard of iced-over highways is a fact, but, if statistical proof is required, it is a simple enough matter to examine hospital emergency room records or vital statistics records to determine what goes on when highways are iced over.

If salt is the fastest and cheapest way known today to reduce the duration of time over which highways are iced, then it also must be the fastest and cheapest way to reduce the hazard. There simply cannot fail to be a link between salt and safety, if in fact salt is the best way known today to accelerate deicing. Statements about the absence of statistical proof linking salt and safety are, along with similar statements on brakes and safety, empty.

Along with the statistical argument, Murray invokes the equally time-worn argument of driver behavior. For ages this has been dragged into accident analyses, even when the direct cause was some glaring engineering deficiency in the highway or the vehicle. But let us assume that driver inadequacy or error is a significant factor. We nevertheless must recognize that, in the highway field, we design to be forgiving of driver error. We do not pass off hazardous conditions by saying the driver should learn how to cope with them. We do not design roads with poor sight distance and leave it to the driver to adjust to the danger. We place guardrails in front of abutments or other exposed rigid objects to protect occupants in out-of-control vehicles, regardless of which factors might precipitate the loss of control—driver error, inadequacies of the vehicle's brakes or suspension, or inadequate skid resistance of the pavement.

Safety is apparently not Murray's primary concern in his classical statistical and driver error arguments, which have historically been used to attack some initiatives to reduce the highway death and injury tolls. Another illustration of his perspective may be seen in the $50 million estimate of annual salt-caused damage to roadside trees, which, in my view of safety, do not belong on road shoulders in the first place. Basic policy of the U.S. Department of Transportation and Federal Highway Administration calls for installation of yielding or breakaway sign supports precisely to eliminate the type of hazard the tree on the shoulder presents to occupants of the out-of-control vehicle. We design the highway environment to be forgiving of driver error, not punishing.

Notwithstanding the fact that there are many species of tree that are not harmed by salt runoff, the decision seems to me to be one of killing people or trees.

There are also highway planning flaws in Murray's work, illustrated by his $500 million estimate of the cost of highway bridge deck corrosion. Half of this is the value of the time motorists waste while bridge decks are being repaired. Not included in this particular entry in his bookkeeping is the value of delay motorists would incur in safely negotiating iced-over bridges. I know of few budget requests—federal, state, or local—for highway capital improvements that do not enter expected benefits in the budget request.

Finally, there is the $2.91 billion as an indicator of the total cost of the environmental impact of highway deicing. Vehicle corrosion accounts for about 74 percent, or $2 billion, of this amount. I question the accuracy of this estimate, but let us assume it is correct. If we add to this the $500 million for bridge deck damage and to the $200 million for the cost of salt and its application, we get about $2.7 billion as the nonecological salt damage, leaving a remainder of $200 million as the cost of ecological damage. This puts the ecological damage from road salting of $2.91 billion off by a factor of about 14.

Few ecologists, I suspect, would consider the automobile and the highways on which they operate as essential to the environment. It is interesting that the automobile seems to have new stature in the environmental protection community.

In summary, the Murray work seeks to assess the costs to society of road salting. I question both the accuracy of the figures and the underlying assumptions of the model, which contains other fundamental peculiarities. However, whether his costs are correct or not is not of great consequence. What is important is that whatever these costs might be, they must be considered in relation to road salting benefits, particularly as no better method for controlling iced-over highways is known today.

Author's Closure

This closure was written in part by Robert Anderson, who was chiefly responsible for the development of the regression analysis.

DISCUSSION BY BELANGIE AND SY

Belangie and Sy have focused their three contentions solely on the method used to estimate vehicle corrosion costs. First, total vehicle corrosion costs can also be derived from an analysis of regional variation in used vehicle prices. Second, the analysis we used to determine the relative contribution of the separate factors leading to vehicle corrosion is incomplete and as such may attribute excessive costs to deicing chemicals. Third, the regression results reported in the EPA report may have contained an error (or errors).

In response to the first point, we observe that the regional variations in used vehicle prices are at best a lower limit on the true losses in economic value. Many used vehicles driven in environments hostile to them (deicing salts, humidity, sunshine, salt spray near oceans) are retired prematurely and never offered for resale simply because the value of the vehicle does not warrant it. Scrapping the vehicle is economically more attractive.

When looking at used automobile prices in a region, one should try to incorporate the near zero value of all prematurely scrapped vehicles. Our approach of looking at advertised asking prices understates the true depreciation of vehicles in these hostile environments.

We also note that most vehicles are not owned and operated in a single city or state for the entire life of the vehicle. Rather, vehicles are exposed to many different environments. The location where the vehicle is offered for sale may or may not be the principal environment in which it was driven. Again, this would cause an understatement of the true depreciation brought about by a hostile environment.

The second point made by Belangie and Sy is that other researchers have noted definite correlations between atmospheric pollution and corrosion and between humidity and corrosion. The fact that these variables were not significant in our regressions implies, in their view, that our equations are deficient—and likely to be attributing excessive portions of the total corrosion to deicing salts.

We agree that one would expect to find humidity and air pollution linked to variations in depreciation rates across cities. We did not measure such an effect because, in our view, the relatively crude data we used did not adequately describe the actual humidity and air pollution to which the vehicle had been exposed during its life. Were much more careful research to be done, the true variations in exposure to pollution, humidity, and other causes of increases in depreciation rates should be measurable.

In any case, the fact that air pollution and humidity did not appear as significant determinants of vehicle depreciation rates does not mean that our estimates for the effect of deicing salts would be overstated. Our estimates are overstated only if the use of deicing salts is positively correlated with either humidity or air pollution.

Although others may want to debate this point, we do not believe that the use of deicing salts is in fact positively correlated on a regional basis to either humidity or air pollution levels.

Therefore, we stand by our estimates as accurate allocations of the measured depreciation rates to the separate determinants of vehicle decay.

Finally, Belangie and Sy raise some concerns about our numerical estimates in the regression equation. Their estimates are based on our data for 41 cities. The EPA report was based on essentially the same data, but for 39 cities. Data on the other 2 cities became available only after the EPA report went to press. As noted by Belangie and Sy, the inclusion of the other 2 cities does increase the coefficients of the two salting variables somewhat, thereby raising the total estimated cost of vehicle corrosion attributable to deicing salts by over $200 million to a total of $2.2 billion.

As far as errors are concerned, there was only one. The Multiple R (0.79) on p. 83 of the EPA report was a misprint and should have been 0.89, thus implying that 79 percent, or $(0.89)^2$, of the variation in the dependent variable is explained by the four independent variables. (Belangie and Sy obtain R = 0.90, apparently because of the two additional data points.)

We reiterate our principal finding that our estimate of the cost of vehicle corrosion attributable to deicing salts is most likely biased downward. The true cost probably exceeds our estimates by a significant margin. Because the budget for the EPA study was very limited, we do not feel that our data are the best obtainable. We would encourage others to continue this line of inquiry in order that the true costs of vehicle decay attributable to deicing salts be estimated and used as one of the key inputs in the formulation of government deicing policies.

DISCUSSION BY BRENNER

In response to the discussion by Brenner, the EPA study was restricted to the analysis of the costs of damage to the total environment, man-made as well as natural. Assessment of benefits was not included, and therefore Brenner's statement that we assigned a zero value to the benefits is misleading. Our only role with regard to benefits was to review the research that had been done.

I have never contended that salt has no safety benefits, but I strongly object to the use of erroneous statistics as proof that "salt and safety are synonomous."

Brenner arrives at a significantly lower cost estimate principally by using other data for used automobile prices and no control for regional variation of factors. This method was discarded by most researchers years ago.

With regard to the magnitude of the benefits relative to the costs, the statement that the benefits are far greater than the costs gives no indication that the right amount of salt is being used. In fact, it might be that far too much is used. In order to determine the best amount, one would need to examine the marginal effect of each unit of salt.

Damage to vegetation is not restricted to trees on the shoulder but may extend 20 to 30 m from the edge of the shoulder (or much more if the runoff is directed away from the road). Furthermore, in urban areas, roadside trees that have died from excessive salt are usually replaced.

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Economic Impacts of Snow on Traffic Delays and Safety

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This paper outlines the effects of snow and snow removal on four types of urban and rural highways that were studied in a 4-year project sponsored by 12 "snow" states and the Federal Highway Administration. The project identified and calculated such road-user costs as delay, volume and speed reductions, and fuel consumption; business costs included losses from such things as absenteeism, tardiness, and spoilage.

State and local governments expend considerable manpower and money annually on snow- and ice-control programs (1). These expenditures on road surface conditions consume as much as 33 percent of some state highway maintenance budgets, although the conditions prevail for only 4 to 8 percent of the total time these roads are in use. Expenditures of this magnitude have traditionally been justified as improved road-user benefits, usually as increased safety and decreased traffic delay during storms.

Increasing snow- and ice-control efforts has immediate consequences on delay, traffic congestion, and the public image of the highway department itself. Some of the effects of this effort are temporary in nature, and a return to normal comes relatively soon; other effects, such as bridge deck damage, vehicle corrosion, and environmental impacts (2), however, are more permanent or even irreversible.

It has traditionally been assumed that the economic effects of snow-control practices on non-road users are negligible compared with those on road users. In terms of benefits, this is probably true. A study undertaken by the American Public Works Association (3, p. 67) to formulate a method for controlling snow and ice resulted in an analysis for comparing the relative merits of varying cost levels. Potential benefits to a community were examined before developing the analysis, but, as finally outlined, road users reaped the major benefits. This paper focuses on business and personal economic losses and delays caused by snow.

PURPOSE OF THE PROJECT

The object of this research was to provide decision makers with the necessary tools to formulate snow- and ice-removal policies that use the resources available. It was also designed to help make decision makers aware of the impacts of snow- and ice-removal policies and their effects on the community and individuals.

Savings in vehicle operating costs are provided by higher levels of snow and ice control. For automobiles, such savings generally mean time saved by driving faster (4). Time savings or losses associated with snow and ice should not be handled in the same way as tangible costs or savings. In general, estimates of the value of time are averages, because time has different values to different people and to the same person on different occasions (4). It is also held, and generally acknowledged, that minute time savings are of less unit value than time savings of considerable amount. For a million people to save 1 min of an hour's trip does not produce the same dollar savings as 100 000 people each making a 10-min savings.

DELAY

Normal Traffic Volume

The difference in traffic volumes between normal and snow conditions will influence the evaluation of the costs of total traffic delay; these two volumes were therefore measured. We selected 12 test locations, where normal traffic volumes had been previously recorded by permanent 24-h traffic counters, and then categorized these normal volumes according to hour of day, day of week, and highway type.

Snow Traffic Volume

Snow traffic volumes were obtained hourly as the number of vehicles crossing continuous traffic counters at the same test sites used for measuring normal volumes. The best estimate of snow volumes was statistically determined by using multiple linear regression analysis with the independent variables

1. Normal traffic volume,
2. Hour of day,
3. Day of week,
4. Mean hourly temperature,
5. Maximum hourly temperature,
6. Minimum hourly temperature,
7. Peak hourly snowfall rate,
8. Second peak hourly snowfall rate,
9. Storm duration up to peak hourly snowfall rate,
10. Storm duration after peak hourly snowfall rate,
11. Total storm duration,
12. Snow accumulation up to peak hourly snowfall rate,
13. Snow accumulation after peak hourly snowfall rate,
14. Total storm snow accumulation,
15. Normal traffic volumes for 5 h before beginning of storm,
16. Normal traffic volumes after storm ends,
17. Change in traffic volumes for 5 h before beginning of storm,
18. Changes in traffic volumes for 5 h after end of storm.

Multiple linear regression analysis and a correlation coefficient matrix for the variables were completed. Regressions were determined on an hourly basis, by five time categories during the day, on a storm basis, for urban highways, and for rural highways.

Highway Delay

Vehicle delay measurements were evaluated for four highway classifications: urban Interstate, rural Interstate, urban secondary, and rural secondary. On rural highways (classified as highways where each driver determines his or her own travel speed), normal speeds are generally higher than on urban highways (classified as highways where the traffic flow determines each vehicle's speed). Because we anticipated that average normal and snow-induced speeds will vary depending on traffic density on any particular highway facility, we analyzed these highways separately. A highway segment could be defined as rural or urban at different times depending on the traffic flow characteristics.

Travel Time and Trip Length

Normal travel time for a particular highway segment is defined as the mean travel time on that segment. For the purpose of evaluating the effects of snow and ice control, normal travel times were estimated, as already stated, for each hour of the day and each day of the week.

Snow-induced delay is the additional time, beyond normal travel time, required for a trip over the highway segment being evaluated. The length of this delay for a particular highway segment equals total delay (Equation 1) multiplied by the ratio of highway segment length to average trip length.

Seven test sections were evaluated for delay. Vehicle speeds on them during snow conditions were monitored at various time intervals during the day. In addition to vehicle speeds during storms, we recorded rate of snowfall, total snow accumulation, pavement surface condition, snow control effort, traffic flow characteristics, ambient air temperature, and percentage of trucks.

All categories for traffic delay are functions of the length of time delay. Delay for any one vehicle is given by

$$\text{Delay} = \text{trip length } [(1/\text{snow speed}) - (1/\text{normal speed})] \quad (1)$$

Speed under snow conditions will vary from vehicle to vehicle, just as it does under normal conditions (dry road). Delay will also vary according to the probability distributions of snow speeds and normal speeds. (Trip length too will vary,' but, because of insufficient data for defining its probability distribution, we assumed it to be constant for given applications.)

Dry road speeds have been observed to generally follow a normal distribution, and we assumed that speeds under snow conditions will follow this same distribution. Accordingly, the probability that the delay D is less than a given value W is

$$F_D(W) = \int_0^{w/t} \frac{1}{2\pi\sigma_S\sigma_N} \int_0^{\infty} \frac{1}{y^2(y+t)^2} \exp\{-0.5[1 - \mu_S(y+t)^2]/S^{(y+t)}\} \left(\exp\{-0.5[(1 - \mu_N y)/\sigma_N y]^2\}\right) \quad (2)$$

where

- t = average trip length,
- σ_S = standard deviation of vehicle speeds for snow conditions,
- σ_N = standard deviation of vehicle speeds for normal road conditions,
- μ_S = mean value of vehicle speeds for snow conditions,
- μ_N = mean value of vehicle speeds for normal road conditions, and
- y = dummy variable.

The function $f_D(W)$ is the density function associated with this delay and is given by

$$f_D(W) = (d/dW)F_D(W) \quad (3)$$

Equation 3 was derived for this study and has been checked by simulation. It enables us to calculate expected comfort, convenience, and lost wage costs, none of which could be done accurately from the single value obtained in Equation 1 by using the mean values for snow and normal speeds.

Mean delay can be computed by

$$\text{Mean delay} = W[f_D(W)dw] \quad (4)$$

The total delay for all vehicles traveling on a given roadway segment for a given hour can be calculated by multiplying mean delay by the monthly, daily, hourly, and volume reduction from snow factors and by roadway segment length and then dividing by average trip length. Then all hours of the storm are totaled.

Traffic Speed Reductions

Figure 1 is a typical representation of normal traffic speeds, snow-induced traffic speeds, hourly snowfall rate, and accumulated snow depth according to time of day on an urban Interstate highway. Before snow began to fall, traffic speeds on the high-density highways began to drop, indicating that impending poor pavement conditions affected traffic speeds. The actual amount of snowfall or accumulation did not significantly affect speeds in the range we evaluated. The presence of snow, however, does seem to reduce speeds by approximately 46 percent.

In Table 1 the speed reduction percentages for an Interstate (62 000 average daily traffic) and a secondary highway (18 000 average daily traffic), under varying snow conditions, are given as an example.

It was not possible to vary the level of service on the test sections experimentally. As a level of service indicator, the differences in roadway surface conditions noted for separate highway segments were dry, wet, wet and snowing, wet and slushy, slushy and sticking, snowing and sticking, or snowing and packed. Thus level of service reduced to the best roadway surface the snow removal operators could achieve.

Figure 1. Snowstorm characteristics and normal speed gradients for urban Interstate highway.

