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Aerial Surveys, Geometrics, Surface Drainage, Ecological Impacts, and Safety Appurtenances

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Contents

Aerotriangulation Research to Reduce Ground Control Requirements

Robert H. Holdridge, Wisconsin Department of Transportation, Madison

Aerotriangulation research conducted by the Wisconsin Department of Transportation to determine the minimum control configuration that would yield accurate photographic control with error-detection capability is described. A new aerotriangulation program package developed for the department's photogrammetry system was used. Conclusions are based on the results of four projects in which 1:3000-scale photography was used at a flying height of 457 m (1500 ft). In the test procedure, an adjustment that used all available control was taken as a basis of comparison in determining a minimum control standard, with redundancy, to allow for detection of single and multiple errors. Accuracies were determined based on the standard deviation of discrepancies among withheld-control points and pass-point movement. The relation between analytical instrumentation, control configuration, and program capabilities has resulted in standards that produce equal adjustments to different projects. The results indicate that the bridging distance between successive vertical wing points and successive horizontal picture points can be as much as six models and all control points can be in double-overlap areas. The new program package has greatly reduced field survey time and increased design flexibility.

An aerotriangulation computer program package has been developed specifically for the analytical photogrammetry system used by the Special Services Section of the Wisconsin Department of Transportation (DOT). This program package consists of integrated computer programs that can perform either fully analytical or semianalytical control extension by using a least-squares simultaneous adjustment of blocks of photographs and several editing routines.

Research was conducted to determine the capabilities and limitations of the new aerotriangulation system and the reduction of ground control allowed in 1:3000-scale photography with a 152-mm (6-in) focal length camera and a flying height of 457 m (1500 ft).

RESEARCH OBJECTIVE

The objective of this research was to determine the minimum ground control configuration required to produce accurate photographic control, in accordance with 1968 Federal Highway Administration standards (1), in 1:3000-scale production projects. Another objective was to determine the ability to detect single and multiple errors in photogrammetric measurements and ground control surveys at various ground control configurations.

SELECTION OF TEST PROJECTS

Four test projects were selected from among projects that had recently been processed by using analytical sequential strip formation and a polynomial strip final adjustment. Since these projects contained many more ground control points than it was believed the new program package would require, they provided an abundance of ground control for use in testing different control configurations and completing the research objectives.

Every attempt was made to select representative projects. Prime consideration was given to such variables as (a) the length of photographic strips, (b) multiple strip configuration, (c) the quality of analytical instrumentation, and (d) the quality, configuration, and type of ground control.

TESTING PROCEDURE

A basis of comparison was needed to test the results of the four production test projects. An overabundance of the ground control required for a simultaneous adjustment made it possible to withhold many ground control points from various adjustments for use as test points.

An adjustment in which all available ground control was used became a basis of comparison with all subsequent test runs for determining a minimum control standard. Control points were withheld from each solution to determine the accuracy of the adjustment. "Pass-point" movement was also determined to indicate the adjusted coordinate strength of pass points used in compilation (a pass point is a photographic control point that is mechanically produced in the analytical process and is usually not identifiable on the ground). A root-mean-square error was calculated for discrepancies among points at which control was withheld and differences of pass-point positions based on a comparison of coordinates for total control adjustments and minimum control adjustments. Additional tests were made by using a vertical ground control point in the center of each model because the policy of the Special Services Section requires these points for stereoplotter indexing.

The capability to detect single and multiple ground control errors was also tested and incorporated into the final control standards for horizontal and vertical control. Analysis of error-detection tests yielded several guidelines that were used to help determine the minimum control configuration. The magnitude of detectable error appears to be 0.18 m (0.6 ft) in coordinates X, Y, and Z. This type of error, however, does not always show up at the point itself within the polynomial or simultaneous adjustments.

RESULTS

Projects

Each of the four projects tested contributed to the development of control standards for production projects that use 1:3000-scale photography:

1. Project 1 (see Figures 1-4)—Three strips were run to form a block configuration of 21 models representative of most flight designs covering a highway corridor. All ground control points were targeted.
2. Project 2 (see Figures 5 and 6)—Two cross-flight strips were run to form a block configuration of 14 models representative of highway interchange areas. Ninety percent of ground control points were targeted.
3. Project 3 (see Figures 7 and 8)—One long strip of 25 models was selected to test the strength of strip formation orientation and simultaneous adjustment. Ninety percent of ground control points were targeted.
4. Project 4 (see Figures 9 and 10)—This 17-model strip was tested because it appears to be the optimum length for maintaining strong analytic orientation during strip formation. This strip was considered to have weak analytic orientation compared with the other three projects tested.

The root-mean-square discrepancies at the control points and pass points for the four projects tested are given in Table 1.

Standards

Figure 11 shows samples of the ground control standards determined by the Wisconsin DOT for 1:3000-scale photography. These standards are summarized below.

Horizontal Control

1. All horizontal control points may be double overlap.
2. Each strip must contain four horizontal points for redundancy.

Figure 1. Total control configuration: project 1, run A.

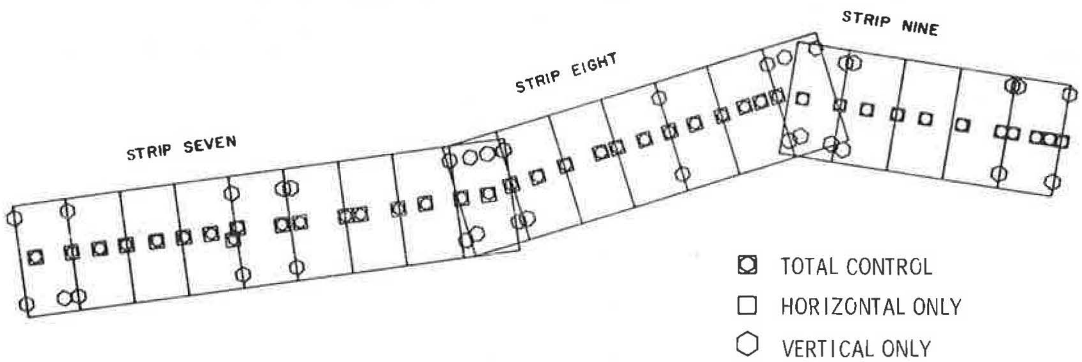


Figure 2. Minimum control without vertical index points and without redundancy: project 1, run B.

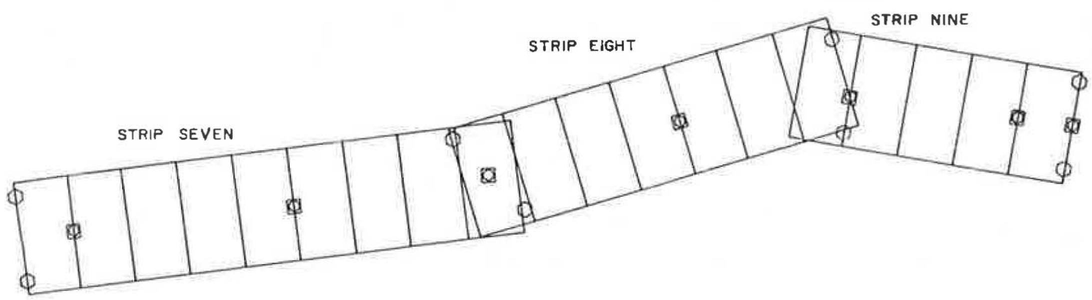


Figure 3. Minimum control with vertical index points and redundancy: project 1, run C.

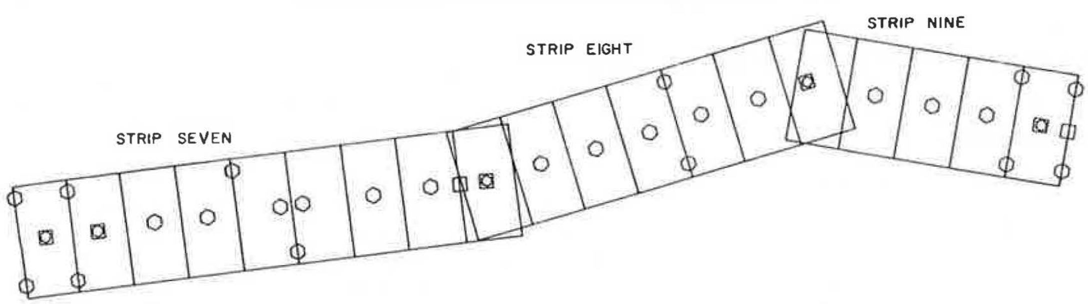


Figure 4. Minimum control with vertical index points and redundancy: project 1, run D.

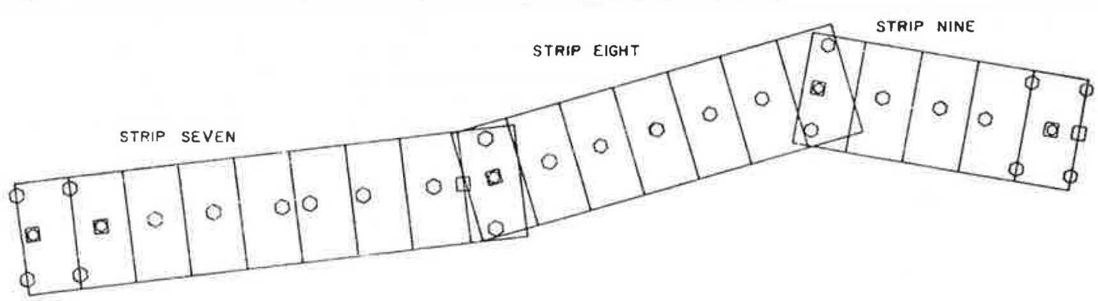


Figure 5. Total control configuration: project 2, run A.

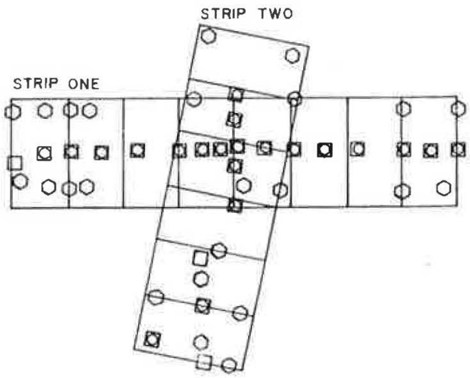


Figure 6. Minimum control with vertical index points and redundancy: project 2, run B.

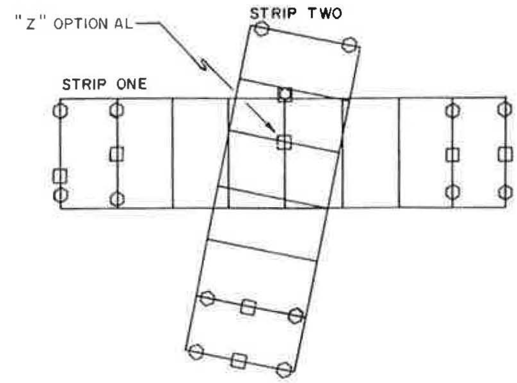


Figure 7. Total control configuration: project 3, run A.

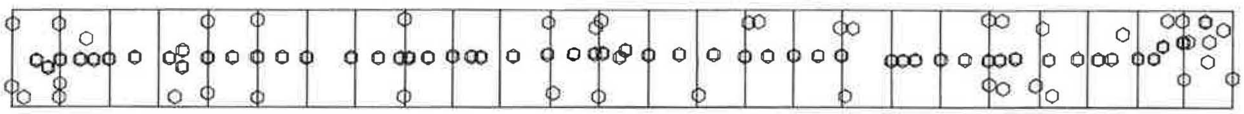


Figure 8. Minimum control with vertical index points and redundancy: project 3, run B.

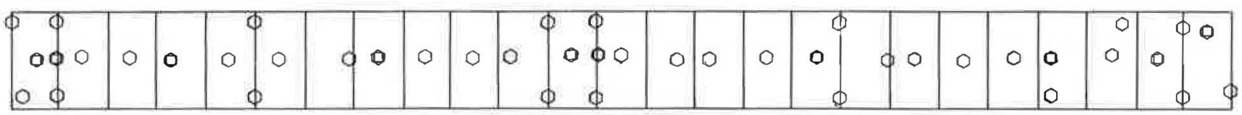


Figure 9. Total control configuration: project 4, run A.

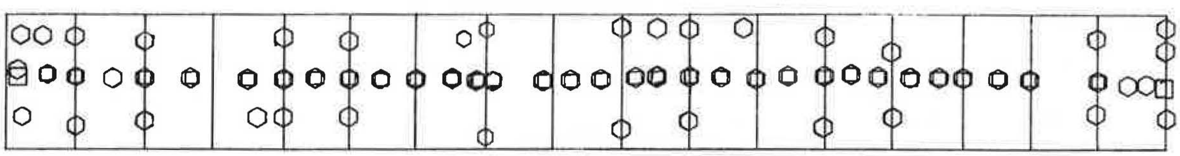


Figure 10. Minimum control with vertical index points and redundancy: project 4, run B.

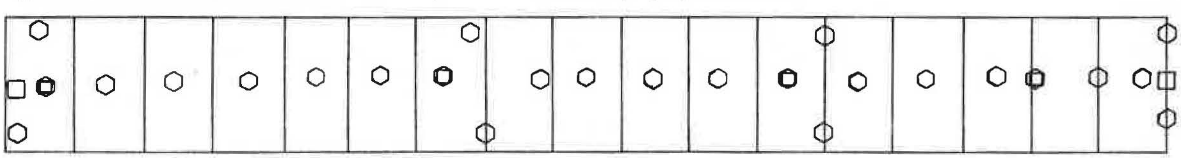
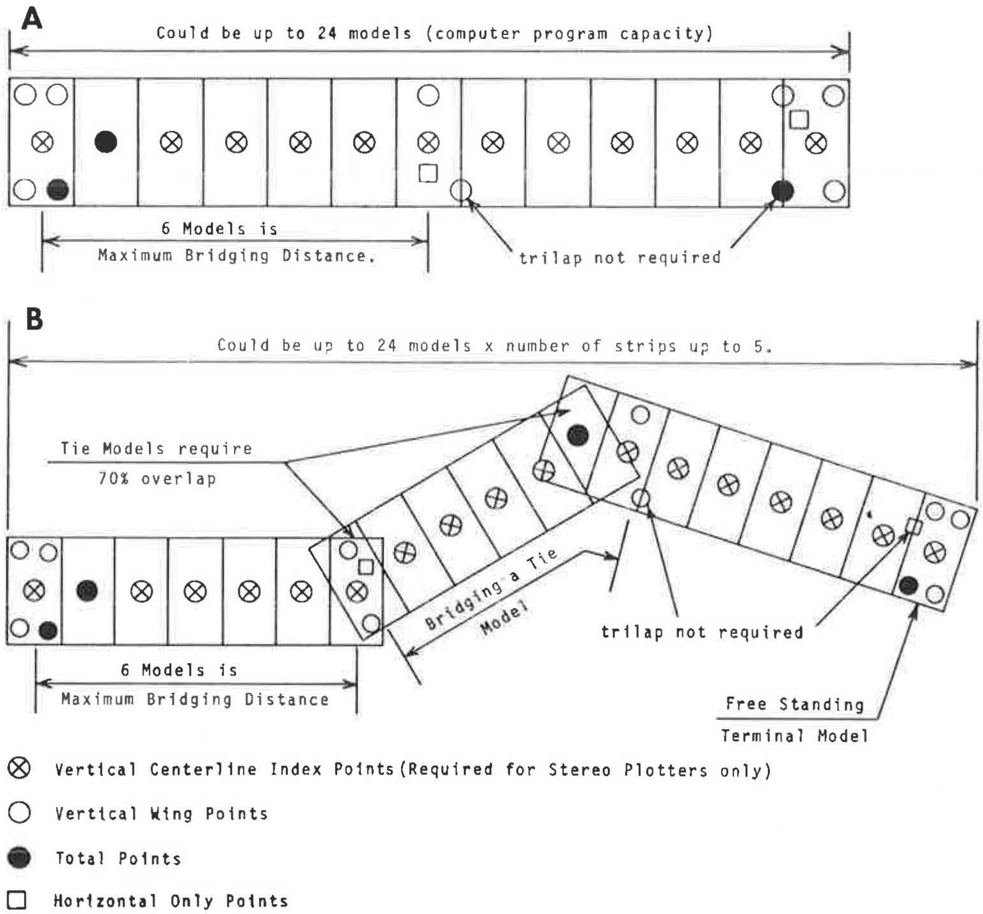


Table 1. Root-mean-square error of control and pass points for coordinates X, Y, and Z.

		Root-Mean-Square Error (m)								
		Control Held			Control Withheld			Pass-Point Movement		
Project	Run	X	Y	Z	X	Y	Z	X	Y	Z
1	A	0.018	0.020	0.027						
	B				0.043	0.030	0.048	0.035	0.025	0.034
	C				0.043	0.043	0.043	0.025	0.027	0.021
	D				0.044	0.044	0.039	0.026	0.026	0.019
2	A	0.016	0.028	0.031						
	B				0.024	0.056	0.038	0.022	0.041	0.026
3	A	0.028	0.026	0.037						
	B				0.033	0.037	0.048	0.019	0.026	0.028
4	A	0.032	0.026	0.029						
	B				0.063	0.043	0.051	0.053	0.038	0.036

Note: 1 m = 3.28 ft.

Figure 11. Ground control points required (a) for single strips and (b) for multiple strips.



3. At least one horizontal point is required in each terminal model of each individual strip of a block solution.
4. The horizontal control-point spacing may be as much as six models or seven triple overlaps (trilaps).
5. "Tie" models (points of overlap between strips) within a strip block type of adjustment must contain at least one horizontal point.
6. All horizontal points may be placed along the centerline of a flight strip or may coincide with wing-point placement (a wing point is a picture point on either side of a strip of photographs, usually surveyed for only vertical control).

Vertical Control

1. All vertical control points may be double overlap.
2. The free-standing terminal models of a strip block type of adjustment must contain total wing-point design and one centerline index point for redundancy.
3. The vertical control-point spacing may be as much as six models or seven triple overlaps when centerline index points exist.
4. Wing points should be located as far out from the center of the model as is physically possible.
5. The flexibility in vertical control design provides the ability to bridge tie models between strips in a block configuration and incorporate this area within the six-model wing-point spacing.
6. The length of individual strips is limited to 17 models because of a deterioration of interior orientation during strip formation, which is not offset by the simultaneous adjustment without use of additional control. Strips that exceed 17 models in length may be split without using full terminal control by treating them as a tie model.

CONCLUSION

Aerotriangulation research conducted by the Wisconsin DOT has been successful in determining new ground control standards for use in large-scale highway mapping and determining pay-quantity cross sections on highway construction projects. The need for sufficient redundant control was incorporated into the standards to make it possible to detect errors during aerotriangulation. These standards for ground control have resulted in a substantial reduction in field work and increased flexibility in the placement of control points.

In the aerotriangulation system developed by the Wisconsin DOT, the average horizontal positional error distribution of photogrammetric control has a standard deviation of approximately 1 part in 10 000 of the project flying height. The error in vertical position has only a slightly larger standard deviation than that in horizontal position. The accuracies obtained are the result of good survey procedures and ground control, properly calibrated analytical equipment, and a simultaneous aerotriangulation adjustment program.

REFERENCE

1. Reference Guide Outline Standards: Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways. Federal Highway Administration, U.S. Department of Transportation, 1968.

Publication of this paper sponsored by Committee on Photogrammetry and Aerial Surveys.

Guidelines for Constructing Local Roads in New York's Adirondack Park

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The Adirondack Park in upstate New York contains more than 23 000 km^2 (9000 miles^2) of public and private lands. Most state-owned land is designated by the state constitution to remain "forever wild", and development of private land is closely controlled by the Adirondack Park Agency, which is part of the executive branch of the state government and also has jurisdiction over construction of new municipal roads and expansions of existing ones. Guidelines that have been developed for use in lieu of review of individual local road projects by the Adirondack Park Agency are presented and discussed. The guidelines are presented in seven categories: (a) planning, (b), alignment, (c) cross section, (d) roadbed construction, (e) riding surface, (f) bridges and culverts, and (g) general construction. Their objective is to ensure that local roads are constructed or reconstructed so that they fit harmoniously into the natural surroundings and impart the feeling of being in a park. Local road standards issued by the American Association of State Highway and Transportation Officials contained some geometric guidelines that were considered inappropriate for widespread use in the Adirondacks.

In 1892, the state of New York established the Adirondack Park, which now consists of >2300 km^2 (9000 miles^2) and is the largest park in the continental United States. About 60 percent of the land is privately owned; the remainder, about 10 000 km^2 (3800 miles^2) is state land that is primarily under the jurisdiction of the New York State Department of Environmental Conservation as part of the Adirondack Forest Preserve. This mixture of public and private lands posed many problems, so in 1968 Governor Nelson A. Rockefeller appointed the Temporary Study Commission on the Future of the Adirondacks to assess and make recommendations for the future use of all lands in the park. The commission's report resulted in (a) the creation of the Adirondack Park Agency (APA), (b) a Master Plan for State Lands, and (c) a land-use and development plan for all private lands in the park.

The Master Plan for State Lands, issued in 1972 by APA, classified all lands and promulgated extensive guidelines for their care, custody, and control. The guidelines for state lands classified as travel corridors called for "parklike" roads that complement the total Adirondack environment. Although the master plan applied only to state lands and therefore to state highways, it also called for the New York State Department of Transportation (NYSDOT) to use its influence over local governments to try to achieve similar objectives for other highway corridors within the Adirondack Park.

In 1976, NYSDOT issued special design standards for state highways in the park. These called for varying clearing limits, back slopes, and ditch depths and for avoiding wetlands where possible so that highways fit harmoniously into the natural surroundings and impart the feeling of being in a park. For reconstruction projects, these new standards will result in a total roadway width—including pavement, shoulder, ditches, and clear area—of only 28 m (92 ft) compared with 40 m (132 ft) for a similar roadway outside the park. For rehabilitation and preservation projects, the total clear width will be only 16.5 m (54 ft).

The guidelines suggested here present similar goals for local roads but have been modified somewhat because of the lower traffic volumes and speeds on these roads. The objective is to construct and reconstruct roads so as to ensure protection, conservation, and enhancement of the parklands. The guidelines emphasize that aesthetics and engineering are mutually dependent and that roads can be built that will be operationally safe and efficient and easier and cheaper to maintain and yet will blend attractively into the surrounding landscape. Figures 1 and 2 show examples of good construction practices in the park, and Figure 3 shows an example of what should be avoided.

Many researchers have questioned the applicability of the American Association of State Highway and Transportation Officials (AASHTO) Geometric Design Guide for Local Roads and Streets (1) and Highway Design and Operational Practices Related to Highway Safety (2) to low-volume rural roads and have suggested lesser standards (3-8). The AASHTO standards for local roads were generally considered to be too costly for roads in the Adirondack Park and to result in an overly wide road section that would not be parklike. Currently, about 50 percent of the 5900 km (3660 miles) of the park's town and county roads have gravel riding surfaces and are less than 4.3 m (14 ft) wide. It was necessary, therefore, to develop new guidelines in which the emphasis would be on minimal disruption of the area surrounding the roadway. These guidelines are presented here in seven categories: (a) planning, (b) alignment, (c) cross section, (d) roadbed construction, (e) riding surface, (f) bridges and culverts, and (g) general construction.

PLANNING

Because of increased concern for the environment, extra precautions must be taken in planning to build or reconstruct roads in environmentally sensitive areas such as the Adirondack Park. During the early stages of a project, adequate consideration should be given to all factors that could influence the location, type, and size of the road. Among these factors are the function of the road, its present and future traffic characteristics (speed, volume, and vehicle type), land use of the adjoining property, snow storage, and the safety of those traveling on the road. These engineering requirements must be integrated with environmental and scenic considerations so that no unnecessary damage is done to the surrounding landscape during construction.

On new construction, or in the reconstruction of a new alignment, the Department of Environmental Conservation and APA can assist in determining the existence or the location of particularly sensitive areas, such as wetlands, habitats of rare or endangered species, historic landmarks (see Figure 4), and forest preserve lands.

ALIGNMENT

The following guidelines are provided for alignment:

1. Alignment between control points should be to as high a standard as is commensurate with the topography, terrain, design traffic, obtainable right-of-way, and preservation and enhancement of the unique character of the park.
2. The road should blend with the terrain. A

Figure 1. Trout Pond Road in Essex County: typical low-volume gravel road with curvilinear alignment and minimal clear distance.

Figure 2. Typical high standard road with adequate lane and shoulder widths and clear distance and revegetated side slopes.

curvilinear alignment (see Figure 5) is visually and functionally preferable to tangents cut through hillsides, which leave unsightly cut slopes or fill slopes (see Figure 6).

3. Wherever possible, alignments should be chosen to bring interesting natural and man-made features into view.

4. Small dips and humps should be avoided in what is actually a uniform grade (see Figure 7), and "broken-back" curves should be avoided in what is actually one long curve (see Figure 8).

5. A sharp horizontal curve should not begin near the top or bottom of a hill. Generally, the horizontal curve should begin before the vertical curve starts and be somewhat longer (see Figure 9).

6. Consideration should be given to providing the best sight distance possible under prevailing conditions of terrain and topography while retaining geometrics appropriate to the park atmosphere. These considerations are of particular importance at intersections, at horizontal curves, at the crest of vertical curves, and especially on paved roads where higher speeds are likely. Opportunities for passing other vehicles should also be provided. The design values for sight distance recommended by AASHTO are given below (1 km = 0.62 mile; 1 m = 3.28 ft):

Figure 3. Example of poor construction practices: excessive clearing, unrelocated utility pole, and no revegetation.

Average Daily Traffic (no. of vehicles)	Maximum Anticipated Speed (km/h)	Sight Distance (m)		
		When Stopping	When Passing	At Intersections
<100	32-48	46-61	NA	61-92
100-400	48-80	61-107	336-549	92-152
>400	>80	131	610	168

High and low sight distances correspond to respective high and low anticipated speeds; e.g., for an average daily traffic of <100 vehicles, the recommended stopping sight distances are 46 m (150 ft) at 32 km/h (20 miles/h) and 61 m (200 ft) at 48 km/h (30 miles/h).

7. AASHTO recommends the following maximum grades for three types of terrain:

Average Daily Traffic (no. of vehicles)	Maximum Grade (%)		
	Flat	Rolling	Mountainous
<100	7	10	12
100-400	7	9	10
>400	6	7	9

CROSS SECTION

Selection of roadway width depends on the type, volume, and speed of anticipated traffic. Safety, environmental protection, and future land use must also be considered. Data for three typical sections (see Figures 10-12) are given in Table 1.

On certain low-volume roads, wider cross sections may be necessary, especially when school buses, recreational vehicles, logging trucks, and other large vehicles will be using the road. At some locations, climbing or passing lanes may be needed or shoulders may have to be wide enough for parking. Widening the riding surface on sharp horizontal curves should be considered wherever it is feasible.

Gravel riding surfaces should have a 4 percent cross slope [4.2 cm/m (0.5 in/ft)] to provide surface drainage. On asphalt surfaces, a 2 percent cross slope [2 cm/m (0.25 in/ft)] is adequate. A 6 percent cross slope [6.35 cm/m (0.75 in/ft)] should be used on shoulders. Cut-and-fill slopes should be 1 percent vertical on 2 percent horizontal or flatter, rock cuts

being generally no steeper than 3 percent vertical on 1 percent horizontal.

ROADBED CONSTRUCTION

Ideally, all roads in the Adirondack Park should be constructed with a 1.22-m (4-ft) high compacted embankment on top of existing ground that has been cleared of trees, stumps, and boulders. The top 0.61 m (2 ft) of embankment should be free of stones larger than 0.15 m (6 in). Excavation should be kept to a minimum but, where cuts are necessary, a 1.22-m ditch normally provides adequate subsurface drainage of the subgrade.

Figure 4. Historic Jay Covered Bridge, built in 1857, which carries Essex County Route 22 over the Ausable River.

Figure 5. Curvilinear alignment on low-volume gravel road.

Figure 6. Tangent section of road cut through hillsides.

The top 0.30 m (12 in) of the roadbed should be constructed with a clean, well-graded compacted gravel subbase material [50.8 mm (2 in) top size, 30-65 percent passing the 6.3-mm (0.25-in) sieve and 0-10 percent passing the 0.075-mm (no. 200) sieve]. This material should be used whether it is to be placed beneath a pavement or as the travel surface. In the latter case, the 50.8-mm top size gravel should minimize potholes and washboards. If 50.8-mm top size gravel is not readily available, other granular materials can be used in the lower 0.20 m (8 in) of the subbase, but these should have no particles larger than 0.10 m (4 in) and no more than 10 percent passing the 0.075-mm sieve.

On some town and county highways, where the cost of such construction may be prohibitive, the recommended first stage of construction is raising the roadbed and surfacing with 0.30 m (12 in) of gravel. The gravel should be clean and well-graded and have a gradation

Figure 7. Avoiding small dips and humps in uniform grades.

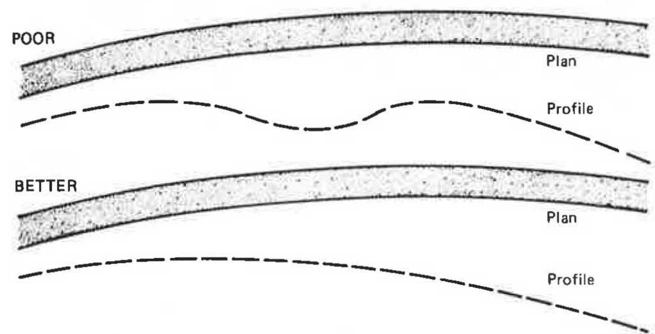


Figure 8. Avoiding broken-back curves.

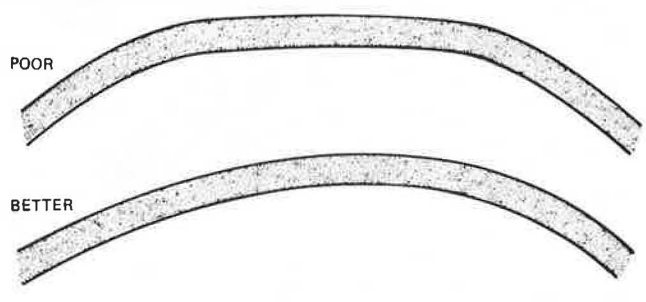


Figure 9. Use of horizontal and vertical curves in combination: Horizontal curve should begin before vertical curve and be longer.

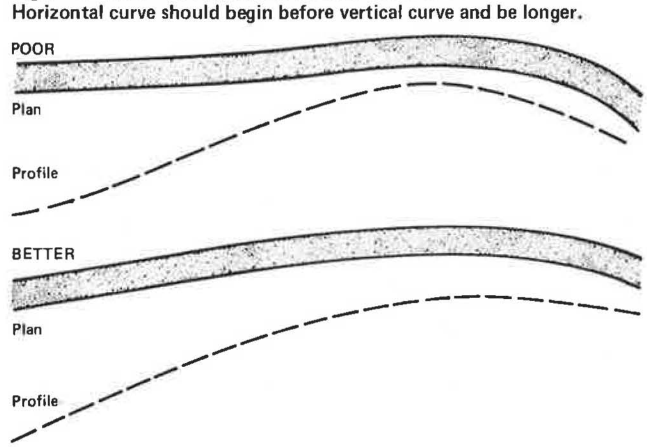


Table 1. Typical cross sections for various road categories.

