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Wetlands, Floodplains, Erosion, and Storm Water Pumping

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Innovative Technique for Preliminary Highway Location

HERMAN D. JUFFER*, KATHERINE L. COWARD, AND LAWRENCE E. FOOTE

In the Trunk Highway 61 preliminary location study conducted by the Minnesota Department of Transportation (DOT), a broad-scale computer mapping technique was used to locate a roadway along a 37-mile portion of the North Shore of Lake Superior, a pristine, wildernesslike area with sensitive social, economic, and environmental conditions. The unique aspect of the preliminary location report was the study of complex interdependent issues in arriving at a workable location solution. The study consisted of three phases: Phase 1 resulted in 14 data maps, 24 analysis maps, and 8 relevant assessment categories. Phase 2, the preliminary location report, identified 68 different segment alternatives through development of a composite assessment map and input from various agencies and interest groups. An alternative evaluation was conducted based on 12 constraint and 2 opportunity maps of 5 segment areas. As a result of the evaluation of route alternatives, three route recommendations were derived from the phase 2 study. These were further narrowed down to one alternative that required categorical exclusion recommendations or preparation of environmental assessments rather than an environmental impact statement (EIS). Several innovative techniques were used on the project: (a) the geographic analysis and alternative evaluation were based on a comprehensive inventory compiled on a 2.66-acre cell size grid; (b) a thorough, complete, and systematic interdisciplinary involvement was initiated early in the process and followed through; (c) this was the first Minnesota DOT scoping document that allowed early elimination of route alternatives and recommended one alternative for detailed study in the environmental process.

The art of regional landscape analysis is as old as mankind. Early man analyzed regional landscapes in order to locate wildlife, building materials, water resources, and housing sites. Since that time, increasingly sophisticated techniques have been developed to use new information and to present it in ways that help man understand the various facets of specific regional landscapes and their complex interrelationships.

The North Shore study is an example of a scoping document, a technique used by the Minnesota Department of Transportation (DOT) that involves developing new methods for data collection, data analysis, and depiction of the resulting information. The scoping process--a first step in the environmental impact statement (EIS) process--provides for early public, agency, and interdisciplinary involvement. It allows for a choice among a small number of logical alternatives and relevant issues to be chosen for further study from a wide range of possible alternatives and impacts.

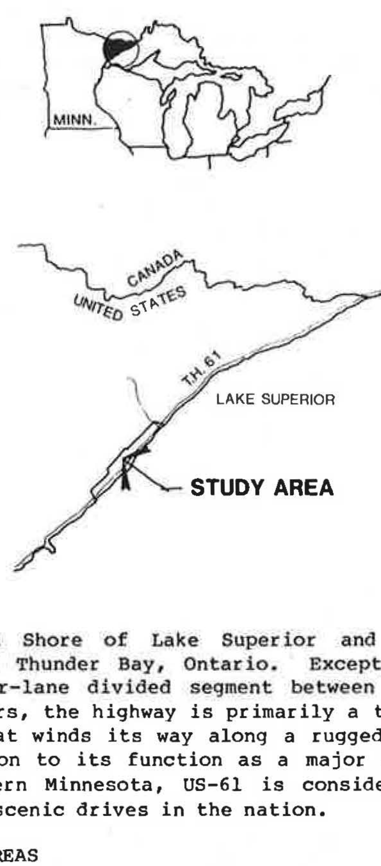
The North Shore study was used to analyze a specific regional landscape, the North Shore of Lake Superior, in view of plans to improve the existing highway corridor of US-61 between Two Harbors and Illgen City, Minnesota. The North Shore of Lake Superior is a pristine, wildernesslike area where economic and social concerns are coupled with sensitive environmental conditions. The issues involved include mobility requirements, critical natural systems, engineering considerations, and aesthetic opportunities. The major groups of users of US-61 include local residents, commercial trucking operations, and large groups of tourists that visit the area primarily during the summer months.

The North Shore study, initiated in 1972, evolved into a three-part study for improving a portion of US-61 between Two Harbors and Illgen City. Phase 1, completed in 1976, was a resource assessment in which resource data were inventoried, analyzed, and assessed. Phase 2 was the preliminary highway location study, completed in 1979. Phase 3 will be the EIS process.

Phase 2 of the study identified 68 alternative segments, which were derived from the composite map of the data collection and from recommendations of various participants and interest groups. The purpose of the phase 2 report, or preliminary location study, was the reduction of these 68 alternative segments into three route alternatives that could be analyzed in greater detail through the EIS process. The computer was used as a tool to highlight potential problem areas. During the EIS process field investigations and data collection will be used to examine the impacts more closely. In this study, problems are identified early to pinpoint constraints that may be brought into the process at a later time.