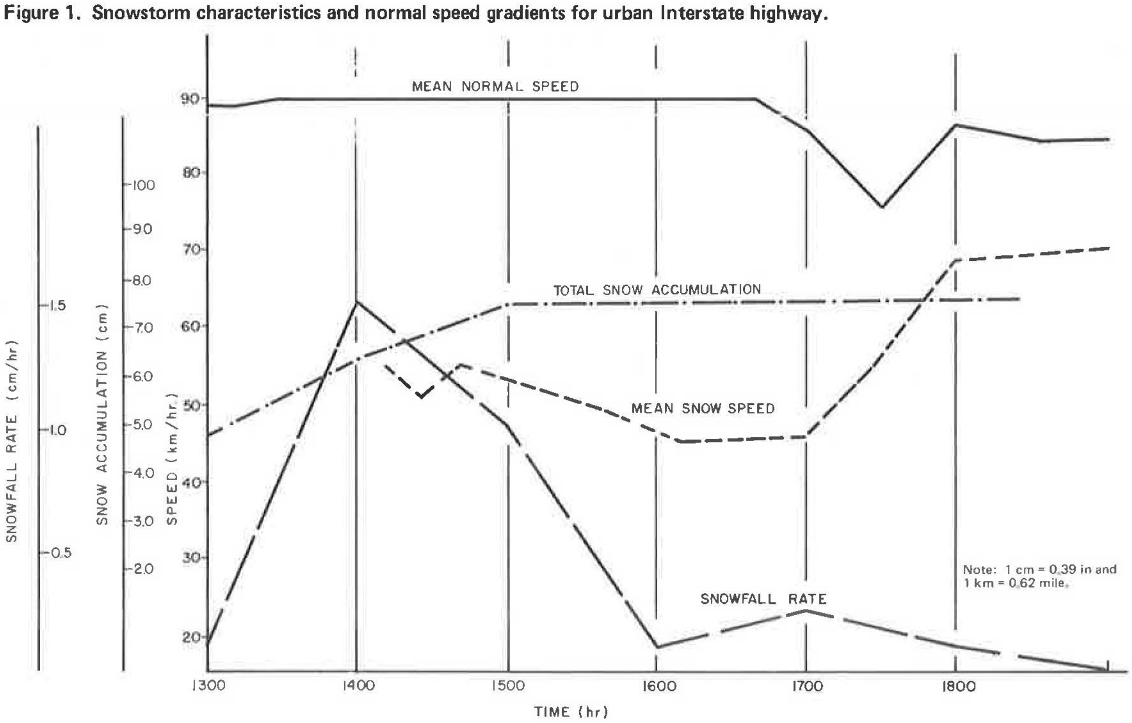
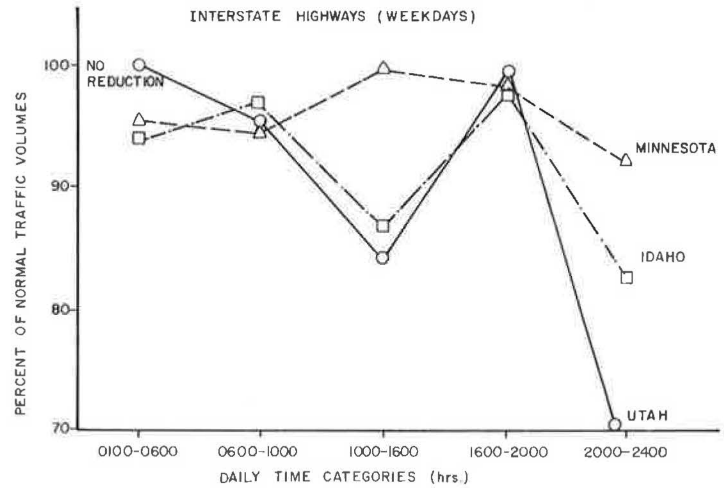


Table 1. Speed reductions and road surface conditions.

Road Surface Condition	Interstate Highway, Uncongested: No. Cars Observed	Reduction (%)	Standard Deviation	Interstate Highway, Congested: No. Cars Observed	Reduction (%)	Standard Deviation	Secondary Highway: No. Cars Observed	Reduction (%)	Standard Deviation
Wet only or dry	19 345	0	—	12 300	16	—	13 200	0	—
Wet and snowing	997	13	5.0	398	21	4.5	676	17	5.2
Wet and slushy	3 888	22	5.1	292	36	4.9	2 978	21	5.1
Slushy and sticking	1 548	30	5.1	518	35	4.8	831	26	4.7
Snowing and sticking	2 194	35	5.2	—	—	—		—	—
Snowing and packed[a]	1 696	42	4.2	—	—	—	—	—	—

[a]1 cm (0.039 in) or less.

Figure 2. Snow traffic volumes as a function of normal traffic volumes on Interstate highways.



Traffic Volume Reductions

Total traffic volumes changed during snowstorms, and traffic volume peaked at different times than during normal conditions. Traffic peaks during a snowstorm also tended to be flatter and longer (Figures 2 and 3) than during normal conditions. A typical volume change is summarized in Figure 4. Volume reductions for different road types were calculated by regression analyses. The equations resulting from the analyses and used to calculate hourly snow traffic volume reductions (HSTVR) are summarized in Table 2, where a is the correlation coefficient and b and c are the individual highway segment constants.

ROAD-USER COSTS

Fuel Costs

Economic estimates for vehicle fuel consumption on snow- and ice-covered highways are derived from Claffey (5). Road surface condition is a significant variable affecting average vehicle fuel consumption and is characterized as very slippery hard-packed snow and ice, hard-packed snow on ice with irregular, bumpy, wrinkled surface, 1.27 cm (½ in) of snow on hard-packed snow, 1.9 cm (¾ in) of snow on hard-packed snow, 2.54 cm (1 in) of snow on hard-packed snow, 3.81 cm (1½ in) of snow on hard-packed snow, or 5.08 cm (2 in) of snow on hard-packed snow.

Dividing the average delay for a particular storm on a highway segment or network into the corresponding highway segment length yields the average operating speed required for a given pavement condition. By applying Claffey's charts, the average operating speed is converted to fuel consumption. Excess fuel consumption yields total fuel consumed in excess of normal consumption. This number converts to a dollar figure by multiplying by the fuel cost. Fuel consumption varies with travel speed, which has been accounted for in Claffey's work.

Fuel consumption resulting from travel on snow- and ice-covered pavements is applied to passenger vehicles, light trucks, and vans. Because we assumed that snow-related fuel consumption for large trucks and tractor-trailer combinations depends on the size of load being hauled, not on pavement condition, we used only percentage of automobiles as an input for estimating additional fuel consumption on snow- or ice-covered pavements. The level of service in terms of snow or ice removal, then, influences fuel-consumption rates of automobiles, and the highway policy and modifications or alterations of it affect the economics of fuel consumption by inducing corresponding changes in surface conditions.

Operating Costs

Vehicle operating expenses such as tires, oil, and vehicle maintenance are not included as a function of snow and ice control. How much of these three items individual automobiles require is usually derived from the distance driven, not the time required to do so.

For commercial vehicles such as trucks, the value of a trip is based on operating time, as are some preventive maintenance activities. For these vehicles, operating costs are assumed to be a linear function of excess delay time.

Fixed Costs

Fixed costs here are insurance, depreciation, taxes, drivers' wages, drivers' welfare, workmen's compensation, social security, and registration. Fixed costs are generally a yearly expenditure not directly dependent on distance driven or the time required. For this reason, automobile costs do not include a portion of fixed costs for snow and ice conditions. Commercial vehicles are assumed to have a profit that either is productivity dependent or varies according to the number of trips that can be made per year. Costs for these vehicles are therefore included.

Additional delays for truck and commercial carriers traversing a highway segment under various snow and ice conditions are estimated from values provided elsewhere (6).

Comfort and Convenience Costs

Comfort and convenience costs are not direct, measurable economic costs, but their importance cannot be ignored. These costs are figured as time delay or time of inconvenience for a motorist. We adopted Thomas and Thompson's (7) evaluation approach, which takes these costs as the amount of money people would pay to

Figure 3. Snow traffic volumes as a function of normal traffic volumes on secondary highways.

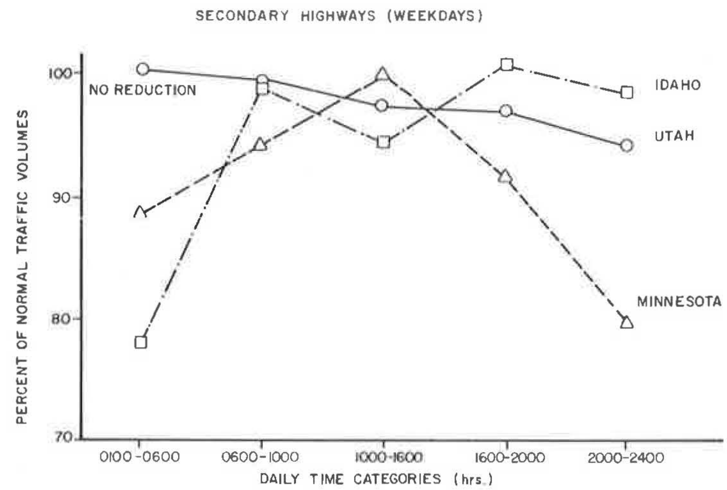


Figure 4. Traffic volume reduction on urban Interstate during a snowstorm.

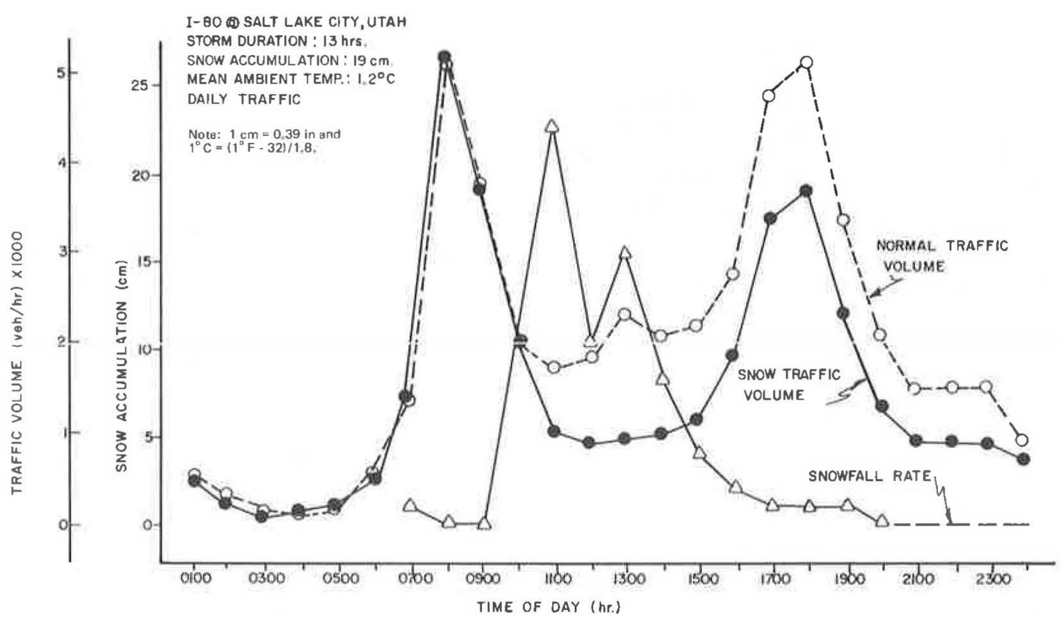


Table 2. Volume reductions from regression equations for various highway types.

Highway Type	State	Average Daily Traffic (1974)	No. Lanes	Weekday HSTVR a	Weekday HSTVR b	Weekday HSTVR c	Weekend HSTVR a	Weekend HSTVR b	Weekend HSTVR c
Urban interstate	Utah	110 000	6	0.80	712	0.61	0.88	153	0.94
Urban interstate	Utah	122 000	6	0.80	712	0.61	0.88	153	0.94
Rural interstate	Idaho	33 800	4	0.80	143	0.78	0.72	–	0.97
Urban interstate	Minnesota	190 000	8	0.91	-122	1.42	–	–	–
Urban interstate	Minnesota	130 000	6	0.72	-482	1.09	–	–	–
Urban secondary	Utah	40 800	4	0.99	–	0.95	0.98	–	0.97
Urban secondary	Utah	34 000	4	0.99	–	0.95	0.98	–	0.97
Rural secondary	Idaho	7 556	2	0.99	-20	1.12	0.87	24	0.76
Rural secondary	Minnesota	48 000	4	0.65	202	0.77	–	–	–

avoid a given delay. An example of personal delay or discomfort cost as perceived by a driver is given in Figure 5. The slopes, used with the probability density function for delay, derive estimates for comfort and convenience costs.

BUSINESS LOSSES

Interviews were conducted with businesses to estimate their losses resulting from snow and ice conditions. For this, the categories considered include absenteeism, tardiness, production losses, deferred sales, spoilage, and recreational losses.

Business types subject to deferred sales, such as car dealers and clothing stores, were found not to suffer permanent economic losses from highway snow and ice control.

Some companies responding to the questionnaire indicated lost sales, but they were not able to identify the magnitude of these losses.

Absenteeism

Large storms can create significant absenteeism, but industry often compensates for this by using employees' sick leave. It is therefore not clear if snow-caused absenteeism does result in real costs to industry that are not accounted for in production losses.

Tardiness

In order to quantify tardiness induced by snowy highways, we need to know the percentage of traffic comprising work-oriented trips. This percentage is a function of time of day, day of week, and highway characteristics.

We standardized length of delay and real tardiness times and their corresponding economic impacts by using questionnaires sent to retail sales companies, wholesale manufacturing companies, mining companies, agriculture cooperatives, small businesses, and large supermarkets. This questionnaire solicited information regarding normal tardiness rates, tardiness rates for specific storm days, and the economic impacts of tardiness on both employer and employees.

The cost of snow-caused tardiness per person can be approximated by lost wages. We questioned industrial engineers of a large national company on their general policy for snow-caused tardiness and found that an employee is docked in pay if late beyond a specific time unless he or she is habitually late. In either case, there is some threshold below which no pay would be deducted. Our estimate of the value for lost wages follows Figure 6.

The expected value of lost wages per worker is then given by

$$\text{Lost pay (K)} = [\text{wage (K)}/60]\left(\int_{TH(K)}^{\infty} Wf_D(W)\,dw - TH(K)\{1 - F_D[TH(K)]\}\right) \quad (5)$$

where wage (K) is the hourly wage for employees in industry type K and TH(K) is the tardiness threshold in industry type K.

The total value of wages lost by tardiness in industry type K is then given by an expression similar to Equation 5 by using the proportion of workers in industry type K, the proportion of these going to work rather than to other destinations, the amount of tardiness attributable to snow conditions, and the threshold value t, below which tardiness has no economic value.

Figure 5. Personal discomfort cost as perceived by motorists.

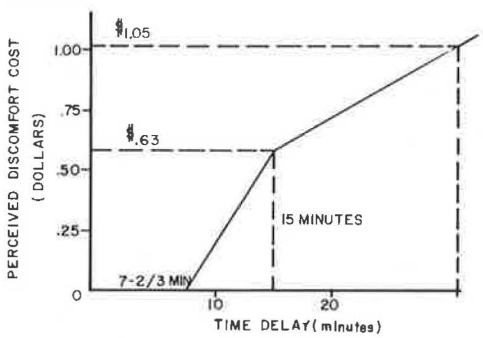
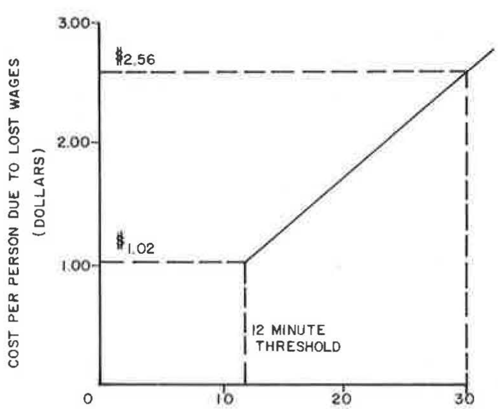


Figure 6. Lost wages per worker as a function of tardiness.