Category	Avg Daily Traffic (no. of vehicles)	Maximum Anticipated Speed (km/h)	Width of Riding Surface (m)	Shoulder Width (m)	Clear Distance[a] (m)	Typical Surface Material
1	<100	32-48	4.3-5.5	0-0.6	3.0	Gravel
2	100-400	48-80	4.9-6.1	0.9-1.5	3.7	Double surface treatment
3	>400	>80	5.5-7.4	1.5-2.4	4.3	Plant mix or road mix

Note: 1 km = 0.62 mile; 1 m = 3.28 ft.

[a]For reasons of safety, the clear distance may be extended to the edge of the right-of-way.

Figure 10. Redmond Road in Essex County: typical category 1 low-volume gravel road.

Figure 11. Essex County Route 24: typical category 2 road with treated riding surface and gravel shoulders.

Figure 12. Herkimer County Route 4: typical category 3 road with paved riding surface and shoulders.

similar to that described above for subbase material. When it becomes necessary to upgrade an existing road because of problems related to frost, drainage, soft soils, or increased traffic, the upgrading should follow these guidelines—that is, raise the grade where possible and use 0.30 m of compacted subbase material.

To provide subsurface drainage, there should be ditches at least 0.15 m (6 in) below the bottom of the gravel. The top and bottom of the ditches should be rounded.

When particularly complex problems are encountered that involve foundation soils, earth or rock slopes, or subbase materials, the NYSDOT regional soils engineer can be consulted.

RIDING SURFACE

Low-volume roadways may be left with a gravel riding surface. When necessary, additional gravel with a top size of 0.05 m (2 in) may be added. For dust control on gravel surfaces, an alternative to oil would be calcium chloride. The riding surface can be upgraded by adding a double surface treatment in which an appropriate bituminous material is used (emulsion is preferred) with no. 1 or no. 1A stone. A more substantial riding surface could consist of a minimum 0.08 m (3 in) of a bituminous-stabilized gravel. This should be covered with a double surface treatment for a wearing course. For roadways that are subject to substantial traffic, a plant-mixed asphalt concrete with a minimum thickness of 0.06 m (2.5 in) should be used.

BRIDGES AND CULVERTS

All new bridge structures should be at least 1.22 m (4 ft) wider than the approach riding surface. For drainage structures with spans of 7.6 m (25 ft) or less, the full shoulder width should be carried. Vertical clearances should be at least 4.25 m (14 ft) over the entire roadway width, and a 0.10- to 0.15-m (4- to 6-in) allowance should be made for resurfacing. The recommended minimum design loading for bridges should be MS-18 (HS-20), particularly for spans of more than 7.6 m (25 ft). Use of materials such as treated timbers and controlled-oxidizing steel will result in functional bridges that blend with the surrounding landscape.

Ideally, culverts under the roadway should have a

Figure 13. Poor construction practices: undercut slopes, no revegetation, and piles of debris along shoulders.

Figure 14. Abandoned borrow pit where no restoration effort was made.

minimum diameter of 0.38-0.45 m (15-18 in). This can be reduced to 0.30 m (12 in) under driveways, if necessary. Culvert ends should not protrude unnecessarily beyond the grade of the slope, and wherever possible they should be concealed by stones to give a natural appearance.

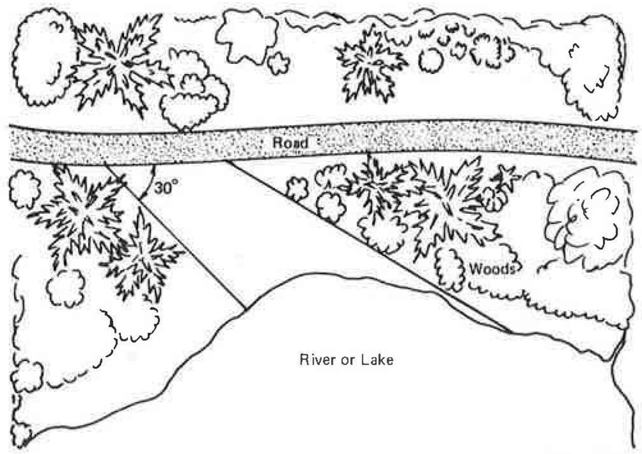
Bridges and culverts should be located along natural drainage channels to be most efficient and to minimize erosion problems. The location of bridges should allow for a smooth approach of horizontal and vertical alignments.

GENERAL CONSTRUCTION

All areas both within and outside the right-of-way that are disturbed or serve as sources of materials (see Figure 13) should be restored to a pleasing and acceptable condition. This applies to borrow pits (see Figure 14), spoil or waste areas, tops of cut slopes, drainage ditches, haul roads, storage areas, and all similar locations. All debris and waste material should be removed from the right-of-way. The objective is to reduce construction scars and to retain and protect the visual quality of the travel corridor.

Construction projects in any road corridor in the

Figure 15. Desirable geometrics for moving vistas.



Adirondacks may encounter highly erodible soils that could affect nearby waterways and adjacent properties. Soil areas that have a high potential for erosion and possible sediment production include (a) earth cut slopes and fill slopes without vegetative cover; (b) earth cuts with slopes steeper than the natural angle of repose of the in-place soils; (c) cut-to-fill transitions; (d) ditches that have steep or long continuous grades and no vegetative, stone, or other protection; (e) inadequate systems for controlling surface water (i.e., shallow ditches and infrequent or undersized culverts); and (f) saturated soil conditions in and around the road (silts, clays, and fine sands). Temporary or permanent erosion controls should be used in these areas (9).

Proper highway design, including rounding the tops and bottoms of earth slopes, encourages vegetation and minimizes erosion. Earth cut slopes and embankment slopes should be seeded and mulched as soon as it is practical to do so during construction to reduce damage by erosion; to minimize sedimentation in nearby streams, lakes, and wetlands; and to minimize damage to adjoining property.

Excessive removal of roadside vegetation should be avoided, but selective thinning should be considered to provide views of bodies of water, streams, wetlands, unique rock formations or landforms (such as mountains), and man-made features. A 30° angle from the direction of travel is the desirable angle for the moving vista (see Figure 15). Trees that are removed should be cut as close to the ground as possible to avoid unsightly stumps along the roadside. In most locations, brush, logs, slash, or other inflammable materials should not be left within 6.1 m (20 ft) of the public right-of-way.

An undulating clearing line for trees (see Figure 16) has a more pleasing appearance than a straight-edged channel. When safety permits, consideration should also be given to preserving important vegetation (such as specimen trees) and landscape features within the limits of construction. However, at intersections and horizontal curves, trees, shrubs, and brush that could obstruct sight lines should be controlled or eliminated if necessary (10).

Wherever possible, utility lines should be one set of poles set on one side of the road. Efforts should be made to locate them so that they will have minimal visual effect. Figure 17 shows the type of placement that should be avoided.

Figure 16. Undulating clearing line along the roadside.

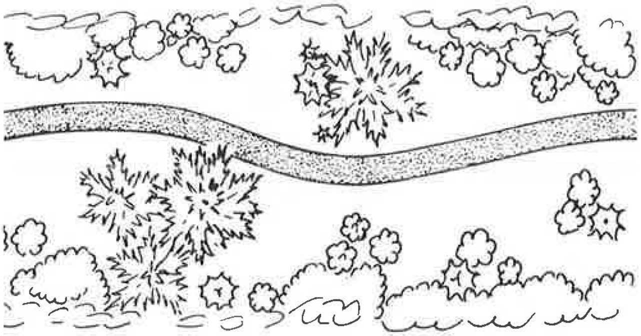
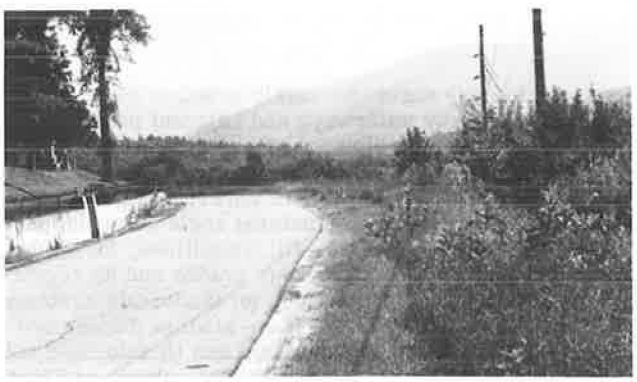


Figure 17. Utility lines and poles obstructing the view along a local road in the high-peaks region.



SUMMARY

The guidelines discussed in this paper were developed to ensure that roads in the Adirondack Park are constructed or reconstructed so as to protect and enhance the parklands. AASHTO standards for local roads, in relation to sight distances, maximum grades, pavement cross slopes, and bridge design loading and widths, were considered necessary for safety and are incorporated into these guidelines. However, AASHTO pavement and shoulder widths and clear roadside area were not considered parklike or cost-effective for these low-volume roads, and lesser widths were adopted. In addition, it is recommended that special precautions be taken to minimize erosion problems and reduce construction scars and at the same time provide safe and efficient roads.

ACKNOWLEDGMENT

I wish to thank the members of the Adirondack Highway Council and the highway superintendents in the 12 Adirondack Park counties for their guidance and constructive comments during the preparation of these guidelines. In addition, the assistance of the Soil Mechanics Bureau and Photolog Unit of NYSDOT is gratefully acknowledged. The guidelines were edited for publication by A. D. Emerich, and design and layout were done by Charlotte J. Ronish and Donna L. Noonan, all of the NYSDOT Engineering Research and Development Bureau.

I especially acknowledge the assistance of Robert E. Longfield, Jr., of Northeast Environmental Design, Woodstock, Vermont, author of The Vermont Backroad (7), which served as a model for these guidelines.

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Abridgment

Decision Sight Distance for Highway Design and Traffic Control Requirements

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A primary feature in the design of a highway is the arrangement of the geometric elements so that there is adequate sight distance for safe and efficient vehicle operation. With this principle in mind, the American Association of State Highway and Transportation Officials (AASHTO) has established guidelines for three important types of sight distance: (a) stopping, (b) passing, and (c) intersection. These established distances, however, are often inadequate for situations in which drivers must make very complex decisions, the development of a potentially hazardous situation is difficult to perceive, or severe braking is inappropriate. For locations where longer sight distances are needed, a review of human factors and aspects of traffic operations shows that sight-distance criteria should be based on the driver's ability to properly react to impending danger. This concept has been referred to as decision sight distance.

Decision sight distance (DSD) has been defined as the distance at which drivers can detect a hazard or a signal in a cluttered roadway environment, recognize it or its potential threat, select an appropriate speed and path, and perform the required action safely and efficiently (1). Research was performed to relate this concept to specific road types, design speeds, traffic operating conditions, geometric features, and driver attributes. The work was done in two phases:

1. In phase 1, a model of the hazard-avoidance process was formulated to be used as a basis for quantifying DSD, and preliminary DSD values were developed based on the average times for the elements of the model derived from literature sources.
2. In phase 2, 19 subjects drove an instrumented vehicle through eight typical highway situations to validate the preliminary DSD values.

HAZARD-AVOIDANCE MODEL

An analytic assessment of the definition of DSD and its components led to the formulation of a model for quantifying appropriate distances. The model outlines a sequential chain of events that occurs in hazard avoidance, starting from detection of the hazard and ending with completion of the avoidance maneuver. This process is adopted and modified from one originally developed by Baker and Stebbins (2), which was in turn later modified by Leisch (3) and Pfefer (4). The steps in the process are briefly described as follows:

1. Sighting (time t_0)—This is the baseline time point at which the hazard is within the driver's sight line.
2. Detection (time t_1)—The driver's eye fixates on the hazard and "sees" it.
3. Recognition (time t_2)—The image on the eye is translated by the brain, and the hazard is recognized or perceived as such.
4. Decision (time t_3)—The driver analyzes alternative courses of action and selects one.
5. Response (time t_4)—The driver initiates the required action.
6. Completion of maneuver (time t_5)—The driver accomplishes a change in the path and/or the speed of the vehicle.

The process as described above is a simple additive model in which the total time from the moment when the hazard is visible to the completion of the hazard-avoidance maneuver equals the sum of the incremental times required for detection (t_0 to t_1), recognition (t_1 to t_2), decision (t_2 to t_3), response (t_3 to t_4), and completion of maneuver (t_4 to t_5).

PRELIMINARY DSD VALUES

Data for quantifying the various components of the model were taken from the existing literature. For a complete discussion of the findings from previous research on the various components of the hazard-avoidance process, the reader is referred to McGee and others (5).

From the literature on DSD parameters, it was clear that there are gaps that make it difficult to quantify distance values for various conditions. Many variables can affect each of the components in the detection-through-maneuver process. These variables can be grouped in the following categories: driver capabilities, design features, and traffic operation factors. Unfortunately, the state of the art is not sufficiently advanced to quantitatively describe how these and other factors may affect each component of the model.

At best, a range of values could be developed by using the literature findings as a basis. Such an approach has been followed in preparing the following table, which gives the preliminary time values for the various elements of the hazard-avoidance model:

Phase	Time (s) Low	Time (s) High
Before maneuver		
Detection and recognition	1.5	2.0
Decision and response	4.2	7.1
Maneuver (lane change)	3.5	4.5
Total	9.2	13.6

As this table indicates, the ranges of values were grouped into two phases:

1. The before-maneuver phase, which consists of (a) detection and recognition and (b) decision and response initiation, and
2. The maneuver phase, in which lane changing was used as the maneuver.

VALIDATION OF DSD VALUES

Since the existing literature was only marginally ade-

Table 1. Recommended DSD values.

Design Speed (km/h)	Time (s) Before Maneuver: Detection and Recognition	Decision and Initiation of Response	Maneuver (lane change)	Total	Decision Sight Distance (m): Computed	Rounded for Design[a]
40	1.5-3.0	4.2-6.5	4.5	10.2-14	113-156	120-160
60	1.5-3.0	4.2-6.5	4.5	10.2-14	170-233	170-230
80	1.5-3.0	4.2-6.5	4.5	10.2-14	227-311	230-310
100	2.0-3.0	4.7-7.0	4.3	11.2-14.5	306-397	310-400
120	2.0-3.0	4.7-7.0	4.0	10.7-14	357-467	360-470
140	2.0-3.0	4.7-7.0	4.0	10.7-14	416-544	420-540

Note: 1 km = 0.62 mile; 1 m = 3.28 ft.
[a] Rounded up to the nearest 10 m for the low value and up or down to the nearest 10 m for the upper value.

quate for quantifying certain portions of the hazard-avoidance process and ultimately for estimating DSD values, field work was performed to operationally validate the preliminary DSD values.

The methodology for conducting this field validation was designed to develop time estimates for the following combinations:

1. Detection and recognition—time elements t_0 to t_2,
2. Decision and initiation of response—time elements t_2 to t_4, and
3. Avoidance maneuver—time elements t_4 to t_5.

Nineteen test subjects drove over a course and responded to certain geometrics (primarily lane drops) that necessitated a change in path and possibly in speed in order to reach the destination objective. The responses sought included the initial sighting of the geometric feature (detection plus recognition), the moment of initiation of a change in path and/or speed (decision plus initiation of response), and finally the time used to complete the maneuver (avoidance maneuver).

Validation of the preliminary DSD criteria developed from the literature would be attained if one or both of the following results were found:

1. The times recommended for the various components of the hazard-avoidance process were replicated by several subject drivers, and
2. At sites where the existing sight distance was shorter than the recommended decision sight distance, drivers could not negotiate the situation safely and efficiently, and, conversely, at sites where the sight distance was equal to or greater than the recommended decision sight distance, drivers had no problem negotiating the required change in path and speed.

The first validation criterion was only partially met. For the detection-plus-recognition phase, times greater than the maximum value of 2.0 s were observed in many cases. However, the high values were not considered to be indicative of the actual time required for this phase of the hazard-avoidance process. In view of the results, a range of 1.5-3.0 s seemed more appropriate. The lower value suggested in the table above appears to be the minimum required, whereas 3.0 s would be required in more complex situations.

The results of the field data for the decision-plus-response time were reasonably compatible with the analytically developed criteria. Although higher times were observed, it is believed that, depending on vehicle speed, the upper range of 6.6-7.1 s is a good design criterion for the more complex situations and that 4.2-4.7 s is adequate for the less demanding situations.

The most nearly replicated time value was the maneuver time. The preliminary DSD criterion allows times of 4.5 s for 48.3 km/h (30 miles/h) to 3.5 s for 112.7-128.7 km/h (70-80 miles/h). Based on the results of the field experiment as well as a reanalysis of data from a previous study, it appears that a value of 4.5 s is appropriate for speeds at least as high as 96.6 km/h (60 miles/h). Design values for higher speeds should probably be 4.0 rather than 3.5 s.

The second of the two validation criteria given above was met. At four sites where the maximum sight distance was greater than the DSD, the subjects successfully negotiated the course; that is, they were able to recognize the potential hazard situation and responded to it safely and efficiently. At the other four sites that had inadequate sight distance, several subjects could not negotiate the sites properly.

RESULTS AND RECOMMENDED VALUES

In view of the findings of the literature synthesis and the results of field validation experiments, it is concluded that the concept of decision sight distance is operationally valid. Drivers do need a sight distance of the roadway that gives them ample time to detect and recognize a potential hazard, decide on the proper course of action, and complete the required maneuver in a safe and efficient manner. This sight distance depends on the driver's ability to process information and to maneuver the vehicle, and these factors in turn are related to the level of decision complexity, the visual clutter, and the surrounding traffic.

From analytic and limited empirical research, it is possible to recommend a range of DSD values. These values, which are given in Table 1, have been divided into before-maneuver and maneuver phases. The before-maneuver phase is the time required for a driver to process information relative to a hazard. It consists of the time needed to (a) detect and recognize the hazard and (b) decide on the proper maneuver and initiate the action. The second phase is the maneuver time. Since a lane change is likely and more time is consumed in changing lanes than in reducing speed, a lane-change maneuver is assumed in Table 1.

The last column in the table gives the recommended DSDs. In the range of values provided, the lower value is the minimum acceptable for situations of moderate complexity or visual clutter and the upper value is desirable for highly complex or visually cluttered situations. Unfortunately, because of the limitations of this study, it is not possible to provide specific criteria for the level of decision complexity or clutter.

For design purposes, DSDs should be from the driver's eye height to an object of zero height, since the driver must be able to see the entire roadway. A higher limit can be used if some other physical feature provides the hazard information to the driver.

RECOMMENDED APPLICATIONS

Two applications of DSD are recommended. First, it should be used in highway design, either for new facilities or reconstruction (improvement) of "below-standard" facilities. The locations where DSD should be applied are generally characterized by conditions that create the potential need for drivers to depart from simple steering and speed-control maneuvers to follow the road. DSD is also recommended for use at special-feature locations where drivers could experience problems in handling information. These locations generally include interchanges—especially freeway-to-freeway—intersections, toll plazas, pavement-width reductions (lane drops), and any other location where unusual or unexpected maneuvers are required.

For all design situations, the higher values are suggested for especially complex areas such as interchanges that have left-hand exits or multiple exits in close proximity. The lower values should be considered minimally acceptable.

The second suggested application is for traffic control techniques at hazardous locations. More specifically, the criteria can be used to determine the need for and location of advance warning signs. In using Table 1 to determine the appropriate DSD, the 85th percentile speed rather than design speed should be used. In addition, although the higher values are recommended for defining the DSD, ranges of values are given to provide the flexibility that is often required in the positioning of advance warning signs.

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**H. W. McGee was with Biotechnology, Inc., when this research was performed.*

Use of Overland Flow in Storm-Water Management on Interstate Highways

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An assessment of the potential of shallow-water ditches and shoulder areas adjacent to roadways for deposition of heavy metals is reported. The metals examined in the field were those that result from automobile emissions and the wear of automotive parts: lead, zinc, copper, chromium, and nickel. Cadmium content was also measured. The highest concentrations of metals were found in roadside plant and animal populations. These, however, contained the least metals in mass. Soils adjacent to the edge of the pavement contained the greatest mass of metals. In general, the topsoil contained higher concentrations of metals than subsurface soils. Lead was shown to be relatively immobilized by the soil, whereas other metals were more mobile. Design equations for estimating the volume of shallow-water storage areas for rainfall excess (runoff) are presented. In general, the use of overland flow with shallow ditch areas was shown to be effective for the control of runoff and its associated pollution content.

Storm-water runoff in the United States is receiving considerable attention, primarily as a result of federal regulations such as the Federal Water Pollution Control Act and the Clean Water Act. Runoff pollutants from highway surfaces had been documented as early as 1957 (1), when high concentrations of lead in soils adjacent to highways were reported. More recent studies (2, 3) have identified zinc, copper, chromium, cadmium, nickel, and other metals in highway runoff waters. On a mass basis, Shaheen (2) compared highway runoff with sanitary sewage for comparable land uses and populations and determined that the masses of lead, zinc, and chromium in highway runoff were, respectively, 1000, 20, and 300 times greater than amounts of those metals encountered in sanitary sewage. Such a comparison should be viewed with caution, however, because the chemistry of highway drainage and its mode of discharge are very different from those of sewage effluent.

The automobile is the predominant source of lead, as well as some other heavy metals, near highways. The combustion of leaded gasoline is generally acknowledged to be the major source of lead, but some lead also results from the wear of tires, in which lead oxide is used as a filler material (2). Zinc also results from tire wear and from the leakage of crankcase oil, in which high concentrations of zinc are used as a stabilizer (1). Chromium, copper, and nickel are produced by the wear of metal plating, bearings, bush-

ings, and other moving parts in the engine (2).

From the viewpoint of environmental health, lead is not required by any form of life and at high levels is known to be toxic to plants, animals, and humans (4). Although zinc, copper, and chromium are essential to most life forms in trace amounts, high levels of these metals have also been shown to be toxic.

The ultimate fate of metals in highway runoff depends to a great degree on the areas immediately adjacent to the paved surface. Environments adjacent to paved roadways are not similar. The shoulder and ditch areas may be impervious or pervious to various degrees. The physical and chemical properties of the soil and the extent of vegetative cover vary widely. In addition, overland flow of water and channelization to a discharge point are options in design. If overland flow is the drainage design, heavy metals in the runoff are exposed to soil, vegetation, and, to some extent, animal life in the shoulder and ditch area. Some metals are removed through this exposure. The extent of removal depends primarily on the characteristics of the right-of-way area. If the drainage design is channelized flow (i.e., concrete drainage channels or deep ditches), there is generally no opportunity for the removal of heavy metals by soil or vegetation. In these situations, "first-flush" volumes of runoff can be diverted to and stored in available ditch areas or specially constructed ponds to reduce the discharge of metals and other pollutants into receiving water bodies.

RESEARCH OBJECTIVES

The major objective of this research was the management of storm-water runoff to reduce concentrations of metals. Based on the results of previous studies, which indicate that soil is a significant "sink" for heavy metals (5, 6), it was postulated that overland flow of storm water from impervious surfaces to a ditch before discharge to lands or surface water bodies adjacent to highway rights-of-way would be effective in reducing concentrations of metals. The overland flow of runoff would promote exposure of metals to the soil and thus make maximum use of the ability of the soil to retain these metals.

To investigate the effectiveness of overland flow in this regard, field sampling and statistical analysis were conducted for shoulder and ditch areas adjacent to several representative highways in central Florida. The specific objectives of these investigations were to determine the following:

1. Metal concentrations in soils, plants, animals, surface water, and groundwater;
2. The relative mobility of metals transported by overland flow by distance from the edge of the pavement and depth into the soil;
3. The total metal-carrying capacity of soils; and
4. The volume of shallow-water retention areas required to remove impurities if channelized flow (not overland flow) is the method of handling storm water.

The last activity in this list recognized the fact that some bridge and roadway drainage cannot be designed to use overland flow.

BASIC DATA RESULTS

Concentrations and masses of metals, as well as estimates of the retention capacity of sinks adjacent to the paved highways, were determined. Eleven sampling sites in east-central Florida were used. The sites were chosen to provide a range of geographic locations, highway ages, number of traffic lanes, drainage and soil conditions, and traffic volumes. The methods and procedures used are described in detail elsewhere (7, 8).

The analysis of metal concentrations in roadside plants, animals, surface water (standing water in roadside ditches or ponds), groundwater, and soil can be used to determine the relative mass of metals in each. This makes it possible to identify the major sinks for these metals in roadside environments. For example, summary statistics for lead concentrations are given in Table 1. The highest concentration of lead was found in animals and the lowest concentration in surface water and groundwater. Comparative data for water from various Florida sources are given below (1 mg/kg = 1 ppm):

	Lead Concentration (mg/kg)	
Water	Average	Range
Groundwater from dry sampling site, I-95, Titusville	0.1	–
Upper Floridan aquifer, Orlando	<0.5	–
Lower Floridan aquifer, Orlando	<0.005	–
Apalachicola River	–	0.0021-0.0062

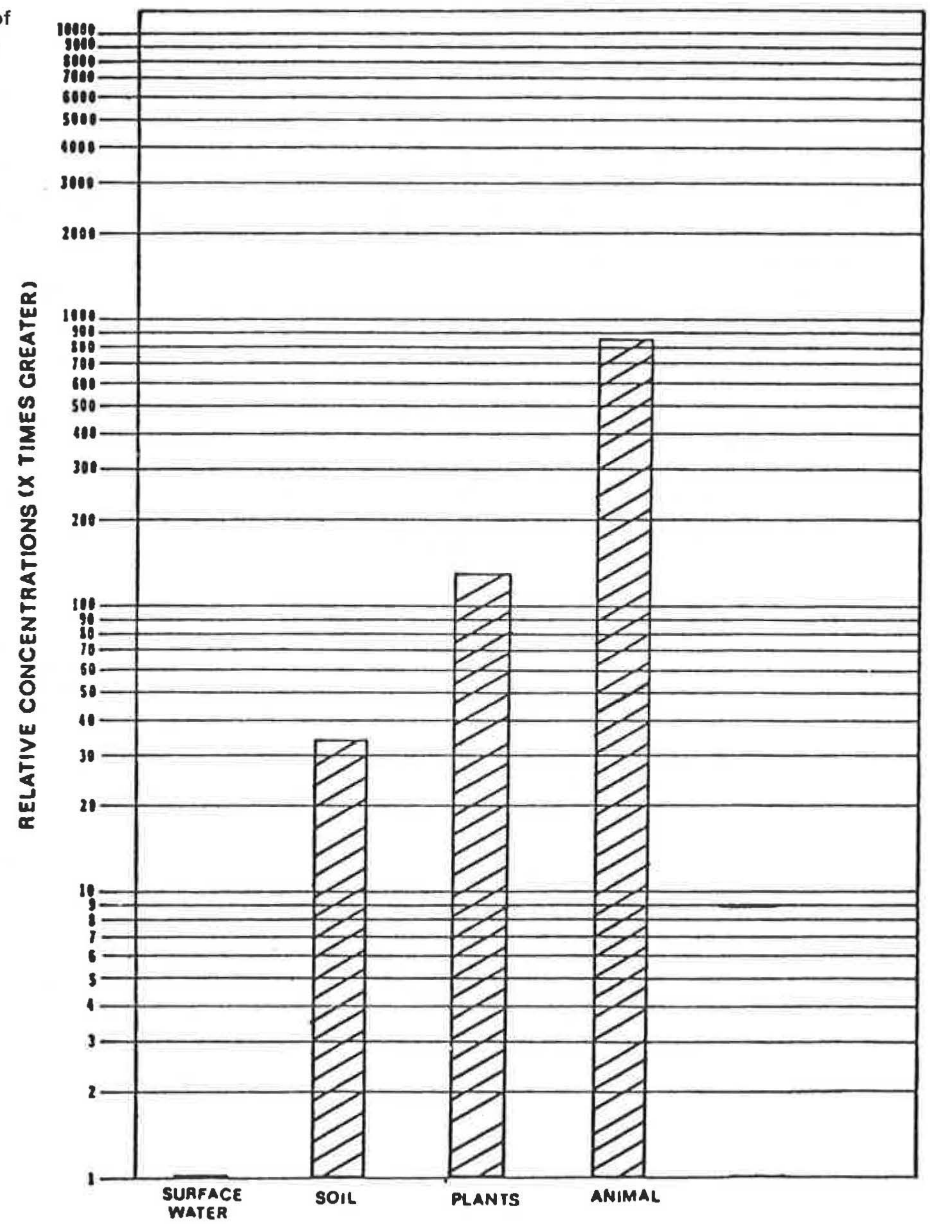
The maximum contaminant level (MCL) specified in the Florida Safe Drinking Water Act is an average 0.05 mg/kg. Clearly, total lead concentrations in some surface water samples exceeded the standards. The dissolved fraction of lead in surface waters, as well as the

Table 1. Summary statistics on lead concentrations.

		Concentration (mg/kg)			
Item	No. of Samples	Range	Average	Standard Deviation	Coefficient of Variation
Surface water					
Total	28	0.0012-0.27	0.218	0.265	122
Dissolved	27	0.009-0.04	0.026	0.041	158
Sediment	3	0.001-5.8	1.949	3.335	171
Soil					
All samples	85	0.16-53.0	7.787	10.600	136
Topsoil	47	0.25-53.0	10.591	11.823	112
13 cm deep	38	0.16-25.0	4.320	9.600	222
Plants					
All samples[a]	17	3.15-65.0	29.57	17.38	59
Dry sites	11	26.4-65.0	27.34	15.46	57
Wet sites	6	3.15-53.4	33.65	16.03	48
Animals					
All samples	4	27.6-429	191.1	176.5	92
Dry sites (grubs)	2	220.5-429	324.8	147.4	45
Wet sites (minnows)	2	27.6-29.2	28.4	1.1	4

Note: 1 mg/kg = 1 ppm.
Lead removable by dilute acid extraction (0.75 normal).
[a]Dry weight.

Figure 1. Relative concentrations of lead in four types of roadside sinks.



measured lead content of a groundwater sample adjacent to I-95, was within standards. The coefficients of variation illustrate that more consistent estimates of the average were possible for animal and plant samples. However, since soil at various depths and distance from the edge of the pavement were included in the average statistics, variability should be expected in the results of the soil analysis.

Relative concentrations of lead from Table 1 are shown in Figure 1. Comparative charts of concentrations of lead, zinc, copper, chromium, cadmium, and nickel all show that the highest concentrations are found in animal life. However, very little animal life was found in the roadside environments investigated. Therefore, if concentrations are converted to mass loading of metals per meter of highway, the greatest quantity of metals is found in the soils. Figure 2 shows the relative mass of lead in each of the potential sinks per meter of highway.