The 37-mile segment of US-61 on which the study focused is shown in Figure 1. The corridor under study is 4 miles wide and is located entirely within Lake County. US-61 is a major roadway paralleling the North Shore of Lake Superior and connecting Duluth to Thunder Bay, Ontario. Except for a 21-mile, four-lane divided segment between Duluth and Two Harbors, the highway is primarily a two-lane facility that winds its way along a rugged shoreline. In addition to its function as a major arterial in northeastern Minnesota, US-61 is considered one of the most scenic drives in the nation.

Figure 1. North Shore US-61 study area in northeastern Minnesota.



PROBLEM AREAS

Portions of the roadway along US-61 are considered problem areas because of hazardous road alignments, inadequate shoulders, and substandard structural capacity. These areas include Silver Creek Cliff, Lafayette Bluff, and Chapin's Curve. Both Lafayette Bluff and Silver Creek Cliff have a sharp right-an-

*Deceased

gle turn of the roadway on the rock bluff. Other problem areas, such as Gooseberry Falls State Park and Split Rock Lighthouse State Park, involve Section 4f requirements (National Environmental Policy Act). Gooseberry Falls State Park has a small wayside on both sides of the highway that poses a safety problem for pedestrians on the US-61 bridge. Problems in the Split Rock Lighthouse area are the curves and inadequate shoulders. Chapin's Curve is another area that has circuitous highway alignment with sharp angle-right turns and a 25-mph speed limit.

PLANNING APPROACH

The flow diagram in Figure 2 shows the phase 1 and phase 2 processes. Phase 1, the resource assessment, was begun in 1972 and was an in-house document completed in 1976. Phase 2 identified the 68 alternatives and separated them by segment. These were then evaluated individually and in combination for potential impact on environmental characteristics.

This resource approach used a computer program called Environmental Planning and Programming Language (EPPL). EPPL is a packaged data storage and manipulation program that lends itself to an interdisciplinary approach.

The computer study was a coordinated effort of the Minnesota DOT Office of Environmental Services and the District Office in Duluth. Inventory and analysis criteria were studied and evaluated by a team of Minnesota DOT specialists in forestry, agronomy, landscape architecture, geology, biology, sociology, economics, engineering, and other areas. Periodic review and coordination sessions were held with the Minnesota State Planning Agency, the Minnesota Department of Natural Resources, and Minnesota DOT District 1. Technical assistance on the computer program was provided by the Land Management Information Center of the Minnesota State Planning Agency.

There are 63,000 computer cells in the study area, which is 37 miles long and 4 miles wide. The unique feature of this study was the use of a grid cell size of 2.66 acres rather than the typical 40-acre parcel that is available on a statewide basis in Minnesota. The smaller cell size of 2-2/3 acres provided a detailed view of the study area and its affected resources. For each route alternative, specific areas of major and severe impact were isolated and identified for further study in the EIS process.

Another computer program used in the planning process was the PLOT 150, which organized data in three-dimensional pictures that clearly depicted each route location and surrounding land forms in aerial perspective (see Figure 3).

The three steps in the phase 1 process included data collection and coding, analysis, and resource assessment. The data collected for the North Shore study included soils, vegetation, water and tributary systems, watersheds, slope, aspect, elevation, unique features, existing land use, recreational land use, land value, roads, and utilities (Figure 2).

Through the use of computer system software, new data variables were created by various combinations of original data according to criteria provided by a variety of resource specialists in the analysis and resource assessment of phase 1. The specialists included wildlife and water-quality people from the Minnesota Department of Natural Resources as well as the Minnesota DOT wildlife biologist. These new variables were then used to assess social, economic, environmental, and engineering constraints considered in the development of alternative route locations in phase 2.

In the planning process for this project, a complete, systematic interdisciplinary approach was a natural because of the sensitive environmental constraints, the protectiveness of other public agencies, and the concerns of local citizens and public interest groups on the North Shore. The computer process required early interdisciplinary input to establish the critical or important factors to be considered for preparation of the constraint and opportunity maps.

The important elements that make interdisciplinary involvement possible go much beyond the National Environmental Policy Act requirement that mandates this. Minnesota DOT was progressive and open in involving nonengineering people in this study. It is advantageous to have an interdisciplinary person who has a basic understanding of transportation problems act as a mediator when disputes arise and as a coordinator with other agencies. This is especially important in controversial projects about which other agencies have raised concern.