Production Losses

Very few industries indicated a production loss as a result of inclement weather or snow-covered highways. In the case of assembly-line work, an employee is not allowed to leave the job until his or her replacement has arrived. For non-assembly-line production, weekly or monthly quotas are standard; daily quotas would be more affected by snow- and ice-induced absenteeism. Some industries, however, did report higher overtime expenditures for extended cold periods.

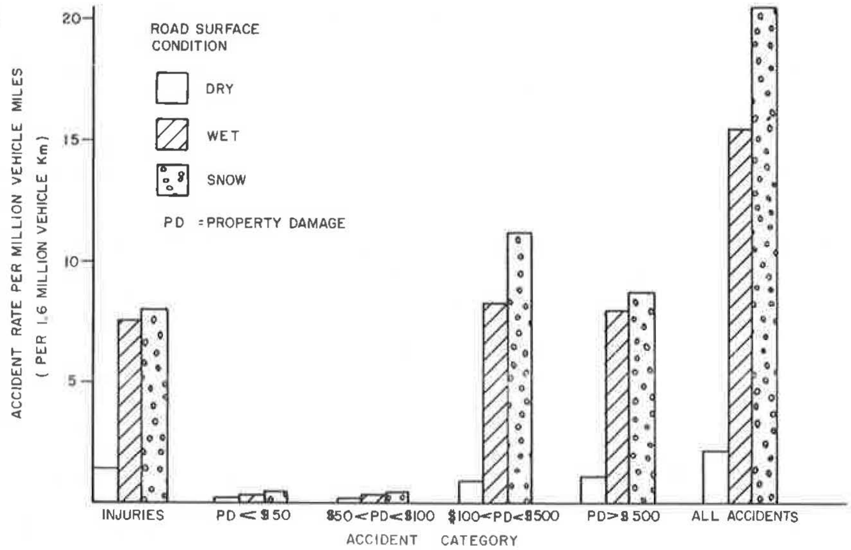
Deferred Sales

Responses from all retail companies indicated an awareness of sales fluctuations on snowy days, but no company gave information or had isolated sales for bad weather conditions. Approximately half of the responses indicated that most sales would be made up on a later date (deferred sales). Thus the quarterly sales index is not significantly reduced. Sales losses resulting from perished goods and impulse buying are not recoverable at a later date, but the magnitude of these losses was not known. Deferred sales, however, comprise the majority of retail expenditures.

Wholesale Sales

Wholesale suppliers of non-perishable goods generally inventory stockpiles on either a quarterly or a demand

Figure 7. 1973 urban accident rates in Utah.



basis. Thus delivery fluctuations on a daily scale as a result of snow do not seem to alter quarterly sales. The majority of wholesale sales losses also fall into the deferred sales category.

Spoilage Costs

Highways that serve as supply routes for perishable products can cause economic losses as a result of snow- and ice-caused closures. Especially susceptible are products delivered daily such as eggs or milk. The dairy industries who returned questionnaires indicated that spoilage was not a major problem and that only on rare occasions did farmers have to dump their milk because of shipping delays. Present snow removal practices have essentially eliminated losses associated with snowstorms, and any losses are associated only with extreme storm conditions.

Wholesalers of perishable goods reported that produce may freeze on trucks during bad weather conditions. They did not distinguish between highway conditions or the cold weather as causing these losses, nor did they indicate the amount of these losses.

Recreational Losses

All recreation industries polled, such as restaurants, theaters, and sporting events, with the exception of sporting franchises with seasonal ticket sellouts, reported a definite decline in business on stormy days. Estimates ranged from 20 to 50 percent reductions on bad days. It was not clear whether the decline was the result of bad weather or anticipated roadway surface conditions.

ACCIDENT ANALYSIS

We selected 21 test sections and compared accident rates under dry road conditions with those under snowstorm or wet and slippery conditions. For each section, traffic volume under normal conditions was estimated by multiplying the average annual daily traffic (AADT) by the monthly volume factor and the number of days in that month. Traffic volumes accumulated during snow and wet conditions were then subtracted, which left the traffic volumes for dry pavement conditions.

Where records indicated that 1.37 cm or more snow had fallen on a given day, the hours during which snow had been falling were determined. Traffic volumes accumulated during these snow hours were then estimated by multiplying the AADT, monthly factor, and the hourly factor and totaling these over the hours of the storm. These volume estimates were then reduced by regression-derived factors relating snow traffic volumes to normal traffic volumes.

Before analyzing accident severity and frequency, one must identify roadway segment length and number of accidents. All highway segments chosen had no major interchanges, weaving areas, or other geometric configurations that would add variables affecting accident rates. In the control segments, of a total of 539 accidents, 248 were on dry roads, 128 on wet roads, and 163 on snow-covered roads.

The severity or type of accident was thought to relate to pavement condition. Therefore, from the accident reports filled out by the investigating officer at the accident site, the estimated severity was classified as property damage less than $50, more than $50 but less than $100, more than $100 but less than $500, or more than $500, and as personal injury (total number per accident), and as a fatality (total number per accident).

Accident rates were determined separately for property damage accidents and for those involving injuries or fatalities. This allowed us to use the same accident twice, because accidents involving injuries or fatalities inevitably involved property damage as well.

Figure 7 illustrates the 1973 urban accident rates in Utah on dry, wet, and snow-covered pavements. Distinctions among property damage, injuries, and fatalities are also shown.

For the economic analysis of this study, estimates of fatalities and injuries were made with no attempt to derive their economic consequences. It was intended that the safety aspect of snow and ice control would be understood on its own merit. Safety together with economics serve to moderate the administrators' decision processes through more complete awareness. The relative importance of the two must be evaluated separately for each application.

CONCLUSIONS

The influence of snowstorms on traffic and traffic patterns was noted to be more pronounced during the first third of a storm. Traffic disruptions seemed to taper off as the storm progressed. This may not hold true for storms of abnormally long duration.

The safety aspect of snow and ice control becomes a minor economic effect for short highway segments or highways without extremely high traffic volumes. Many vehicle-kilometers must be traveled during and shortly after a storm for the small difference in accident rates for various road surface conditions to have a measurable effect.

User savings as a result of snow- and ice-control activities should be made by means of a comprehensive economic analysis that includes the costs of providing higher levels of service. The most extreme service for snow and ice control will still result in user costs. It is not anticipated that for every dollar saved in user costs, a dollar's expenditure is justified to control snow- and ice-covered highways. An incremental benefit-cost analysis is required to determine the point of diminishing returns. Even in this case, the costs summarized should be used as a tool to tender responsible administrative decisions.

REFERENCES

1. S. Kinosita and E. Akitaya. Classification of Snow and Ice on Roads. Snow Removal and Ice Control Research. HRB, Special Rept. 115, 1970.
2. B. H. Welch and others. Economic Impact of Highway Snow and Ice Control State-of-the-Art. Federal Highway Administration, Rept. FHWA-RD-77-20, Sept. 1976.
3. B. C. Butler Research Associates. Economics of Snow and Ice Removal in Urban Areas. American Public Works Association, Special Rept. Project 114, Ohio State Univ., 1965.
4. Highway Finance and Benefits. HRB, Highway Research Record 138, 1966.
5. P. J. Claffey. Running Costs of Motor Vehicles as Affected by Road Design and Traffic. NCHRP, Rept. 111, 1971.
6. W. G. Adkins, A. W. Ward, and W. F. McFarland. Values of Time Savings of Commercial Vehicles. NCHRP, Rept. 33, 1967.
7. C. Thomas and G. I. Thompson. The Value of Time by Trip Purpose. Stanford Research Institute, Menlo Park, CA, SRI Project MSU-7362, FH-11-6881, Oct. 1970.

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Impact of Highway Deicing Salts on Rural Stream Water Quality

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This study examines chloride concentrations in small rural streams receiving runoff from highways treated with deicing salts. Sampling points were established near Jamesville, New York, on Butternut Creek and some of its smaller tributaries. Higher mean chloride levels were found downstream of the highway than upstream. Downstream dilution reduced mean chloride levels in approximately inverse proportion to the additional watershed area; the higher variability was still evident. The chloride level in highway runoff was correlated by linear regression with the level in the receiving stream. Other relations were developed by investigating downstream dilution, stream confluence, temperature, and recent salt applications. Precipitation and temperature seem to act as controls on the release of salts from the highway area into natural drainage systems. There are also indications that much salt can be temporarily stored in the roadway vicinity until it rains. Intensive sampling showed that chloride concentrations can vary significantly in a matter of hours. And, when salt infiltrates the soil, stream chloride levels vary long after the season of salt applications.

The purpose of this study is to provide quantitative, preliminary data for assessing the impact of highway deicing salts on rural stream water quality. Published data of this type are currently rather limited.

Samples taken from rural streams both upstream and downstream of a salted highway were checked for differences in chloride levels attributable to highway runoff. Also investigated were chloride levels in the highway runoff. Also investigated were chloride levels in the highway runoff itself, the effects of dilution as watershed area increases downstream of the highway, and environmental factors affecting salt delivery to the streams.

STUDY AREA DESCRIPTION

The study area is in the Butternut Creek watershed, just east of Jamesville, New York. I investigated Butternut Creek, which has a drainage area of 48.2 km^2 (18.6 miles^2) at the sampling sites, and two small tributaries, stream I, which has a watershed of 1.8 km^2 (0.7 mile^2), and stream II, which drains 1.8 km^2 (0.7 mile^2). These tributaries were further subdivided into basins Ia and Ib, and IIa and IIb in Figure 1, where the sampling sites are the lettered circles.

Butternut Creek in this area runs through a glacial valley on the northern fringe of the Allegheny Plateau. Butternut Valley has a local slope of about 1 percent; the tributaries sampled slope from 5 to 10 percent diagonally down the valley side. All the streams studied receive deicing salt from US-20, which traverses the area in an east-west direction.

Land use is primarily agricultural, but there are woods and permanent pastures, where slope or soil or both precludes efficient cultivation. The area is free of high-density land uses, and residential development is confined to scattered one-family dwellings.

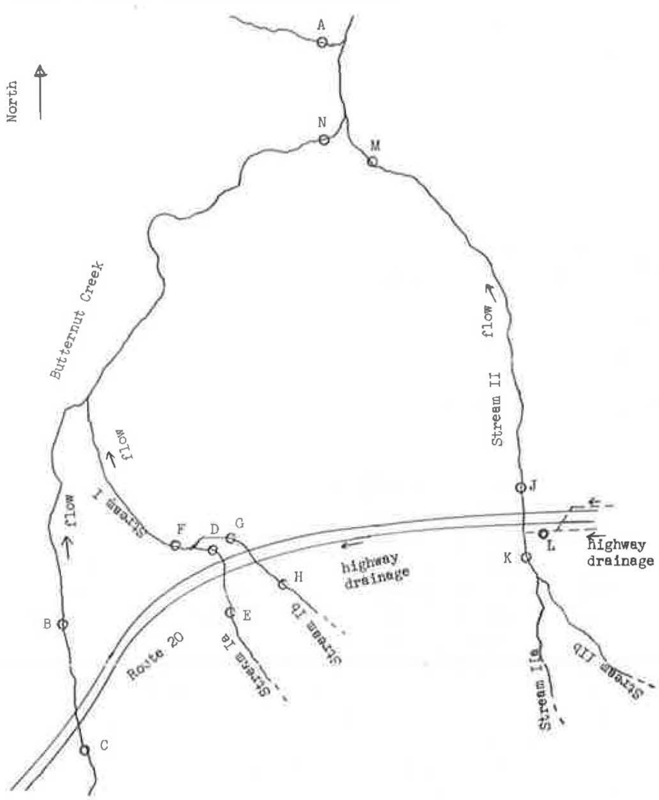
Stream I's watershed includes 0.8 km (0.5 mile) of US-20 and 1.1 km (0.7 mile) of unpåved town road. Stream II's watershed has 1.3 km (0.8 mile) of US-20, 1.7 km (0.8 mile) of paved county road, and 0.8 km (0.5 mile) of unpaved town road. Deicing salt effects from the town and county roads near the upper watershed boundaries are considered minimal.

SAMPLING ANALYSIS AND COLLECTION SITES

Samples were collected on 43 occasions, from January 25 to July 12, 1975, at 14 sites, but not at every site on every occasion. Originally, I intended to concentrate on a stream the size of Butternut Creek, but early results indicated no measurable impact from the US-20 effluent. Thus, most of the conclusions of this study are based on studying the small tributaries.

I chose to test with a chloride field test kit that uses potassium chromate for the indicator reagent and silver nitrate for titration. I was concerned not with minute changes in salt concentrations but with grosser changes that have greater potential impact on stream communities. I took 10 ppm as the lower limits of this testing procedure (1) and as indicative of 10 ppm or less. Samples containing more than about 1000 ppm were diluted to 1 in 4 or 1 in 10 with distilled water for analysis.

Figure 1. Sketch of principal sampling sites.



Climatological data from stations located about 13 km (8 miles) away were used as indicators of general conditions and trends. U.S. Geological Survey flow data from a gauging station on Butternut Creek about 3 km (2 miles) downstream of the study area were obtained as a flow regime indicator.

Highway salt inputs were estimated from records kept by the Onondaga County Highway Department, which maintains US-20 in the study area. These records indicated that about 30 Mg (33 short tons) of sodium chloride were applied to each 1.6 km (1 mile) of US-20 from January 1, 1975, to the end of the season's salting program.

RESULTS OF THE SAMPLING

Basic Analysis and Description of the Data

The range, mean, standard deviation, coefficient of variation, and number of samples taken at each sampling point for chloride levels are given in Table 1. The sampling points are grouped generally by stream or tributary.

Site A was sampled to indicate possible chloride inputs from Onondaga County Route I, which parallels Butternut Creek through the study area. Judging from the levels found at points C, B, and N on Butternut Creek, inputs from all sources in the study area, including Onondaga County Route I, were not sufficient to raise chloride concentrations by a measurable degree. In fact, mean concentrations upstream of suspected inputs are actually higher than downstream means, although by only 1 ppm, which is not considered meaningful with the type of testing I performed. In any event, the inputs certainly have very little impact on Butternut Creek.

Sampling points E and H provide background levels for tributary stream I. One of these locations was usually sampled on any given occasion. The means are in good agreement at 11.8 and 11.2 ppm respectively. On five occasions when stream IIa was tested separately from stream IIb, the mean was 12.4 ppm, which is reasonably close to the levels found in Ia and Ib for a local background concentration. The threshhold value of the testing equipment may have concealed an even lower actual level for these sites.

Point D is downstream of the highway on tributary I. Even though it received drainage from only 137 m (450 ft) of highway, it had an average concentration of 15.3 ppm, or 3.5 ppm higher than point E, which is upstream of the road. Applying the F-test (as outlined by 2, chapter A1) to the 21 occasions when both points were sampled at the same time indicates statistical significance at the 1 percent level. Based on this, it appears that 137 m (450 ft) of salted highway produced a mea-

Table 1. Chloride concentration, variability, and number of samples at principal sites.

Location	Sample	No. of Samples	Range (ppm)	Mean (ppm)	Standard Deviation (ppm)	Coefficient of Variation (%)
County Road	A	15	12.5 to 25	18.5	4.5	24
Butternut Creek	B	15	15 to 25	19.0	3.6	18
	N	24	12 to 25	19.1	4.1	21
	C	23	15 to 27.5	20.0	4.2	20
Stream I	E	21	10 to 20	11.8	2.5	20
	H	11	10 to 20	11.2	3.5	30
	D	33	10 to 20	15.3	3.4	27
	G	29	35 to 235	93.4	52.0	55
	F	22	15 to 70	29.8	14.7	48
Ditch	L	43	20 to 5500	448.0	900.0	198
Stream II	K	29	18 to 30	24.2	3.6	15
	J	40	22.5 to 165	40.4	23.7	58
	M	30	15 to 55	27.6	7.5	27

surable effect on this stream of about 1.2 km^2 (0.5 $mile^2$) of watershed.