The assumptions used to convert average concentration to mass per meter of roadway were average conditions at the sampling sites along I-75, I-95, FL-50, FL-405, and US-1. The shoulder width, not including the ditch area, was calculated as approximately 12.25 m (40 ft). The weight of soil in an area that measures 12x1 m (40x3.3 ft) and is 15 cm (6 in) deep is approximately 2.2 Mg (2.4 tons). Samples of grass were weighed and, for a 12-m^2 (130-ft^2) area, the weight was estimated at 5.91 kg (13 lb). "Grubs" (believed to be june bug larvae) or resident animals were difficult to find. A total of twelve 1-m^2 (11-ft^2), 15-cm-deep areas were examined. Approximately 4 grubs/m^2 (0.4 grub/ft^2) were found. The average weight per animal was about 0.35 g. The volume of water per meter of highway was estimated at 560 L (148 gal), which assumes a 0.3-m (1-ft) average depth 2 m (6.5 ft) wide. Thus, by knowing concentration and mass of media, the mass of metals can be calculated. Example calculations for copper are as follows (1 mg/L = 0.125 mg/gal; 1 mg/kg = 1 ppm; 1 kg = 2.2 lb):

Type of Sink	Calculation per Meter of Roadway		Mass (mg)	Ratio
Water	0.033 mg/L x 560 L	=	18.48	23

Type of Sink	Calculation per Meter of Roadway		Mass (mg)	Ratio
Soil	0.688 mg/kg x 2200 kg	=	1514	1892
Plants	36.3 mg/kg x 5.91 kg	=	214	268
Animals	43.4 mg/kg x 0.018 kg	=	0.80	1

The weight given for animals is the average weight of grubs and minnows per meter.

These results indicate that the soil is the major sink for heavy metals in roadside areas. This is consistent with the findings of several other studies of the fate of highway-related heavy metals. Studies by Motto and others (9), Lagerwerff and Specht (10), Singer and Hanson (11), and Olson and Skogerboe (12), to name but a few, have shown elevated levels of lead and zinc in roadside soils. Concentrations of lead as high as 7000 mg/kg have been reported (12). A study of the mass inputs and outputs of highway-related lead in urban and rural basins was performed by Rolfe and Jennett (6), who calculated that 75 percent of the total lead input by automobiles in the urban basin was leaving that basin by way of streamflow but that only 2 percent was leaving the rural basin as streamflow. They concluded that the soil was accumulating a large portion of the lead input to the rural basin and, based on measurements, estimated that the equivalent of 30 years of automobile lead emissions were contained in the soil.

Evidence developed in our study shows that, once heavy metals are retained by the soil, they are effectively immobilized and generally do not leach downward. This was found to be true particularly for lead. Other metals appeared to be somewhat more mobile in the soil so that some leaching could occur. At every sampling site, metal concentrations decreased with both depth into the soil and distance from the edge of the pavement. Table 2 gives an example of lead concentrations in surface and subsurface [10-15 cm (6-8 in)] soils at edge-of-pavement and ditch locations. These lead concentrations represent the total lead extractable by concentrated acid solution (previous results are for dilute acid extractions). Table 2 also gives comparative results from four other studies that appear to be consistent with the results of the study discussed in this paper.

Figure 2. Relative masses of lead in four types of sinks per meter of highway.

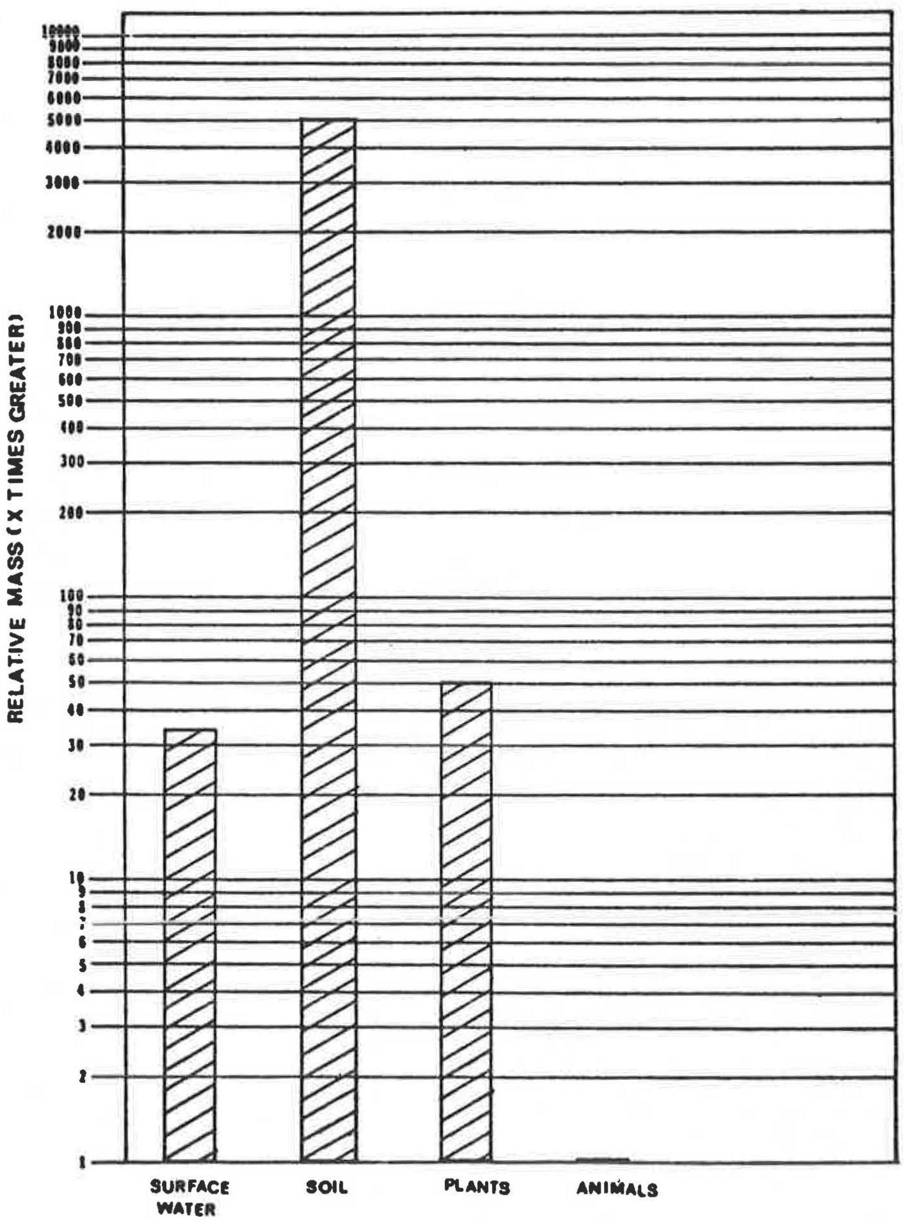


Table 2. Total lead content in roadside soil and grass as a function of distance from traffic and depth in soil profile.

Location	Distance from Road (m)	Dry Weight of Lead (mg/kg) Grass	Soil Profile Layer 0-5 cm	5-10 cm	10-15 cm
West of US-1, near Plant	8	68.2	522	460	416
Industry Station, Beltsville, MD	16	47.5	378	260	104
	32	26.3	164	108	69
West of southbound lanes,	8	51.3	540	300	98
Washington-Baltimore	16	30.0	202	105	60
Parkway, Bladensburg, MD	32	18.5	140	60	38
West of I-29, Platte City,	8	21.3	242	112	95
MO	16	12.5	140	104	66
	32	7.5	61	55	60
North of Seymour Road,	8	31.3	150	29	11
Cincinnati, OH	16	26.0	101	14	8.2
	32	7.6	55	10	6.1
All sites, edge	0	50.4	822	-	60
Central Florida, ditch[a]	12	27.5	365	-	180

[a] Buildup of humus materials on bottom of ditch.

Table 3. Probable soil components or metal forms in various ranges of soil density.

Density Range (g/cm^3)	Probable Soil Components and Metal Forms
<1.5	Organic matter and organically bound metals
1.5-2.0	Organic matter and possibly clay with adsorbed organic matter; metals organically bound or adsorbed on clay minerals directly
2.0-2.5	Some organic matter, light minerals, and light clays; metals organically bound or adsorbed on clay minerals; Cr may be present as a precipitate
2.5-2.9	Bulk of the inorganic soil components, including sand, clay, silt, and other minerals; very few organics likely to be present; metals likely to be adsorbed by clay minerals or in precipitated form (Cr and Zn)
2.9-3.3	Dense minerals and possibly clays with adsorbed heavy metals; probably no organics; metals in adsorbed or precipitated form (Cr and Zn precipitates have densities in this range but not Pb)
>3.3	Dense minerals, possibly some clay with adsorbed heavy metals; metals probably in precipitated form (Pb, Zn, Cr)

ESTIMATES OF METAL-RETAINING CAPACITY OF SOIL

Now that soil has been identified as a major sink for highway-related heavy metals and it has been shown that heavy metals, once retained by the soil, are effectively immobilized, it would appear that using overland flow of runoff, to promote exposure to the soil of dissolved or suspended heavy metals, is probably an effective means of removing these materials. First, however, the capacity of the soil to retain heavy metals must be determined. To estimate this capacity, two basic approaches could be used:

1. The stoichometric approach, in which the chemical principles of the reactions that take place between soil and heavy metals are used to calculate a theoretical maximum capacity, and
2. The empirical approach, in which statistical correlations between soil properties and capacity for metal retention are used.

In this work, we first used the stoichometric approach. Findings made during the course of the study indicated, however, that the empirical approach might be more practical. Nevertheless, an explanation of the stoichometric approach is provided here to point up the difficulties involved in making estimates of soil retention capacity and because it leads to some key findings on interactions between soils and heavy metals.

Stoichometric Approach

To understand the stoichometric approach, it is first necessary to understand that there are a number of simultaneous chemical and physical reactions by which soil is able to retain heavy metals. Examples of these include adsorption, ion exchange, chemical precipitation, and organo-metallic-complex formation. Each reaction has its own kinetics of metal retention and its own saturation capacity and is affected by a unique set of environmental parameters, such as temperature, pH, and soil moisture.

The bulk metal-retaining capacity of the soil is the sum of the retention capacity attributable to each of the individual reactions. If sufficient information were available, this could be calculated theoretically by using basic chemistry considerations. Because of the number of reactions and their complexity, it would be impossible to make such calculations if all were of equal, or nearly equal, importance to overall metal-retaining capacity. But, if a single reaction could be isolated as most important, efforts could be focused on the chemistry of this reaction. For example, if it could be determined that the majority of lead entering the soil formed a specific compound—say, lead sulfate—then the lead-retaining capacity of the soil could be estimated based on knowledge of the reaction between lead in the runoff and sulfate in the soil.

This rationale was used in attempts to identify the most important reactions between soil and the heavy metals in runoff. It was determined that physically separating field soil samples into components according to density was the most useful way to accomplish this. Soil samples were separated into density ranges. Then, based on observations and tests of the separated fractions and knowledge of the density of the various compounds or complexes of soil and heavy metals, the probable chemical forms present in each density range were assessed. These data are given in Table 3. Identification of compounds by this method is by no means absolute, but it can be used as an indicator.

Soil fractions from each density range of a sample were analyzed for lead, zinc, and chromium content. If the greatest quantities of a metal were consistently found in a single density range, it could be postulated that that metal was generally forming one of the compounds given for that range in Table 3. An example of such an analysis is given below for an edge-of-pavement

surface sample (1 g/cm^3 = 0.036 lb/in^3; 1 mg/kg = 1 ppm):

Density Range (g/cm^3)	Weight Fraction of Soil (%)	Lead Amount (mg/kg)	Lead Percentage of Total
<1.5	0.23	1 610	1.8
1.5-2.0	1.9	1 750	4.4
2.0-2.5	10.5	531	2.9
2.5-2.9	86.0	57	14.8
2.9-3.3	0.53	7 560	7.9
>3.3	0.84	71 300	68.2

As indicated, about 68 percent of the lead in that sample was in the most dense fraction. According to Table 3, this would mean that the lead was probably in the form of an inorganic lead compound rather than associated with soil particles or organic material.

Analysis of other edge-of-pavement surface samples produced similar results for lead. It is also interesting to note that Olson and Skogerboe (12) performed similar tests on highway soils and obtained comparable results. They carried the analysis one step further and found that the lead in the most dense fraction was in the form of lead sulfate.

In our study, however, analysis of samples from other locations in the right-of-way (i.e., subsurface and ditch samples) obtained very different results. In these samples, the metals were distributed generally throughout the density ranges. In some samples, the least dense fraction [<1.5 g/cm^3 (<0.054 lb/in^3)], which was determined to be primarily organic matter, contained substantial amounts of the metals. Therefore, in looking at the right-of-way as a whole, no single reaction can be identified as most important for the retention of heavy metals by the soil. It is for this reason that it is not practical to use the stoichometric approach.

Some important conclusions can, however, be drawn from these analyses:

1. It is clear that reactions between soils and heavy metals are site specific. This means that, in areas where removal of metals by the soil is critical, site-specific studies should be performed.
2. The organic portion of the soil is, in many cases, very important to its ability to retain heavy metals.

Empirical Approach

The second approach to estimating metal-retaining capacity, the empirical approach, has been illustrated by Zimdahl and Skogerboe (13). They found from laboratory tests that the capacity of a particular soil to adsorb lead can be reasonably predicted based on a correlation equation that involves cation exchange capacity (CEC)—the ability of the soil to adsorb cations, such as heavy metals—and pH. This equation, determined from an analysis of the lead-fixation capacity of 18 soils, is

$$N = 2.81 \times 10^{-6}\ CEC + 1.07 \times 10^{-5}\ pH - 4.93 \times 10^{-5} \qquad (1)$$

where

N = moles of lead per gram of soil at saturation,
CEC = cation exchange capacity of the soil (meq/100 g), and
pH = soil pH in units.

A regression coefficient of 0.971 was obtained, and the calculated values of N generally agreed within 10-20 percent with experimentally determined values.

In light of the finding that interactions between soil and heavy metals depend greatly on site-specific factors, it is felt that estimates of the metal-retaining capacity of soil that are based strictly on laboratory studies should be used with extreme caution. If the reactions that occur are site specific, so will be the metal-retaining capacity. The best means of estimating this capacity would probably be a combination of laboratory and field tests. Batch tests and column (breakthrough) tests should be conducted on soil samples taken from representative areas adjacent to the highway under consideration.

However, as a "first-cut" estimate of the lead-retaining capacity of the soils studied, the regression equation developed by Zimdahl and Skogerboe (13) was used. Values for CEC and pH for each sample were entered, and capacity N was calculated to range from 3.97×10^{-5} to 7.74×10^{-5} moles Pb/gram. The corresponding concentrations range from 8220 to 16 030 mg/kg. This means that, if the soils analyzed in this work behaved like those tested by Zimdahl and Skogerboe (13), they would have an additional capacity to fix lead that would range from 10 to 500 times their existing lead content.

STORAGE VOLUME FOR RUNOFF

Hydrologic Considerations

To limit the quantity of pollutants discharged from a highway right-of-way, the infiltration characteristics of the shoulders can be used to retain some metals. The more mobile metals and other potential pollutants can be retained in shallow-water ditches, where most metals will deposit in the sediment. When shoulders or earthen areas are not available for overland flow, such as rainfall excess (runoff) from bridge or limited right-of-way areas, then retention ponds with underdrains or natural percolation would be valuable for treating the first flush of storm water. The major question is the dimensions of these ditches or ponds that should be allocated for retention of storm waters.

Other methods of storm-water treatment are available and should be examined before the use of shallow-water ditches or construction of retention ponds.

A general equation to express the allocation of waters within a highway right-of-way would be

$$R_T = R_{RW} + R_O \qquad (2)$$

where

R_T = volume of total runoff that requires treatment,
R_{RW} = volume of runoff from the right-of-way, and
R_O = volume of runoff from outside the right-of-way.

At the beginning of a storm, precipitation infiltrates into the ground and is stored in surface depressions or otherwise abstracted. Rainfall intensity and distribution vary during a storm, producing variable quantities of runoff. Eventually, a saturation level is reached and runoff water is equivalent to precipitation. Thus, the highway shoulders and ditches receive runoff at a variable rate, but the volume of runoff for a storm can be predicted. Depending on the water table and soil-water conditions, runoff waters will percolate or remain on the surface as excess (runoff). The factors that affect the amount of runoff from a given area are the intensity and duration of rainfall, the characteristics of the soil drainage, the amount of vegetative

cover, the amount of impervious surface (i.e., pavement), and topographic characteristics (e.g., slopes and depressions).

Example 1

Many mathematical formulas have been developed to model rainfall-runoff relations. One such formula has been developed by the Soil Conservation Service (SCS) of the U.S. Department of Agriculture (14) for use on urban watersheds. This formula is also useful for predicting roadway runoff and is applied here as an illustrative example.

Consider a section of roadway 28.7 m (94 ft) wide with 10.4 m (34 ft) of paved, otherwise impervious, area. The length of roadway drainage is 739 m (2424 ft) (like an I-95 area). The entire area drains as overland flow into an outfall at the lowest elevation.

The runoff from this highway section during a storm event can be calculated as follows by using the SCS procedure (14) (since the SCS formula is based on U.S. customary units of measurement, no SI equivalents are given in these calculations):

1. From an evaluation of soil type, soil moisture, ground cover, and percentage of impervious area, a weighted curve number is established. The curve number is used to estimate the potential for infiltration and the total infiltration or saturation capacity of a soil. For the roadway under consideration, the soil is assumed to be type B (moderately well drained) under average moisture conditions (condition 2). The ground cover is assumed to be fair grass over 50-75 percent of the pervious area. The percentage of impervious area is 36 percent. The weighted curve number (CN) is determined as follows:

Type of Land	Percentage of Total Area	CN	Weighted CN
Pavement	36	90	32
Pervious (grassed slope and ditch)	64	69	44
Total			76

2. Total storage S′ (initial abstraction, infiltration, and evapotranspiration) is then estimated by using the formula

$$S' = (1000/CN) - 10 = (1000/76) - 10 = 3.16 \text{ in} \tag{3}$$

3. By using this storage term, runoff Q is calculated as follows:

$$Q = (P - 0.2S')^2/(P + 0.8S') \tag{4}$$

where P is the precipitation in inches. Substituting the value obtained for S′,

$$Q = (P - 0.63)^2/(P + 2.53) \tag{5}$$

and, for a 3-in rainfall (once-in-10-years storm),

$$Q = (3 - 0.63)^2/(3 + 2.53) = 1.01 \text{ in} \tag{6}$$

4. If it is decided to store the 1.01 in of runoff, the volume of storage required is traditionally calculated by using the formula, volume = area × runoff (in) ÷ 12 (in/ft); i.e.,

$$\text{Volume} = (2424 \times 94) \times 1.01/12 = 19\ 178 \text{ ft}^3 \tag{7}$$

or 0.44 acre-feet.

Volume Calculations Considering Antecedent Conditions

The problem with the calculation given above is that the volume is considered sufficient to store all the runoff water for each storm event. During the rainy season, storms of different quantities may occur each day over a long period of time, perhaps weeks. Water in the storage area will be displaced into adjacent lands or surface water bodies, and this displaced water will carry pollutants. If the stored water can be treated before the next storm event that produces runoff, the storage capacity of the holding area will be available.

Work completed by Wanielista (15, 16) for the East Central Florida Regional Planning Council indicated that diverting first-flush storm waters into percolation ponds and underdrained storage areas and use of overland flow with percolation are cost-effective procedures. Since these land areas are already available in most roadway rights-of-way and thus additional land purchases may not be necessary, diversion for percolation, underdraining, or overland flow would be even more cost-effective.

Coaxial graphs to aid in computing the volume of treatment (storage) as a function of watershed area, soil percolation, curve number, and depth of pond have been developed. These equations considered the stochastic conditions of rainfall and used data from three weather stations in Florida, one in Illinois, and one in New York. The efficiencies obtained did not vary by more than 5 percent. Because the coaxial graphs were difficult to use and were not available for watershed areas larger than 60 hm^2 (150 acres), they were extrapolated for larger watersheds, and a series of equations were developed by using bivariate regression analysis. The correlation coefficients for the bivariate equations were never less than 0.97. The equations for estimating "pond" volume for good percolation-type soil drainage conditions in ditch or pond areas are given in Table 4, where

V_I = basin volume for impervious area, 5-ft depth (acre-feet);
A = contributing watershed area (acres);
V_5 = basin volume at 5-in depth (acre-feet);
CN = composite curve number;
V_D^A = volume of basin at depth D in type A soil (acre-feet);
V_m = minimum basin volume (acre-feet);
D = depth of basin (feet);
DI = diversion volume (in); and
12 = conversion factor (inches to feet).

It has been suggested, as part of portions of the research not included here, that shallow aerobic water conditions be maintained for purposes of hydrocarbon degradation (e.g., gas, soil, and grease). A 0.3-m (1-ft) deep ditch adjacent to a roadway would most likely be aerobic and percolate fast during and after runoff conditions. Other depths—up to 1.5 m (5 ft) for type A soils and 0.9 m (3 ft) for type D soils—are also possible. These maximum depths were established to drain the areas by percolation to prevent mosquito-breeding problems.

If overland flow into ditches parallel to the roadway surface is the design and the first 2.5 cm (1 in) of every storm-water runoff can be stored and treated, then the quantity of pollutants removed from direct surface discharge to adjacent lands and water bodies is about 99

Table 4. Calculation of pond volume for type A soils.

Diversion Volume (in)	Impervious Watershed (5-ft-deep pond)	Composite Land Use		
		5-ft-Deep Pond	1 ft < Pond Depth < 5 ft	Pond Depth = 1 ft
0.25	$V_I = 0.016(A)^{1.28}$			
0.50	$V_I = 0.046(A)^{1.18}$			
0.75	$V_I = 0.09(A)^{1.11}$	$V_5 = V_I\,[0.59 + 0.37\,(CN/100)]$	$V_D = V_1 + [(V_5 - V_1)/4](D - 1)$	$V_1 = (A \times DI)/12$
1.00	$V_I = 0.14(A)^{1.07}$			
1.25	$V_I = 0.20(A)^{1.04}$			

percent of the yearly runoff mass. This level of treatment is more efficient than most advanced wastewater treatment processes for industrial and sewage wastes. But the shoulders and ditches must be designed to always retain the first runoff volume.

When conduits are used to transport a number of first flushes entering the conduits at various areas (and times), the concept of first flush is no longer valid because the pollution concentrations are random. The efficiencies calculated for such a case from cumulative runoff distributions for associated rational runoff coefficients are given below (c is the rational coefficient in Q = ciA):

Efficiency of Conduit System with No First Flush (%)			Diversion Volume (in)
c = 0.8	c = 0.4	c = 0.2	
96	95	90	1.25
95	93	82	1.00
93	90	72	0.75
90	82	60	0.50
82	60	40	0.25

These efficiencies would be applicable to large sewered areas.

This table was developed by using rainfall data obtained at the Tallahassee, Florida, airport. In a comparison with data obtained at the Orlando Jetport, these data produced the lowest efficiency of storm-water treatment. Orlando has fewer storms, and they are of less intensity, duration, and quantity than those at Tallahassee. The efficiencies obtained for the Orlando area are estimated to be at least 2-4 percent higher than those given above.

Example 2

For the previous example, taking into account the factor of percolation, calculate the required ditch volume if the ditch is, on the average, 1 ft deep and the percolation rate is estimated at a minimum of 1 in/h. It is desired to store and treat at least 1 in of runoff. By using Table 4, calculate the needed volume as follows (in U.S. customary units):

$$V_I = 0.14(A)^{1.07} = 0.14(5.23)^{1.07} = 0.82 \text{ acre-feet} \quad (8)$$

or 35 760 ft^3;

$$V_5 = 0.82\,[0.59 + 0.37(76/100)] = 0.72 \text{ acre-feet} \quad (9)$$

or 31 380 ft^3; and

$$V_I = [5.23(1)/12] = 0.44 \text{ acre-feet} \quad (10)$$

or 19 168 ft^3. Therefore, at a 1-ft depth, the area of ditch is 19 168 ft^3. For a 2440-ft-long area, the width of the ditch is 2.4 ft.

Overland flow of highway runoff into a ditch parallel to a roadway can be designed by using these equations. If no percolation is available in the ditch area, consideration should still be given to overland flow and possibly a 1-m (3.3-ft) deep ditch occupying a larger area. It is believed that the metals from highway runoff can be retained for the most part in the right-of-way areas if good percolation-type soils are used in the shouldered ditch areas.

CONCLUSIONS AND RECOMMENDATIONS

Soil, plant, animal, groundwater, and surface water samples were obtained from the shoulder and ditch areas adjacent to Florida highway rights-of-way. These samples were analyzed for metal content and hydrocarbon degradation. During this research, which extended over an 18-month period, 2221 metal determinations were done. Eleven field sites were used to represent aerobic water environments, partly anaerobic water environments [1-3 m (3.3-10 ft) deep], and different soil conditions.

The findings of previous research, which provided valuable guidance, can be summarized as follows:

1. Rates of deposition of metal from automobiles and the quantities of these pollutants in runoff waters have been well documented.
2. The transport of heavy metals by overland flow results in large amounts of these metals coming into contact with the soil, where they are generally retained. The relatively high concentrations of these metals found in soils adjacent to roadways are evidence of this.
3. When sewage is treated by land spreading, metals are found primarily in the soil.
4. The important interactions between soils and heavy metals that are involved in the retention of these metals in the soil are likely to include adsorption, ion exchange, redox reactions, and "coordinate" chemical reactions, including precipitation and complex formation.
5. Soil properties generally considered to be important in the retention of heavy metals are pH, cation exchange capacity, clay mineral content, and organic matter content.
6. Lead is particularly significant as a public health problem.
7. In laboratory studies, the capacity of soil to retain lead can be reasonably predicted by pH and cation exchange capacity. Under actual field conditions, however, many other variables may affect this capacity.

The research reported in this paper produced the following results:

1. The highest concentrations of metals along highway rights-of-way are found in animal life. However, more than 10 m^2 (1076 ft^2) of topsoil had to be "turned over" to collect enough soil insects (grubs) for one metal-detection analysis.
2. The greatest mass of metals was found in the

soil of shoulder areas—approximately 5000 times more than that found in animals.

3. Lead concentrations were considerably higher in surface soils [top 2-3 cm (0.8-1.2 in)] than in subsurface soils [15-20 cm (6-8 in)] at the same location. This was not consistently found for the other metals. Therefore, lead is generally immobilized in roadside soils.

4. The concentration of metals in the soil decreased with distance from the edge of the pavement.

5. Based on a regression equation developed by Zimdahl and Skogerboe (13), which related pH and cation exchange capacity to lead retention of a soil, the soils tested have an additional capacity to retain lead that is more than 10 times their existing lead content.

6. The organic matter in soil is important and correlates well with the ability of soil to retain metals. In general, however, other factors are also important, such as clay minerals, pH, and chemical reactions. No one removal mechanism or soil characteristic was determined to be the most important. The types of reactions that can occur are likely to be site specific.

For areas where surface water discharges from highway rights-of-way to adjacent lands are limited, the following recommendations are made for design and maintenance of the land and water. The underlying concepts for these recommendations are based on percolation rates of the land and interactions between metal and soil:

1. Rainfall excess (runoff) should be directed as overland flow as much as possible to promote water percolation and removal of metals.

2. Runoff can be diverted from open or closed conduits into shallow holding areas to remove specified quantities of pollutants.

3. A "muck blanket" should be spread on the soil before vegetation is planted to promote removal of metals.

4. Subsurface soil should be alkaline to promote removal of metals. Organic matter and clay minerals also aid in the removal of metals.

5. Soils adjacent to pavements need to be replaced periodically because of metal saturation. Care should be exercised in the disposal of these soils.

When overland flow of surface runoff waters is not possible, the first flush of runoff water can be diverted for treatment. The volume of water diverted depends on the discharge limitation. Treatment can be accomplished by storage and percolation in median or shoulder areas. Formulas for determining diversion volume and the percentage efficiency of diversion systems are given in this paper.

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The opinions, findings, and conclusions expressed in this paper are ours and not necessarily those of the Florida Department of Transportation.

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Stability of Stream Channel Patterns

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Many stream channel classifications have been developed based on stability considerations. Previous classifications can be categorized grossly as based on characteristics of straight, meandering, and braided planimetric patterns. A classification scheme is presented that divides stream patterns into five types: (a) straight, (b) straight with sinuous thalweg, (c) meandering, (d) meandering with the development of midchannel bars, and (e) braided. In the proposed scheme, the observed relative stability of the various patterns decreases from pattern 1 to pattern 5. Observed stability can be explained by theoretical stability analysis, but basic theoretical rules must be applied with caution because of the complexity of stream behavior.

When highway engineers plan a bridge across a stream, a major problem is to assess the stability of that stream. Because all streams are undergoing continuous changes, it is valuable for the transportation engineer to know the relative stability of different stream channel patterns in order to assess the stability of the bridge crossing. A stable stream is defined as one whose bed and banks are spatially fixed. For transportation engineers, this means that a stable cross section can be treated as a rigid boundary for design purposes. The purpose of this paper is to discuss the relative stability of different channel patterns based on current knowledge.

Stream behavior is an extremely complex problem, and it is difficult to formulate rules that can be universally applied. The general stability relations described here should, therefore, be applied with caution.

Geomorphologists use field evidence to trace the history of channel developments, since they are particularly interested in long-term effects. Engineers, on the other hand, use a theoretical base to investigate what would happen to streams under certain conditions of change. From conceptual models, they predict short-term (100-year) effects. Geomorphologists usually try to determine why a stream changes from the way it changes (why from how). Engineers try to predict how a stream will change from theoretical considerations (how from why). In this paper, we first discuss stream stability from the observational, or geomorphologist's, viewpoint and then discuss stability analysis of stream channel patterns from the engineer's viewpoint.

STREAM CHANNEL PATTERNS

Stream channel patterns are the cumulative results of a combination of climatic, geologic, topographic, hydrologic, and human disturbance factors. Basically, there are only three types of patterns: straight, meandering, and braided.

1. A straight channel has straight and parallel banks, and flow within the channel is mainly in the longitudinal direction.
2. A meandering channel [see Figure 1 (1)] consists of many bends separated by short, straight reaches, or "crossings", between two bends. The secondary currents in each stream bend are significant enough to cause modifications of the bend. Usually, there are deep scour holes at the outer channel bend and the water near the inside bend is rather shallow. The channel bends may or may not be symmetrical.
3. A braided stream [see Figure 2 (1)] usually has a large width-to-depth ratio, and there are always many small channels that have developed within the main channel.

Both meandering and braided channels can develop into anabranched channels, which are subsidiary channels that diverge from a stream and eventually rejoin it.

Sometimes, the distinctions among these three basic channel patterns are not clear. For example, one reach of a stream can exhibit both meandering and braided patterns, or a stream may consist of straight, meandering, and braided reaches at different locations. A

Figure 1. Meandering reach of Clark's Fork of the Yellowstone River showing two recently formed oxbow lakes produced by meander cutoff.

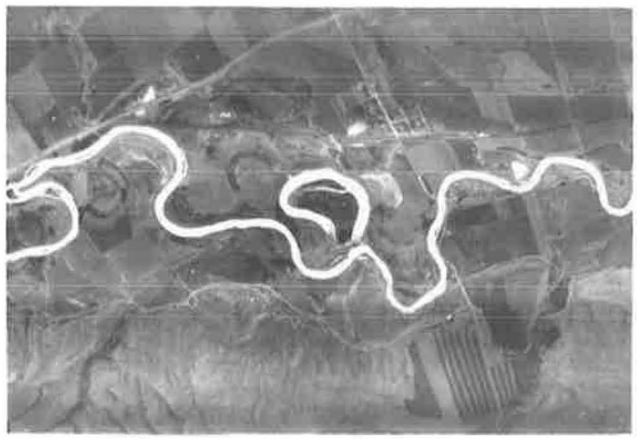


Figure 2. Braided reach of the Yellowstone River.

stream reach may appear to be braided at low flow stages, and yet its overbank flow may be contained within two relatively straight banks. In both cases the stream appears to be straight at flood stage. A few rivers are straight because they have stable banks and gentle gradients adjusted to the water and sediment load supplied—for example, the lower Mississippi River on the delta below New Orleans.