BROAD-SCALE RESOURCE ASSESSMENT

The broad-scale resource assessment described the environmental setting for the entire study area. Eight resource assessment maps from the phase 1 study were used to identify the natural and cultural features, social patterns, and economic factors that would affect highway relocation.

The following series of six resource assessment maps represented factors that place constraints on highway development: community cohesion, displacement, public facilities and services, water and noise sensitivity, conservation and preservation, and engineering considerations. Two additional types of maps were used to determine the desirable location opportunities for development of the scenic highway: viewing opportunities and scenic quality. Examples of the engineering and viewing opportunities maps are shown in Figure 4.

The example of water-quality analysis shown in Figure 5 illustrates the water sensitivity assessment. The most sensitive water system was a stream with erodible areas in the headwaters area of the North Shore. This information formed the basis for the water data in the computer printout in Figure 5. In the computer map, the darker the tone, the more environmentally sensitive or more constrained the area is; the lighter the computer tone, the more opportunity there is for route location.

The factors that went into the conservation and preservation evaluation included analysis maps that displayed land systems, wildlife habitats, sensitive land uses, and fire hazard areas. An overview of the conservation and preservation resources indicated the many constraints near the shoreline. Another important assessment was the viewing opportunities or scenic aspects map (Figure 4), which indicated those areas as points of interest that can be seen within the specified "viewshed" (a viewshed is anything visible from a given observer point within a stated distance). The lightest-toned areas on the computer map contain the greatest number of visible features. The maximum number of views seen from any observer location is three.

The types of views analyzed were (a) views of special features, including man-made sites such as Split Rock Lighthouse, and natural features such as waterfalls, cascades, and trout streams; (b) views of land forms, including Pallisade Head, Silver Cliff, islands, and other geologic formations; (c) views along the shoreline, defined as the edge of Lake Superior; and (d) the view to the horizon, defined as 1 mile into Lake Superior from the shoreline. The lighter the tone, the more opportunity

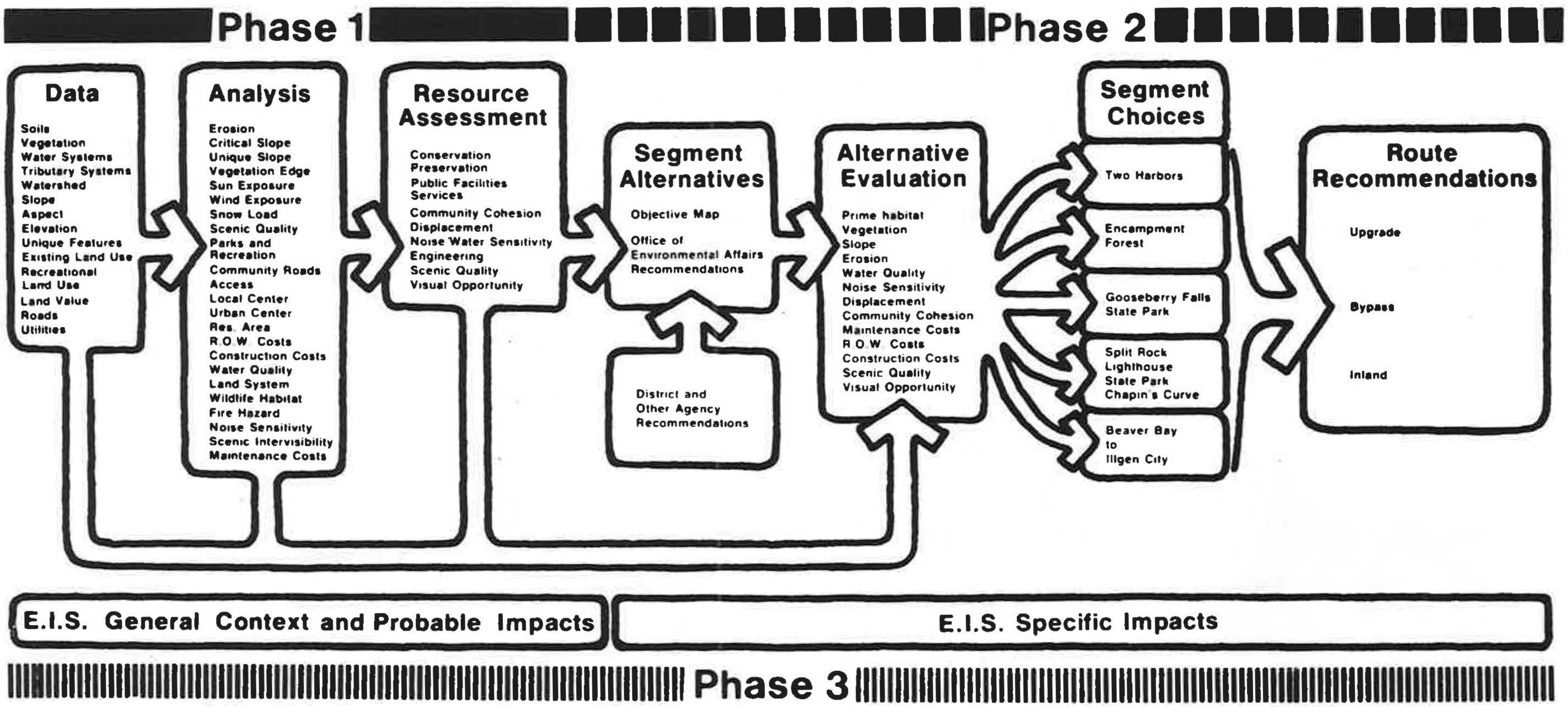
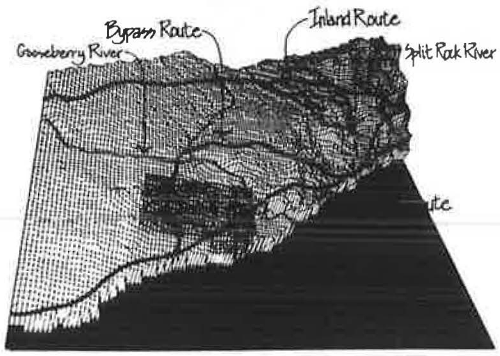