The effect of highway salting on the small stream Ib [about 0.5-km^2 (0.2-$mile^2$) watershed] is more obvious from the sample means. The background level is 11 to 12 ppm (based on sites E and H), but the downstream sample average (site G) is 93.4 ppm, with a high of 235 ppm. I also noted that the variability was greater here than at the upstream sites. Even after stream Ib has joined stream Ia, which is about 2.5 times larger, the variability of the receiving stream (I at point F) remains high, although the mean drops considerably. The impact of this greater variability might be as important an ecological consideration as the actual means. Stream Ib receives runoff from approximately 671 m (2200 ft) of highway.

A similar procedure for testing upstream and downstream of the highway was used on tributary stream II, which drains about 1.8 km^2 (0.7 $mile^2$) before reaching the road. It is estimated that 1250 m (4100 ft) of highway drains into this stream. Stream II differs somewhat from stream I in that it receives highway effluent at one localized discharge point via a concrete-lined ditch. (Some seepage into the soil is possible before the effluent reaches stream I's channel.) Sampling point L, at this discharge point, provides a measure of chloride concentrations in the highway runoff just before it enters the stream. The samples show this runoff to have highly variable chloride levels and a standard deviation almost double the mean. A comparison of points K (upstream) and J (downstream) reflects the effects of this discharge. The mean levels increase from 24.2 to 40.4 ppm, and the variability is notably greater, as was true of stream I.

Point M on stream II was tested for the effects of dilution on chloride levels. This point, about 1829 m (6000 ft) downstream of point J, has approximately double the watershed area. Mean chloride levels here have dropped considerably. The variability has also dropped, but not to as great an extent as the mean.

Interrelations Among Sampling Points

I speculated at the beginning of the study that runoff from salted highways would raise chloride levels in receiving streams in some predictable fashion. Examination of the statistics in the previous section indicates the magnitude of some impacts, but further analyses using correlation coefficients and regression equations are valuable in considering the predictability of some of the variables. The results of these analyses are presented in Table 2, where x is the independent variable, y is the dependent variable, n is the number of paired samples, r is the correlation coefficient, and $y = a + b(x)$ is the regression equation [$1°F = 1°C(1.8) + 32$].

Of the variables with sufficient numbers of paired data sets, those that I tested were chosen to demonstrate the effects on chloride levels of (a) stream confluence, going from point G to point F, (b) downstream dilution, going from point J to point M, and (c) highway runoff entering receiving streams, considering points L and J as well as points L and G.

The effects of temperature and salt applications were also considered by testing those factors relative to chloride levels at point L, the highway runoff. Point L was chosen because a strong relation was found between this runoff and the receiving stream. Predicting the chloride levels at point L is therefore an important step in predicting stream levels.

The analyses were prepared following methods outlined by Riggs (2) and Freese (3). However, some errors of measurement are probable, and the data may not be normally distributed. Therefore some of the underlying assumptions of correlation and regression analysis are probably imperfectly satisfied. Riggs (2), in his discussion of the use of statistics for hydrological analyses, notes that the non-normality effects commonly found are not sufficient to prohibit meaningful use of such analyses.

Some of the relations found were expected. For instance, sample G is from stream Ib, a relatively salt-laden, small tributary. Sample F is on stream I, just downstream of the input from Ib, and would be expected to be influenced by levels at G, as the analysis indicates. It was also considered likely that downstream dilution does reduce concentrations and would be indicated by a relation between point J and point M farther downstream. The regression of the data examined (29 sets of samples at each point) showed t-test significance at the 0.1 percent level. The stream at point M has about double the watershed area found at point J.

A strong relation was found between the input of point L, the highway drainage ditch, and point J, just downstream. The t-test of the regression indicated significance at the 0.1 percent level, which was not necessarily expected, because the runoff characteristics of the highway effluent compared with the natural flow in the receiving stream were an unknown factor. The possibility of point L's having value as a general index of highway salt delivery was also considered by comparing the L values with those found at point G on stream Ib. The relation was weaker than that between the road runoff and the actual receiving stream but was still adequate for the t-test on the regression to indicate significance at the 2 percent level.

If we assume, then, that the chloride concentration of receiving streams is closely related to the concentration of chloride in highway runoff, we should next consider factors influencing those runoff concentrations. Factors of potential importance include (a) time and quantity of salt applications, (b) temperature, (c) snow, ice, and water availability for formation of chloride solutions, (d) effects of plowing and traffic on movements of salty material to highway drainage facilities, and (e) rainfall flushing action. Athough the present study results were not as concrete as I would have liked, they do permit some speculations for some factors.

Table 2. Correlations and regressions among sample points for chloride values.

x	y	n	r	a	b	t-Test Significance Level (%)
Point J	Point M	29	0.79	17.1	0.232(x)	0.1
Point L	Point J	32	0.90	29.97	0.0232(x)	0.1
Point G	Point F	22	0.95	7.69	0.26(x)	0.1
Temperature (°F)	Point L	27	0.37	58.9(x)	-585	10
Salt applied (3 d before sample)	Point L	27	0.15	—relationship not indicated—		
Point L	Point G	18	0.56	0.104(x)	84.3	2

Salt Applications

The first few years of salting will establish an equilibrium in the roadside soil, and it is likely that salt applied during a given year will leave the sub-basin during that time in approximately the same quantities. Previous studies have noted a good correlation between long-term salt use and long-term average chloride concentrations in the receiving streams (4).

In the short run, however, this study does not indicate an immediate relation. For instance, no statistical relation was found in the amount of salt applied during a 3-d period before a sample of highway runoff was collected, at least not by using simple analysis (Table 2). At some time interval salt application must become significant, but it appears that for particular occasions during the salting season, the salting operates as much as a prerequisite as an immediate cause of fluxuating chloride levels.

Nevertheless, inspection of the data does show occasions when heavy applications occurred for a period of several days preceding the sample, and the effluent seemingly responded. For instance, about 2 Mg/km (3.5 tons/mile) were applied during the 10 d before the February 5 reading of 1875 ppm at point L, and about 4.5 Mg/km (8 tons/mile) for a similar period before the February 7 reading of 1750 ppm (point L). The highest reading of 5500 ppm (also point L) on February 16 likewise followed a period of heavy salting and marked a rise in temperature after a relatively cold period. It may be that climatic or other natural conditions and trends generally control the release of chlorides from the roadway vicinity, while the amount depends on salt applications of the near past.

Temperature

The test of temperature and runoff chloride level showed t-test significance at a 10 percent level (Table 2). This test employed rather imperfect temperature data [mean daily temperatures at Tully, New York, over 12 km (8 miles) away, and about 150 m (500 ft) higher], so the level of significance is weighted as a potentially positive indicator.

A rise in temperature facilitates melting of snow and ice and the movement downslope of the water and salt solution. The highest sample concentration during this study was 5500 ppm or 0.55 percent, which has very little effect on the melting point of ice. It seems unlikely, then, that salt lowered the melting point or had much effect on the stream. A little salt will enhance mechanical movement of snow and ice from the roadway to the ditch and shoulder area by traffic and plows, but temperatures near or above the freezing point are probably more important in initiating runoff from the system.

Rain

Rain that fell during some of the sampling occasions appeared to raise chloride levels. For instance, on January 25 no heavy salting had occurred for several days, but the chloride level was 1000 ppm (concentrations in this discussion refer to point L), more than double the mean observed at this point. It was raining lightly at the time. A more convincing example of this rain effect occurred on March 29. On this occasion, a sample taken at 9:30 a.m. contained 950 ppm of chloride, whereas one taken at 3:00 p.m. the previous day contained 92 ppm; a sample concentration at 6:00 p.m. on March 29, just hours after the 950 ppm sample, contained 125 ppm. It had started raining about half an hour before the 950 ppm sample and quit about noon the same day.

No salt had been applied for three days, so the rain appears to have triggered this higher sample, sandwiched between two lower ones. On April 20, 12 d after the last salting, a sample registering 82 ppm was taken. Before this, on March 16 and 18, levels were down to 20 and 25 ppm respectively, but the rain apparently influenced the sample on March 20. Temperatures had previously risen well above 0°C (32°F), and no obvious factor except the rain could explain the upsurge in chloride level.

Roadside Samples

Some samples of snow and slush in the roadway vicinity were tested, and these results reflect, at least partially, the above hypothesis concerning temperature or other natural factors.

Six snow or slush samples ranging from 25 to 8500 ppm of chloride were taken near the roadway or in the ditch. On most occasions these levels were above that of the runoff at point L, and, if the concentration of the liquid portion is considered when the snow or ice is only partially melted, the chloride levels are even higher. For instance, the slush sample, which measured 8500 ppm when totally melted, was tested (the liquid portion) when it was about 30 percent liquid and 70 percent ice; the concentration then was 22 000 ppm.

The concentration of the initial brine formed as the salt first dissolves can be expected to be very high, but such levels were not found at point L.

This indicates relatively salt-free snow and ice in the catchment area, probably transport and dilute runoff from the highway drainage system. Even in the salted areas enough melting probably also precedes runoff conditions to dilute the effluent considerably from the initial chloride levels on the road surface. The roadway area draining to point L includes some high shoulders, and melt from snow samples 1.5 to 3 m (5 to 10 ft) up the bank measured 60 ppm and 12 ppm of chloride on the two occasions when such samples were taken. These samples indicate a source of relatively uncontaminated runoff material to dilute the saltier discharge from the road.

Intensive Sampling

Six samples were taken between 9:00 a.m. on March 26 and 3:00 p.m. on March 28. Conditions included 3.3 cm (1.3 in) of snow on March 25 and 469 kg (1042 lb) of salt applied on March 26, all before 3:00 p.m. according to the records. There was no further precipitation or salting during this sampling. Results at point L, together with temperatures, are presented below [1°C = (1°F - 32)/1.8].

Date	Time	Chloride Concentration (ppm)	Temperature (°C)
3/26	9:00 a.m.	250	-9.1
3/26	3:30 p.m.	340	+0.6
3/26	11:00 p.m.	82	-12.2
3/27	4:00 a.m.	70	-7.8
3/27	10:00 a.m.	65	-7.7
3/28	3:00 p.m.	92	-0.6

The rise of 90 ppm from 9:00 a.m. to 3:30 p.m. on March 26 was probably caused by the daytime temperature increase and by the freshly applied salt. By 11:00 p.m. that night the concentration had dropped considerably and continued to drop, apparently in response to colder temperatures.

This drop was caused not merely by depletion of salt present, as evidenced by the rise to 92 ppm the next day (March 28), when temperatures rose even though no new salt had been spread. Then, as mentioned previously, it rained on March 29, and concentrations rose to 950 ppm at the 9:30 a.m. sampling and fell again after the rain stopped. Chloride levels in the receiving stream (stream II), measured at point J, followed the trend established at point L.

Residual Effects of Salting

The last salting for the winter of 1974 to 1975 occurred, according to county records, on April 8, the end of a major snowfall of 58 cm (23 in). Fourteen samples were collected after this date, mostly in April, but extending to July 12.

Stream Ib strongly suggests residual effects of winter salting. On April 9, the chloride level was 115 ppm and dropped fairly steadily to 35 ppm, which was recorded on the last four sampling dates. The chloride level upstream of the highway during this time ranged from 10 ppm to 15 ppm but did not exceed 12 ppm after April 11.

As previously noted, the highway runoff to this stream is not guttered all the way to the channel; apparently chloride has ample opportunity to infiltrate the soil and to percolate out at some later date. It should be noted in connection with the lower summer flow levels that the same chloride concentration does not imply that the same amount of salt was delivered to the stream but merely that it is in the same proportion relative to stream flow.

Thus data indicated that progressively less salt reaches the stream as summer lengthens. Road salting influencing groundwater and effluent streams beyond the salting season has been previously reported.

PROJECTION OF MEAN AND PEAK CHLORIDE LEVELS

If chloride levels in roadside soils can be assumed to be in equilibrium, then in most cases estimating mean chloride outputs, such as an annual mean concentration, on a long-term basis, is fairly straightforward. This can be calculated on the basis of the amount of salt applied in a basin and the amount of flow that will be dissolving and transporting that salt.

Although such average figures can be calculated fairly simply, the variation of chloride levels around their means has not been reported. Comments on this topic have been generally limited to observations of seasonal variations, such as concentrations during spring high flows compared with summer low flows.

VARIABILITY ESTIMATES

In this study the coefficient of variation was calculated for each sampling site. At site G, this coefficient was 55 percent, and at site J it was 58 percent. This is in contrast to 20 and 15 percent for the respective upstream sampling stations. Thus, highway salting is apparently responsible for approximately tripling or quadrupling the variability of chloride concentrations in the subject streams.

Halbritter (5) showed that the coefficient of variation increased from 24 percent at the control site to 54 percent after two salted highways intersect. This approximate doubling of the variability occurred in conjunction with a rise in mean levels from 9.6 to 17.9 ppm.

Another variability index is the ratio of the maximum observed value to the mean. For site G these values are 235 and 93 ppm, showing the high value to be 2.5 times the mean. At site J, a similar procedure, dividing 165 ppm by the mean of 40 ppm, shows the high value to be 4.1 times the mean.

In the absence of sampling, such an index could be useful in estimating probable peak concentrations. For instance, a mean chloride level could be estimated according to projected salt use and stream flow volumes. Then a factor of four or five, applied to the estimated mean, would represent the probable peak. The actual factor, of course, may require modifications as more data become available.

CONCLUSIONS

When salt is applied to highways as part of a deicing program, most of it can be expected to be dissolved in the runoff water and to leave the drainage area. If a chloride equilibrium has been reached in the nearby soil, salt output of the system essentially equals the salt applied—at least at the magnitudes of current deicing programs. This salt can be detected by comparing concentrations upstream with those downstream of the highway; the result, taken as a long-term mean (minimum of several months, say), depends primarily on the amount of salt applied in the basin and on the flow volume of the stream.

In addition to raised mean chloride level, I found that the variability in chloride levels was three to four times as great downstream of the highway as upstream. Also, peak values were two to four times the mean.

The chloride levels downstream of highway inputs were decreased by dilution. Chloride concentration decreased in roughly inverse proportion to increased area contributing to flow.

The runoff from the salted highway was such that the effect on chloride levels in the receiving stream could be predicted fairly confidently by a linear regression equation. Initial brine levels on the roadway were very high, but such solutions were diluted considerably before reaching the stream.

The timing of the highway runoff relates to natural environmental factors such as temperature and precipitation. Until runoff occurs, the salt is probably stored in the immediate vicinity of the roadway. I observed that rainfall seemed to initiate peaks or upsurges of chloride concentrations in the highway effluent and receiving streams.

The few samples gathered in the late spring and summer from stream Ib indicate that deicing salts were stored within the drainage basin well beyond the salting season. Levels downstream of the highway were approximately 25 ppm higher than those upstream in this small waterway. This increase was last observed on July 12, 1975.

Butternut Creek, which drains about 51.8 km^2 (20 $miles^2$) upstream from the study site, was not affected to a detectable degree by the deicing salts applied to US-20 and to other highways in the study area. Measurable increases in chloride concentrations were detectable in the smaller tributaries tested, which ranged from 0.5 km^2 (0.2 $mile^2$) of drainage area to 3.4 km^2 (1.3 $mile^2$).

ACKNOWLEDGMENTS

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REFERENCES

1. R. H. Hawkins. Street Salting and Water Quality in Meadowbrook, Syracuse, New York. Proc., Street Salting—Urban Water Quality Workshop, State Univ. of New York College of Environmental Science and Forestry, Syracuse, 1975.
2. H. C. Riggs. Some Statistical Tools in Hydrology. United States Geological Survey Techniques of Water-Resources Investigations, Book 4, 1968.
3. F. Freese. Elementary Statistical Methods for

Foresters. U.S. Department of Agriculture, Agriculture Handbook No. 317, 1974.
4. W. H. Walker and F. O. Wood. Road Salt Use and the Environment. HRB, Highway Research Record 425, 1973, pp. 67-76.
5. S. J. Halbritter. Effects of Street Salting on a Small Stream in Central New York. State Univ. of New York College of Environmental Science and Forestry, Syracuse, 1975.