The basic causes that initiate meandering are still not entirely clear, but they are probably numerous. Shen (2) and Callander (3) have summarized these causes as follows:

1. Development of secondary currents as a result of (a) dynamic stability of flow (3), (b) rotation of the earth (4, 5), and (c) differences in roughness between bed and bank (6);
2. Disparity between hydrologic and topographic conditions [according to Schumm and Khan (7), if the valley floor on which the river flows is, as a result of past hydrologic conditions, steeper than necessary for the modern water and sediment load, meandering occurs as a natural way for flow to seek a lesser slope; they also found that the addition of 3 percent kaolinite in suspension would enhance and cause the development of a meandering channel];
3. A lateral disturbance, which can be caused by such factors as a tributary or a difference in the soil between the left and right banks (8); and
4. "Erosion-deposition processes tending toward the most stable form in which the variability of certain essential properties is minimized" (9).

According to Lane (10), there are two primary causes of braided streams:

1. The stream may be supplied with more sediment than it can carry (overloaded), and part of that sediment may be deposited.
2. Steep slopes and high sediment loads and velocity can cause a wide, shallow channel in which bars and islands readily form (11).

Shen and Vedula (12) have presented an explanation of braided channels based on consideration of bank erosion and sediment transport. The basic principle is that in a narrow stream the entire bed can act as a unit to aggrade or degrade according to the difference between the sediment supply and the capability of the flow to transport it. However, when a stream cross section is too wide (because of excess bank erosion during high flow or weak bank resistance or both), the entire channel cross section cannot act as a single unit, and thus part of the wide channel may be covered by numerous small channels and result in a braided stream.

PREVIOUS STUDIES OF CHANNEL PATTERNS AND OTHER PROPERTIES

Channel Patterns

Various investigators have classified channel patterns based on direct observation. Table 1 gives a brief summary of these studies and the specific items investigated.

Kellerhals and others (19) described channel patterns in a different manner, dividing them according to three main headings: (a) a type that consists of straight, sinuous, irregular, irregular meander, regular meander, and tortuous meander; (b) channel islands, subdivided into occasional, frequent, split, and braided islands; and (c) channel bars, subdivided into none, side bar, point bar, channel junction bar, midchannel bar, diamond bar, diagonal bar, and sand wave. Table 1 gives our arbitrary interpretation of their classification, according to the normal divisions of straight, meandering, transitional, and braided (Kellerhals and others actually classified the braided pattern as a subset of straight or meandering patterns).

Brice and others (20) suggested a new criterion by which a channel is classified into five categories: (a) equiwidth, point-bar stream; (b) wide-bend, point-bar stream; (c) braided, point-bar stream; (d) fully braided stream; and (e) anabranched stream (Table 1).

Other Channel Properties

In 1863, Ferguson noticed that meander wavelength bears a relationship to channel width. Since then, various channel properties, including width, depth, meander wavelength and amplitude, sinuosity, bend characteristics, and braiding, have been investigated by many researchers by using conventional statistical

Table 1. Classification of channel patterns by various authors and some specific factors investigated.

	Channel Pattern			
Author	Straight	Meandering	Transitional	Braided
Lane (10)		X		Caused by (a) steep slope and (b) aggradation
Leopold and Wolman (13)	X	X		X
Schumm (14)	X	Regular, irregular, tortuous	X	Straight (island)
Popov (15)	Alternate bars			Midstream bars
Culbertson and others (16)		Point bars (a) of uniform width and (b) wider at bends		Point bars, islands
Chitale (17)[a]	X	Regular, irregular, simple, compound	X	X
Garg (18)		Uniform width, point bar		Point bar, bar or island
Kellerhals and others (19)	X	Irregular, regular, tortuous	Sinuous, irregular	
Brice and others (20)[b]	Equiwidth, wide-bend	Equiwidth, wide-bend, braided	Equiwidth, wide-bend	Wide bend, with (a) braided and (b) fully braided

[a] Chitale (17) defines two major categories: single-channel (straight, meandering, and transitional) and multiple-channel (braided).
[b] Brice and others (20) also define another type of channel: anabranched.

Table 2. Channel properties investigated by various authors.

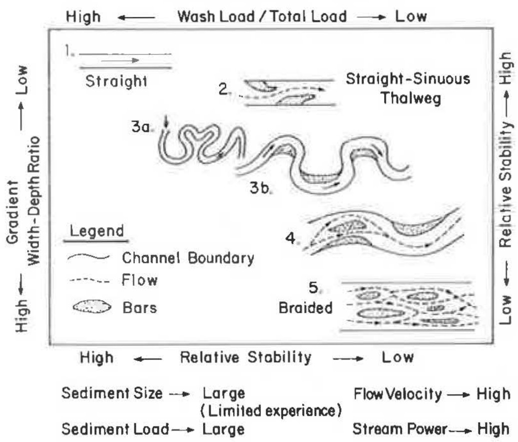
Property	Author	Property	Author
Channel width and depth	Schumm (14, 21-23) Leopold and Maddock (24) Bray (25)	Sinuosity	Langbein and Leopold (9) Schumm (14, 21-23) Chitale (17) Bray (25) Inglis (27) Ackers and Charlton (30) Ferguson (31, 32) Daniel (33)
Wavelength and amplitude of meander	Dury (11) Leopold and Wolman (13) Schumm (14, 21-23) Jefferson (26) Inglis (27) Ferguson[a] Carlston (28) Speight (29) Ackers and Charlton (30) Ferguson (31, 32)	Bend characteristics	Leopold and Wolman (13) Inglis (27)
		Braiding	Brice (34) Howard and others (35) Krumbein and Orme (36)

[a]See Inglis (27).

Table 3. Classification stability of alluvial channels.

Pattern No.	Channel Pattern	Channel Stability
1	Straight; equiwidth, with straight thalweg	Stable
2	Straight, with sinuous thalweg	Generally stable, but thalweg shift and bar migration occur
3a	Meander; equiwidth with small point bars	Stable; chute cutoffs can occur
3b	Meander; wide bends, with large point bars and cut bank outside of meander	Relatively stable because of chute and neck cutoffs and shift and growth of meander
4	Meander-braid transition; large point bars with frequent chute cutoffs	Unstable
5	Braided, with multiple thalwegs that shift and many shifting bars and islands	Unstable

Figure 3. Classification and stability of alluvial channels and variables that affect channel patterns.



methods. Some have attempted to establish empirical selection between channel morphology and hydrology. The channel characteristics studied are given in Table 2.

CLASSIFICATION AND RELATIVE STABILITY OF ALLUVIAL CHANNELS

There are three major categories of stream channels: (a) bedrock, (b) alluvial, and (c) partially controlled. The bedrock channel is fixed in bedrock and is stable over the time span that concerns engineers. The alluvial channel is formed in sediment that has been transported by the stream, and therefore channel morphology and the alluvium reflect the type of sediment load transported. It is this group of channels that is classified in this paper. The third group requires brief mention because it is predominantly an alluvial channel that encounters bedrock and older resistant alluvium in its course and is, at least locally, influenced by this encounter. For example, the channel may be locally fixed in position by resistant materials, which may significantly alter the local meander pattern.

Classification

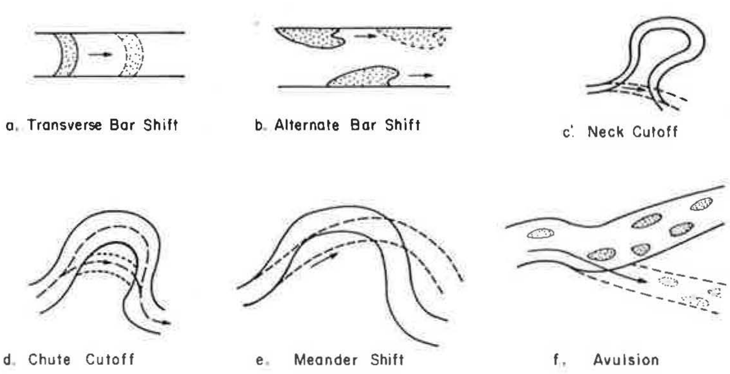
We propose a tentative classification of the patterns of alluvial stream channels that relates to the stability of the patterns. The five patterns given in Table 3 have been selected as including the types of streams common in the United States. Obviously, some channel types span a great range of patterns; for example, pattern 3 channels can have a wide range of sinuosity.

In order to provide information on the stability of alluvial channels, classification of such channels should be based not only on channel pattern but also on the variables that affect channel morphology. Many empirical relations demonstrate that channel dimensions are attributable mostly to water discharge, whereas channel shape and pattern are also related to the type and amount of sediment load that moves through the channel. Geomorphic history is also important because it can determine the slope of the valley floor or alluvial plain on which the stream flows. For example, some very straight rivers, such as the Illinois River and the Mississippi River below New Orleans, are flowing on alluvial surfaces that are relatively flat; on the other hand, the most sinuous reach of the Mississippi (Greenville Bends) is localized on the steepest part of the valley floor below the confluence of the Arkansas River (23).

Stream channels form a continuum, from straight to meandering to braided, that can be illustrated by the five patterns shown in Figure 3. These patterns illustrate the range to be expected in nature. The variables that influence pattern characteristics and the relative stability of the patterns are also indicated in Figure 3.

Since this classification is designed to indicate the relative stability of a river, it should be especially useful to transportation engineers. For instance,

Figure 4. Typical types of channel changes.



although a pattern 1 straight channel may occur only rarely, when it is found it means that the banks are relatively stable and will erode slowly. In the case of a pattern 2 straight channel, the outer banks are relatively stable but the thalweg between them can shift. In designing a pier, the shifting of flow within the channel must be accounted for.

Channel patterns 3 and 4 are meandering channels in which the outer banks are not straight and parallel and may shift frequently. In this case, the shifting of the thalweg and the banks follows a more or less regular pattern. Most rivers meander, and we show the continuum of meandering channels by only three patterns: patterns 3a, 3b, and 4.

Some investigators have divided braided channels into several groups. For instance, Brice (34) has subdivided braided channels into those with submerged bars and those with islands. The changes of the thalweg within the cross section of a braided channel cannot be predicted, and bank cutting is usually relatively rapid. Submergence or emergence of an island is very much a function of the frequency of flow. Since all of these factors cause similar problems for highway engineers in designing bridges across streams, the braided category is not subdivided here.

The basis of classification is the type of sediment load transported by the channel. The absolute quantity of water and sediment that moves through the channel can be less important than the type of sediment load. For example, in nature there are large and small channels of each of the types shown in Figure 3, and in each case the large channel forms in response to a larger water discharge but the pattern itself and the shape of the channel depend on the proportion of the total sediment load (silt, clay, sand, and gravel) that is wash load (silt and clay) or bed-material load (sand and gravel) (23).

Observed Stability Tendencies

The following relations were established from investigations of the pattern, dimensions, and shape of sand-bed streams of the Great Plains of the western United States. A summary of the results can be found elsewhere (23).

When bed-material load is small, wash load is a large part of the total load and the channel is narrow and deep (with a width-depth ratio of <10) and, depending on valley slope, the channel can be straight (pattern 1) or very sinuous (pattern 3a). When the percentage of bed-material load is intermediate, the width-depth ratio is less and sinuosity is between about 2.0 and 1.3 (pattern 3b). This sandy channel may also be relatively straight, but the thalweg or the deepest part of the straight channel may be sinuous (pattern 2).

As the proportion of bed-material load increases, width-depth ratio increases (to >40) and sinuosity is low. There is a tendency for multiple thalwegs to form (pattern 4). The greatest development of channels and bars occurs in the braided channel (pattern 5) when the ratio of bed-material load to total load is high.

As Figure 3 shows, not only does the channel pattern change from pattern 1 to pattern 5 but other morphologic aspects of the channel also change; that is, for a given discharge, gradient and width-depth ratio increase. In addition, peak discharge, sediment size, and sediment load will probably increase from pattern 1 to pattern 5. Naturally, with such geomorphic and hydrologic changes, hydraulic differences can be expected, and flow velocity, tractive force, and stream power increase from pattern 1 to pattern 5. Obviously, then, channel stability decreases from pattern 1 to pattern 5, patterns 4 and 5 being the least stable.

In nature, there is a continuum of patterns between patterns 3 and 4 of decreasing sinuosity, increasing gradient and width-depth ratio, and decreasing bank and channel stability. A comparison of aerial photographs with Figure 3 should provide a means of evaluating the relative stability of river channels.

Suspended-load channel patterns 1 and 3a are associated with small amounts of bed-material load; as a result, the banks tend to be relatively stable because of their high silt-clay content and thus the channels are not characterized by serious bank erosion or channel shift. Bars may migrate through the channel of pattern 1 (Figure 4a), but this should not create undue instability. Neck cutoffs are characteristic of pattern 3a (Figure 4c); they can be anticipated by inspecting the shape of the meanders.

Channels with higher bed-material loads and banks that contain less cohesive sediment are less stable than suspended-load channels (patterns 3b and 4). Alternate bars migrate through the low-sinuosity channel (pattern 2), causing alternating reaches of stable and eroding banks. In these cases, meander growth and shift are characteristic (Figure 4c), and chute cutoffs (Figure 4d) reveal a tendency for the development of multiple thalwegs.

When channels have large loads of coarse sediments, bank sediments are easily eroded, gravel bars and islands form and migrate through the channel, and avulsion (Figure 4f) may be common (pattern 5). As the channel straightens, meander shift and cutoffs are absent.

Because the preceding discussion relates entirely to stable alluvial channels, the changes indicated for each channel type are typical and can be expected under

all conditions. However, when the sediment load or the discharge transmitted by channels is altered, channels respond by either eroding or depositing sediment, and they become unstable. But, because channels are composed of sediments of different degrees of stability and because the ways in which they erode and deposit and transport sediment are different, their responses to an altered hydrologic regime are also different (23). In each case, a reduction of sediment load will tend to produce scour and perhaps an increase in sinuosity. As load increases, aggradation and braiding occur, and meander cutoffs cause a decrease in sinuosity.

Channel patterns are frequently altered by human intervention, usually by straightening the channel by means of cutoffs or complete channelization. The straightened channel can then be very unstable as it attempts to resume its former gradient and pattern. Care should therefore be taken in evaluating the stability of straight channels: Natural channels should be stable, but artificial channels may have a propensity for major change.

An anastomosing, or anabranched, system is actually a multiple-channel system (23, p. 155), and such a system can be composed of any of the channel types shown in Figure 3. The anabranches will be sinuous or alluvial plains, straight deltas, and braided or alluvial fans. Therefore, the class function of Figure 3 can also be applied to an anastomosing or anabranched pattern.

Analysis of Stability Tendencies

Strictly speaking, the rate of sediment transport cannot be correlated with channel stability but, as a general rule, streams that have high transport rates tend to be relatively less stable. As Figure 3 shows, the type of sediment load, sediment size (with which there is only limited experience), flow velocity, stream power, width-depth ratio, and channel gradient is related to channel pattern and the relative stability of alluvial channels. In this section, additional, variable factors are examined from the viewpoint of the mechanics of flow. For convenience, the order of the factors is changed slightly. Width-depth ratio can be treated as a result of instability and is not commented on here.

Sediment Transport

In sediment load we include flow conditions, channel gradient, and sediment size. A completely stable channel is one in which the flow is not able to move the sediment on either the banks or the bed. However, aggradation could alter the shape of the channel.

The rate of sediment transport can be determined by using the following ratio:

$$R_1 = F_1/F_2 \tag{1}$$

where F_1 is the fluid force acting with a particle and F_2 is the resistance force of the particle to flow. R_1 can also be shown as a Froude number based on shear velocity and sediment particle size. Shields (37) determined that, if $R_1 < 0.06$ in turbulent flow with cohesionless sediment, no sediment particles can be moved by the flow. In any case, a low R_1 value indicates a low rate of sediment transport and a relatively stable channel, whereas a high R_1 value indicates a relatively high rate of sediment transport and a relatively unstable channel.

For a relatively narrow range of sediment sizes (e.g., the channel used in Figure 4), an increase in flow velocity and stream power will have the same effect as an increase in the rate of sediment transport, and channel stability will decrease. This situation may be worsened if an increase in flow rate carries the flow into "upper-regime flow", where the occurrence of antidunes creates a great deal of turbulence and further weakens the channel banks. There were no upper-regime flows in the observations used to formulate Figure 3.

An increase in channel gradient has almost the same effect as an increase in the Froude number of the flow. It is quite reasonable, therefore, that an increase in sediment load, flow velocity, stream power (increasing stream power is almost the same as increasing flow velocity), or channel gradient would have the same effect on channel stability as an increase in the rate of sediment transport. The few observations of a decrease in channel stability with an increase in sediment size are not easily explained. Several other factors may be involved in these cases.

Type of Sediment Load

The influence of type of sediment load on the stability of a stream can be analyzed as follows:

1. The patterns shown in Figure 3 are dimensionless and therefore do not depend on the quantity of water and sediment moved through the channel. If discharge were constant for each pattern, however, the quantity of bed load moved through the channels would indeed be related to the channel pattern. For the great range of channels of greatly differing dimensions, the significant factor is the proportion of wash load and bed load moved through the channels (14, 23) and the type of load. A high ratio of wash load to total load indicates that the proportion of bed load to total load is small, and a narrow, deep, straight channel (patterns 1 and 2) or a sinuous, low-gradient channel (pattern 3) can move the sediment load. When the ratio of wash load to total load is low, the proportion of bed load to total load is high, and a shallow, wide, relatively straight channel (pattern 4) or a braided, steep-gradient channel (pattern 5) is required for transport of this sediment load.
2. The type of sediment load determines the nature of the sediment that composes the bed and banks of a channel. When the ratio of wash load to total load is high, the bank will contain large amounts of silt and clay and they will be cohesive. A channel with cohesive banks is relatively stable.
3. According to Simons and others (38), the presence of fine sediment will have the same effect as a change of fluid viscosity because both will affect the fall velocity of sediment. When there are very large concentrations of fine material, the dune bed (in the lower flow regime, when $R_1 \leq 0.10$) may become plane. A reduction in the transport of bed material will occur, and the channel could become more stable, as shown in Figure 3. In the upper flow regime, the reduction in fall velocity of sediment particles that results from the addition of fine sediment may change a standing-wave flow to an antidune flow or increase the activity and turbulence of an antidune flow. These changes increase resistance to flow, increase the transport of bed material, and decrease channel stability. This case is outside the range of observations used in determining Figure 3.

Additional Factors

Other factors that affect channel stability are not discussed here in detail since their influence varies from

site to site. These factors include the ratio of bank resistance to bed resistance, the ratio of fluid force to resistance, gradation of sediment material, bank seepage, vegetation, and water temperature. Any of these factors may be of paramount importance at specific sites, and their importance can be determined by thorough design investigations.

METHODS OF IDENTIFYING AND EVALUATING CHANNEL PATTERNS

Several techniques of identifying channel patterns and evaluating their stability are available to the transportation engineer. These methods can be grouped according to whether they are based on existing conditions, historical information, or prediction. Essentially, two kinds of pattern changes may occur: (a) intrinsic (changes inherent in the stream system itself, such as cutoffs, channel migration, and avulsion) and (b) extrinsic (changes that are the result of external factors such as changes in sediment supplies and human intervention).

Existing Conditions

Existing channel patterns and possible pattern changes are determined by using the following methods:

1. Direct observation—Low-altitude flights and ground visits are necessary to identify existing channel patterns. Aerial photography is probably the least expensive way of studying channel patterns.
2. Analysis of data—Analysis of bank stability, geologic setting, variation of flow magnitude, sediment transport characteristics, and geomorphic factors can be useful in determining possible changes in channel patterns.

Historical Information, Records, and Research

The following types of information can be used to identify changes in channel patterns:

1. Records such as newspaper reports, railroad company files (if there was a railroad nearby), church records, and court cases are possible sources.
2. Several sets of aerial photographs taken at different times will clearly document channel changes. In many areas, aerial photography was begun in the mid-1930s.
3. Visits with local residents can be helpful in determining previous channel changes. Many of these "stories" should be verified with other sources if possible.

Predictions

Of utmost importance to the transportation engineer is the ability to predict changes in the pattern and location of streams. Rather rapid and otherwise unexpected changes are likely to occur in response to natural and human disturbances of the fluvial system. Among the disturbances that should be considered in making predictions are the following:

1. Natural disturbances include (a) geologic processes that are discontinuous with respect to time and (b) short-term episodic phenomena. Examples of the former include large-magnitude flooding, mass wasting, and stream capture. Another problem area is engineering design based on nonrepresentative data that result from short-term climatic cycles. It is difficult to anticipate channel changes produced by these disturbances.
2. Man-made changes in the drainage basin (land-use changes), in the stream channel (engineering projects), and in retention areas (reservoirs) may cause significant response in a channel. Alteration of vegetation, surface materials, and landforms and the installation of storm sewers change water yield, the timing and magnitude of flood peaks, sediment yield, channel geometry, snow accumulation and melt, the water table configuration, and other important components of the hydrologic cycle. In this regard, land use can be expected to have potentially profound downstream effects on stream channels. Alteration of stream courses by channelization, straightening, and the construction of streamside structures (e.g., dikes, levees, and bridges) can be expected to affect the channels and has the potential for changing stream patterns. Interbasin water transfer projects and impoundments of water in reservoirs can be expected to increase the erosion of stream beds and banks by increasing annual runoff and decreasing upstream sediment load, respectively.

Accurate predictions for a specific reach of channel can only be made after thorough analysis by engineers and geomorphologists. The actual methods of analysis are beyond the scope of this paper.

SUMMARY

This paper proposes a scheme to classify alluvial channel patterns into five types. The observed relative stability of these various types of patterns is given, and the applications of the classification are presented. Although it has been shown that the observed stability can be explained by theoretical stability analysis, readers must be cautious in applying these rules because stream behavior is extremely complex.

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Ecological Effects of Highway Fills on Wetlands: Examples from the Field

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To establish a more comprehensive base of empirical evidence on interactions between highways and wetlands, case studies were conducted at eight highway sites in wetlands at various U.S. locations. Selection of the study sites was based in part on a comprehensive review of literature on highway-wetland interactions. The conclusions drawn from the case studies also reflect information obtained from the literature review. Since all case studies involved analyses of wetlands in which highway-induced changes had already taken place, the effects that were observed or deduced were largely limited to major, long-standing changes resulting from the continuining physical presence of a highway facility. Four classes of effects appear to dominate: (a) altered drainage, the most common effect; (b) interrupted tidal exchange, a manifestation of altered drainage found in tidal areas; (c) physical obliteration of wetlands resulting from the placement of highway fill or disposal of dredged material; and (d) habitat creation, included not because of its general occurrence but as an example of what can be done in many instances.

Wetlands are among the most biologically productive ecosystems in the world, often surpassing forests and managed farmlands. Like all ecosystems, wetlands can be partly understood by studying their most prevalent or dominant vegetation and interrelated animals and microbes. These biological features are dependent on the physical environment and thus vary widely according to climate, physiography, and soils or other substrate features. But, regardless of other variations in physical conditions, wetlands occur because there is water present on a permanent or reliably recurring basis above or within the surface soils, and it is the presence or absence of this water that exerts the greatest single influence on wetland ecology.

By altering the hydrologic regime, and through other means, the placing of highway fills on wetlands can have significant physical and biological effects on the natural environment and the ecological processes of the affected area. In addition to the direct physical alterations that result from construction activities, there are often physical, chemical, and biological effects that extend well beyond the construction and right-of-way corridor. This cascading of effects beyond the immediate site of impact is much more likely to occur when wetlands (in contrast to uplands) are involved, since wetlands are, almost by definition, those units of the landscape that receive, detain, retain, and discharge both surface and groundwater flows. As such, each wetland may reflect even the smallest change in the waters that feed it, and these changes may in turn be transmitted to other wetlands downstream.

Both coastal and inland wetlands have been well studied by biologists for many decades. The resulting literature provides a comprehensive, but essentially static, picture of wetland biota in terms of individual species and biotic communities and their distribution by geographic area and type of wetland. Within the past decade or so, the study of wetland dynamics—the interaction of wetland species and biotic communities over time—and associated changes in wetland productivity has been greatly aided by improved scientific theory and extensive use of computer simulations. From these relatively recent advances in ecosystem modeling there has emerged a growing understanding of how wetlands interact with other ecosystems and of the relationship between wetlands and local and regional hydrology.

In an effort to establish a more comprehensive base of empirical evidence on highway-wetland interactions, a set of case studies was conducted at highway sites in wetlands across the United States. The choice of field sites and the conduct of the studies were strongly influenced by the resources available to the investigators and by the lack of documented information on which instructive case studies could be based. There are few instances in which the physical and biological characteristics of a wetland have been documented before, during, and after construction of a highway facility. Since our resources did not permit the acquisition and analysis of primary data, we were almost wholly dependent on secondary information, supplemented by our own after-the-fact, on-site observations. Further contributing to the problem is the fact that relatively few highways are built in pristine wetlands—that is, wetlands unaltered by the prior construction of railroad embankments, water control and drainage structures, and other works of man. Thus, the effects of the highway are often confounded, if not totally obscured, by the effects of prior, or subsequent, alterations. For these reasons, what is reported here is a forced compromise between what we would have preferred ideally and what was actually possible.

As can be seen in the map of the lower 48 states shown in Figure 1, a reasonable approximation to geographic comprehensiveness was achieved. The eight case study sites range from Oregon to Massachusetts and from Minnesota and North Dakota to Florida. A wide range of wetland classes is represented, including examples of both tidal and inland situations. The studies include a variety of highway types, from gravel roads 50 years old or older to Interstate highways. Nevertheless, vast areas of the country, particularly the West Coast and south-central and southwestern regions, are not represented. However, the nature of the effects that are reported on is such that, in many instances, experience in one area can be transferred to another.

The various study locations and wetland classes and the primary effect or effects caused by the highway in question are given below:

Location	Type of Wetland	Effect of Highway
Whately, Massachusetts	Borrow pit ponds	Habitat creation
Fairfield, Connecticut	Estuarine salt marsh	Interrupted tidal exchange
Philadelphia, Pennsylvania	Freshwater marsh	Obliteration by fill
South Florida	Freshwater marsh and wooded swamp	Altered drainage
Roscommon, Michigan	Wooded swamp over organic soils	Altered drainage
Northeast Minnesota	Wooded swamp over organic soils	Altered drainage
Jamestown, North Dakota	Prairie potholes	Altered drainage, obliteration
La Grande, Oregon	Cattail-bulrush marsh	Altered drainage

Figure 1. Case study sites.

The information derived from each case study is organized according to type of ecological effect. Emphasis here is on those effects that tended to predominate in several studies or that are most graphically illustrated at one or more study sites. Since all of the case studies involved a retrospective analysis of a wetland in which highway-induced changes had already taken place, the effects that were observed or deduced were largely limited to major, long-standing changes resulting from the continuing physical presence of a highway facility. There was no opportunity, given the nature of these studies, to observe such significant, but relatively transitory, effects as erosion and sedimentation resulting directly from ongoing construction activities. The absence of such effects from these case studies, therefore, reflects not their lack of importance but the practical limitations of the case studies themselves.

Of the many effects of highways on wetlands that were identified in the case studies, four classes of effects appear to predominate:

1. The most common effect by far is altered drainage; indeed, to a greater or lesser extent, all of the case studies (with the exception of the Whately borrow pits) provide evidence of this class of impact.
2. In total area, the effects of altered drainage are manifested as interrupted tidal exchange, and this class of effect is placed in a separate category.
3. The physical obliteration of wetlands as a result of the placement of highway fill or disposal of dredged material was also a commonly identified effect of highway construction.
4. The fourth class of effect, habitat creation, is included not because of its general occurrence but as an example of what can be done in many instances.

ALTERED DRAINAGE

Wetlands are defined in various ways; in each case, however, it is the presence of water in and on the soil that is critical to the definition—and the existence—of wetlands. Thus, for example, the U. S. Fish and Wildlife Service defines a wetland as "land where water is the dominant factor determining the nature of soil development and types of plant and animal communities living at the soil surface" (1). The U. S. Army Corps of Engineers defines wetlands as "those areas that are inundated or saturated by surface or groundwater at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions" (2).

The hydrologic regime is the controlling feature in wetland ecology, and alterations in this regime will have profound effects on the environment. Wetland plant communities are dependent not only on the presence of water but also on the frequency and amount of inundation. Changes in community structure can be expected when any of these characteristics are altered. The exact nature of the change will depend on the new water regime, the species composition of the former community, the available seed sources, and other factors. In most parts of the nation, the occurrence of water in wetlands is the result of seasonal patterns of precipitation, freezing, thawing, and the rate at which plants use water (transpiration). The duration and timing of these influences are a stochastic phenomenon particular to a given geoclimatic area. Thus, the concept of "normal flow pattern" in wetlands is understood as a pattern of flow probability with volume, time, location, and duration of occurrence.

Highways and analogous structures impound the flow of surface water and groundwater to a greater or lesser degree and thus tend to raise water levels on the upflow side of the structure and lower levels on the downflow side. Conversely, directing water around and at specific places through the structure may concentrate the flow from the affected watershed to specific aquifers, channels, and wetland areas, thus raising water levels in those areas. These effects are usually most apparent where sheet flow is intersected by the highway embankment. Attempts to accommodate the interrupted flow by means of culverts are generally successful only in protecting the highway structure. The change from diffused to concentrated flow results in a significant disruption of the hydrologic regime, which is reflected in alterations in the associated ecological system. Two types of sheet flow should be distinguished: surface and subsurface.

Subsurface Flow

Subsurface sheet flow is essentially ubiquitous, representing as it does the manner in which groundwater typically moves through the earth. But it is in organic soils, which underlie many classes of wetlands, that the interruption of subsurface flows often presents a problem. The Roscommon, Michigan, and northeastern Minnesota study sites exemplify this situation. In both cases, the highway embankment interrupted the movement of subsurface flows, which led to the elevation of the water table on the upstream side of the highway. Where culverts were installed, they were generally placed too high and too far apart to significantly affect the upstream buildup of water.

Large areas of the several states and Canadian provinces that border the Great Lakes are overlaid to various depths with peat and other poorly drained soils. These areas are typically dominated by various species of wetland conifers, the harvesting of which is a major economic activity. Extensive damage to timber has been observed on the upstream side of highways and other embankments crossing peat wetlands. The U. S. Forest Service estimates that in northern Minnesota alone more than 30 000 areas of swamp conifers have been so affected.

The nature of the problem is shown in Figures 2 and 3, which are taken from the Minnesota study. Figure 2 is a planimetric view of a typical wetland crossing and the area in which flooding occurs. The road is shown running parallel to the contours of the land and positioned so as to cross the wetland at its narrowest point—a typical situation. The cross-hatched area upland from the highway represents the location of inundation caused by the blockage of drainage. Because of the geometry of the basin, the extent of this zone of inundation is usually considerably larger than the area immediately adjacent to the road.