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Advance Traffic-Control Warning Systems for Maintenance Operations

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This paper discusses the effects of sign size, height of sign installation, and sign legend on driver responses measured by speed, conflict, and queuing parameters. Effects of flashing chevrons were also evaluated in terms of these responses. The experiment was conducted on two-lane highways and the Interstate system at four locations. The conclusions, based on the analysis and evaluation of the various responses using standard statistical procedures, are that (a) speed decrease at two-lane locations was greater for the 0.76-m (30-in) signs than either the 0.91-m (36-in) or the 1.22-m (48-in) signs; (b) at Interstate locations, the 0.91-m (36-in) sign yielded better overall response than the corresponding 0.76-m (30-in) signs; (c) installation height of 0.31 m (1 ft) and 1.52 m (5 ft) and sign legend did not indicate any statistical difference in the measured response; (d) two-way flashing chevrons greatly enhanced the obedience of the driver to warning signs; and (e) differences in responses by location can be discussed in terms of traffic volume and the motorists' attitudes toward signing in general.

The types of advance traffic-control warning systems used during maintenance operations generally include signs and such supplements as flags and flashing lights. Basically, these warn, control, protect, and expedite the flow of traffic and provide safe work areas.

An effective warning system should, among other things, command attention and convey a simple, clear message. Guidelines for the design and placement of various signs have been drawn up (1). Design is specified in terms of size, shape, color, and so forth; placement must be within the driver's core of vision to allow adequate time to respond.

Kentucky (2) has reported on effects of sign color on traffic response at construction sites. However, information on driver response to sign size and placement height is lacking. In recent years, the tendency has been toward larger signs and higher mounting. Although there may be some innate justification for this tendency, the effects of sign design parameters on driver response should be quantitatively evaluated.

This study is an attempt to measure, quantitatively, the effect of sign size and sign height on driver response during maintenance operations involving lane closures in rural areas.

PURPOSE AND SCOPE OF THE STUDY

In this study we attempted to evaluate the effects of advance traffic-control warning systems on traffic flow and driver alertness by varying sign size, sign height, and sign legend, by using signs with and without flashing (attention-getting) devices, and by altering traffic situations in rural areas.

All maintenance operations lasted long enough to require single-lane closure. On the Interstate system, closure was limited to the outside lane only, and the number of vehicles required to merge was larger than for any other lane closure. In rural areas, higher volumes are generally encountered in this lane. All maintenance work zones were limited to less than 91.4 m (300 ft).

The scope of the study did not include observations of variables such as roadway alignment, weather, or terrain. Likewise, situations arising from detours and sight obstructions were also eliminated from the study design. In order to reduce maintenance scheduling problems, use of simulation or "dummy" maintenance operations was allowed, but care was taken to ensure that such simulations duplicated actual closures.

PROCEDURE

Experimental Design

The factors generally considered to affect driver alertness (message comprehension) for Interstates are (a) sign size, which varies 0.76 m (30 in), 0.91 m (36 in), and 1.22 m (48 in); (b) sign height, which varies 0.31 m (1 ft) and 1.52 m (5 ft) from ground elevation to the bottom of the sign; (c) specific, general, or diagrammatic sign legend; and (d) trailer-mounted flashing chevrons. For two-lane roads, only (a) and (b) affect alertness. These are our independent variables.

Our dependent or response variables for Interstates included (a) average speed in critical zone, (b) traffic conflicts, and (c) number of vehicles properly queued in the travel lane between the last sign and the first cone taper. For two-lane highways, only (a) and (b) were dependent variables.

Experiments were conducted at four different locations for each type of facility. All test sites were in rural areas having traffic volumes greater than 1000 vehicles/d. Not all combinations of independent variables were compared across all test sites because of scheduling constraints. For example, diagrammatic signs were tested at one site only. Likewise, for another location, data for flashing chevrons could not be obtained because of an electrical malfunction. Figure 1 is a flow chart of the experimental design for the Interstate system.

Sign System Layout

Figures 2 and 3 show the three-sign series for the two-lane road and the Interstate system respectively. These are minimum standards as defined in the Louisiana Department of Highways Maintenance Traffic Control Handbook (3).

Sign legend, as an independent variable, was evaluated on the Interstate system for the message on the third sign only, RIGHT LANE CLOSED 1000 FT. This message was considered specific. The corresponding signs with general (MERGE LEFT) and diagrammatic messages are also shown in the layout. As mentioned before, the diagrammatic message was evaluated at one location only.

The location of the trailer-mounted flashing chevrons is also shown in the layout. Table 1 lists the variables we evaluated at each of the locations on the Interstate system. In this table one location involved a dummy, but at all other locations maintenance crews were removing a distressed segment of the pavement section and replacing it. Use of flashing beacons on work vehicles was prohibited at all locations.

Measurement of Parameters

Radar spot speeds were measured in two zones (simultaneously) at each location throughout the evaluation period. One zone was located approximately 3.2 km (2 miles) in advance of the first sign; the other lay between the last sign and the first cone taper. The latter was considered the critical zone. The first zone provided us with normal highway speeds of drivers unaware of the impending roadway operation ahead.

The magnitude of speed reduction in the second zone was taken as one of the indicators of effective signing schemes and, therefore, increased driver responsiveness or obedience to the corresponding sign size, height, and legend.

Traffic Conflicts

This parameter was classed, according to Kentucky (2), as a forced merge or a complete stop. On two-lane

Figure 1. General layout of experimental design for Interstate locations.

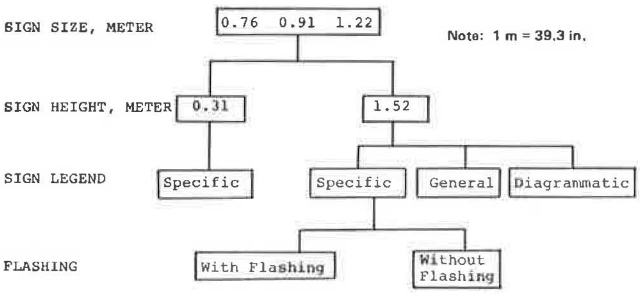


Figure 2. Sign scheme for two-lane system.

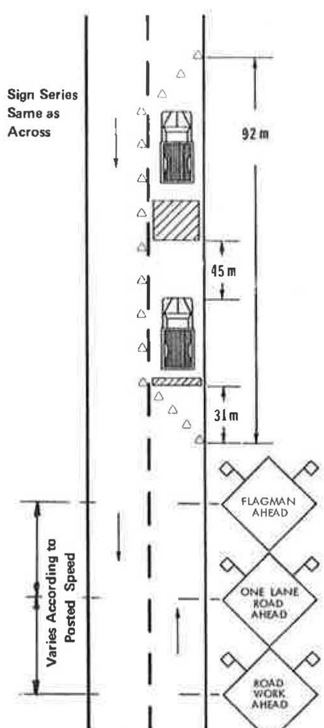


Figure 3. Sign scheme for Interstate system.

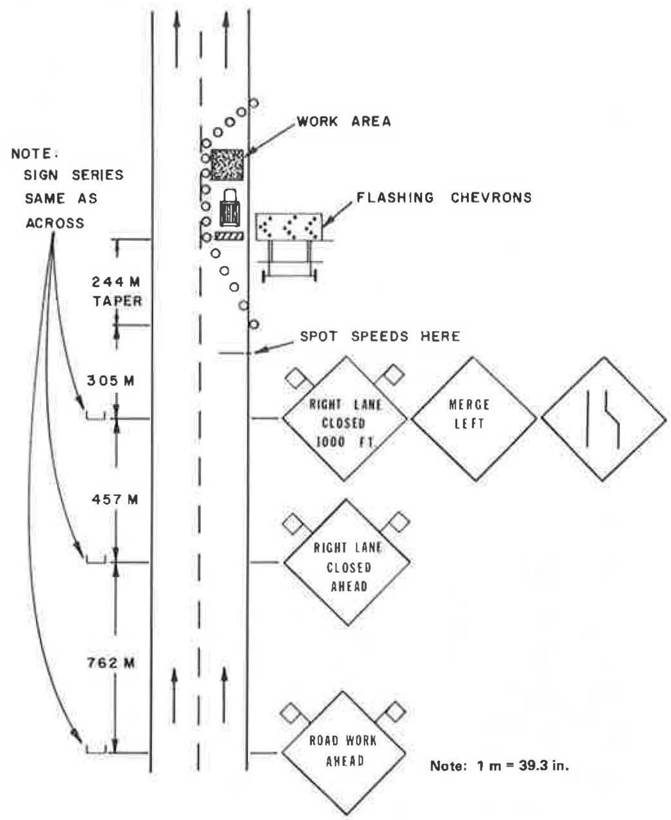


Table 1. Variables evaluated at four locations.

Sign Size (m)	Sign Height (m)	Legend (last sign only)	Location 03	Location 04	Location 07	Location 62
0.76	0.31, 1.52	Specific	Yes	Yes	Yes	Yes
	1.52	Specific with chevrons	Yes	Yes	No	Yes
	1.52	General	Yes	Yes	Yes	Yes
	1.52	Diagrammatic	No	No	No	Yes
0.91	0.31, 1.52	Specific	Yes	Yes	Yes	Yes
	1.52	Specific with chevrons	Yes	Yes	No	Yes
	1.52	General	Yes	Yes	Yes	Yes
	1.52	Diagrammatic	No	No	No	Yes
1.22	0.31, 1.52	Specific	Yes	Yes	Yes	Yes
	1.52	Specific with chevrons	Yes	Yes	No	Yes
	1.52	General	Yes	Yes	Yes	Yes
	1.52	Diagrammatic	No	No	No	Yes

Note: 1 m = 39.3 in.

roads, categorizers confined themselves to abnormal brake application only.

The above parameters were measured with a time-lapse camera and a video system inconspicuously mounted in a van in the tapered zone (Figure 3). An observer recorded measurements for two-lane roads between the last sign and the first cone taper. Figure 4 is a photograph of the video system.

Queuing of Vehicles

This variable was also measured with the video equipment. An increase in the number of vehicles properly queued in the inside (open) lane indicated effective signing and, consequently, the effect of the independent variables.

Traffic Split and Volume Data

Traffic volume at each location was obtained while we were evaluating the signs. For the Interstate system, the traffic split between the two lanes, or the percentage of vehicles traveling in each lane at the test site, was measured after the evaluation period. These parameters were also measured with the video system. The split information was used to correct for the queuing parameter.

Sample Size

The sampling at each test site had to be accomplished within the time limits imposed by the maintenance operation. Therefore, for each sign scheme, measuring parameters was continued for preestablished time spans. The time span for each location was established in a way that provided some uniformity in the traffic volume or the number of vehicles arriving at that maintenance site. In all cases, however, this number of vehicles was to be no less than 25 for speed measurements. We considered this a necessary prerequisite for adequate statistical evaluation of data.

Figure 4. Video system.

Table 2. Summary of significance levels.

	Speed Reduction		Interstate	
Factors	Two-Lane	Interstate	Queuing	Conflict
Location (L)	p = 0.01	p = 0.05	p = 0.05	p = 0.001
Size (S)	p = 0.01	p = 0.05	p = 0.05	p = 0.05
Height (H)	NS	NS	NS	NS
Interactions				
L × S	p = 0.05	NS	p = 0.01	NS
S × H	NS	NS	p = 0.01	NS
L × H	NS	NS	NS	NS

Note: NS = not significant.

DATA ANALYSIS AND EVALUATION

Data were evaluated in terms of the number of vehicles responding to a given sign scheme as measured by the response parameter defined in the previous section. Vehicles were categorized as automobiles or trucks, and all were converted to percentages of the total for that category. The percentage of vehicles properly queued was determined after appropriate correction factors were applied for the split that prevailed during that sign scheme evaluation period.

The data were analyzed by using analysis of variance, which is basically just what the name implies—a partitioning of the variance of an experiment in order to test whether certain factors introduced into the design actually produce significantly different results in the variable. For example, does sign size affect driver message comprehension as measured by some variable? Does the legend or message on the sign increase driver awareness of the existing situation? In each case the point is to test whether the effect of the factor (sign size) on the variable measured (speed, conflict) is real when compared with the random variations in the system. Table 2 lists the various factors considered in the statistical analysis and their significance with respect to the measured differences in the parameters.

STUDY FINDINGS

Effects of Size and Height on Measured Responses

Two-Lane System

Table 3 lists the distribution of speeds for the two zones and the difference between these speeds for each sign scheme. Figures 5 and 6 are bar charts comparing the mean data for different locations according to sign size and height, respectively.

The variance analysis indicated that the decrease in speed in the critical zone was statistically significant, which means that people do respond to warning signs. However, the statistical significance of the location factor (Table 2) indicates that different populations react differently to warning signs.

The height of the sign installation did not have any significant effect on driver response.

The most surprising finding in the analysis is reflected by sign size. At all locations, this factor indicated better response to smaller sizes than to larger ones, and two of the locations (08 and 62) indicated a statistically significant difference in the measured parameter. No explanation for such data can be offered here, except that sign size and sign height may not be effective criteria for providing increased awareness during maintenance operations on two-lane systems. Generally, local traffic predominates on rural roads, so such responses might become habitual. This fact is emphasized by data for district 58, where the difference between sizes is insignificant.

Another aspect used for evaluating various sign schemes was the total absence of conflict. Furthermore, the normal speed of travel as measured in advance of the critical zone was far below the posted speed of 88 km/h (55 mph). This too may be attributable to habitual response or to local drivers using the facility.

In summary, then, on two-lane rural roads the increase in sign size or sign height does not improve driver response to impending roadway maintenance operations.

Table 3. Speed versus sign measurements for two-lane system.

Location	No. of Vehicles	Sign Size (m)	Sign Height (m)	Speed Before Critical Zone (km/h) Mean	SD	Low	High	Speed in Critical Zone (km/h) Mean	SD	Low	High	Speed Difference (km/h)
07 (ADT 1050)	30	0.76	0.31	67.6	12.4	43	79	53.8	11.0	34	79	13.8
			1.52	68.1	12.2	32	100	56.0	12.4	34	89	12.1
		0.91	0.31	68.1	12.1	40	90	53.0	9.8	34	81	15.1
			1.52	66.7	15.3	19	97	55.7	10.1	34	84	11.0
		1.22	0.31	68.0	12.7	35	92	58.3	12.1	35	93	9.7
			1.52	69.9	8.2	53	82	59.4	7.6	48	74	10.5
08 (ADT 2600)	40	0.76	0.31	79.7	9.2	63	101	52.2	11.3	32	81	27.5
			1.52	83.4	7.7	71	98	55.4	9.5	39	77	28.0
		0.91	0.31	82.1	9.3	58	105	68.4	9.0	52	92	13.7
			1.52	80.4	9.8	64	113	59.9	10.8	39	82	20.5
		1.22	0.31	75.7	8.7	61	110	63.0	9.7	39	82	12.7
			1.52	77.0	9.7	58	97	67.8	11.6	45	98	9.2
58 (ADT 2040)	25	0.76	0.31	81.6	14.2	57	116	61.5	8.2	48	76	20.1
			1.52	81.6	14.2	57	116	60.5	10.0	37	76	21.1
		0.91	0.31	80.4	14.7	47	103	59.6	9.7	40	77	20.8
			1.52	80.4	14.7	47	103	60.7	9.2	34	82	19.7
		1.22	0.31	80.5	10.5	63	103	62.6	7.6	48	76	17.9
			1.52	80.8	11.0	45	101	60.7	9.8	39	82	20.1
62 (ADT 1316)	30	0.76	0.31	69.1	11.1	52	92	49.6	6.4	39	64	19.5
			1.52	67.1	12.2	37	89	48.8	7.1	37	68	18.3
		0.91	0.31	61.8	7.9	45	81	51.9	7.1	39	71	9.9
			1.52	63.6	11.0	42	85	53.8	9.5	39	85	9.8
		1.22	0.31	65.9	9.8	45	84	55.9	7.9	47	84	10.0
			1.52	65.5	13.4	48	97	57.2	8.5	45	76	8.3

Note: 1 m = 39.3 in and 1 km/h = 0.621 mph.

Figure 5. Effect of sign size on mean speed reduction at two-lane locations.

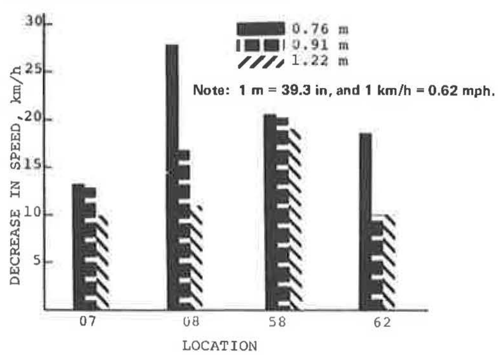
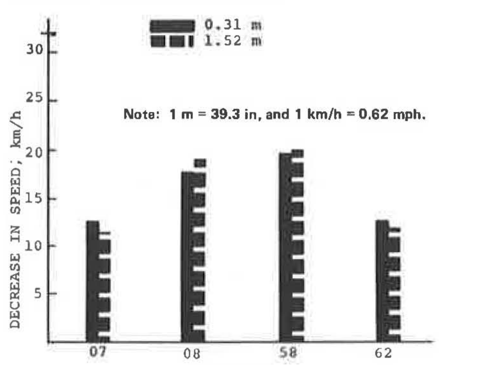


Figure 6. Effect of sign height on mean speed reduction at two-lane locations.



Interstate System

Table 4 is a listing of data on the speed parameter for each sign configuration at different locations. Figures 7 and 8 compare data on size and height, respectively, for each location.

The reduction in speed in the critical zone is statistically significant for all locations except district 62. Likewise, the size factor was also significant at the 5 percent level. However, the trend is inconsistent and is location dependent. This means that driver attitude may be confounding the effectiveness of the independent variables. Only one location (district 07) indicated the expected trend in Figure 7. In district 62 an increase in speed in the critical zone was observed and is shown as a zero decrease in the figure. Overall, the 0.91-m (36-in) sign provided better or increased awareness than either of the other two, but the difference in response variables for height was statistically insignificant.

The mean decrease in speed for each sign scheme cannot be considered to be of any recognizable magnitude. The small decrease that did exist can be attributed to the present legal posted speed of 88 km/h (55 mph), which is considerably lower than the 112 km/h (70 mph) speed limit of 3 years ago. At the higher speeds Kentucky (2) reported a mean decrease in speeds of about 24 km/h (15 mph) during their evaluation of sign color schemes on the Interstate system.

The response variables queuing and conflict are two of the primary indicators of effective signing for single-lane closures on the Interstate system. Table 5 contains comparative queuing data for various sign configurations for all locations, and Figures 9 and 10 present the corresponding graphic evaluation. The queue data were provided by applying the split factor prevalent at the time the particular sign configuration was evaluated. In other words, the data account for vehicles in the travel (open) lane before they arrived at the last sign.

Only one location, 03, yielded generally consistent results, namely increasing obedience to signing with increasing size and height. But results at location 62 contradicted the expected norm, although the 1.52-m (5-ft) height did show better response than the corresponding 0.31-m (1-ft) height.

No significant difference in queuing was indicated by the analysis for any of the factors except size and location. Ideally, adequate advance-warning signs before a lane closure should result in increased queuing in the open lane between the last sign and the beginning of the cone taper. However, increasing traffic volume makes the available gap for merging more difficult and results in drivers becoming trapped in the closed lane. This ultimately increases the occurrence of conflict, which is generally created by forced merging. This fact is emphasized by data for locations 03 and 04 in Figure 10. The former had a smaller proportion of queuing and a correspondingly more frequent occurrence of conflict (Figure 12). Likewise, the latter location had a high percentage of queued vehicles and correspondingly few conflicts. The many conflicts at location 07 can be attributed to high traffic volume.

Detailed conflict data are presented in Table 5, and Figures 11 and 12 show corresponding bar charts. The difference in this parameter was significant with respect to size variable at the 5 percent level. This is readily seen in Figure 12. The total absence of conflicts at location 62 can be attributed to the high proportion of queuing (although not as high as at 04).

The specific legend as used in one evaluation (RIGHT

Table 4. Speed versus sign measurments and legend type for Interstate locations.

	Sign			Before Critical Zone					In Critical Zone					
					Speed (km/h)					Speed (km/h)				
Location	Size (m)	Height (m)	Legend	No. of Vehicles	Mean	SD	Low	High	No. of Vehicles	Mean	SD	Low	High	Speed Difference (km/h)
03 (ADT 13 000)	0.76	0.31	Specific	65	86.6	7.4	71	105	50	83.7	6.9	68	98	2.9
		1.52	Specific	62	87.8	8.5	64	113	50	85.2	7.9	64	103	2.6
		1.52	Chevron	38	89.9	5.8	79	100	50	81.2	8.1	66	100	8.7
		1.52	General	40	89.5	7.4	69	110	50	81.0	8.2	56	93	8.5
	0.91	0.31	Specific	35	87.9	8.5	76	122	50	81.8	7.9	56	103	6.1
		1.52	Specific	58	88.6	5.8	74	100	50	84.4	9.5	61	105	4.2
		1.52	Chevron	67	89.4	7.6	64	106	50	80.5	8.7	60	100	8.9
		1.52	General	74	87.6	7.7	61	111	50	84.2	7.2	66	98	3.4
	1.22	0.31	Specific	55	89.0	6.1	74	103	50	83.1	6.0	68	97	5.9
		1.52	Specific	60	84.9	7.1	64	98	50	83.4	8.7	61	101	1.5
		1.52	Chevron	69	87.8	6.4	72	110	50	82.4	8.4	66	103	5.4
		1.52	General	40	85.3	8.5	68	105	50	82.9	8.1	64	97	2.4
04 (ADT 11 000)	0.76	0.31	Specific	115	87.3	9.2	66	126	42	82.3	8.2	69	101	5.0
		1.52	Specific	90	86.2	7.7	64	122	34	80.0	7.4	63	95	6.2
		1.52	Chevron	60	83.4	7.4	64	101	42	76.5	6.8	61	90	6.9
		1.52	General	80	90.5	9.1	71	124	42	84.7	8.7	64	103	5.8
	0.91	0.31	Specific	100	86.5	8.4	63	105	42	79.9	9.0	61	100	6.6
		1.52	Specific	90	86.8	9.5	61	111	42	80.5	7.6	64	105	6.3
		1.52	Chevron	80	88.1	8.0	71	110	42	81.6	7.9	52	95	6.5
		1.52	General	75	86.0	6.4	72	103	42	82.4	6.9	68	101	3.6
	1.22	0.31	Specific	80	88.6	7.2	66	113	50	85.3	6.9	66	101	3.3
		1.52	Specific	80	87.3	6.8	69	104	65	82.9	6.3	72	97	4.4
		1.52	Chevron	80	88.2	6.9	72	106	66	83.6	7.2	71	98	4.6
		1.52	General	80	89.9	7.1	74	110	68	86.5	6.6	64	100	3.4
07 (ADT 15 000)	0.76	0.31	Specific	40	83.6	7.4	71	103	64	81.6	8.7	56	97	2.0
		1.52	Specific	40	85.0	6.3	71	95	59	83.6	7.6	66	103	1.4
		1.52	General	40	86.1	7.4	71	105	65	84.9	6.9	72	103	1.2
	0.91	0.31	Specific	60	86.8	8.4	58	110	68	82.9	6.0	67	103	3.9
		1.52	Specific	70	86.0	7.6	66	110	68	83.7	6.1	64	95	2.3
		1.52	General	66	89.4	7.4	68	103	64	83.7	8.4	63	111	5.7
	1.22	0.31	Specific	70	89.0	10.3	68	119	74	84.9	9.7	56	103	4.1
		1.52	Specific	60	87.9	8.5	63	110	68	81.3	8.2	56	98	6.6
		1.52	General	61	85.8	8.7	71	110	68	85.0	9.5	58	118	0.8
62 (ADT 6 200)	0.76	0.31	Specific	60	90.3	8.9	76	130	63	87.4	7.9	64	105	2.9
		1.52	Specific	60	90.5	10.5	58	116	56	90.3	8.7	71	124	0.2
		1.52	Chevron	60	90.7	10.1	76	118	55	88.9	7.6	76	108	1.8
		1.52	General	60	87.0	9.5	61	113	52	89.2	8.7	68	116	-2.2
		1.52	Diagram	58	86.2	9.8	53	114	60	89.0	7.7	66	118	-2.8
	0.91	0.31	Specific	76	87.1	9.2	61	108	55	89.0	7.1	76	110	-1.9
		1.52	Specific	80	88.6	9.2	61	122	55	86.5	10.5	55	119	2.1
		1.52	Chevron	90	88.2	11.3	53	118	52	86.2	10.5	58	116	2.0
		1.52	General	103	85.5	9.0	64	111	53	88.2	9.7	61	108	-2.7
		1.52	Diagram	75	87.0	9.8	68	119	54	86.3	8.9	69	114	0.7
	1.22	0.31	Specific	64	89.4	8.5	64	110	53	88.6	8.7	69	113	0.8
		1.52	Specific	62	89.2	7.9	71	110	50	87.1	7.1	71	103	2.1
		1.52	Chevron	79	91.0	9.0	68	113	53	85.5	9.2	63	110	5.5
		1.52	General	70	88.2	10.5	58	111	52	87.3	8.1	61	106	0.9
		1.52	Diagram	52	78.1	15.6	42	122	51	89.4	10.5	64	110	-11.3

Note: 1 m = 39.3 in and 1 km/h = 0.621 mph.

Figure 7. Effect of sign size on mean speed reduction at Interstate locations.

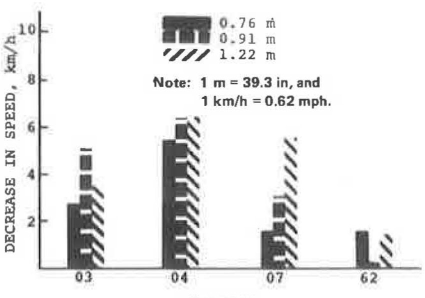
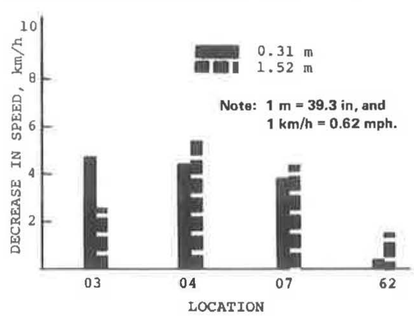


Figure 8. Effect of sign height on mean speed reduction at Interstate locations.



LANE CLOSED 1000 FEET) warns the motorist of something some distance farther up the road. It does not explain specifically what he or she should do. The action he or she should take, once this warning sign is encountered, is strictly left to his or her discretion. Diagrammatic signs also fall into this category. The general legend (MERGE LEFT) employed here is somewhat more particular in that it orders the motorist to take specific action.

Data collected for this variable indicated inconsistency within the location and no significant difference in any of the response variables. Location 62, however, showed a more pronounced effect on the queuing with general and diagrammatic messages than with the specific message (Table 5).

Flashing chevrons are attention-getting devices and can actually be categorized in the legend variable group. The equipment, shown in Figure 16, was a 1.07 by 1.98-m (3.5 by 6.5-ft) trailer-mounted panel of amber lamps that were activated in a series of two-directional

Table 5. Queue and conflict versus sign measurements and legend type for Interstate locations.

	Sign						Queued Vehicles		Conflicts	
Location	Size (m)	Height (m)	Legend	No. of Automobiles	No. of Trucks	Total Vehicles	No.	Percentage	No.	Percentage
03	0.76	0.31	Specific	68	17	85	12	14.1	12	14.1
		1.52	Specific	85	18	103	28	27.2	10	9.7
		1.52	Chevron	80	32	112	37	33.0	5	4.5
		1.52	General	78	22	100	26	26.0	9	9.0
	0.91	0.31	Specific	97	31	128	40	31.3	12	9.4
		1.52	Specific	101	20	121	46	38.0	6	5.0
		1.52	Chevron	110	45	155	77	49.7	4	2.6
		1.52	General	130	32	162	82	50.6	12	7.4
	1.22	0.31	Specific	61	15	76	34	44.7	5	6.6
		1.52	Specific	82	37	119	63	52.9	8	6.7
		1.52	Chevron	67	25	92	42	45.7	5	5.4
		1.52	General	75	15	90	36	40.0	3	3.3
04	0.76	0.31	Specific	121	41	162	91	56.2	3	1.9
		1.52	Specific	124	34	158	66	41.8	7	4.4
		1.52	Chevron	92	40	132	72	54.5	2	1.5
		1.52	General	126	29	155	91	58.7	2	1.3
	0.91	0.31	Specific	138	30	168	101	60.1	1	0.6
		1.52	Specific	151	37	188	106	56.4	3	1.6
		1.52	Chevron	150	39	189	105	55.6	4	2.1
		1.52	General	171	27	198	68	34.3	2	1.0
	1.22	0.31	Specific	130	47	177	93	52.5	0	0.0
		1.52	Specific	116	25	141	54	38.3	1	0.7
		1.52	Chevron	121	43	164	98	59.8	2	1.2
		1.52	General	131	44	175	80	45.7	3	1.7
07	0.76	0.31	Specific	114	24	138	58	42.0	19	13.8
		1.52	Specific	98	22	120	53	44.2	9	7.5
		1.52	General	80	17	97	39	40.2	14	14.4
	0.91	0.31	Specific	126	15	141	66	46.8	15	10.6
		1.52	Specific	84	17	101	53	52.5	7	6.9
		1.52	General	167	16	183	72	39.3	29	15.8
	1.22	0.31	Specific	115	17	132	72	54.5	4	3.0
		1.52	Specific	196	28	224	101	45.1	39	17.4
62	0.76	0.31	Specific	133	26	159	76	47.8	0	0.0
		1.52	Specific	127	36	163	92	56.4	0	0.0
		1.52	Chevron	90	21	111	92	82.9	0	0.0
		1.52	General	107	18	125	68	54.4	0	0.0
		1.52	Diagram	91	24	115	72	62.6	0	0.0
	0.91	0.31	Specific	62	14	76	33	43.4	0	0.0
		1.52	Specific	123	40	163	80	49.1	0	0.0
		1.52	Chevron	95	23	118	72	61.0	0	0.0
		1.52	General	92	18	110	68	61.8	0	0.0
		1.52	Diagram	112	25	137	69	50.4	0	0.0
	1.22	0.31	Specific	77	23	100	41	41.0	0	0.0
		1.52	Specific	69	27	96	46	47.9	0	0.0
		1.52	Chevron	59	19	78	57	73.1	0	0.0
		1.52	General	102	31	133	101	75.9	0	0.0
		1.52	Diagram	68	10	78	43	55.1	0	0.0

Note: 1 m = 39.3 in.

Figure 9. Effect of sign size on queuing.

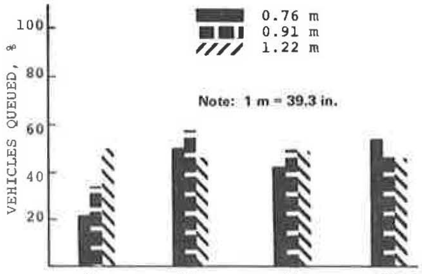


Figure 10. Effect of sign height on queuing.

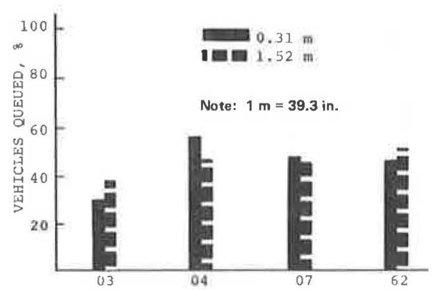


Figure 11. Effect of sign size on conflict.

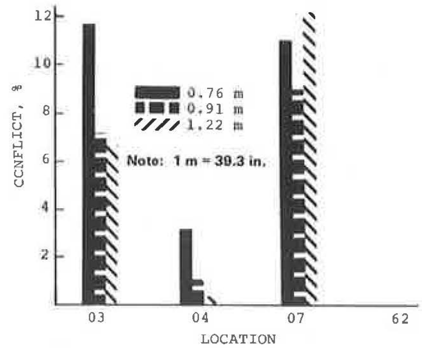


Figure 12. Effect of sign height on conflict.