Figure 3 shows a profile view of the area shown

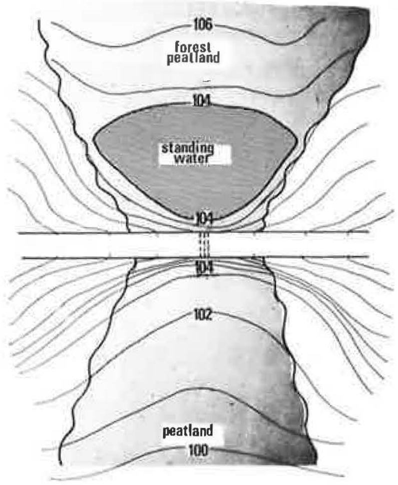


Figure 2. Planimetric view of typical peat wetland crossing showing area in which flooding occurs.

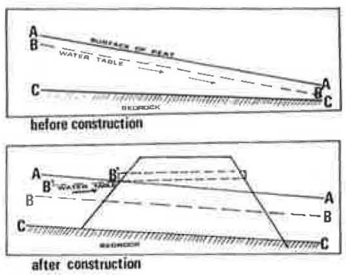


Figure 3. Hydrologic problem and water table relations in roads crossing peat wetlands approximately parallel to the contour.

planimetrically in Figure 2. In part A of the figure, the level and direction of flow of the water table are shown by the broken line from B to B. In undisturbed peat wetlands, the water table tends to be located 10-50 cm below the surface during much of the growing season, and it is in this zone that most of the horizontal movement of water through the soil takes place. Below 50 cm or so, the soil is consolidated because of the weight of the overlying soil, and little, if any, movement of water occurs in this near-impermeable medium.

Part B of Figure 3 shows the effect that the construction of the road has on the level of the water table. In the first instance, the road fill acts as a dam, blocking the movement of water along and below line B-B. The peat immediately below the fill is consolidated by the weight of the overburden, and this effectively reduces its hydraulic conductivity to zero. Since only the upper 50 cm of the peat is hydraulically active, almost any combination of fill, displacement, and consolidation will impede the movement of water across the line of fill regardless of the depth of the peat layer. The water that is impounded by the fill will rise at the embankment to the level of the culvert; the level of the water table will assume a position along line B^1-B^1, intersecting the undisturbed water table B-B some distance uphill from the road.

The invert elevation of the culvert relative to the undisturbed water is the most important design feature affecting the inundation caused by construction of the road. Typically, the culvert is located at or above the point of intersection of the embankment with line A-A so as to intercept surface flow in a more or less defined channel. Although this practice may suffice to prevent overtopping of the roadway by storm runoff, it does not provide adequately for subsurface flow, which is the primary means of drainage in peat wetlands during much of the year. Flooding and damage to timber in the area above the road are almost inevitable. (Highway-induced flooding may also have a beneficial effect on wildlife by providing open water, standing dead trees, and other favorable habitat. Because of these and other wildlife benefits, fish and wildlife agencies periodically request the creation of highway impoundments.)

The extent of the damage that results from highway-induced flooding and the road design and location features most frequently associated with such damage were investigated in 1965 by the North Central Forest Experiment Station of the U. S. Forest Service (3). A systematic random sample of 70 forest wetland crossings in seven contiguous counties in northeastern Minnesota was studied to determine the extent of damage done to trees by the damming effect of roads. The roads studied had all been in place for a considerable period of time, many for 50 years or more, and all could be classified as low standard in terms of design features such as width, surface type, foundation, and drainage. Eighty percent of the crossings were on peat, the rest on mineral soil. The timber involved was mainly black spruce and tamarack. Northern white cedar, black ash, red maple, and associated species were also present.

Tree damage was classified into three categories: (a) no damage, (b) trees killed or weakened within 15 m of the road, and (c) trees killed beyond 15 m of the road. Of the 70 sites included in the sample, 55 percent showed some degree of damage. All of the 39 crossings that showed damage were within 45° of being parallel to the contours along which they ran; that is, they tended to run across rather than along the drainage flow. The apparent rise in water table in the zone of most serious tree damage averaged about 28 cm for the 39 crossings, the greatest rise being 76 cm.

A comparison of U. S. and European practice in the design of road drainage features in organic soils suggests several steps that can be taken to prevent the damming action of roads that cross peat wetlands. European practice calls for the excavation of a collector ditch along the upstream side of the road. Culverts are placed so that their inverts are at or near the bottom of the collector ditch, 1 or 2 m below the original level of the swamp. A discharge ditch, perpendicular to the roadway, to carry water beyond the influence of the road is also recommended. A perpendicular entrance ditch on the upstream side of the road and an additional discharge ditch running parallel to the downstream side may also be used. Figure 4 shows the location of the various ditches (3).

The location of these drainage features in profile and their effect on the water table are shown in Figure 5. The dimensions and location of the various drainage features will, of course, vary with conditions at the site.

The converse of the blocked-drainage problem observed in the Michigan and Minnesota cases is found at Ladd Marsh, in La Grande, Oregon, where the water table has been lowered as a result of highway construction activities. The manner in which both the surface and subsurface water regimes have been altered by highway activities at this site is shown in Figure 6. The Foothill-Ladd Canyon Road was constructed with borrow. Drainage for the road is provided by a ditch along the uphill (west and south) side of the road. In the absence of lateral culverts through the road embankment, the surface sheet flow that formerly fed the marsh is now diverted around it. A secondary source of sheet flow from the north is intercepted by a preexisting drainage ditch at the end of the marsh. The deprivation of water for the marsh is compounded by the borrow ditch along the west side of I-80N, which has lowered the subsurface water table in the marsh

Figure 4. Location of various ditches to ensure proper drainage of peat wetland road crossing.

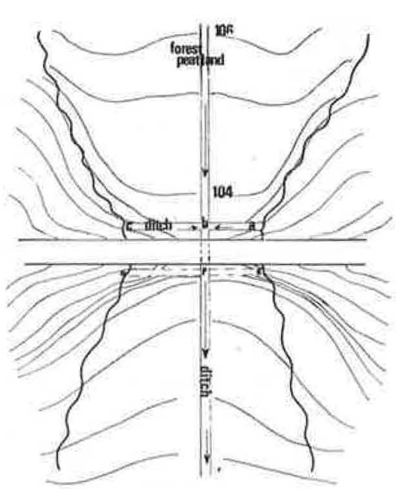


Figure 7. Everglades and Big Cypress wetland complexes in southern Florida.

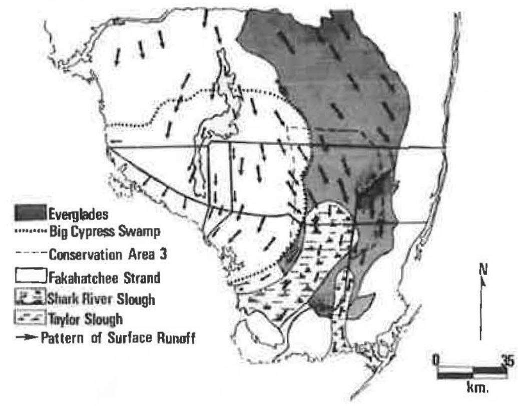


Figure 5. Collector ditch.

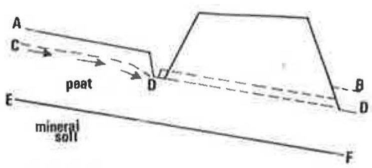
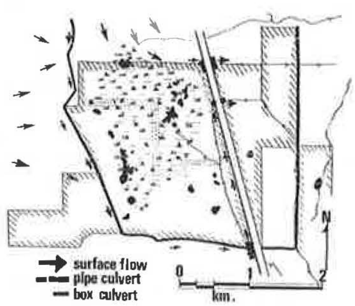


Figure 6. Ladd Marsh, Oregon.



by reducing the base grade at which groundwater discharges.

The combined effect of these hydrologic changes has been to lower the water table in the marsh during all seasons, thus changing the vegetation from wetland to upland and slowly eliminating areas of open water that are important to waterfowl and other marsh animals. Despite attempts by the Oregon Game Commission to divert runoff back into the marsh, over the past 29 years wetland area has been reduced by 15 percent.

Diffused Surface Flow

The effects of highway construction on wetlands that are dependent on diffused surface drainage are most dramatically illustrated in the Everglades and Big Cypress complexes of South Florida. The marsh and swamp systems of these two vast wetlands receive the bulk of their water from surface runoff and sheet flow over shallow aquifers. The natural flow is generally from the latitude of Lake Okeechobee on the north, south through the Everglades and Big Cypress, and exiting through the mangrove forests that border Florida Bay and the Gulf of Mexico (see Figure 7). Although highways occupy only a minute fraction of this vast area and generally do not block critical flow points, their construction has contributed to significant changes in portions of the region.

These changes stem principally from two aspects of the highway construction process. In the first instance, the presence of roadway embankments across the axis of primary flow has the potential for blocking sheet drainage, much as in the cases of the Minnesota and Michigan peat wetlands discussed earlier. In Florida, however, the bulk of the flow is above the aquifer and can often be effectively accommodated by proper culverting. Where this has been done—as in the case of FL-27 between the Everglades Park entrance and Flamingo—the drainage-related impact of the highway embankment is negligible. Where provisions for surface drainage are not adequate—as along the western portion of Alligator Alley—backwater conditions develop on the upstream side of the highway and are reflected in the drawdown of the water table on the downstream side. These hydrologic alterations inevitably produce changes in vegetation.

The effect of highways in the Everglades-Big Cypress environment is very much a function of the presence or absence of drainage canals associated with the construction of the roadway embankment. Unlike FL-27, which was constructed with borrow from sites well removed from the right-of-way, the other Florida roads in our study were constructed on fill obtained from borrow-canal spoil excavated along the right-of-way. Although these canals are used as flood-control structures at critical periods during the year, they also serve to collect sheet drainage on the upstream side of the highway and to facilitate the reestablishment of sheet flow on the downstream side. Where the combination of parallel canals and adequate culverts at all natural drainage channels is sufficient to reestablish natural drainage immediately downstream from the highway—as along the Tamiami Trail and the eastern portions of Alligator Alley—little if any ecological effect is observed.

It is important that a balance be struck between too little and too much drainage. For example, connecting the borrow canal along Alligator Alley to the south-running Barron River and Turner River Canals has accelerated flows south of the alley and directed water away from the historic pattern. Similarly, the borrow canals along FL-29 and FL-840A, which run parallel to the north-south drainage axis between Alligator Alley and the Tamiami Trail, intercept substantial amounts of surface water and divert it directly to the estuaries of Florida Bay. There is evidence that the reduction in freshwater head that has resulted from this diversion

has led to inland migration of saltwater.

The Flamingo Road demonstrates that, if highways must be constructed across sheet-flow wetlands, extensive use of culverts without borrow canals can have minimal effects. The original construction of the Tamiami Trail suggests that borrow canals in combination with culverts need not have adverse effects but that they lay the foundation for subsequent disruption of the wetland ecosystem. Alligator Alley illustrates that, even when attention is given to the effect on hydrology, modern techniques of road and borrow-canal construction need further refinement if uniform desirable results are to be obtained. FL-29 and FL-840A are examples of a straightforward effort to use a combination of canals and no culverts in conjunction with a spoil-bank base to drain a wetland, with questionable economic and adverse ecological effects.

Appropriate drainage features are obviously necessary for avoiding ecological damage in the Everglades-Big Cypress environment. But, given the destructive effects of many of the activities associated with roads such as the Tamiami Trail, it is clear that proper drainage alone will not suffice. Rigid control of access is also essential to preventing adverse impacts and further inappropriate uses of these vast wetlands. Reconstruction of Alligator Alley to Interstate standards will provide an opportunity to develop a major demonstration of compatible road construction and wetland protection.

INTERRUPTED TIDAL EXCHANGE

Interruption, by embankments and other structures, of the tidal exchange in coastal and estuarine marshes is an important, special case of altered drainage. The twice-daily ebb and flow of tidal waters is essential in maintaining the level and salinity of these salt and brackish marshes and in carrying detritus from marsh areas to the marine environments that rely on this source of nutrient. Highway drainage structures are typically designed to accommodate storm flow from upland sources. The capacity of the culvert, its invert elevation, and the erosion protection measures associated with it are characteristically based on the need to convey storm waters that flow down from upland locations. Insufficient attention is often directed to the passage of tidal waters, which, during part of their cycle, move through these structures in a direction opposite to the design flow. The result of such "unidirectional design" is often the alteration of the tidal regime within the marsh.

The ecological effects of restricted tidal exchange are illustrated by the Fairfield, Connecticut, study. Pine Creek Marsh is a 9.6-km^2 estuarine salt marsh on Long Island Sound in the town of Fairfield. A storm- and flood-control dike was constructed in 1970. The location of this dike and changes in the dike and culvert system that have been proposed in an effort to mitigate effects of tidal interruption on the marsh are shown in Figure 8.

The interruption of tidal flows by the dike has resulted in an average 23-cm reduction in tidal elevations within the marsh and a reduction of as much as 46 cm during spring tides. Groundwater levels have receded well below that required to support two common species of marsh grasses. The reduction in tide height has reduced the total marsh area that is exposed to saltwater and, in combination with lowered groundwater levels, has resulted in a shift of vegetation from salt marsh to fresh; at the landward margins of the marsh, upland species have begun to replace the wetland biota. Where upland or freshwater species have become established, the peat soils of the original salt marsh have become compacted and less permeable. As a consequence, even if current efforts to restore the historic tidal hydrology are successful, the capacity of the marsh to support salt marsh species would be reduced.

The restriction of saltwater intrusion into an estuarine system by a highway acting as a barrier to or restrainer of tidal inundation will greatly affect a wetland. Where saltwater intrusion is prevented by a highway barrier, plant populations will show slow but significant changes. Many estuarine plants actually grow well in freshwater but cannot compete successfully with freshwater species in that environment because of slow growth and lack of viable seeds. Some estuarine plants require salt for growth and will die in freshwater conditions. The estuarine species of macroscopic algae and microscopic diatoms will be replaced by freshwater species.

PHYSICAL OBLITERATION

The physical obliteration of wetland habitat by the highway embankment itself is an unavoidable consequence of that form of construction. When such loss of habitat is unacceptable—for example, when a unique wetland may be lost or rare or endangered species may be threatened—rerouting of the highway or open-pile construction may be the only alternative. However, there are many situations in which loss or alteration of habitat through physical obliteration extends well beyond the highway fill. The construction of I-95 through Tinicum Marsh in Pennsylvania is a case in point (4).

Tinicum Marsh occupies the lowlands along Darby Creek in Delaware and Philadelphia Counties in southeastern Pennsylvania. Before construction of I-95, the marsh covered about 200 km^2 between PA-291 and the Tinicum Wildlife Preserve of the city of Philadelphia (see Figure 9). Though the marsh area has been greatly disturbed and considerably reduced in size over the past 300 years, it was, and still is, an important, tidally inundated freshwater environment. No rare or endangered species consistently live or breed in Tinicum Marsh, but the habitat itself is rare, being the last remaining tidal wetland in the state of Pennsylvania.

In the initial planning phase for construction of I-95, a compromise between the Pennsylvania Department of Highways and local conservation groups in 1963 provided for the routing of the highway along the southern edge of the marsh, where it would have interfered least with tidal flows and would have obliterated the least amount of marsh habitat. This compromise, however, was not included as a restriction when construction bids were advertised in 1968. The project contractor, unencumbered by the earlier compromise, negotiated contracts with the private owners of the marshland to obtain sand and gravel lying under the marsh for roadbed fill. These contracts also obligated the contractor to fill other parts of the marsh to a level above the highest tide so that light industrial facilities, high-rise apartments, or shopping centers could be erected. Even though this filling was not a direct result of roadbed construction, the entangling contracts tied it intimately with highway construction. The location and extent of the marsh areas destroyed or altered by these related activities are shown in Figure 10.

CREATION OF NEW HABITAT

Highway construction may result in the creation as well as the destruction of wetland habitat. Habitat creation is often the unplanned result of borrow-pit excavation or the inadvertent blockage of surface or subsurface drainage by a highway embankment. Increasingly, however, provisions for the creation of new habitat are being in-

Figure 8. Proposed restoration of Pine Creek Estuary, Fairfield, Connecticut.

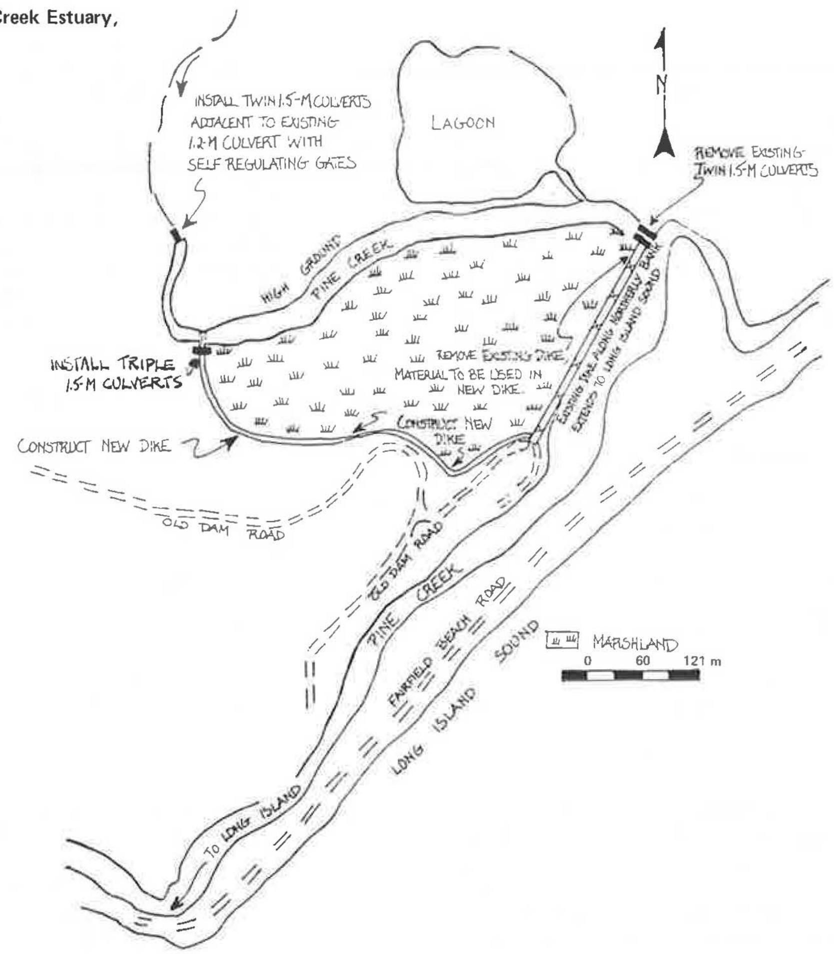


Figure 9. Pennsylvania's Tinicum tidal marshlands in 1968.

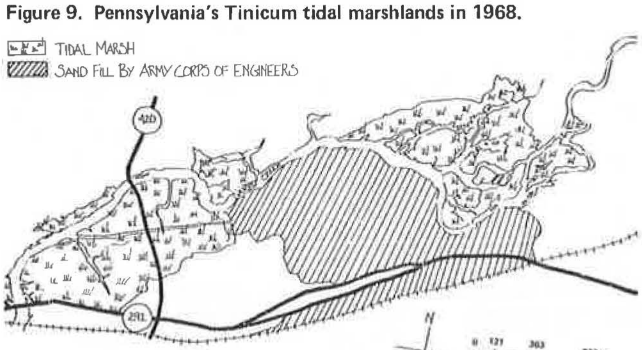
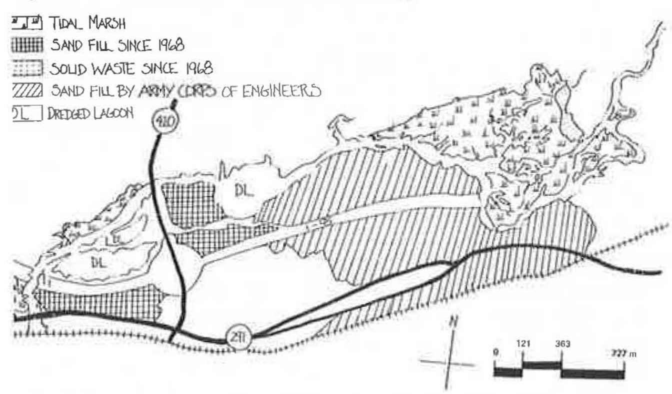


Figure 10. Tinicum tidal marshlands in 1971.



corporated explicitly in highway location and design plans as highway agencies gain experience with the advantages that can accrue from such practices to both highway users and the public at large. Early and continuing coordination between highway agencies and state natural resource or wildlife departments is essential if the full benefits from the creation of new wetland habitats are to be realized. Of the many highway features and construction activities that can be used in this capacity, three—use of ditches and culverts, construction of borrow pits, and disposal of dredge material—appear to offer the most extensive opportunities for habitat creation. The material dredged from a wetland or removed from an upland site can be used to create new wetlands or extend existing ones.

The ecological uncertainties involved in the accidental creation of wetland habitat are exemplified by a series of borrow pits excavated in 1959 and 1960 along I-91 in Whately, Massachusetts (5, 6). Figure 11 shows the location of the larger of these pits and identifies four that are the subject of the present case study. In this instance, location of the various borrow areas and the excavated configuration of each pit were dictated by the availability of appropriate borrow material and ease of excavation and haul to the construction site. Consideration was not given to the ecological potential of one location or excavation procedure relative to another.

The four borrow-pit ponds are very similar in most respects. All four ponds have predominantly granular bottoms (as would be expected, given the purpose for which they were excavated), and all four are fed mainly by groundwater supplemented to a degree by surface drainage from I-91 and the surrounding fields. The principal differences between the ponds lie in their basin configuration (a and b have greater average depths and considerably less shallow margin than do c and d), the composition of the surface flows into them, and, directly related to these two factors, their biological productivity.

Borrow-pit productivity is strongly related to nutrient availability, substrate composition, runoff characteristics, and basin morphology. Because of the nature of borrow pits, the bottom sediments are primarily sand and gravel, contributing little to wetland fertility. Upland seepage and runoff is characteristically slow and, although it does exert a major control over water chemistry, it usually does not encourage rapid eutrophication. Growth of aquatic vascular plants depends on the extent of shallow areas in the basin. Borrow-pit morphology often results in either extensive shallows or no shallows at all.

Of the four sites discussed, 91 South and 91 North exhibited the highest productivity. The 91 South pond exhibited extensive growths of algae and a dense population of cattail (Typha spp.). Both shallow water and agricultural drainage contributed to this condition. Shallow water was also a contributing factor in the higher productivity of 91 North, but the absence of agricultural drainage leads to most of the pond's energy being cycled through the benthic flora rather than through an extensive plankton population. The 91 Swim pond lacked both shallow areas and nutrient-rich upland drainage. As a result, this impoundment was more deficient in plant nutrients (oligotrophic) than 91 North and 91 South. In addition, because of its use as a swimming area, 91 Swim was highly turbid throughout the growing season, which inhibited the penetration of light in the water column. Drainage entering 91 Woods came primarily from a pine-mixed hardwoods swamp, and the resultant pH range was 4.9-3.7. This pond had very little emergent vegetation, a very small plankton population, and no fish. All of this emphasizes the importance of site to subsequent wetland characteristics.

In marked contrast to the strict highway-function orientation that characterized construction practices a decade ago, an increasing number of highway agencies are making explicit provision for creating or replacing wetland habitat in the course of highway construction. The practices of the North Dakota State Highway Department (7) and the state of Minnesota are instructive in this regard.

A number of artificial wetlands have been created by the North Dakota State Highway Department as an integral part of the construction of I-29. Borrow areas are designed specifically to create marsh habitat and include the flat slopes and shallow areas necessary for the establishment of marsh vegetation. Figure 12 shows the type of plan and basin configuration used. It is reported that, within one year after construction, marsh vegetation appeared along the periphery of the borrow area and waterfowl were observed.

In addition to making good use of the opportunities for wetland creation provided by borrow excavation, the North Dakota State Highway Department has used a substantial number of highway embankments as dams for the deliberate impoundment of surface drainage. The management of the lakes and other wetlands so created is coordinated with the North Dakota State Water Commission and State Game and Fish Department to provide the fullest possible ecological and recreational benefits from these areas.

The policies of the state of North Dakota with regard to the replacement of wetland habitats lost through highway construction are also worthy of note. A memorandum of understanding between the North Dakota State Highway Department and the U. S. Fish and Wildlife Service establishes a basis of exchange for the replacement of wetlands beyond the highway right-of-way covered by easement agreements between the U. S. Fish and Wildlife Service and private owners. Wetland filled or drained as a result of highway construction is replaced by alternative land as agreed to by both agencies. Exchange options are reviewed during the loca-

Figure 11. I-91 borrow-pit ponds, Whately, Massachusetts.

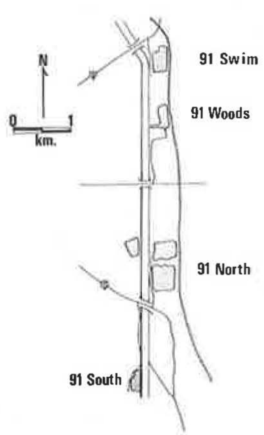
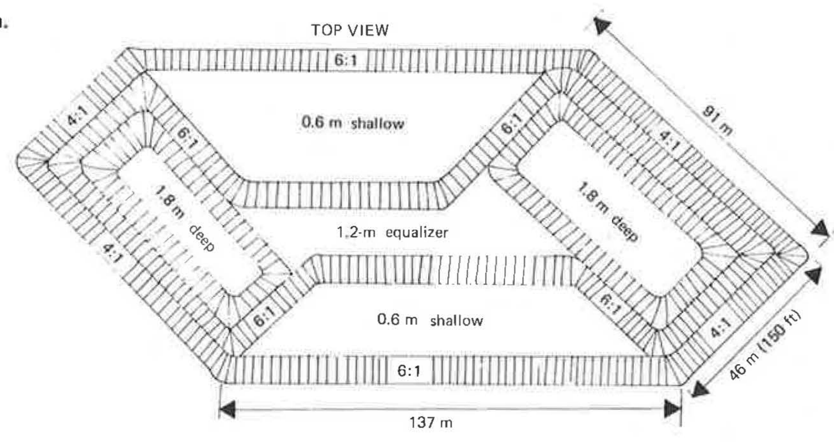


Figure 12. Basin configuration used in North Dakota.



tion and predesign phase of the project, the range of options is narrowed during the preliminary design phase, and the final choice is made in conjunction with the final project design. Twelve types of wetland for which replacement may be required under the federal-state agreement are specified, along with six replacement options. The ratio of replacement area to affected area varies depending on the type of wetland that is to be lost, the type of area with which it is to be replaced, and the biotic region in which the replacement land is located. Replacement ratios range from 0.25, when the replacement wetland is considered to be of a higher type than that to be replaced, to 8.0, when tame grassland is to replace prime wetland in another biotic region. The Minnesota Department of Transportation (DOT) has also entered into such a memorandum of understanding with the U. S. Fish and Wildlife Service and uses borrow pits and control structures to create new habitat.

CONCLUSIONS AND RECOMMENDATIONS

The effects that highway fills have on the wetlands in which they are constructed depend primarily on the extent to which the surface and subsurface hydrology of the affected wetland is disturbed. Standard design and construction practices do not appear to provide adequately for the unimpeded flow of groundwater through the fill. The result is that the water table on the upstream side of the embankment is raised, and this leads to the destruction of timber and to other ecological changes. In coastal wetlands, drainage facilities designed to accommodate surface flow from upland sources often do not adequately handle tidal ebb and flow; the level and salinity of the waters within these tidal marshes are reduced, which results in changes in marsh biota. Restriction of tidal exchange also reduces the amount of nutrients that can be exported from the marsh to other, dependent, environments.

The extent of physical obliteration of wetlands that results from embankment construction and disposal of dredge spoil can be limited by careful location and construction practices. With the assistance of ecologists and other environmental specialists, highway construction can also be designed to create new wetlands. The extent of damage or enhancement that results from highway construction is, in the final analysis, determined not so much by the nature of wetlands or by the construction process itself but by the perceptions and objectives of those responsible for location and design decisions.

Our understanding of the effects on the hydrologic regime of various highway construction activities and design features is incomplete, especially with reference to induced changes in the movement of groundwater at the local and regional scale. Our knowledge of how the ecology of specific types of wetlands will respond to a given change in the hydrologic regime is also badly deficient. It is recommended, therefore, that the following steps be taken to increase our knowledge of the effects of highway construction on local and regional hydrology and the response of various wetland ecosystems to changes in the hydrologic regime:

1. Research should be undertaken to further our knowledge of the geophysical factors that govern the movement of surface and subsurface waters at both the local and regional scale. Particular emphasis should be given to studies of groundwater movements at the regional scale.

2. Studies of local and regional hydrology, including both surface and subsurface flows, should be incorporated in the preliminary engineering studies that precede highway location and design decisions.

3. Research should be undertaken to increase our understanding of the responses of various wetland ecosystems to the changes in the hydrologic regime associated with each wetland class.

Identifying and assessing the probable effects that highway activities will have on wetlands require the application of knowledge that is in a state of active evolution. Considerable progress has been made in recent years in understanding how wetlands function and how highways and other engineered works affect those functions; considerable further progress is both necessary and feasible. For many decades highway engineers have been refining and applying their knowledge of soils, hydrology, and other elements of the geophysical environment to the construction of structurally sound and economically efficient highway facilities. It is now essential that this knowledge be more fully merged with that of biologists, ecologists, and other natural scientists so that the integrity of the environment through which a highway passes is as carefully protected as the integrity of the highway itself.

ACKNOWLEDGMENT

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The investigators on the case studies were as follows: Whately, Massachusetts, borrow-pits synthesis by Richard Newton, adapted from theses by Heusmann (5) and Moulton (6); report on Fairfield, Connecticut, and investigations of Jamestown, North Dakota [with reference to reports by Nilson (7)], and La Grande, Oregon, by Carl A. Carlozzi; Philadelphia report by Richard Clarke, adapted from reports by McCormick (4); South Florida investigation performed by Joseph Larson, with reference to unpublished data compiled by Frank C. Craighead, Sr.; Michigan investigation performed by Dale F. Cope, with assistance from the research team at the Department of Natural Resources, Michigan State University, and especially Phillip B. Davis; northeastern Minnesota investigation performed by Paul W. Shuldiner, with reference to research by Stoeckeler (3) (special thanks to Mary Quilling, who located sites and preliminary contacts). The original figures were drawn by Caren M. Caljouw.

The opinions and conclusions expressed or implied are ours and not necessarily those of the sponsors or the individual states that participate in the National Cooperative Highway Research Program.

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State Practice and Experience in the Use and Location of Truck Escape Facilities

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One phase of a study undertaken to develop warrants for the use and location of truck escape ramps is described. A questionnaire submitted by mail to state highway agencies sought information on (a) the type and number of escape facilities constructed, (b) variables considered in determining the need for escape ramps, (c) factors that affect ramp location, and (d) operational experience with escape ramps. The study results indicated that, although most ramps are located on four-lane divided and two-lane highways, they can also be found on three-lane routes, in medians, and at the end of freeway off-ramps. Only two states indicated that a rational technique was used to determine the need for ramps. Both techniques made use of accident rates. Other important factors in determining the need for escape ramps included length and percentage of grade, percentage of trucks, and conditions at the bottom of grades. Topography was cited as the primary factor in ramp location. Examples of satisfactory and unsatisfactory ramp location are described.