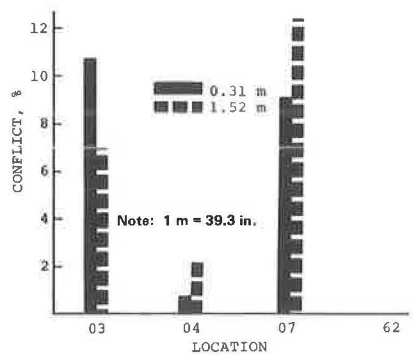


Figure 13. Effect of sign size and legend and flashing chevrons on mean speed reduction for Interstate locations.

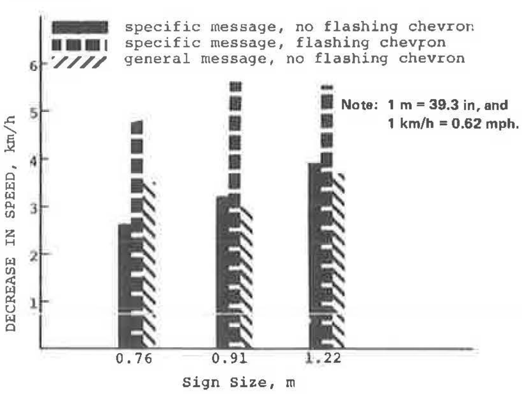
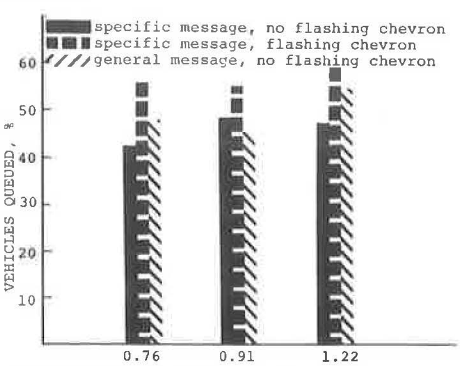


Figure 14. Effect of sign size and legend and flashing chevrons on queuing for Interstate locations.



Note: 1 m = 39.3 in.

Figure 15. Effect of sign size and legend and flashing chevrons on conflict for Interstate locations.

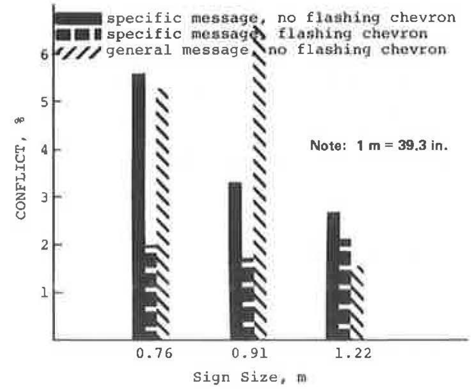


Figure 16. Trailer-mounted flashing chevrons.

arrows. We used the left-directional chevrons in our evaluation.

The differences in mean speeds and queuing were significant at the 5 percent level. Measured conflict was slightly less for flashing than for nonflashing conditions, but not to any statistical extent. The greatest difference in queuing was observed at location 62, where 72 percent queued with the chevrons as against 51 percent without.

General Discussion

Figures 13, 14, and 15 show comparative data for the study variables pooled according to size, over all locations, for speed, queue, and conflict, respectively. These data represent weighted averages and are presented here to reflect the overall trend of the effects of sign size, legend, and attention-getting devices on driver response as measured by speed, queuing, and conflict in the critical zone. Although the effect of location on response predominated, we had to assume that such variations among locations represent variations caused by psychological factors, which are random.

There is a significant variation in driver awareness of sign size. The 0.91-m (30-in) signs yielded a greater number of conflicts than the other two sizes; likewise, the queuing for this size was the least of the three. The effect of sign legend is insignificant. The most significant difference in driver obedience was observed with the flashing signs used to supplement the normal signs. The presence of such an attention-getting warning system emphasizes the sign messages of some activity that is going on. Disrespect for standard maintenance signs has some validity, because it is not uncommon for motorists to encounter such warning signs that accompany a total lack of maintenance activity at the specified distance. Directional flashing signs give motorists a genuine warning of the situation ahead.

We realized an added advantage of such devices during the actual setup of the barricade and zone taper. Although flagmen and flashing beacons on work vehicles have proved effective during this initial setup, the safety provided by the use of these flashing chevrons, coupled with the reduction in total time required for installing them, was significantly better.

SUMMARY AND CONCLUSIONS

In the preceding sections we attempted to evaluate, quantitatively, the effects of certain variables defined by sign size, type, and legend on driver response as measured by speed, conflict, and queuing parameters. Effects of flashing signs were also evaluated in terms of the above responses. The experiment was conducted at four locations on two-lane highways and on the Interstate system that required single-lane closures during maintenance. The seven conclusions that follow are based on the analysis and evaluation of the various responses using analysis of variance.

1. Motorists do respond to advance-warning signs, as was indicated by reduced speeds in the critical zone. However, this reduction is much more pronounced for two-lane roads than for the Interstate system.
2. The height of the sign does not indicate any statistical difference in any of the measured responses for either two-lane roads or for the Interstate system.
3. For two-lane roads there was a recognizable difference in speed reduction for the three sign sizes. However, the 0.76-m (30-in) sign yielded better response (greater speed reduction) than either the 0.91-m (36-in) sign or the 1.22-m (48-in) sign.
4. The significant difference in the dependent variables caused by location factor can be attributed to the traffic volume parameter, in addition to the driver's attitude toward signing in general.
5. At Interstate locations the 0.91-m (36-in) signs yielded better results than the 0.76-m (30-in) signs. The difference between the 0.91-m (36-in) size and the 1.22 m (48 in) was negligible.
6. Driver response to sign legend was statistically insignificant.
7. Flashing chevrons greatly enhanced the obedience of the driver to warning signs and also provided greater safety to the work force and the motorists during both initial sign installation and subsequent maintenance activity.

ACKNOWLEDGMENTS

We are grateful for the assistance and cooperation provided by the district maintenance engineers and their personnel during the study evaluation. Appreciation is also expressed to John Melancon and Veto Yoches, who managed this study during various phases, and to many other support personnel who helped in the successful completion of the study. The study was conducted under the Louisiana HPR program in cooperation with the Federal Highway Administration. The opinions, findings, and conclusions expressed in the paper are ours and not necessarily those of the State of Louisiana or the Federal Highway Administration.

REFERENCES

1. Manual on Uniform Traffic Control Devices for Streets and Highways. Federal Highway Administration, U.S. Department of Transportation, 1971.
2. W. M. Seymore, R. C. Deen, and J. H. Havens. Traffic Control for Maintenance on High-Speed Highways. TRB, Transportation Research Record 484, 1974, pp. 24-35.
3. Maintenance Traffic Control Handbook. Louisiana Department of Highways, 1973.

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Risk Assessment for Solving Transportation Problems

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Battelle, Pacific Northwest Laboratories, is currently conducting a research program sponsored by the Energy Research and Development Administration to assess the risk of transporting energy materials. The risk assessment model, although originally developed for use in analyzing shipments of radioactive materials, can be used to evaluate the risk of shipping any hazardous material. This paper briefly reviews the risk assessment method and describes how it can be used to solve hazardous materials shipping problems.

A clear understanding of the safety aspects is essential to planning and regulating the transport of potentially hazardous materials. Research programs are one method of improving the level of understanding.

Since 1972, Battelle, Pacific Northwest Laboratories (PNL), has been conducting a transportation safety studies program for the transportation branch of the Division of Environmental Control Technology of the U.S. Energy Research and Development Administration. The initial purpose of the program was to develop and use a model to assess the risk associated with the shipment of radioactive materials. Recently, this program has been expanded to include transport of all potentially hazardous energy-related materials, both nuclear and nonnuclear. Risk analysis was chosen for use in assessing safety because it allows us to predict the consequences of releases of hazardous materials in relation to how often accidents might be expected to occur.

A National Transportation Safety Board special study (1) pointed out the variety and inconsistency of current regulations governing the transport of hazardous materials via various transport modes. The desirability of determining the relative levels of risk for various commodities is clearly shown in this study.

BACKGROUND ON RISK ASSESSMENT

Risk, as used in the context of this paper, is defined as the magnitude of a possible loss multiplied by the expected frequency of loss occurrence. Two measures of risk are useful in assessment, (a) the total risk, obtained by adding up the risk associated with each particular loss, and (b) a risk spectrum (Figure 1).

To illustrate, we will assume that the consequence of interest is the number of fatalities expected from accidents. The expected frequency of N or more fatalities is plotted as a function of N. The risks associated with two activities are truly similar if they have the same total risk (risk magnitude) and the same risk spectrum.

In the past, safety studies have used only historical data and previous accident-free experience to assess the safety associated with transport of potentially hazardous materials. The method we developed supplements this technique but does not replace it. Risk assessment techniques permit proper consideration of both past and possible accidents.

ASSESSMENT TECHNIQUE

PNL has already completed a number of risk assessments (2, 3, 4) to provide the background needed to demonstrate the usefulness of risk assessments. The technique developed for use in those studies will be reviewed, using examples from the studies themselves.

The risk analysis method comprises four steps: (a) system description, (b) release sequence identification, (c) release sequence (and severity) evaluation, and (d) risk calculation and assessment. The risk analysis model has been described in detail by Hall and McSweeney (2) and will be treated only briefly here.

Figure 1. Sample risk spectrum for plutonium shipment in early 1980s.

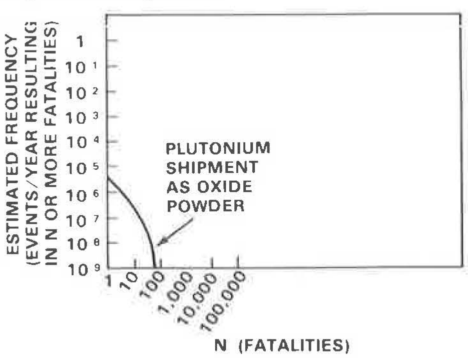


Figure 2. System description for plutonium shipment analysis.

WHAT: ______ PuO_2 POWDER

HOW MUCH: ____ 18 Mg OF Pu, 2.55 kg PER CONTAINER

WHEN: ______ 1980

AVERAGE SHIPMENT DISTANCE: ____ 2372 km

HOW: ______ EXCLUSIVE USE, ESCORTED TRUCK 100 kg Pu PER TRUCK IN 39 6M CONTAINERS

Note: 1 Mg = 2200 lb and 1 km = 0.6 mile.

System Description

The system description can be considered the what, how, when, and where step. A risk assessment is no better than the kind of information known about the system through which the material is being shipped. Most of this information is already known or easily available. The seven components of a complete description of the system generally are

1. Quantifying projected industry characteristics;
2. Specifying amounts, origins, and destinations of materials to be shipped;
3. Specifying the material's basic characteristics;
4. Specifying the transport mode and carrier;
5. Specifying the container type and amount per container;
6. Calculating the number of shipments required; and
7. Specifying route and restrictions and population and weather zones.

A portion of the system description used in the risk assessment of plutonium shipment by truck is shown in Figure 2.

Release Sequence Identification

Materials become a safety concern only when the barrier(s) between them and people are breached. Hazardous materials are shipped in containers that isolate them from the human environment, so the first step is to identify the possible ways the materials could be released during transport. The components of the system description provide most of the information needed to identify possible release sequences. Although many techniques can be used to identify release sequences, the most complete listing is obtained by working backwards from a postulated release through the chains of events or failures that caused the breach. We used a deductive method called fault-tree analysis to identify release sequences because we felt that this method decreased the likelihood of overlooking any important sequence.

As an example, in the risk assessment of a plutonium shipment by truck, the system description specified that the material would be shipped in a closed van in a U.S. Department of Transportation (DOT) Specification 6M container (Figure 3). A release of material into the environment would therefore require a simultaneous breach of four barriers—the sample can, the 2R inner container, the outer drum, and the van, as shown in Figure 4.

These barriers are shown on the second level of a fault tree in Figure 5. The fault tree is further developed below this second level to a point where probabilities can be assigned to various events that take part in breaching the barriers. The fault tree therefore shows all the possible ways that each barrier can fail during transport.

As an example of how the risk assessment method determines the likelihood of events that have not occurred, we will consider the 2R container. No 2R (inner) container has ever failed during transport; therefore, no data exist on its failure probability. However, the probability of failure can be determined from other information. Let us consider, for instance, the likelihood that the 2R container will fail during an accident.

The probability that a truck will be in an accident is known from accident data, and the force required to cause failure of the 2R vessel can be found by testing. The probability that the accident forces will exceed that level can also be derived from analysis of accident data. By using the above information, we can then determine the probability that the 2R container will fail in an accident.

The same type of development is also used for other failure paths. In addition to accident-caused failures, releases caused by packaging errors or as the result of normal transportation forces, such as jarring and vibration, can also be analyzed.

The technique described above is the key to the entire risk analysis. Estimates of risk can be made for events that have never happened, and each possible release sequence is detailed in a way that provides the specific conditions required for a release. This detailed analysis of each release sequence is, as we shall see later, very helpful in analyzing transportation safety.

Figure 3. Specifications for 6M container.

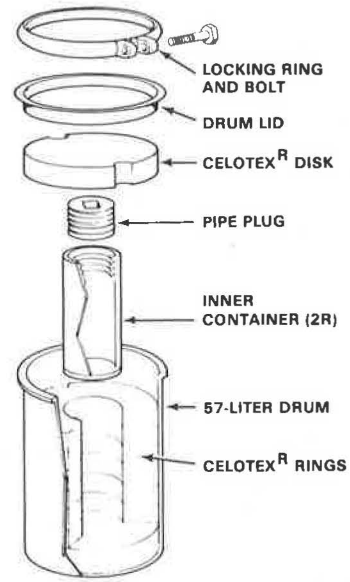
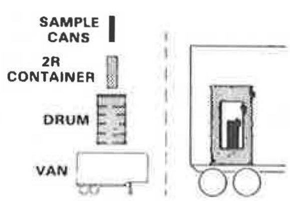


Figure 4. Barriers to plutonium release from van.



Release Sequence Evaluation

The third step in risk assessment is determining the severity of each individual release sequence. Since the severity of each sequence will be different, estimates of the amount of material that will be released in each postulated release sequence must be made. For instance, in shipping gasoline, a release sequence involving a leaking valve will probably release significantly less material than one in which the entire side of the tank is damaged. Where there are no accident history data, we can instead analyze the behavior of the hazardous material under the conditions of the postulated release sequence.

Risk Calculation and Assessment

The final step in the analysis is determining the consequences of each postulated release sequence, relating it to its respective occurrence rate, and combining these individual risks to obtain an indication of the total risk system.

In order for a particular material to be injurious to people, it must reach them after it is released. This might be a matter of a few feet (as in a gasoline spill) or several miles (as in an airborne release of a powder). Therefore, any evaluation of the consequences of a release should include such aspects as weather, population distribution near the release site, and the health effects of the particular released material. After all of the individual sequence risks have been combined, a risk spectrum can be determined.

APPLYING THE TECHNIQUE

A basic result of the risk assessment procedure is the determination of the overall risk. Comparisons of the relative safety of an activity can be made by expressing alternatives in terms of risk. For example, PNL has been analyzing plutonium shipments via truck (2), rail (3), and air (4).

In each case, the system description was the same except for mode of transport. This way each mode could be assessed and compared with the others to determine relative safety. The results (Figure 6) showed that the

Figure 5. Fault tree for plutonium shipment release sequences.

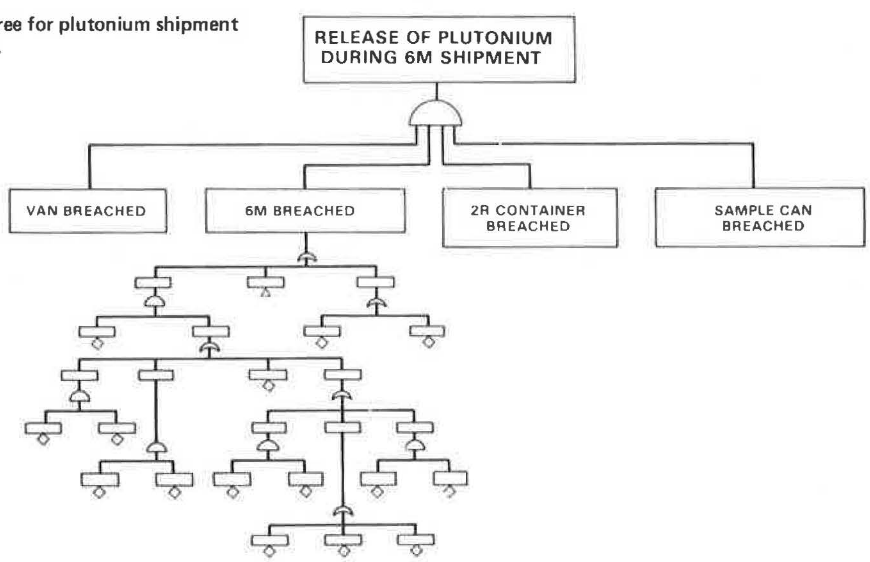


Figure 6. Risk spectra for plutonium shipments by three modes in early 1980s.

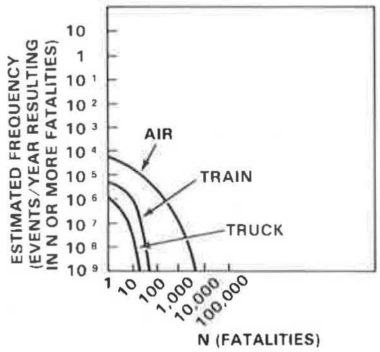


Figure 9. Risk comparison between existing and proposed regulations.

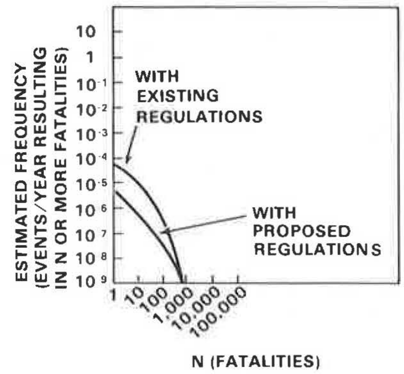


Figure 7. Risk spectra for plutonium shipments by truck in early 1980s.

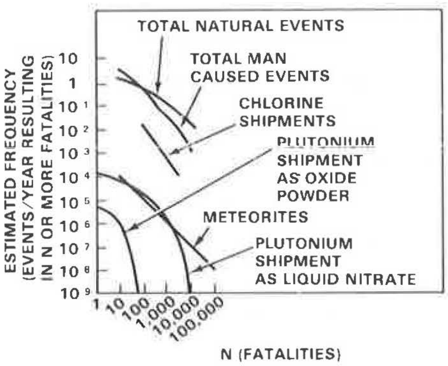
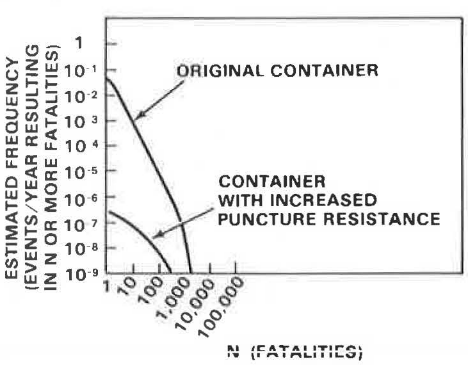


Figure 8. Risk comparison between old and redesigned containers.



risks of shipping plutonium on trucks and trains were about equal and that the risk by air was higher although still very low. Using risk assessment technique in this manner would show the shipper of hazardous material the safest possible mode.

Risk in the shipment of a hazardous material can also be compared to other risks to which human society is exposed. As an example, the risk of shipping plutonium is compared with other known risks in Figure 7. As can be seen, the risk of transporting plutonium at 1980 shipping levels is much lower than that of shipping chlorine.

It is also possible to compare the relative safety of shipping the hazardous material in different physical forms (solid, liquid, gas) by using the same method. Other things such as route changes, changes in the type of container used, and amounts per shipment can be analyzed as well.

A second important feature of the risk assessment technique is that the main contributors to overall risk can be identified. During the analysis, every possible release sequence (combination of events leading to a release) is outlined, and each element of the sequence is assigned a probability. We can then state the probability that a sequence will occur. At the same time, release fractions (the amount of material released during a particular sequence) and consequences are determined for each release sequence. If we multiply release sequence probability by its consequences, we can get an indication of that particular sequence's contribution to overall risk. The release sequences that make the greatest contribution to overall risk can thus be identified. These contributors can then be used to signal areas where modifications to reduce overall risk and increase safety could possibly be made. Combining this with other information on costs and benefits can lead to the best ways to increase the safety of hazardous material shipment.

For example, analysis might reveal a puncture of the container as the highest risk contributor. Then a redesigned container that is more puncture resistant could give new failure thresholds for reevaluating risk (Figure 8). A new risk spectrum showing the effect of the new container would indicate a decrease in overall risk.

The technique can also evaluate proposed changes in safety regulations. The merits of proposed regulatory changes can be assessed before implementation in terms of the overall transportation risk of a particular material. This is done by first finding the risk involved in shipping the material in the conventional manner. Next, a second risk analysis is made by using the proposed rule change (for instance, a requirement for greater wall thickness in containers). A comparison of the risks can then show what effect the new regulations will have on shipment safety (Figure 9). If the new regulation is found to significantly reduce risk, then further consideration should be made to implement it. By using the risk assessment technique, regulatory changes can be made on the basis of reduction in risk.

By comparing the risk spectra of various transportation modes, we can see whether one mode is being over-regulated in relation to others. The differences in safety of the various modes can also be compared.

The risk assessment technique can be valuable for solving a variety of transportation safety problems, of which we have presented a few of the more significant ones.

REFERENCES

1. Risk Concepts in Dangerous Goods Transportation Regulations. National Transportation Safety Board, U.S. Department of Transportation, Rept. No. NTSB-STS-71-1, 1971.
2. R. J. Hall and T. I. McSweeney. An Assessment of the Risk of Transporting Plutonium Oxide and Liquid Plutonium Nitrate by Truck. Battelle, Pacific

Northwest Laboratories, Richland, WA, BNWL-1846, Aug. 1975.
3. R. J. Hall. An Assessment of the Risk of Transporting Plutonium Oxide and Liquid Plutonium Nitrate by Train. Battelle, Pacific Northwest Laboratories, Richland, WA, BNWL-1996 (draft), June 1976.
4. T. I. McSweeney and J. F. Johnson. An Assessment of the Risk of Transporting Plutonium Oxide by Cargo Aircraft. Battelle, Pacific Northwest Laboratories, Richland, WA, BNWL-2030 (draft), July 1976.

Publication of this paper sponsored by Committee on Transportation of Hazardous Materials.

Abridgment

Transport of Hazardous Materials and Docket HM-112

Alan I. Roberts, Office of Hazardous Materials Operations, U.S. Department of Transportation

HM-112 is a serial docket number assigned to an omnibus regulatory action on several hundred different subjects pertaining to the U.S. Department of Transportation's (DOT's) regulations on the safe transport of hazardous materials. The principal matters addressed in the action are

1. Consolidation of DOT's hazardous materials regulations into a single volume;
2. Allocation of one part addressing hazardous materials communications, documentation, marking, labeling, and placarding;
3. Realignment of the regulations applicable to certain hazardous materials that are consumer commodities;
4. Elimination of all regulations pertaining to certain materials;
5. Complete reissuance and restatement of the modal regulations pertaining to transport of hazardous materials by air, rail, and water;
6. Addition of four new classes of materials, or other regulated materials (ORM), to be subject to certain regulations when transported by air or water or both;
7. Requirement that all materials classed as class B poisons and those materials in other classes also meeting the definition of class B poisons be labeled to identify their hazards even in quantities previously exempt from labeling requirements; and
8. Many other changes necessary to unifying and clarifying DOT's hazardous materials regulations.

This amendment is probably the most significant action taken over the past 60 years. It is important because it brings all the department's regulations together into a single volume. It also improves the safety regulations pertaining to the safe transport of hazardous materials by making them as intermodally compatible as practicable. All persons concerned with the department's regulations—shippers, carriers, or emergency, regulatory, or enforcement personnel—will agree that this is an important rule-making action.

The impact of HM-112 is best judged by the people affected by the regulations adopted under the Docket, who agree that consolidation is a benefit. Now they need only deal with a single volume when they class a material, determine its required packaging, marking, and labeling, prepare shipping documents, and identify transport vehicles regardless of the mode or modes to be used.

The new hazardous materials table set forth in Section 172.101 applies to four modes of transport for the first time in 60 years of regulation. Furthermore, consolidation eliminated more than 700 pages of federal regulations and thereby the need to wade through three different volumes to find the applicable requirements on transport of hazardous materials. Of further benefit was the elimination of requirements that were incompatible for movement between modes. In the past, regulations addressed requirements in different places and not only were inconsistent in several areas but also failed to recognize intermodal movements.

For example, there were different placarding requirements for rail and highway for 40 years. When a tractor semitrailer moved to a rail yard, the placarding on the trailer was not appropriate for its transport aboard a rail car. Worse yet, though, was the situation for intermodal transport involving carriage aboard vessels. The system failed to recognize intermodal container movement, which has become a very important form of transporting all kinds of goods in commerce. Under the new system, the shipper knows how to label the package, mark the contents on the outermost packaging, prepare documentation, and apply placards to freight containers that will be transported by one or a combination of modes. These requirements are now set forth in part 172 of DOT's hazardous materials regulations.

Another important fact is that now the labeling and placarding system can be considered consistent with the international standards and provisions for the additional communication required by some international regulatory bodies, such as the Intergovernmental Maritime Consultative Organization.

It has been estimated that more than $60 billion worth of retail consumer commodities sold annually in the United States fall within the hazardous materials definitions set forth in the regulations. All aerosol products and such products as nail polish, aftershave lotion, paints and related materials, and many cleaning compounds are classed as hazardous under the regulations.

In 1972, a notice was published in the Federal Register requesting public participation and comment on whether some form of adjustment should be made in the regulations as they apply to these materials. Many responses supporting the contention that these materials were in some ways overregulated were received. Comments along these lines were also received from the president of the New York City Fire Fighters Union; this suggested that certain adjustments should be made, particularly in the area of documentation requirements.

After the comments were carefully considered, a notice was issued, on January 24, 1974, that removed certain regulatory requirements pertaining to the transport of all hazardous materials covered by small quantity partial exemptions in the regulations. After the comments on the notice were evaluated, it was decided that the final regulations on some materials, such as those packaged and distributed in a form intended or suitable for sale through retail sale agencies or instrumentalities for consumption by individuals for the purpose of personal care or household use, be limited. Also included in the provision were drugs and medicines.

One of the important aspects of an action of this type is the fact that a number of transportation requirements addressing a large block of materials posing a very limited transport hazard were removed. Greater emphasis can now be placed on those materials remaining under regulation. Until now, much time and effort was spent on these retail materials. Basically, the consumer commodity provision makes certain requirements of shippers and places limitations on quantity, type of packaging, package marking, and certain special qualifications. When the shipper has met these requirements, he or she may offer them for transport by air, highway, rail, or water in the same fashion, except that shipping documents are required for air shipments. Virtually no regulations, other than incident reporting and air documents, apply to transport of such materials by carriers.

During the rule-making procedure, it became clear that there were several materials being regulated that did not warrant treatment as hazardous. It is interesting to note, for example, that before July 1, 1976, it was illegal to transport an inflated truck or bus tire unless the tire was a part of the vehicle in which it was placed. Many carriers transport tires to tire banks nationwide, and, since the standard truck tire is inflated to 550 kPa (80 lbf/in^2) or more, its contents conform to the definition of a compressed gas under the regulations. No provision was made for such transport in the regulations. Although the enforcement personnel were not citing people for transporting inflated tires, a federal regulation prohibiting such transport created a risk for unknowing individuals doing so.

Other materials considered were carbonated beverages, air conditioners, refrigerators, and tennis balls of internal pressures exceeding the definition of a compressed gas. Tennis balls, in terms of quantity and form, hardly pose a risk that should be regulated for purposes of transportation safety. Another example is film manufactured today, which is of the safety type. Therefore extensive modifications and deletions in the regulations were made for transporting film. The same treatment was also given to flash bulbs, which today are not considered to pose the same kinds of hazard that bulbs made many years ago did.

For several years the National Archives Service has been recasting rules for carrying hazardous materials by air, rail, and water. It is important that the regulations be set forth clearly and concisely, so that those who ship or transport hazardous materials can easily inform themselves of what is required of them. Over the past 60 years, the regulations have been amended on a piecemeal basis but never completely overhauled in terms of layout or manner of expression. In the 200-word paragraphs of the old regulations, much could be missed or misunderstood. Appropriate enforcement of regulation requirements not clearly understood is a problem. This effort to codify and state them clearly has been very effective. The transfer of the regulations pertaining to the transport of hazardous materials by water, for example, resulted in the elimination of more than 700 pages of previous regulations. The recodification and restatement effort should be completed in the next 2 years.

Four new classes of materials (ORM) will be subject to certain regulations when transported by air or water or both. Each ORM class is covered by a suffix, A, B, C, or D. An ORM-D material is one identified by the shipper as a consumer commodity. The other three letters designate materials that can create certain irritating or toxic effects (ORM-A), can cause destruction to the structure of an aircraft (ORM-B), or have certain properties warranting specific attention when transported in large volumes aboard vessels (ORM-C). The ORM-C designation is the principal recipient of requirements previously in the Hazardous Articles class in the Coast Guard regulations. A typical material under the latter would be castor beans, which pose a hazard when transported in large volumes aboard vessels.

Simply stated, the ORM classes address certain materials that warrant a limited amount of regulation usually applicable to only one mode of transport.

All class B poisons, and those materials in other classes also meeting the definition of a class B poison, must be labeled to identify their poison hazards, even when they are in quantities previously exempt from labeling requirements. In the past, the regulations pertaining to the labeling of class B poisons applied only if the material was a designated class B poison.

There was previously no dual labeling requirement for flammable liquids that also met the definition of a class B poison. Historically, with few exceptions, flammability took precedence over toxicity in determining the class of a material, because the greatest transport problem has been fire. There have been very few transport accidents resulting from poisoning. This does not mean that we should ignore the potential hazard of poisons during transport. One major regulation pertaining to the transport of poisons is the prohibition against their being in the same cargo compartment or vehicle with materials marked as or known to be foodstuffs, feed, or any other material intended for consumption by humans or animals.

The new regulations require that the poison label be placed on any package containing a material that conforms to the definition of a class B poison, regardless of class and quantity. The only exception pertains to drugs intended for consumption by humans.

These new requirements for poisons, along with those pertaining to hazardous materials communications, reflect one of the most significant increases in the level of regulation imposed under Docket HM-112.

In the eyes of a safety professional, Docket HM-112 brings many improvements, especially communications requirements, to the hazardous materials transport safety program. Of shippers of industrial chemicals and petroleum products, for instance, it requires more effort to assure compliance with several new and some improved old requirements.

Many have expressed appreciation of the benefits of the consolidated regulations. The Docket has given the shippers of consumer commodities, drugs, and medicines relief from the documentation requirements that caused them considerable problems in the past.

Carriers reactions to the Docket are mixed. Motor carriers must convert to the new placarding format. Rail carriers are required to follow revised car placement. Train crews must also carry documents indicating the locations of all placarded cars in their trains. Overall, the standardized documentation and placarding requirements will improve the ability of carriers to carry out their responsibilities.

The view of the public is probably the most difficult to measure, because the general public is not aware of most programs. As new regulations are implemented, they will need to be instructed on one communication scheme rather than two for placards on vehicles, and instructions on how to read shipping papers will be provided. In the past there was no format for hazardous materials descriptions.

Although this review has been general in nature, the importance and impact of HM-112 have been outlined.

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