On long, steep highway downgrades, there is the possibility of brake failure on large commercial vehicles. In such situations trucks often accelerate uncontrollably, endangering not only the lives of truck drivers but the lives of occupants of other vehicles as well. Residences and business enterprises adjacent to or at the foot of long, steep downgrades may be damaged or destroyed by runaway vehicles. A large percentage of runaway-vehicle accidents result in fatalities.

Highway agencies in states that have roadways in rugged terrain (primarily the Appalachian region and the mountainous western states) have attempted to mitigate the problem by using various types of truck escape facilities. Until recently, there had been little formal research and development in the design and construction of truck escape ramps. Since the mid-1970s, however, there has been increasing interest in all facets of truck escape facilities.

No single type of truck escape facility has been adopted nationwide, but four general types of escape facilities can be identified (see Figure 1): (a) ascending-grade ramps, (b) horizontal-grade ramps, (c) descending-grade ramps, and (d) sandpiles.

The ascending-grade ramp is probably the most common type of escape facility now in use. In general, these ramps consist of a roadway that is composed of layers of loose gravel or uncompacted sand and ascends at a very steep grade, using the force of gravity to stop moving vehicles. The length of ascending-grade ramps tends to vary considerably, depending on percentage of grade, the type of aggregate used in the arrester bed, and the land available for ramp construction.

Horizontal-grade ramps are the newest type of truck escape ramp to be constructed. They use only the resistive force of the aggregate arrester bed to stop vehicles. Horizontal-grade ramps are primarily used where the terrain precludes the construction of other types of ramps.

Descending-grade ramps, like horizontal-grade ramps, depend entirely on the resistance of the aggregate bed to stop runaway vehicles. Because of the adverse effect of the negative grade, they are generally longer than ascending or horizontal ramps.

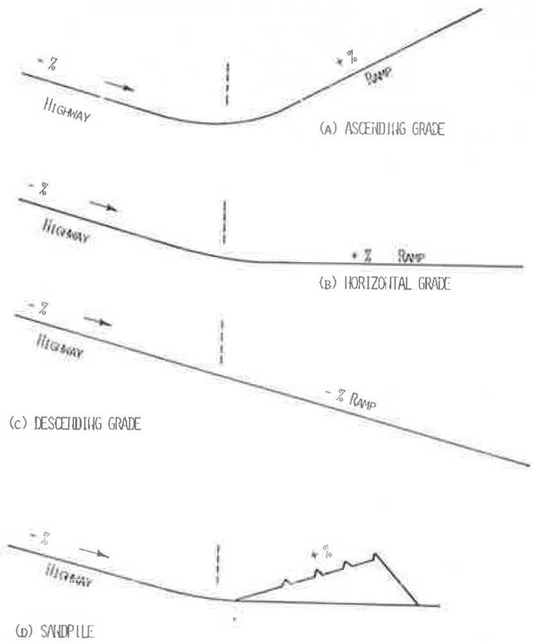
Of the four types of escape ramps, sandpiles are probably the easiest and least expensive to construct. An inclined pile of loose, dry sand provides the resistive force. The use of sandpiles is currently confined to several eastern states.

The state of the art of escape-ramp construction has advanced in recent years, but the same cannot be said of escape-ramp warrants. In most cases, the use and specific location of truck escape facilities are based on subjective judgment rather than formal engineering analysis. As resources for highway construction and maintenance become more limited, a "seat-of-the-pants" approach to locating and installing truck escape facilities is no longer justified. There is a need to develop methodologies by which optimum use and location of truck escape ramps can be determined.

The West Virginia Department of Highways, in cooperation with the Federal Highway Administration (FHWA), has sponsored a research project at West Virginia University, the overall objective of which was to develop warrants for the use and location of truck escape ramps. To accomplish this general objective, a number of detailed objectives were developed. These included

1. Use of a mail questionnaire to determine the experiences and practices of state highway agencies in re-

Figure 1. Four basic types of truck escape facilities.



lation to truck escape ramps,

2. Use of a second questionnaire to determine truck drivers' perceptions of the runaway-vehicle problem,

3. Collection of accident data for locations where there were frequent accidents involving runaway vehicles,

4. Performance of statistical analyses of the accident data to determine significant factors in runaway-vehicle accidents, and

5. Development of warrants for the use of truck escape ramps based on the collected data.

Very little published information was available on escape-ramp warrants or locational criteria, and it was felt that correspondence with state highway agencies would be a source of data and insight on this topic. This paper describes the development of the mail questionnaire for state highway agencies and discusses the results obtained. The other objectives of the project are currently being addressed in ongoing research efforts.

LITERATURE REVIEW

Present highway design criteria are not very specific about truck escape ramps. The 1974 edition of the American Association of State Highway and Transportation Officials' (AASHTO's) Highway Design and Operational Practices Related to Highway Safety (1) states only that special consideration should be given to providing escape areas for trucks on long, steep downgrades. Escape ramps are not mentioned at all in the 1965 edition of AASHTO's A Policy on Geometric Design of Rural Highways (2). The forthcoming revised edition of the AASHTO "Blue Book" will contain material on truck escape ramps, but this information will deal with the design and construction of escape facilities rather than with warrants or locational criteria. In a 1974 highway accident report (3), the National Transportation Safety Board recommended "establishment of design policy for long/steep grades that will ensure provision of escape routes when the character of the grade has a potential for contributing to the generation of runaway vehicles."

Since no national standards or policies existed for truck escape facilities, an extensive search was made of the highway engineering literature for published information on escape ramps. It was hoped that examination of case studies or data dealing with existing ramps might provide insight concerning ramp use and location. Before the mid-1970s, published information on truck escape ramps was limited to a few articles in highway engineering journals (4-7). The first comprehensive survey of truck escape ramps was that by Versteeg and Krohn (8), who noted a number of considerations that should go into the design and construction of such ramps:

1. The length of ramp depends on ramp geometry and the aggregate used.
2. The ramp should be wide enough to accommodate more than one vehicle.
3. The aggregate used for the ramp should be free draining and clean.
4. Anchors are required to secure tow trucks when they remove vehicles from the bed.
5. Surfaced road is needed, adjacent to the ramp, for use by tow trucks and maintenance vehicles.

In addition to discussing ramp characteristics, Versteeg and Krohn (8) described escape ramps used by state highway agencies throughout the country. In certain cases, locational criteria were mentioned. For example, two ascending-grade escape ramps were installed on the westbound lanes of I-80N at Emigrant Hill in Oregon. The first ramp was located approximately 3.2 km (2 miles) from the summit, the second 6.4 km (4 miles) below the summit. Experience indicated that the lower ramp was used far more (91 percent of total ramp use) than the upper ramp. This seems to indicate that drivers stay with their out-of-control vehicles as long as possible.

Brittle (9) has presented a history of sandpile escape facilities in Virginia. Along US-52 at Fancy Gap Mountain, out-of-control trucks were being driven into maintenance stone stockpiles along the 7 percent, 6.4-km (4-mile) long downgrade. It was suggested that similar stockpiles be placed at strategic locations to provide an escape mechanism for runaway vehicles. Since the downgrade lane was on the outside of the mountain, sandpiles could only be placed where there was sufficient space. Ascending-grade escape ramps were considered, but out-of-control vehicles would have to cross opposing traffic to enter the ramps.

Williams of the Tennessee Department of Transportation (DOT) is currently preparing a state-of-the-art survey of escape-ramp design, construction, and operation. In field visits, Williams has been able to obtain detailed information on most escape ramps in the United States. When his report is published, it will be an important reference for highway engineers involved with truck escape facilities.

The literature review on truck escape facilities showed that, although advances have been made in ramp design and construction in recent years, there are still no criteria for determining the need for escape ramps, and ramp location is usually based on finding a convenient site that will minimize earthwork and construction costs. Apparently no attempt has been made to develop a rational cost-benefit procedure for locating escape ramps. For these reasons, a survey was made of state highway engineers to determine the factors that are currently being considered in ramp installation.

DEVELOPMENT OF QUESTIONNAIRE

Shortly after the decision was made to use a mail questionnaire to obtain information from state highway agencies, Williams of the Tennessee DOT distributed his

own brief questionnaire on the use of vehicle arrester beds. In this questionnaire, highway departments were asked (a) whether emergency escape ramps were used in a given state, (b) if there was sufficient operational experience to evaluate the effectiveness of the ramps, and (c) if design plans or reports were available on existing ramps. The main purpose of Williams' questionnaire was to lay the groundwork for future field visits. To avoid duplication of effort, I obtained the results of the Tennessee study and compared them with my anticipated needs before developing the questionnaire described in this paper.

Response to the Tennessee questionnaire indicated that 23 highway agencies in the United States had constructed or were planning to construct truck escape ramps. States that have or plan to build truck escape ramps are given below:

State	No. of Ramps	State	No. of Ramps
Alabama	1	Pennsylvania	9
Alaska	2	Puerto Rico	4
California	3	South Dakota	2
Colorado	3	Tennessee	1
Hawaii	4	Texas	2
Idaho	6	Utah	2
Kentucky	1	Vermont	1
Montana	2	Virginia	10
Nevada	1	Washington	1
New York	2	West Virginia	2
North Carolina	3	Wyoming	3
Oregon	5		

To reduce the number of questionnaires to be completed by highway agency engineers, it was decided to send questionnaires only to these 23 states. In an effort to minimize internal transmittal of the questionnaire within state agencies and ensure that the appropriate person in each agency received it, cover letters were addressed to the persons who had responded to the Tennessee DOT questionnaire.

The questionnaire sought information on (a) the type and number of escape facilities constructed; (b) how the need for escape ramps is determined, i.e., what variables are considered in determining whether an escape facility should be installed; (c) factors that enter into the decision on where to locate such facilities; (d) operational and accident experience with the ramps; and (e) techniques for monitoring use of the ramps.

QUESTIONNAIRE RESULTS

Responses were received from 19 of the 23 state highway agencies that received the questionnaire. In addition to returning the questionnaire, many states sent accident records, unpublished reports, photographs, and plans and specifications for existing or planned truck escape ramps. The results obtained from the completed questionnaires are discussed below.

Types of Ramps

Engineers of the various state highway agencies were asked to list the types of escape facilities used and the number of each type that had been installed. These results are given below:

Type of Escape Facility	Number Existing and Planned in United States	Number of States Using
Gravel bed	11	5
Ramp	39	13
Sandpile	11	3
Total	61	19

Since the questionnaire did not list specific types of facilities, a wide variety of responses was expected (it was felt that listing types of escape ramps on the questionnaire form might create confusion in that the type of ramp visualized by the investigator might differ from that visualized by the respondent). However, only the three types of escape facilities given above were listed by respondents. A total of 61 escape facilities had been built or were planned in the 19 states that responded. As the third column in the table indicates, some states use more than one type of ramp. Since different terminology is used in different regions of the United States, gravel beds and ramps probably refer to the same type of facility. Sandpile ramps appear to be used exclusively in the East (in North Carolina, Pennsylvania, and Virginia).

When asked about typical sections and profiles for escape facilities, most respondents referred to plans and drawings, which they included with the questionnaire. No new or unique approaches to ramp design and construction were found in the questionnaire results that had not been found in the literature review. The state-of-the-art report by Versteeg and Krohn (8) gives detailed information on ramp cross sections and profiles.

Highway agencies were asked to indicate whether escape facilities were on two-lane roads or divided highways. Forty-two of the 61 escape ramps were classified according to type of roadway. However, Pennsylvania and Virginia, which have the largest number of escape ramps (9 and 10 ramps, respectively) did not give detailed breakdowns. Out of 42 ramps, 26, or 62 percent, were located on divided highways; 15, or 36 percent, were located on two-lane roads; and 1 was on a three-lane undivided highway. Both Pennsylvania and Virginia indicated that they had ramps on two-lane and four-lane divided highways, and Virginia noted having ramps on three-lane undivided routes. But the exact number of ramps in each of these categories is not known.

From the data, it is not possible to determine the reason for the large number of ramps on four-lane divided highways. Four-lane divided highways would normally be expected to have better geometrics than two- or three-lane roadways. However, higher speeds and greater traffic volumes may increase the probability of runaway-vehicle accidents. There is also the possibility that the four-lane divided highways were built recently and it may have been convenient and economical to include escape ramps at the construction stage.

Ramp Warrants

In response to a question as to whether or not a rational technique was used in determining the need for escape facilities, only 2 of the 19 responding states, or 10.5 percent, said yes. Colorado indicated that a three-year study was made of accident history at problem locations. Oregon stated that the traffic engineering branch maintained current accident records. When high-accident locations were identified, a detailed study was made to identify causes and potential solutions.

Those states that do not have a rational technique for determining the need for escape facilities were asked to list the variables they considered in determining whether an escape facility should be installed. Figure 2 shows the factors cited by state highway agencies and the number of agencies that cited them. Several states noted certain factors as being more important than others, but lack of additional data made it difficult to assign a numerical weight to the importance of each factor. Each factor listed by the states was thus given equal weight in formulating the plot shown in the figure.

Several states added comments that might be useful

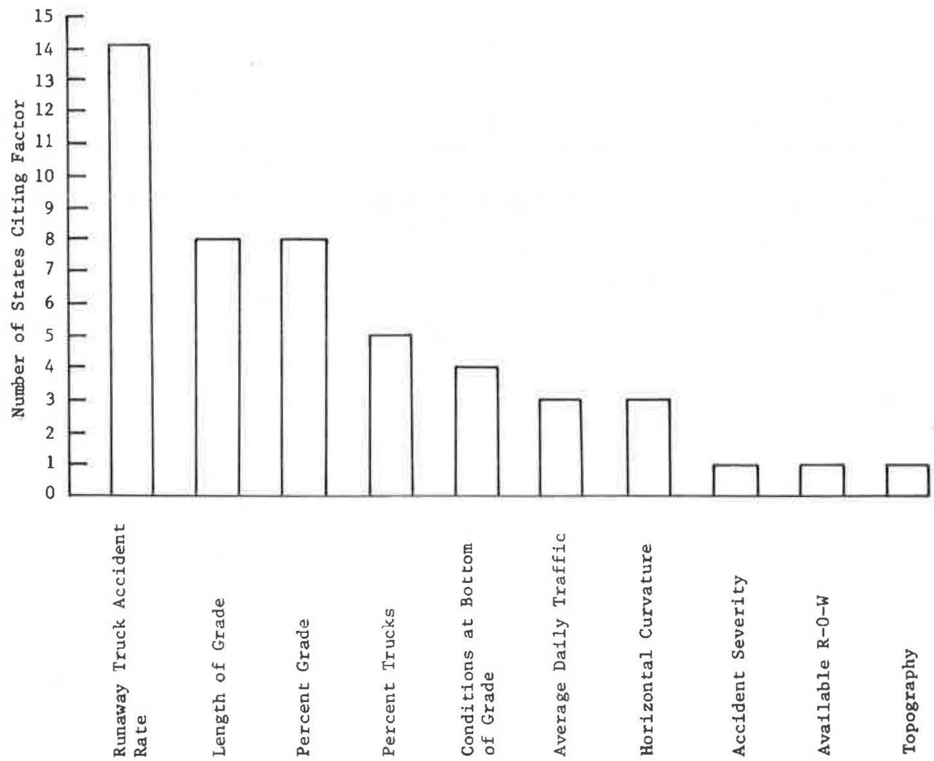


Figure 2. Factors considered by state highway agencies in determining the need for truck escape facilities.

to engineers who deal with escape ramps. One state highway agency noted that percentage and length of grade should be considered in designing the length and grade of the escape ramp. It was also felt that the horizontal alignment of the roadway should be considered in designing the width and alignment of the escape ramp. One engineer noted three criteria that should be used to determine whether an escape facility should be installed:

1. Is there a problem with runaway trucks?
2. Can the problem be corrected by signing or delineation?
3. If the problem cannot be corrected by signing, where should the facility be built to best fit the conditions?

It was noted that escape ramps were built so infrequently that each case was considered on its own merits.

In still another instance, an agency noted that, although no rational criteria existed, the two escape ramps in the state were both installed at high-accident locations where there was a long downgrade leading to a T-intersection in the center of a small community. The respondent stated that both projects were initiated to solve a demonstrated problem rather than to analytically appraise a design.

Among other states that discussed the problem of determining the need for escape ramps, one southern state listed six factors as important and then stated that "all are important and are used in determining a need. A definite problem exists when there is a combination of long, steep grade and long, continuously curving alignment. In hot weather, most all trucks will exhibit some type of brake problems on this type of highway." On the other hand, one engineer noted that "truck escape ramps are constructed based upon the incidence of truck accidents downgrade on any hill if the conditions are applicable for their use. Percent grade, length of grade, or horizontal alignment are not in themselves criteria for the construction of a truck escape ramp."

Ramp Location

Engineers were asked to specify factors that enter into the decision on where to locate an escape facility once it is determined that one is needed on a particular downgrade. Figure 3 shows the factors cited and the frequency at which states cited them. The most frequently mentioned factor was topography because of its obvious effect on construction costs. Horizontal alignment and accident location were the second most frequently cited factors. Several respondents noted that ramps should be located upgrade from sharp horizontal curvature, since runaway trucks would not be able to negotiate these curves at high speeds. Since runaway-vehicle accidents are usually run-off-the-road accidents caused by failure to negotiate horizontal alignment, accident location is probably closely related to horizontal alignment. Right-of-way availability, truck driver input, speeds of out-of-control vehicles, and length of grade were each cited once.

Several states cited specific examples of ramp locations. Some of the more important ones are discussed here.

Hawaii stated that the preferred location was near the end of a downhill tangent where there was a curve to the left and that the ramp grade was at least 15 percent uphill. Because Hawaii does not have a readily available source of rounded aggregate for the arrester bed, it is very important that the escape ramp have an uphill grade.

Idaho attempted to place its first escape ramp on a given grade about 0.8-1.2 km (0.5-0.75 mile) from the summit of the grade. The second ramp was then located approximately 3.2 km (2 miles) downgrade from the first ramp. The suitability of the topography usually dictated the exact location. Oregon indicated that 6.4 km (4 miles) from the summit of a grade was the most desirable location. New York attempted to locate escape facilities as close as possible to the bottom of downgrades (assuming that poor horizontal alignment did not force the truck off the road prior to this point), whereas

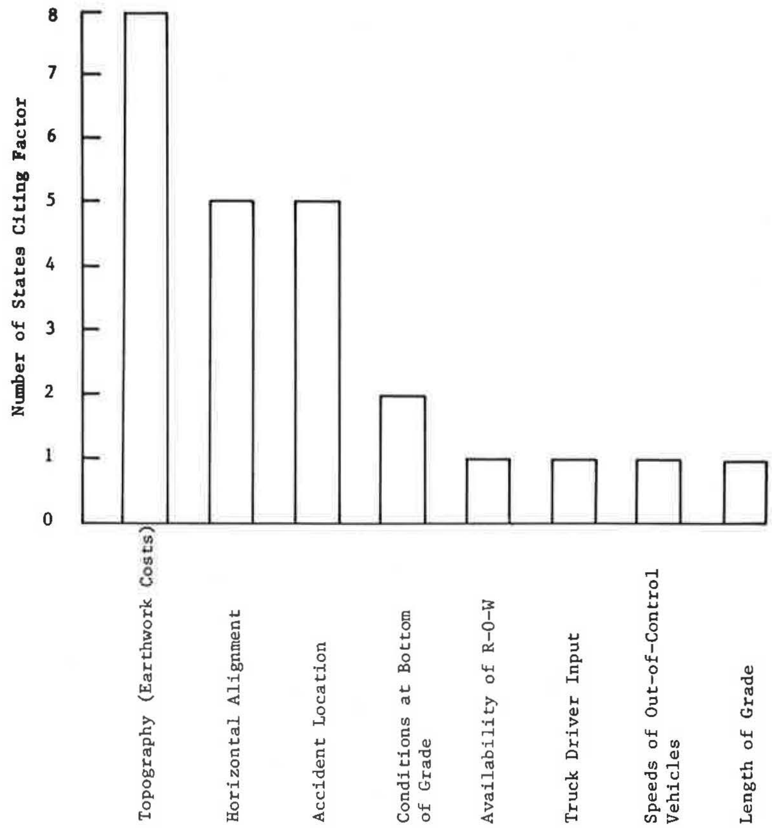


Figure 3. Factors considered by state highway agencies in determining the location of truck escape facilities.

Colorado noted that each site required an individual study based on grade, alignment, and accident reports.

Respondents were asked to indicate the number of escape facilities in each of six specified locations. The results are given below:

Location of Escape Facility	Number of Facilities
On tangent	25
On curve	29
In cut	29
On fill	15
In median	1
Left side of highway	1
End of off-ramp	2

An unknown number of ramps were double counted; for example, a ramp located in a cut section on a tangent would receive two citations.

The greatest number of ramps were located on curves and/or in cuts. Ramps on tangents were also commonly used. Although ramps on tangents may be more desirable from the viewpoint of the driver, the widespread use of ramps on curves is not surprising. A large percentage of highway kilometers in mountainous terrain consists of horizontal curvature. In addition, the fact that a roadway is in a cut makes it easier to achieve the desired uphill gradient for the escape ramp.

Fifteen escape facilities were located on fills, and many of these were sandpile ramps. It will be recalled that Virginia developed sandpile ramps for use in locations where inadequate space existed for a typical escape ramp. Thus, a common location for sandpile ramps is on small fills where the downgrade lane or lanes are on the outside of the mountain.

Two states, California and Virginia, have escape facilities at the end of freeway off-ramps. California also has a ramp in the median. An escape ramp at Parley's Canyon in Utah is also located in a median but, since Utah did not return the questionnaire, this facility is not included in the results given in the table above.

It is interesting to note that, based on the results of the questionnaire, there is only one escape facility in the United States that is located on the left side of a two-way highway: Wyoming constructed a left-hand ramp on a low-volume route. The reluctance of state highway agencies to use left-hand ramps is probably attributable to serious questions about liability (should a runaway truck heading for an escape ramp strike an oncoming vehicle), signing problems, and the reaction of truck drivers to this type of ramp.

Operational Experience with Escape Ramps

States were asked whether any of the six suggested ramp locations were unsatisfactory in terms of operational or accident experience. Eight states indicated that there were no operational or accident problems with the ramps. Five states did not respond to the question, which could be interpreted to mean that they did not have sufficient data to reach a conclusion. Six states noted problems with their escape facilities. These difficulties are discussed here in the hope that similar problems can be avoided in the future.

Idaho, North Carolina, and Pennsylvania cited problems with aggregate arrester beds. In Idaho, one ascending-grade ramp was surfaced with compacted gravel. After stopping on this ramp, trucks rolled backwards and jackknifed. Pennsylvania also stated that there is a need for suitable arrester beds on ascending-grade ramps to prevent trucks from rolling backwards. Another problem experienced in both Pennsylvania and North Carolina was freezing of arrester

beds or sandpiles. North Carolina experienced problems with compaction of sandpiles. These are essentially construction and maintenance, rather than locational, problems in that they would occur whether the ramp was located on a fill, in a cut, on a tangent, or on a curve.

Several states mentioned operational problems with escape facilities. Idaho indicated that one ramp had a problem with poor visibility. At one Tennessee location, where a ramp exited the roadway at the right-hand side and the main line curved to the left, some vehicles at night mistook the escape ramp for the through roadway.

North Carolina noted that terrain conditions have limited the length of sandpiles. The piles work satisfactorily for trucks traveling slower than 129 km/h (80 miles/h), but they would have to be longer to bring a truck traveling faster than 129 km/h to a complete stop. Tennessee also stated that some vehicles have penetrated barriers at the end of ramps.

New York reported that, since an escape ramp was installed in 1964 on NY-10 at Richmondville, four trucks have lost control and entered the community. Two trucks have overturned on a lawn near the center of the village, and two trucks have gone through a T-intersection. To eliminate this situation, NY-10 will be relocated. Although New York furnished the plans for this escape ramp, no geometric data—such as location of summit or percentage or length of grade—were available. However, since the escape facility is located approximately 1.9 km (1.2 miles) upgrade from the town of Richmondville, it is probable that drivers of out-of-control vehicles bypass the escape ramp in an attempt to "ride out" the grade.

States were asked whether use of escape ramps was monitored in a formal manner. If a formal monitoring program was in effect, the states were asked to describe the reporting procedure. Fifteen of the 19 responding states indicated that they did not formally monitor the use of escape ramps. Three states did have a formal monitoring procedure, and one state did not respond to the question.

North Carolina reported using three different monitoring mechanisms: The highway patrol completed accident report forms for each incident, a traffic services technician prepared special reports on ramp use, and at one ramp there was surveillance by a vehicle-actuated time-lapse camera. In Tennessee, accident reports were filled out by state police, and special forms were completed by maintenance foremen. Colorado indicated only that ramp use was monitored by state police.

Although they indicated no formal monitoring program, several states described procedures that they used to gain information about the use of escape ramps. California noted that the escape ramp on old CA-99 was monitored for 18 months by a radar unit to determine entry speeds. The maximum speed recorded for a ramp entry was 113 km/h (70 miles/h). Speeds claimed by drivers were consistently 8-32 km/h (5-20 miles/h) higher than those recorded by the radar unit.

Oregon's monitoring program consisted of observation by maintenance crews on routine field duties and by state police. Oregon also noted that, since most trucks were equipped with citizens band radios, truck drivers contacted tow trucks on their own. Wyoming stated that ramp use was indicated by a broken wire and tire marks; no further explanation was provided.

States that indicated that they did not formally monitor escape ramps were asked to state whether they felt there was significant unreported use of the ramps. Of the 15 states that did not monitor ramp use, 7 stated that there was not significant unreported use of the ramps, 2 indicated that there was significant unreported use, 3 did not know, and 3 others did not respond.

The final question on the form asked engineers of state highway agencies whether before-and-after accident data were available for escape facilities. Seven states have collected such data: Colorado, North Carolina, Pennsylvania, Tennessee, Texas, Virginia, and Washington. Ten states had no data available, and two states did not respond to the question. Colorado, North Carolina, Virginia, and Washington sent accident reports and accident statistics to the investigator. Some of this information will be used in later stages of the research on warrant development. Pennsylvania did not furnish specific data but stated that accident experience indicated a reduction in the number of downgrade truck accidents in most cases. It was noted that some truck drivers still show ignorance of the escape ramp as a safety device.

SUMMARY AND CONCLUSIONS

Based on the results of a questionnaire regarding the practices and experiences of state highway agencies with truck escape ramps, a number of conclusions can be drawn. Escape ramps are felt to be effective as an accident countermeasure and are becoming more common on long, steep downgrades. Several types of ramps are used, the specific type for a given application depending primarily on site conditions. Most ramps have been constructed on four-lane highways. Two-lane highways ranked second in terms of frequency. Escape ramps have also been built on three-lane undivided routes, in medians, and at the end of freeway off-ramps.

Only two of the states that responded to the questionnaire stated that they had a rational technique for determining the need for an escape facility. Both used accident rates to detect problem locations. Among those states that did not use a rational criterion, accident rate was also the most frequently mentioned factor. Other important factors appeared to be length and percentage of grade, percentage of trucks, and conditions at the bottom of grades.

Responding states indicated that topography was the primary factor in determining ramp location because of the direct relationship between topography and earthwork costs. Horizontal alignment and accident location were also cited frequently by states as factors considered in locating escape ramps. Almost as many ramps have been built on tangents as on curve sections, but nearly twice as many ramps are in cuts as on fills. The widespread use of ramps in cuts is attributable to more favorable conditions for ramp grade and difficulties in constructing ramps on the outside of mountains.

The questionnaire results tend to confirm the fact that most escape ramps are installed on the basis of judgment rather than formal engineering analyses. Responses indicate that, as problems are encountered with specific ramp designs or locations, modifications are made and thus the state of the art is advanced.

In view of the various procedures used to determine the need for and location of escape ramps and the variation in experience with ramps, there is a need to develop methodologies by which optimum use and location of truck escape ramps can be determined. Elements of risk and economic costs and benefits should be included in any such methodology. The fact that several states have already collected before-and-after accident data on long, steep downgrades will help in the development and verification of rational criteria for escape ramps.

Based on the results of the questionnaire discussed in this paper, several additional recommendations can be made. There appears to be wide variation among states in the signing of escape facilities. Sandpile, escape ramp, runaway truck ramp, and other terms may be used to refer to essentially similar facilities. Some

states indicate the type of ramp surfacing on advance signs whereas others do not. Such variation may create uncertainty on the part of truck drivers when they are faced with using escape ramps. It is recommended that more uniform signing policies be developed.

Although there is only a limited amount of information on truck escape ramps in the technical literature, it is apparent from the results of the questionnaire that many states have conducted research on the topic. Much of this work involves selection and testing of arrester-bed aggregate and construction and maintenance policies. There is a need for better dissemination of information on studies that deal with truck escape ramps. Personnel of state highway agencies should report on the results of their research and development activities so that duplication of effort can be avoided.

ACKNOWLEDGMENT

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Performance of a Gravel-Bed Truck-Arrester System

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The testing of a truck-arrester system that consists of a 158-m (528-ft) long bed of 0.6-m (2-ft) deep screened gravel, backed up by an array of 88 sand-filled plastic barrels, is described. The system was constructed on NY-28 east of Utica, New York, on a steep downgrade where geometric restrictions precluded building a conventional uphill escape lane. Three trial runs with a 16 650-kg (37 000-lb) dump truck, at speeds of 34, 66, and 90 km/h (21, 41, and 56 miles/h) demonstrated the ability of the gravel bed to stop runaway vehicles. The decelerations experienced in these tests were similar to those experienced in panic stops on dry pavement.

Truck escape lanes are constructed so that runaway trucks descending long, steep grades can stop safely. These lanes, which sometimes use uphill ramps to decelerate trucks, may also contain loose sand or gravel to increase deceleration by imparting drag forces to the wheels of the vehicle (1). A device with steel nets and cables was once designed for an installation in Puerto Rico but was apparently never constructed.

A long history of runaway trucks led to the construction of an escape lane on the downslope of what is locally known as Vickerman Hill on NY-28 near the village of Mohawk, New York, 16 km (10 miles) east of Utica, under a New York State Department of Transportation (NYSDOT) contract. Selection of a design for this escape lane was complicated by site geometry. The village is located in a valley, and NY-28 descends on a long downgrade. Just south of the village limits, at the site of the escape lane, the downgrade is 10 percent. Because the highway is in a sidehill cut with the downhill lane on the fill side, an uphill ramp would require placement of excessively high fill. Thus, another design approach was necessary. A steel-net system was considered, but the idea was abandoned because of potential maintenance difficulty and a lack of data on the performance of such a system.

The design finally selected consists of two stages: a gravel arrester bed and an array of sand-filled plastic drums (see Figure 1). The 158-m (528-ft) long gravel bed—5.4 m (18 ft) wide at the entry point and tapering to 3.6 m (12 ft) near the end—consists of screened, rounded pea gravel (see Figure 2). The depth of the gravel increases from 0 to 0.6 m (0-2 ft) in the first 15

Figure 1. Plan and profile of arrester system (not to scale).

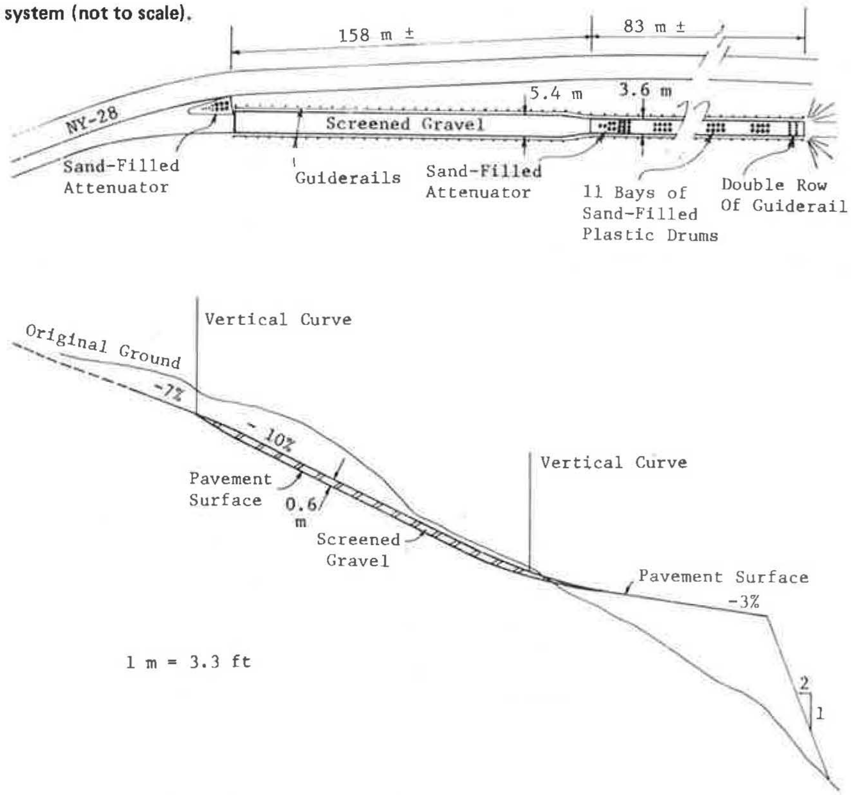

Figure 4. Approach view of arrester system showing impact attenuator to left of entrance.

m (50 ft), and the 0.6-m depth is maintained throughout most of the length, tapering back to 0 m at the other end. The approach pavement, composed of asphalt concrete, continues beneath the gravel for the entire length of the bed. The gradation of this material, sampled at 15-m (50-ft) intervals along the bed, is as follows (1 mm = 0.039 in):

	Percentage Passing		
Sample No.	25.4-mm Sieve	12.7-mm Sieve	6.3-mm Sieve
1	100.0	97.5	3.9
2	100.0	98.7	3.4
3	100.0	99.4	11.6
4	100.0	99.5	9.9
5	100.0	98.2	1.8
6	100.0	99.2	2.0
7	100.0	99.4	1.0
8	100.0	99.0	1.2
9	100.0	98.5	1.3
10	100.0	98.5	1.2

Earlier research (2, 3) had verified the concept of stopping heavy trucks by using gravel beds and indicated that a deceleration of about 0.3 *g* would be developed by this material. Trucks entering at the design speed of 128 km/h (80 miles/h) would thus exit from the 158-m (528-ft) long gravel bed at about 72 km/h (45 miles/h). Although the ramp is intended for a maximum truck weight of 36 000 kg (80 000 lb), no weight factor was considered because these earlier studies showed the deceleration rate caused by the loose gravel to be relatively independent of vehicle weight.

The second stage consists of 88 sand-filled plastic drums arranged in 11 consecutive bays (each 4 barrels long and 2 abreast), which is intended to stop large trucks that have continued through the gravel bed. These barrels are placed on 46-cm (18-in) high corrugated-metal-pipe pedestals to match their centers of mass with those of large trucks. Heavy-post corrugated-beam guiderail is installed in two separate rows at the end of the barrels to stop any vehicles that might penetrate the two stages (see Figure 3).

An installation of heavy guiderail extends the entire length of both stages to contain vehicles within the gravel bed, provide protection against jackknifing, and ensure that both rows of sand barrels are struck simultaneously. Heavy steel posts 3.6 m (12 ft) long (W6x8.5) were driven 2.1 m (7 ft) into the ground at 0.95-m (3.125-ft) centers. Three rows of 10-gage corrugated steel beams were bolted to the posts to a height of 1.5 m (5 ft).

Conventional impact attenuators designed for automobiles and light trucks are used at two locations. The first protects motorists in passenger vehicles from the guiderail ends at the entrance to the installation (see Figure 4), and the second, placed just beyond the gravel bed, ensures that any automobiles or light trucks that run through the gravel will be stopped before they strike the raised barrels.

Extensive signing is used along the route to alert drivers to the device. To minimize the chances of brake

Figure 2. Screened gravel used in arrester bed.

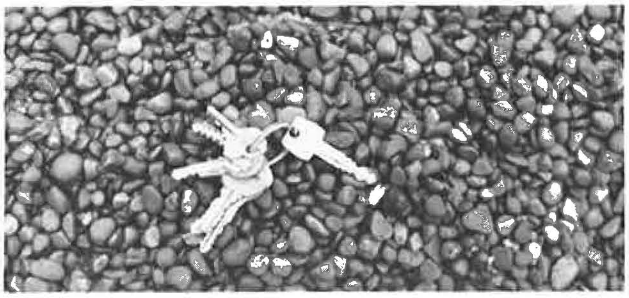

Figure 3. View of arrester system in escape lane showing heavy-post guiderail and sand barrels.

failure and ensure that the transmission is in low gear, trucks are required to stop at a specially constructed turnout before descending the grade.

TEST PROCEDURES

Because truck escape lanes are rarely encountered in New York State, and because of the unique design of this installation, three full-scale tests were conducted to demonstrate its function to potential users and to obtain data to refine future designs. Local officials and representatives of the news media were present to witness the third, highest-speed test.

A two-axle, 3.8-m^3 (5-yd^3) dump truck with dual rear wheels was loaded with sand to a test weight of 16 650 kg (37 000 lb) and driven into the arrester bed at speeds of 34, 66, and 90 km/h (21, 41, and 56 miles/h). The truck was a 1968 International Fleetstar, model 2110, with a 427-cm (168-in) wheelbase, 11.00-22 size tires, and a 213-cm (84-in) track width.

Because the performance of sand-barrel attenuators had been established, no attempt was made to test the system at speeds high enough to go completely through the arrester bed and strike the barrels. The benefits of such high-speed impacts would not justify the potential high cost of damage to the vehicle and the sand barrels or the risk of personal injury to the driver.

Several steps were taken to ensure the driver's safety: A shoulder harness and seat belt were installed, and padding was added to the interior of the vehicle in several places. All unnecessary control knobs, levers, and other protrusions were removed from the cab, and the dump body was welded to the frame to prevent movement in case of a very fast stop or rollover. Finally, the driver wore a safety helmet during each test.

The three tests were conducted during late December 1977 and early January 1978. Subfreezing temperatures and the presence of snow and ice from winter storms had resulted in a frozen crust of undetermined depth on the gravel bed before the first tests. Deicing chemicals were applied to the bed one week before the first two tests, which left the gravel thawed but wet for the three runs.

A radar unit located behind the first group of sand barrels measured the speed of the truck as it entered the arrester system. Penetration into the arrester bed was measured after each test. The principal data sources were two high-speed 16-mm movie cameras, one mounted in the vehicle's path behind the first group of sand barrels and another mounted 45 m (150 ft) normal to the vehicle's path 6 m (20 ft) upstream from the gravel bed. References for time-displacement data were targets mounted on the truck roof, range poles placed at 1.5-m (5-ft) intervals along the ramp, and a time reference of $\frac{1}{120}$ s printed on the film by a lamp inside each camera. Film speed varied through each run as the cameras built up speed. Using 30 m (100 ft) of film at speeds between 800 and 1000 frames/s, the cameras filmed for about 5 s. The film of each run was projected on a screen, and a time-distance chart was developed. Corrections for angular displacement of the cameras were applied. Because resolution of details was difficult from the film, distance could not be measured precisely over small time periods (50 ms). In addition, the timing light malfunctioned on the 90-km/h (56-mile/h) run, so only an approximate time-distance chart could be developed by matching the film speed to the two previous runs. For the two slower runs, speed and position were determined from the chart at 0.5-s intervals, but in both cases film ran out before the truck stopped. Average deceleration during the final portion of the run was thus computed from the measured stopping distance and velocity when the film ran out, as measured from the chart.

The time reference computed for the 90-km/h (56-mile/h) run was somewhat inconsistent. This is attributed to slower performance of the cameras in cold weather at the time of the test. A regression curve was fitted to the time-distance data, but its accuracy appears questionable. Thus, decelerations for this run were computed from the measured stopping distance.

DISCUSSION OF RESULTS

Run 1

At a speed of 34 km/h (21 miles/h), the truck entered the gravel bed with the transmission in neutral and the clutch engaged. Brakes were not applied during the run, and the truck was free wheeling. It followed a straight path with no observed yaw, pitch, or roll. The driver provided no steering other than a firm grip on the steering wheel and reported complete control. The penetration of the truck into the gravel bed, which was measured between the beginning of the gravel bed and the front bumper of the truck when stopped, was 24 m (81 ft). Computed average deceleration for this stopping distance was 0.18 *g*. Vehicle position, speed, and deceleration data at 0.5-s intervals were as follows:

Time (s)	Accumulated Distance (m)	Velocity (km/h)	Avg Acceleration (*g*)
0.00	0	34	—
0.50	4.5	38	+0.3
1.00	10	42	+0.2

Figure 5. Truck tires embedded in gravel after first test run.

Figure 6. Final position of truck after run at 90 km/h (56 miles/h).

Time (s)	Accumulated Distance (m)	Velocity (km/h)	Avg Acceleration (*g*)
1.50	16	34	-0.5
2.00	20	21	-0.7
2.45	22	18	-0.3
3.62	24	0	-0.4

Each average acceleration value was computed over the preceding time interval. At the 2.45-s point, film ran out; subsequent data were calculated from the measured stopping distance.

The largest recorded deceleration—0.7 *g*—occurred during the time interval of 1.50-2.00 s. The film indicated a slight acceleration as the truck first entered the gravel. Deceleration did not begin until about 1 s after the truck entered the arrester bed, in this case 10 m (34 ft) into the gravel. A front-end loader pulled the truck out of the gravel after the test. At first, the tow vehicle was unable to move the truck because the wheels were partially buried in the gravel (see Figure 5). But, after the gravel was raked smooth around the truck wheels, the truck was pulled from the bed by the front-end loader, the truck assisting under its own power. The test vehicle was undamaged by the test and by subsequent removal from the bed.

Run 2

At a speed of 66 km/h (41 miles/h), the truck again entered the bed with transmission in neutral, clutch engaged, and no brakes applied. The truck again ran straight without any steering control other than the driver's firm hold on the steering wheel. Stopping distance between the beginning of the gravel and the front bumper was 53 m (177 ft), resulting in a computed average deceleration of 0.32 *g*. Vehicle position, speed, and deceleration data were as follows:

Time (s)	Accumulated Distance (m)	Velocity (km/h)	Avg Acceleration (*g*)
0.00	0	66	–
0.50	9	67	0.0
1.00	18	67	0.0
1.50	27	59	-0.4
2.00	35	53	-0.4
2.20	38	40	-0.6
4.55	53	0	-0.6

Again, each average acceleration value was computed over the preceding time interval. Film ran out at 2.20 s, and subsequent data were calculated from the measured stopping distance.

As in the 34-km/h (21-mile/h) run, deceleration did not begin until the truck was about 1 s, or 18 m (61 ft), into the gravel. Beyond that point, a more or less uniform deceleration of 0.4-0.6 *g* was experienced until the vehicle stopped. The truck tires were embedded about 30 cm (12 in) deep and were shoveled clear before the truck was pulled from the gravel bed by the front-end loader. Some gravel that had lodged in the truck brake drums was removed before the final test; otherwise, the truck was again undamaged.

Run 3

In the 90-km/h (56-mile/h) test, the truck entered the arrester bed with the transmission in neutral, the clutch engaged, and using no brakes. As the truck approached the gravel bed, a pedestrian ran in front of the vehicle, causing the driver to alter his course slightly. Consequently, the truck entered the gravel bed at a slight angle to the centerline. This was the only one of the three tests in which steering input was required to control the truck. As it traversed the gravel bed, a cyclical pitching motion and some yaw developed. Although the driver reported that considerable effort was required, he was able to control the truck without contacting the side barriers. The pitching motion was apparently initiated by the buildup and subsequent vaulting of gravel in front of each wheel. The final position of the truck is shown in Figure 6.

Because a reliable time reference could not be established for this run, as explained earlier, only average deceleration over the entire length is reported: 0.35 *g* over the 90-m (300-ft) run. Although examination of these time-distance data does not permit precise determination of speed and acceleration for 0.5-s intervals, this run appears to be similar to the other two in both respects. Again, deceleration did not begin until the truck was some distance into the gravel, and the maximum decelerations over 0.5-s intervals appear to be similar to those experienced in the first two runs. After this third test, the truck was again pulled out of the arrester bed by the front-end loader and found to be undamaged.

DISCUSSION OF RESULTS

Analysis of the data films, observations of the test runs, and the driver's reports all indicate that this truck-arrester system can safely stop heavily loaded vehicles traveling at highway speeds. The average decelerations

measured in these tests over the entire stopping distances are similar to those reported by Jehu and Laker (2) but, because the truck did not begin to decelerate until it was some distance into the arrester bed, average decelerations over 0.5-s intervals were two to three times the overall average. All decelerations were well below the level likely to cause bodily injury, but the controlling factor for deceleration in this design was the prevention of load shift or fifth-wheel failure, for which no established criteria could be found. The maximum deceleration observed over a 0.5-s interval—0.7 *g*—is similar to that produced by hard braking on dry pavement. Therefore, it is concluded that the decelerations experienced in entering this arrester bed are no more critical than those experienced in a panic stop on a dry pavement.

Control of the vehicle presented no problem during the lower-speed runs, and with some difficulty the driver was able to control the vehicle on the 90-km/h (56-mile/h) run. The reaction of single-unit vehicles at higher speeds could be expected to be somewhat more severe, but the side barriers are designed to prevent excessive yaw motion and keep the vehicle within the arrester bed. Because of the very flat angle at which any contact with the barrier could result, the possibilities of severe damage to the vehicle or injury to the driver seem remote. Because no tests were conducted on articulated vehicles, it is not possible to predict the performance of the arrester bed in stopping them. However, since the gravel would apply drag forces on the trailer wheels as well as the tractor and because the side barriers are designed to prevent jackknifing, this design appears to be capable of safely stopping articulated vehicles. This is confirmed by results achieved with gravel-bed arresters elsewhere.

Based on the yaw motion experienced at 90 km/h (56 miles/h), a narrower width for the installation might be helpful in preventing excessive yaw in higher-speed runs or jackknifing of articulated vehicles. This could be accomplished by quickly narrowing the distance between the side barriers after the entrance to the chute.

Some difficulty was experienced in removing the truck from the arrester beds. With very heavy vehicles, removal could be extremely difficult, especially if a vehicle cannot assist under its own power. Tow anchors should thus be provided upstream from the entrance to the arrester bed to permit a heavy-duty wrecker to winch out vehicles trapped in the gravel.

Maintenance requirements after impact are minimal for the gravel arrester bed. The gravel was simply raked smooth by hand after the truck was removed. It took only a few minutes to ready the bed for the next run. No gravel was thrown outside the arrester bed on any of the runs. Freezing of the gravel in cold weather does present a problem. Although no tests were run for verification, it seems likely that the retarding forces developed by the gravel would be greatly reduced if the gravel were frozen. Thus, it would appear to be advisable to use deicing chemicals to maintain the gravel in an unfrozen state.

CONCLUSIONS

Based on the three test runs reported here, the following conclusions appear to be warranted:

1. The gravel arrester bed safely stopped the 16 650-kg (37 000-lb) vehicle at speeds up to 90 km/h (56 miles/h).
2. Average deceleration over 0.5-s intervals and the overall average decelerations experienced in the test runs were no greater than would be experienced in a panic stop on a dry pavement.
3. The arrester bed appears to be capable of stopping single-unit trucks at higher speeds, although contact with the side barriers might occur. Based on these tests and other results reported elsewhere, it appears that articulated vehicles would also be safely stopped by this design.
4. Substantial yaw was observed at 90 km/h (56 miles/h). Narrowing the chute width from 5.4 m (18 ft) might be helpful in preventing exessive yaw and jackknifing.
5. A suitably located tow anchor is probably necessary to remove very heavy vehicles from the gravel bed.
6. Post-impact maintenance requirements for the gravel bed are minimal.
7. Application of deicing chemicals appears to be necessary to prevent freezing of the gravel bed in winter.

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The conceptual design of the arrester system was developed by the Engineering Research and Development Bureau and the Soil Mechanics Bureau of NYSDOT. The test vehicle was driven by Richard N. Simberg, regional director of transportation for Region 2 of NYSDOT (Utica). The vehicle was prepared by the Region 2 equipment management facility under the direction of regional equipment manager Anthony N. Slezak. Maintenance employees from the NYSDOT residency in Herkimer, directed by resident engineer Robert Farrington, assisted in site preparation, traffic control, and removal of the test vehicle during the test runs. William McEachon, engineer in charge during construction, was also very helpful during the tests. Senior engineering technicians Robert P. Murray and Peter D. Kelly of the Engineering Research and Development Bureau assisted in data collection and analysis.

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Abridgment

Strategy for Selection and Placement of Highway Guardrails

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A strategy is presented in this paper by which guidance can be provided in selecting and placing guardrails. Figure 1 shows a roadside condition in which a hazardous obstacle, such as a fixed object or embankment, warrants the installation of a guardrail [warranting procedures are discussed elsewhere (1-3)]. Normally, the guardrail in Figure 1 would be of a standard type for the state and would be installed at a customary distance S from the edge of the pavement. Other than these standard procedures, no rational method is known by which it can be determined whether the distance S or the guardrail type is optimal from the standpoint of cost-effectiveness. In view of the current problems of increasing highway construction costs and limited available funding, cost-effectiveness has become a significant consideration in the selection and design process, particularly with regard to rural, low-volume highways.

Based on a cost-effectiveness analysis, the technique presented here might be used in setting policy for guardrail selection and placement as follows:

1. For a preselected type of guardrail, what distance S in Figure 1 would be optimal? This might affect a state's current specifications for guardrail placement.
2. For a preselected distance S in Figure 1, which of several possible state guardrail types should be installed? How would the selected design compare with designs in common use in other states? This might warrant consideration of changing a state's standards of guardrail design.
3. For the site conditions shown in Figure 1, what distance S and what type of guardrail would be most efficient from a cost-effectiveness standpoint? By comparing the results for the several warranted sites along a stretch of roadway, a common offset distance and guardrail type could be selected for use. Alternatives could be a common guardrail type with different optimal offsets or—though probably undesirable—different optimal distances and guardrail types at each site.

BASIS FOR THE STRATEGY

The basis for the strategy presented here is the small COCOST computer program that was produced under a recent Federal Highway Administration (FHWA) contract to develop a cost-effectiveness model for guardrail selection (4). For given roadway conditions at a site, the program computes comparative cost-effectiveness values (state cost, societal cost, total cost, and benefit/cost ratio) and ranks the 11 selected guardrail types shown in Figure 2 and described in Table 1.

The COCOST program was prepared with the intent of producing a program that is flexible but is as simple to use and as easy to implement as possible. In the example of the COCOST input worksheet shown in Figure 3, it can be seen that several variables are included to provide flexibility but the inputs are simple to prepare by using familiar engineering terms and format (since the program is formulated in U.S. customary terms, Figures 3 and 4 include U.S. customary measurements). If preselected representative inputs are acceptable to the user, only four cards per set are required, the second of which is a blank card. Card sets are simply stacked so that as many cases as desired can be run.

To facilitate adaptation to a particular computer, both CDC and IBM operational versions of the program have been prepared. These are available from FHWA, along with documentation and a user's manual. The computer program is small, and run times are minimal.

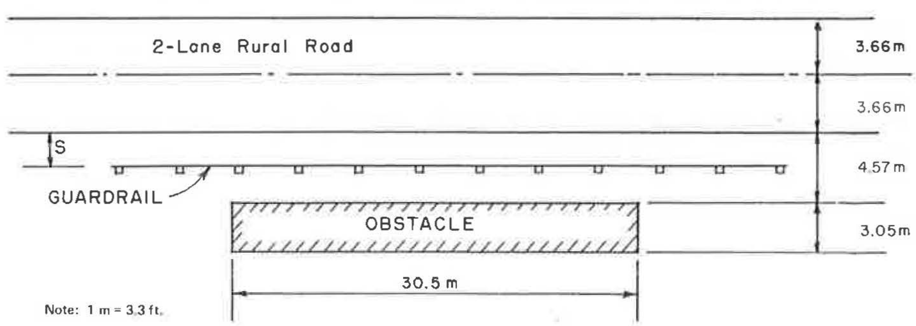
Interpretation of the computer output is not difficult. In the typical COCOST output sheet shown in Figure 4, it can be seen that the guardrail types are ranked by state cost, societal cost, total cost, and benefit/cost ratio in the order of decreasing preference (the best is number 1 and the worst is number 11) with their corresponding values. Thus, the G2 system is the preferable guardrail for this site from the standpoint of societal cost, total cost, or benefit/cost ratio. The analyst can select the cost-effectiveness measure that he or she wishes to use.

APPLICATION OF THE STRATEGY

The first step in applying the strategy to the conditions of Figure 1 involves setting the guardrail at various distances from the edge of the pavement (shoulder widths). For an obstacle exposure length of 61 m (200 ft) and various shoulder widths, the required length of guardrail is as follows (1 m = 3.3 ft):

Distance from Edge of Pavement (m)	Length of Guardrail Required (m)
0.9	95
1.2	91

Figure 1. Sample case of guardrail placement.



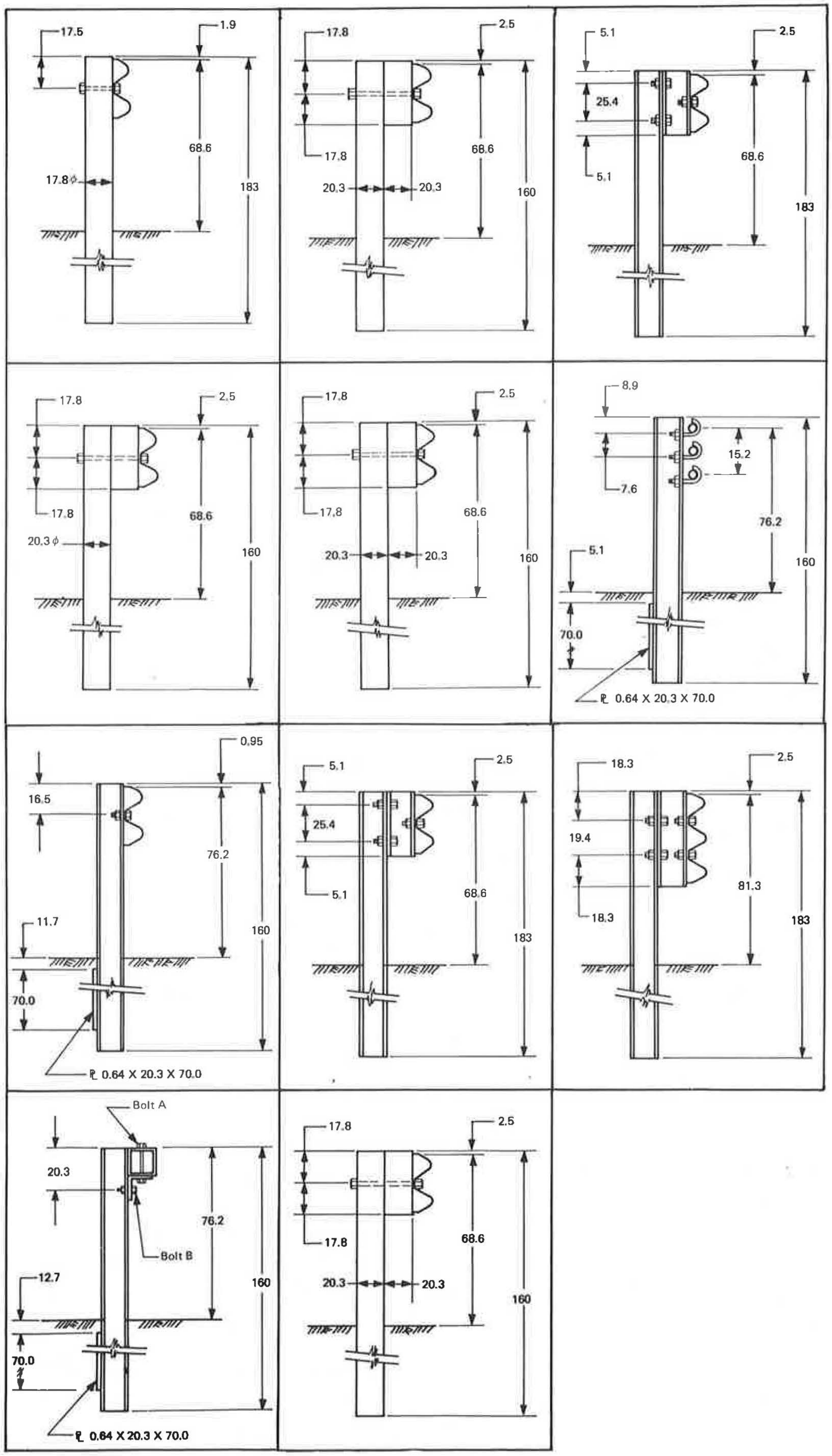


Figure 2. Eleven selected guardrail types.

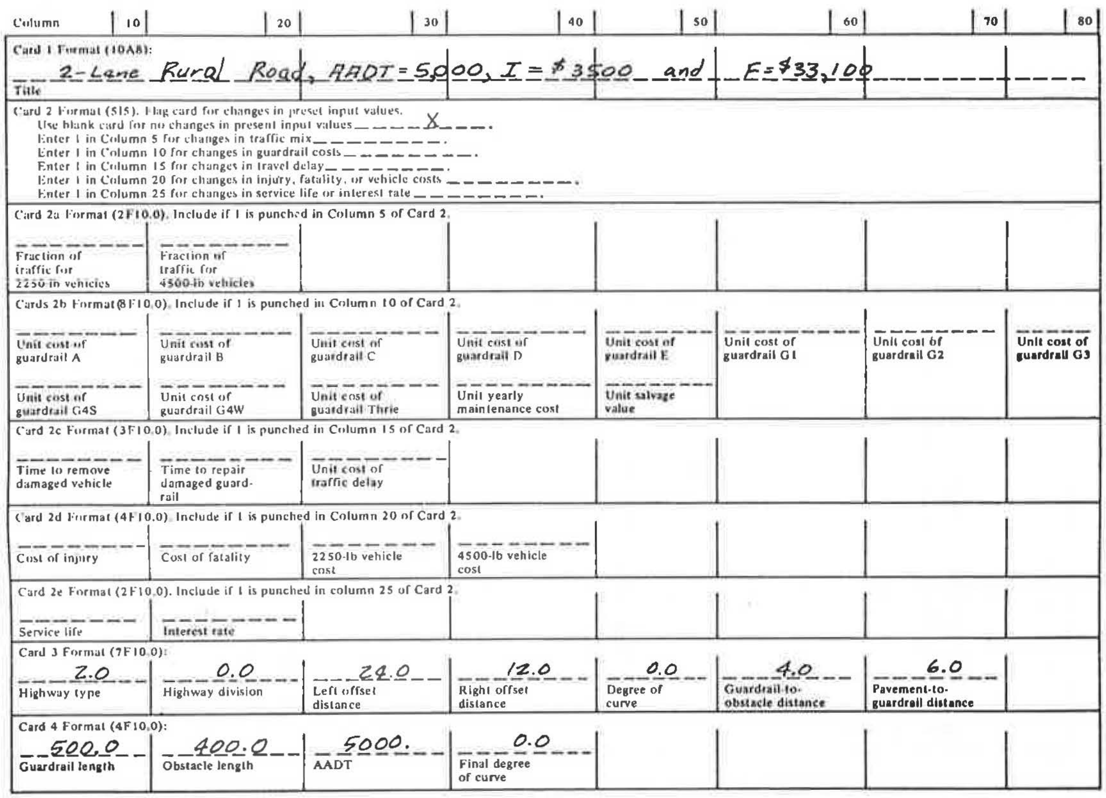
1 cm = 0.394 in.

Table 1. Description of 11 selected guardrail types.

Design	Beam	Blockout	Post: Type	Post: Spacing (cm)	Bolt
A	12-gauge W		17.8-cm diameter, wood	381	0.64-cm diameter, pipe insert in post
B	12-gauge W	20.3x20.3 cm, wood	20.3-cm diameter, wood	190	1.59-cm diameter
C	12-gauge W	20.3x20.3 cm, wood	20.3x20.3 cm, wood	381	1.59-cm diameter
D	12-gauge W	15.2x20.3 cm, wood	15.2x20.3 cm, wood	190	1.59-cm diameter
E	12-gauge W	Charley	Charley	190	1.59-cm diameter
G1	Three 1.91-cm cables		S3x5.7[a]	488	0.79-cm diameter
G2	12-gauge W		S3x5.7[a]	381	0.79-cm diameter
G3	TS6x6x0.1875[a]	S3x5.7[a]	S3x5.7[a]	183	Bolt A, 0.95-cm diameter; bolt B, 1.27-cm diameter
G4S	12-gauge W	W6x8.5[a]	W6x8.5[a]	190	1.59-cm diameter
G4W	12-gauge W	20.3x20.3 cm, wood	20.3x20.3 cm, wood	190	1.59-cm diameter
Thrie	12-gauge Thrie	W6x8.5[a]	W6x8.5[a]	190	1.59-cm diameter

Note: 1 cm = 0.039 in.
[a]Standard rolled shapes of American Institute of Steel Construction, including U.S. customary units of measurement.

Figure 3. COCOST input worksheet.


Column 10 20 30 40 50 60 70 80

Card 1 Format (10A8):
2-Lane Rural Road, AADT = 5000, I = $3500 and E = $33,100
Title

Card 2 Format (5I5). Flag card for changes in preset input values.
Use blank card for no changes in present input values X
Enter 1 in Column 5 for changes in traffic mix
Enter 1 in Column 10 for changes in guardrail costs
Enter 1 in Column 15 for changes in travel delay
Enter 1 in Column 20 for changes in injury, fatality, or vehicle costs
Enter 1 in Column 25 for changes in service life or interest rate

Card 2a Format (2F10.0). Include if 1 is punched in Column 5 of Card 2.
Fraction of traffic for 2250-lb vehicles | Fraction of traffic for 4500-lb vehicles

Cards 2b Format (8F10.0). Include if 1 is punched in Column 10 of Card 2.
Unit cost of guardrail A | Unit cost of guardrail B | Unit cost of guardrail C | Unit cost of guardrail D | Unit cost of guardrail E | Unit cost of guardrail G1 | Unit cost of guardrail G2 | Unit cost of guardrail G3
Unit cost of guardrail G4S | Unit cost of guardrail G4W | Unit cost of guardrail Thrie | Unit yearly maintenance cost | Unit salvage value

Card 2c Format (3F10.0). Include if 1 is punched in Column 15 of Card 2.
Time to remove damaged vehicle | Time to repair damaged guardrail | Unit cost of traffic delay

Card 2d Format (4F10.0). Include if 1 is punched in Column 20 of Card 2.
Cost of injury | Cost of fatality | 2250-lb vehicle cost | 4500-lb vehicle cost

Card 2e Format (2F10.0). Include if 1 is punched in column 25 of Card 2.
Service life | Interest rate

Card 3 Format (7F10.0):
2.0 Highway type | 0.0 Highway division | 24.0 Left offset distance | 12.0 Right offset distance | 0.0 Degree of curve | 4.0 Guardrail-to-obstacle distance | 6.0 Pavement-to-guardrail distance

Card 4 Format (4F10.0):
500.0 Guardrail length | 400.0 Obstacle length | 5000. AADT | 0.0 Final degree of curve

Figure 4. COCOST output for 2.4-m (8-ft) shoulder.

SUMMARY OF RESULTS EXAMPLE GUARDRAIL PLACEMENT CASE - 8-FT SHOULDER

PAVEMENT-TO-GUARDRAIL DISTANCE = 8 FT GUARDRAIL-TO-OBSTACLE DISTANCE = 7 FT

PAVEMENT-TO-OBSTACLE DISTANCE = 15 FT SOCIETAL COST WITH NO GUARDRAIL = $ 3561.28

AADT = 5000 GUARDRAIL LENGTH = 256 FT OBSTACLE LENGTH = 200 FT

DEGREE OF CURVE		PREFERRED GUARDRAIL ORDER BY INDICATED CRITERIA 1	2	3	4	5	6	7	8	9	10	11
0	STATE COST	G1	G2	A	C	B	E	D	G4W	G4S	THRIE	G3
	VALUES	712.90	1032.90	1071.30	1144.30	1455.30	1480.90	1519.30	1583.30	1583.30	1839.30	3375.30
	SOCIETAL COST	G2	G4S	D	THRIE	A	E	G3	C	B	G4W	G1
	VALUES	667.38	1483.47	1531.06	1634.85	1644.65	1661.54	1826.96	1901.89	2142.90	2147.36	3374.85
	TOTAL COST	G2	A	D	G4S	C	E	THRIE	B	G4W	G1	G3
	VALUES	1700.28	2715.95	3050.36	3066.77	3101.19	3142.44	3474.19	3548.20	3730.66	4087.75	5202.26
	BENEFIT/COST	G2	A	C	D	G4S	E	THRIE	B	G4W	G3	G1
	VALUES	2.80	1.74	1.38	1.34	1.31	1.28	1.05	.97	.84	.51	.26

Table 2. Results of COCOST analysis to determine the most effective guardrail system for various shoulder widths and costs.

Shoulder Width (m)	Societal Cost		Total Cost		Benefit/Cost Ratio	
	Type of Guardrail	Value ($)	Type of Guardrail	Value ($)	Type of Guardrail	Value
0.9	G2	910	G1	1868	G1	2.96
1.2	G2	857	G1	1761	G1	3.16
1.5	G2	799	G2	1957	G1	2.96
1.8	G2	755	G2	1868	G2	2.52
2.1	G2	704	G2	1777	G2	2.66
2.4	G2	667	G2	1700	G2	2.80[a]
2.7	G2	627	G2	1623	G2	2.94
3.0	G4S	1302	A	2422	G2	2.17
3.4	G4S	1223	A	2293	A	2.32
3.7	Thrie	1243	C	2576	A	2.04

[a]See Figure 4.

Distance from Edge of Pavement (m)	Length of Guardrail Required (m)
1.5	87
1.8	84
2.1	81
2.4	78
2.7	75
3.0	73
3.4	70
3.7	68

The various exposure lengths given are determined by using the technique described by Calcote (4).

Now assume that the average annual daily traffic for the road is 5000. The next step is to run the COCOST program for each of the 10 cases given above. A typical output sheet, for a 2.4-m (8-ft) shoulder and 78-m (256-ft) guardrail, is shown in Figure 4.

From the results of the COCOST runs, values can be extracted and tabulated to answer the questions posed at the beginning of this paper. For example, Table 2 gives the most cost-effective systems for each shoulder width. Note that type G2 at S = 2.7 m (9 ft) gives the best results for both societal cost and total cost, whereas the cheaper G1 system at S = 1.2 m (4 ft) produces the best benefit/cost ratio.

Suppose that a state has a preferable guardrail type—say, a G4S system—and wishes to select the optimal distance from the pavement edge. It would be of interest to compare the chosen guardrail type with the other 10 types by the cost-effective measure of benefit/cost ratio. The table below gives the results for the G4S guardrail that can be obtained from the COCOST outputs (1 m = 3.3 ft):

Shoulder Width (m)	Benefit/Cost Ratio	Rank
0.9	0.90	5
1.2	0.95	5
1.5	1.02	5
1.8	1.11	5
2.1	1.20	6
2.4	1.31	5
2.7	1.41	5
3.0	1.53	5
3.4	1.64	5
3.7	1.18	8

Note that the optimal distance from the pavement edge is 3.4 m (11 ft) and the highest benefit/cost ratio is 1.64. At that point, however, the G4S guardrail ranks only fifth among the 11 included systems.

CONCLUSIONS

This paper presents a strategy that can provide guidance in the selection and placement of warranted highway guardrails. By using the results of the COCOST computer program, cost-effectiveness tabulations can be prepared to determine (a) optimal placement distance and guardrail type and (b) the ranking of a selected standard guardrail type against the 10 other types included in the program.

As in any cost-effectiveness program, certain limitations exist because of the complexities of the problem, the meager data available, and the assumptions that must be made to fill the gaps. Thus, the COCOST program is intended only as a guide and not as a substitute for engineering judgment. The program does, however, provide a basis for selecting and placing guardrail that has not been available before. In the examples shown here, representative average values have been used for guardrail installation, repair, and maintenance costs. The interested reader is encouraged to obtain the program and documentation and to use his or her own local costs in arriving at quantities for analysis purposes.

ACKNOWLEDGMENT

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Impact Behavior of Small-Highway-Sign Supports

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The results of a series of tests conducted to evaluate the impact performance of widely used and promising new support systems for small roadside signs are presented. All systems were single-post installations. Tests were conducted in accordance with current nationally recognized guidelines, and results were evaluated in terms of American Association of State Highway and Transportation Officials performance specifications. It is concluded that, with the advent of smaller vehicles, small, single-post roadside sign installations can no longer be considered an insignificant hazard. In the tests, many currently used support systems proved acceptable by current performance specifications whereas others were shown to be totally unacceptable and some were what can be termed marginally acceptable. Support systems with breakaway or fracture mechanisms performed much better than base-bending or yielding supports.

A recent survey (1) shows that a variety of systems are used to support small roadside signs. As a result of this survey, it became evident that the crashworthiness of most small-sign supports was unknown. Although many sign-support systems have been crash tested, almost all of the tests have used automobiles that weighed 1453 kg (3200 lb) or more. [A summary of crash tests of sign supports conducted prior to the work reported in this paper is given by Ross and others (1, Appendix B).] Current guidelines (2) recommend that the impact performance of a sign support be evaluated by using a compact vehicle, or its equivalent, with a weight of approximately 1022 kg (2250 lb). The use of smaller automobiles in crash-test evaluations was precipitated by the current trend to smaller and more economical vehicles.

To evaluate currently used sign-support systems and promising new systems, a series of test programs have been undertaken. This paper presents the condensed results of 22 tests sponsored by the Federal Highway Administration (FHWA) (3, 4) and 13 tests conducted by others (5-9). All tests involved installations that had a single support (single-post installations represent approximately 75 percent of all roadside sign installations).

Use of the results of these tests is not limited to state highway or transportation agencies. Although vehicle operating speeds are generally lower in city and county jurisdictions, a sign support can still be hazardous in these areas, especially to occupants of small vehicles. It is important to note that a sign support can be more hazardous at low than at high vehicle speeds. Supports that fracture or break away on impact are generally more hazardous at low speeds, whereas those that yield or bend are generally more hazardous at high speeds. This does not mean, however, that yielding supports are necessarily safer at low speeds than systems that break. Clearly, an agency should be aware of the impact performance of candidate support systems for expected operating speeds.

SUMMARY OF CURRENT SIGN-SUPPORT SYSTEMS

The steel U-post, or flanged channel post, is the most widely used sign support in the United States (1). The next most popular types are the wood post, the steel pipe, and the steel tube post, respectively. Together these four types comprise more than 95 percent of all systems used. An extruded aluminum type X post is also being used to a limited degree. Cross-sectional views of the five basic post types are shown in Figure 1. Rolled-steel shapes with breakaway slip bases are used to some extent, primarily on Interstate systems with controlled access.

Promising new systems have also evolved during the past few years. These include a frangible coupling for use with the steel U-post and a lap-spliced, bolted-base design for the steel U-post that uses a post-stub combination.

TESTING AND EVALUATION

Performance Specifications

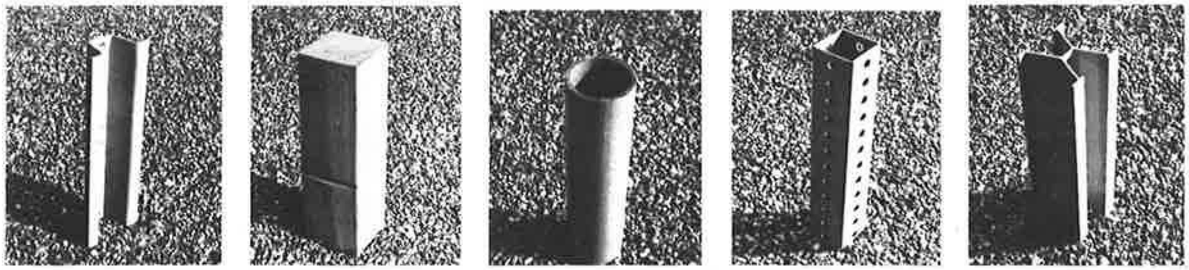
According to the American Association of State Highway and Transportation Officials (AASHTO) (10), "Satisfactory dynamic performance is indicated when the maximum change in momentum for a standard 1020-kg (2250-lb) vehicle, or its equivalent, striking a breakaway support at speeds from [32 km/h to 97 km/h] 20 mph to 60 mph does not exceed [5 kN·s] 1100 pound-seconds, but desirably does not exceed [3.4 kN·s] 750 pound-seconds." In the AASHTO specification, "breakaway supports" is used as a generic term to include all types of sign supports, whether the release mechanism is a slip plane, plastic hinges, fracture elements, or a combination of these. The specification states that "breakaway structures should also be designed to prevent the structure or its parts from penetrating the vehicle occupant compartment" (10). It also alludes to the unacceptability of vehicle rollover after impact.

Transportation Research Circular 191 (2) provides recommended guidelines for crash-test evaluation of a given highway safety appurtenance. With regard to sign supports, it contains recommended test-site soil conditions, vehicle size and impact conditions, procedures for data acquisition and reduction, and performance criteria. The performance criteria given in the circular for sign supports are essentially the same as those given by AASHTO (10). The procedures recommended in the circular were closely followed in the crash tests reported here.

Test Results

Table 1 gives a summary of 22 crash tests sponsored by FHWA (3), and Table 2 gives a summary of recent crash tests of single-post installations sponsored by other agencies. With the exception of test M-13 in Table 2, test vehicles consisted of 1971-1973 Chevrolet Vegas that weighed approximately 1022 kg (2250 lb). In each test, the lower edge of the sign panel was approximately 1.83 m (6 ft) above grade. Soil at the test site conformed to recommended guidelines (2). The types of posts evaluated are categorized as follows.

Figure 1. Five basic types of support posts for small highway signs.



Steel U-Post | Wood Post | Standard Steel Pipe Post | Square Steel Tube Post | Aluminum Type X

Table 1. Results of single-post crash tests sponsored by FHWA.

Test	Material and Type of System	Impact Speed (km/h)	Size of Post	Windshield Broken	Change in Vehicle Momentum (kN·s)
1	Wood, southern pine	34.1	10.2x10.2 cm	No	2.20
2	Wood, southern pine	104.3	10.2x10.2 cm	Yes, by panel	2.17
3	Steel U-post, billet steel	33.5	4.5 kg/m	No	1.44
4	Steel U-post, billet steel	98.5	4.5 kg/m	No	4.31
5	Steel U-post and stub (billet steel) with frangible coupling	35.2	4.5 kg/m	No	1.21
6	Steel U-post and stub (billet steel) with frangible coupling	106.4	4.5 kg/m	Yes, by panel	1.30
7	Square perforated steel tube, post and stub	98.8	6.4x6.4x0.34 cm	Yes, struck by hood	2.54
8	Aluminum type X	102.5	3X	No	1.88
9	Steel U-post, back to back (billet steel)	98.5	8.9 kg/m	Yes, struck by hood	10.21
10	Standard steel pipe	30.4	6.4-cm diameter	No	4.01
11	Standard steel pipe	98.8	6.4-cm diameter	Yes, due to rollover	5.68[a]
12	Wood, southern pine	33.3	10.2x15.2 cm, nominal	No	2.38
13	Steel U-post (rail steel)	102.7	4.5 kg/m	Yes, by panel	1.16
14	Standard steel pipe, post and stub, with breakaway collar	32.7	6.4-cm diameter	No	3.64
15	Standard steel pipe, post and stub, with breakaway collar	101.9	6.4-cm diameter	No	1.72
16	Standard steel pipe, post and stub, with breakaway collar	30.9	6.4-cm diameter	No	2.90
17	Steel U-post, braced-leg design (billet steel)	32.0	3-kg/m post, 3-kg/m brace	No	3.55
18	Standard steel pipe	90.9	5.1-cm diameter	No	2.10
19	Steel U-post, braced-leg design (billet steel)	97.5	3-kg/m post, 3-kg/m brace	Yes	2.40
20	Steel U-post, back to back (rail steel)	108.3	8.9 kg/m	Yes	3.18
20A	Steel U-post, back to back (rail steel)	101.2	8.9 kg/m	Yes	3.04[a]
21	Steel U-post, back to back (experimental billet steel)	93.2	8.9 kg/m	Yes	1.95

Note: 1 km = 0.62 mile; 1 kN·s = 223 lbf·s; 1 cm = 0.39 in; 1 kg/m = 0.67 lb/ft.
[a] Vehicle rolled after impact.

Table 2. Results of single-post crash tests sponsored by agencies other than FHWA.

Source	Test	Material and Type of System	Impact Speed (km/h)	Size of Post	Windshield Broken	Change in Vehicle Momentum (kN·s)
Effenberger and Ross (5)	3491-1	Steel U-post and stub (rail steel) with bolted connection	36.5	4.5 kg/m	No	0.86
	3491-2	Steel U-post and stub (rail steel) with bolted connection	95.9	4.5 kg/m	No	0.81
	3491-3	Steel U-post and stub (rail steel) with bolted connection	27.7	4.5 kg/m	No	1.67
	3491-4	Steel U-post and stub (rail steel) with bolted connection	26.7	4.5 kg/m	No	1.63
Ross and Walker (6)	3636-1	Steel U-post back to back (rail steel)	30.3	8.9 kg/m	No	3.67
	3636-3	Steel U-post back to back (rail steel)	101.4	8.9 kg/m	Yes	4.52
Mohrig and Ross (1)	3683-1	Aluminum type X post	33.0	6X	No	3.73
	3683-2	Aluminum type X post	96.7	6X	Yes	1.83
Walker and Ross (8)	3775-1	Square perforated steel tube, post and stub	31.1	5.1x5.1x0.27 cm	No	1.11
	3775-2	Square perforated steel tube, post and stub	97.5	5.1x5.1x0.27 cm	Yes	0.48
	3775-3	Square perforated steel tube, post and stub	32.8	6.4x6.4x0.34 cm	No	2.87
	3775-4	Square perforated steel tube, post and stub	101.2	6.4x6.4x0.34 cm	No	0.75
Kimball and Michie (9)	M-13[a]	Wood post with weakened section (drilled holes)	32.2	15.2x20.3 cm, nominal	N/A	1.28

Note: 1 km = 0.62 mile; 1 kN·s = 223 lbf·s; 1 kg/m = 0.67 lb/ft; 1 cm = 0.39 in.
[a]Test conducted with soft-nose pendulum.

Wood Posts

Tests 1, 2, 12, and M-13 involved wood posts. In tests 1, 2, and 12, the posts had no breakaway or weakening devices. In tests 1 and 2, the posts were "rough cut" and had full cross-sectional dimensions. In test 12, the post had standard dressed size dimensions of 14.0x8.9 cm (5.5x3.5 in). In test M-13, holes were drilled in the post near the groundline to effect breakaway on impact.

Steel U-Posts

Tests 3, 4, 9, 13, 20, 20A, and 21 involved full-length steel U-posts. There were two basic types of post material and two basic designs. In tests 3, 4, 9, and 21, the posts were hot rolled from billet steel. Of these, the material used in tests 3, 4, and 9, taken from commercially available stock, was considerably more impact resistant than that used in test 21. Post material in test 21 was of an experimental nature and was provided by Armco Steel Corporation of Middletown, Ohio, a producer of billet-steel U-posts. Use of the "experimental posts" in test 21 was precipitated by adverse results in tests 4 and 9. Further discussions of the material properties of yielding or base-bending metal posts are presented in subsequent sections of this paper.

Posts in tests 13, 20, 20A, 3636-1, and 3636-2, taken from commercially available stock (Franklin Steel Company of Franklin, Pennsylvania), were hot rolled from rail steel. In test 20, the intended impact speed was 96.5 km/h (60 miles/h), and the actual speed was approximately 107.8 km/h (67 miles/h). Test 20A was a repeat of test 20 at a lower speed.

The support in tests 3, 4, and 13 consisted of a single 4.5-kg/m (3-lb/ft) post. In tests 9, 20, 20A, 21, 3636-1, and 3636-3, the support consisted of 4.5-kg/m (3-lb/ft) posts bolted together to form a single back-to-back design that weighed 8.9 kg/m (6 lb/ft).

Steel U-Posts with Special Features

Three designs in which the steel U-post was used as a basic component were evaluated. In the first of these, a frangible breakaway coupling was used as a connection between a steel U-post stub and a steel U-post signpost. This coupling was evaluated in tests 5 and 6. In tests 17 and 19, an installation with a vertical U-post and a U-post back or knee brace was evaluated. This design is widely used in the state of Arkansas.

In tests 3491-1 through 3491-4, a stub-signpost design was evaluated. The main feature of this system is a lap-spliced bolted connection at the stub-signpost interface and a retainer-spacer strap. Tests of this concept have also been conducted on multiple-post sign installations (6).

Standard Steel Pipe

Tests 10, 11, and 18 involved full-length standard steel pipe. An anti-twist plate was welded to the base of the post in each case.

Standard Steel Pipe with Breakaway Coupling

Tests 14, 15, and 16 involved standard steel pipe with a standard threaded pipe collar at the base. The collar and a short pipe stub were embedded in a concrete footing. This support system is used primarily by the state of Texas. In test 16, a slight change in the embedment depth of the collar reduced damage to the installation from impact.

Square Steel Tubing

Tests 7 and 3775-1 through 3775-4 involved a square perforated steel tube stub-signpost design.

Aluminum Post

Tests 8, 3683-1, and 3683-2 involved an aluminum post with a cross section similar to that in a back-to-back steel U-post design. The post in test 8 was a type 3X, and in tests 3683-1 and 3683-2 the post was a type 6X (the size designations of the manufacturer, Magnode Products, Inc., of Trenton, Ohio).

Analysis of Tests

Analyses of the test results show that two systems clearly do not meet AASHTO performance specifications: namely, the 6.35-cm (2.5-in) diameter standard steel pipe and the 8.9-kg/m (6-lb/ft) back-to-back billet-steel U-post. Both are the base-bending or yielding type of post with no breakaway mechanism. In the past, when large automobiles were more predominant, this type of sign could be easily ridden down. Now that the small-automobile population has become significant, the base-bending type of post is of much greater concern, especially at higher impact speeds.

To improve the impact behavior of the billet-steel U-post, a steel alloy that exhibited brittle fracture during laboratory impact load tests was developed. The mechanical and chemical properties of this material, and all other metal posts tested, are described by Ross and others (3). Test 21 was scheduled to evaluate the impact behavior of this material under full-scale conditions. The post in test 21 was identical to that in test 9 except for the alloy. Comparison of tests 9 and 21 shows that severity of impact was significantly reduced by the new material: The post fractured in test 21 but did not in test 9. Research is still under way to determine an alloy that not only meets safety performance specifications but also is cost effective in terms of production and field application.

Four supports had a change in momentum above the desirable limit but below the upper limit. These were the 6.35-cm (2.5-in) standard steel pipe with a breakaway coupling (test 14), a 2.98-kg/m (2-lb/ft) steel U-post system composed of a vertical post and a back brace (test 17), a 4.5-kg/m (3-lb/ft) full-length steel U-post (test 4), and an aluminum type 6X (test 3693-1).

Test 16, a test of the same design as test 14, involved a minor change in the embedment procedure (3). The change in momentum in test 16 was well below the desirable limit. The change in momentum for this system at a high-speed impact (test 15) was also well below the desirable limit.

For the steel U-post system with a vertical back brace, change in momentum was only slightly greater than the desirable limit. The change in momentum for this system for a 96.5-km/h (60-mile/h) impact was considerably below the desirable limit. Note that posts in this system were from the same type of billet steel used in posts evaluated in tests 3, 4, and 9.

The steel in the U-post evaluated in test 4 was identical to that used in test 9. The comments made on test 9 would therefore be applicable to test 4.

The other system in the "gray area" was the aluminum type 6X post. In this case, change in momentum was above the desirable limit for the low-speed impact and well below the desirable limit for the high-speed impact. Acceptance of this system from the standpoint of safety performance would seem appropriate, since the difference between the actual change in momentum and the desirable limit in the low-speed test is not believed to be excessive.

With regard to trajectory hazard, there were no penetrations into the passenger compartment of the test vehicle by panel or post in any test. In several tests, however, the windshield was broken, usually when the panel and post rotated down into the windshield. In some cases, the hood of the vehicle was pushed back into the windshield. In some tests, the windshield was only cracked, whereas in others it was shattered and dished. In test 20, the panel and post struck the roof and left a considerable dent in the passenger side of the vehicle. However, the impact speed in test 20 was higher than that called for in current test procedures (2).

Many factors influence the trajectory of a sign-support system. These include type and size of vehicle, impact speed, soil conditions, type of support, mounting height of panel, type of panel, and type of post-to-panel attachment. The sequential photographs of the tests indicate that, if a full-sized rather than a compact-sized automobile had been used, windshield contact would have occurred in some tests. The converse of that is true in other tests. Likewise, if the panel had been mounted higher, the windshield of the compact automobile would probably not have been contacted in certain tests but probably would have been with a full-sized automobile.

The above factors notwithstanding, it was concluded after careful analysis of each test that the penetration problem can be minimized by adequately attaching the panel to the post. In general, impact will accelerate the post and panel, causing the post to bend and the panel to rotate downward toward the hood. If the post fractures or a breakaway device releases, the post and panel are also accelerated in the direction of vehicle travel. To reduce the chance of penetration, it is important that the panel remain with the post during this initial contact so that its velocity relative to that of the vehicle is minimized. It should be noted that keeping the panel on the post will not necessarily prevent windshield breakage.

For some designs, the trade-off for a low change in momentum may be a broken windshield. This can be seen by comparing the results of tests 4 and 13. Although the test conditions and post designs and sizes were very similar, the windshield was shattered and dished in test 13 and unbroken in test 4. The change in momentum was 1.16 kN·s (255 lbf·s) in test 13 and 4.3 kN·s (950 lbf·s) in test 4. In test 13, the post fractured and the post and panel rotated down into the windshield. In test 4, the post wrapped around the hood of the vehicle before being ridden down, without fracturing.

Sign-panel accelerations were approximated by analysis of high-speed film. Combined accelerations up to 40 *g*, acting both perpendicular and parallel to the face of the sign, were calculated. The highest acceleration occurred in the base-bending posts that did not fracture. Even with a factor of safety of two, design of an adequate attachment should not be dif-

ficult, especially with lightweight aluminum panels. Attachment load is determined by simply multiplying the weight of the panel by 40 and that by the desired factor of safety. Tensile and shear load per fastener would equal the attachment load divided by the number of fasteners. Washers should be used as needed to prevent pullout of the nut and bolted head through the panel and post.

Vehicle rollover occurred in tests 11 and 20A. Before these two tests are discussed, it should be noted that in each test the initial contact point on the vehicle was approximately 38.1 cm (15 in) either left or right of the center of the bumper. In addition to a longitudinal force, this produced a moment on the vehicle about the yaw axis. In test 11, impact caused the vehicle to pitch down, yaw, and roll. After loss of post contact, the vehicle went into a significant yaw and roll motion that resulted in complete loss of stability. It rolled three times before coming to a stop and was a total loss. In test 20A, the post fractured and was carried along with the vehicle for a distance. When the brakes were applied, the panel slid off the hood onto the ground in front of the vehicle. Applying the brakes also caused the vehicle to begin a yawing motion. When the panel and post were hit by the front of the vehicle, the panel dug into the soil, resisting vehicle motion. This tripped the vehicle, and it rolled over twice.

Analysis shows that the rollover in test 11 was initiated during impact with the post and was therefore what may be termed repeatable, whereas the rollover in test 20A was caused by events that occurred after impact—i.e., the panel tripping the vehicle after hitting the ground. One can only speculate about the probability of occurrence of the test 20 A type of rollover, but it is believed to be very low. Note that tests 20 and 20A were very similar except that in test 20 the impact speed was higher. The vehicle did not roll in test 20.

Although rollover did not occur in test 4, the vehicle appeared to be unstable. After impact, the vehicle began to yaw and roll; then the cable guidance applied a steer correction that stabilized the yaw and roll motions and apparently prevented rollover.

Toughness or ductility during impact was found to be a key factor in the severity of impact for base-bending posts. Posts that exhibited brittle fracture during impact offered considerably less resistance than those that underwent large deformations and yielding without fracturing. Good correlation was found between the impact behavior measured by Charpy impact tests and that observed in the full-scale crash tests. It was found that posts that fractured during full-scale tests had Charpy fracture energy values less than 28 J/cm^2 (1600 in·lbf/in^2), and posts that did not fracture had energy values greater than 44 J/cm^2 (2500 in·lbf/in^2). The details of post material properties and Charpy test results are given elsewhere (1).

CONCLUSIONS

The following conclusions can be drawn based on the tests described in this paper:

1. With the advent of smaller vehicles, the small, single-post roadside sign installation can no longer be considered an insignificant hazard. Although many currently used support systems were proved acceptable by current change-in-momentum performance specifications, others were shown to be totally unacceptable. Some were what can be termed marginally acceptable. Support systems with breakaway or fracture mechanisms performed much better from a change-in-momentum standpoint than the base-bending or yielding supports.
2. In the 22 full-scale tests conducted in this study and 13 tests conducted by other agencies and summarized here, there was no clear intrusion by the test article or the vehicle structure into the passenger compartment. However, in several tests, the windshield was struck by either the sign panel or the vehicle hood (as it was pushed back), and damage ranged from only cracks to a large dish in the windshield. Breakage occurred in high-speed tests only. It is concluded that trajectory hazard can be minimized by designing the panel-to-support attachment so that the panel remains with the sign support after impact. Even so, for some support systems the trade-off for a low change in momentum may be a broken windshield.
3. Vehicle rollover occurred in two tests, and in another test the test vehicle appeared near rollover. In all tests the contact point was either left or right of the center of the front bumper. In addition to a longitudinal force, this eccentricity of loading produced a twisting moment on the vehicle that tended to spin it sideways. Since off-center hits undoubtedly occur in practice, careful consideration should be given to off-center impacts in future tests of sign and luminaire supports. For a given size of post, the potential for rollover increases as vehicle size decreases.
4. Charpy impact tests were conducted on specimens from base-bending posts to determine why some posts fractured during full-scale tests and others did not. Posts that did not fracture caused considerably higher changes in momentum than posts of comparable size that did fracture. Based on the Charpy tests, post fracture can be anticipated for a high-speed impact if the fracture energy is less than 35 J/cm^2 (2000 in·lbf/in^2) at 65.6°C (150°F), provided, of course, the post is not larger than the limits determined here.
5. Adequate panel-to-post attachment can be achieved if the fasteners can carry a total tensile and shear working load equal to 40 times the weight of the panel. Tensile and shear load per fastener should equal the total force divided by the number of fasteners.

A breakdown of the crash-test performance of widely used single-support systems, as well as promising new systems, is given in Table 3 in terms of the following AASHTO change-in-momentum limits:

1. Acceptable—change in momentum of less than 3.4 kN·s (750 lbf·s),
2. Marginally acceptable—change in momentum greater than 3.4 kN·s (750 lbf·s) but less than 5 kN·s (1100 lbf·s), and
3. Unacceptable—change in momentum greater than 5 kN·s (1100 lbf·s).

Note that the limiting sizes in the acceptable category in Table 3 are not necessarily the maximum sizes that will satisfy the AASHTO specification. These limits are based on current test results. Future tests—if and when they are performed—may show that larger sizes of some designs are acceptable.

Crash tests of the slip-base breakaway design (11-13) and the load-concentration-coupler design (14, 15) have shown that these systems can easily meet current performance specifications for single-post installations. Most of the referenced tests involved installations with multiple supports much larger than those that would typically be used in a single-post installation. Slip bases are commonly used with standard steel pipe and rolled-steel shapes. The load-concentration coupler is typically used with rolled-steel shapes.

Table 3. Acceptability of various single-post systems according to AASHTO change-in-momentum specifications.

Acceptibility Category	Type of Post	Maximum Dimensions	Test No.
Acceptable	Steel U-post		
	Rail steel with lap-spliced bolted-base assembly	6 kg/m[a]	3491-1, 3491-2, 3491-3, 3491-4
	Post with frangible coupling at base	4.5 kg/m[b]	5, 6
	Full-length rail steel	4.5 kg/m	13
	Full-length "experimental" billet steel	8.9 kg/m, two 4.5-kg/m posts back to back	21
	Wood, grade 2, southern pine (or equivalent)		
	No breakaway or weakening device	10.2x15.2 cm, nominal	1, 2, 12
	Holes at base for breakaway mechanism	15.2x20.3 cm, nominal	
	Pipe		
	Full-length standard steel with no breakaway or weakening device	5.1-cm inside diameter	18
	Standard steel with breakaway coupling	6.35-cm inside diameter	14, 15, 16
	Square steel tube	6.35x6.35x0.34 cm	7, 3775-1, 3775-2, 3775-3, 3775-4
	Aluminum, full-length	Type 3X	8
Marginally acceptable	Steel U-post		
	Full-length rail steel	8.9 kg/m, two 4.5-kg/m posts back to back	20, 20A, 3636-1, 3636-2,
	Billet-steel vertical post and billet-steel back brace	3 kg/m each	17, 19
	Full-length billet steel	4.5 kg/m	3, 4
	Aluminum, full-length	Type 6X	3683-1, 3683-3
Unacceptable	Steel U-post, full-length billet steel	8.9 kg/m, two 4.5-kg/m posts back to back	9
	Pipe, full-length standard steel	6.35-cm inside diameter	11

Note: 1 kg/m = 0.67 lb/ft; 1 cm = 0.39 in.
[a] Maximum size crash tested.
[b] Maximum size for which couplings have been crash tested.

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