Figure 2. Study flow diagram.

Figure 3. Three-dimensional aerial perspective showing each route location.



there was for views of these various types; the darker the tone, the less view there was from the area.

Another assessment map, the engineering map, is a composite of right-of-way, maintenance, and construction factors that may potentially contribute to the costs of improving US-61. The intent of this map is to point out areas that are the most acceptable in terms of relative cost and those that should be avoided. The darker tones reflect constraints on highway location with respect to soils, vegetation, water systems, slope, land use, and land values. The engineering map revealed no definite solution to route location because the above constraints are common throughout the region. Those areas with the greatest number of potential cost constraints included the northern portion of the site, land in and around Encampment Forest, and major river crossings. The fewest constraints were found in some of the flatter inland areas.

This concluded the phase 1 process and was printed in the phase 1 report, the resource assessment.

COMPOSITE OBJECTIVE MAP AND SEGMENT ALTERNATIVES

In the phase 2 process, the first step was the determination of segment alternatives. These were derived from a variety of sources. The first desirable route locations were selected by connecting low-impact areas (shown by lighter tones) on a composite map called the objective map (see Figure 6). The composite map combined all of the site data collected in phase 1, giving equal weight to each resource category.

Alternatives were also obtained from other agencies, including the Minnesota Department of Natural Resources, Minnesota DOT District 1, and the Minnesota DOT Office of Environmental Services, and through comments received at a public meeting held on the North Shore in August 1978. Similar route alternatives were consolidated, and the resulting network of potential highway locations consisted of 68 segments.

To simplify the process of evaluating the large number of alternatives, a conceptual framework was developed that combined the segments into three concepts. These concepts were presented through a conceptual framework diagram (see Figure 7), which illustrates

1. An upgrade concept, in which the existing highway corridor is used, improvements are made only at key problem areas, and realignments are kept as close to the existing highway as possible;

2. A bypass concept, in which the existing highway corridor is followed but major problem areas are avoided by going around them; and

3. An inland concept, in which the new route is separate from the existing route and parallels the interior uplands.

SEGMENT RESOURCE ASSESSMENT

In another effort to simplify the study process, the route was divided into five segments or geographic sections: Two Harbors, Encampment Forest, Gooseberry Falls State Park, the Split Rock Lighthouse/Chapin's Curve area, and Beaver Bay to Illgen City. For each segment or section of the route, impact maps were used to identify potential impacts or constraints occurring within the proposed route corridor segments (see Figure 8). Only major and severe constraints were considered for wildlife habitat, slope, erosion, water quality, and engineering impacts or constraints were used. Areas with two or three viewing opportunities and excellent or very good scenic qualities were used as a criterion for determining aesthetic areas. Parks and residential land use potentially affected by unwanted or excessive noise were considered noise sensitive. Coniferous vegetation, which correlates with deer habitat and is visually important, was considered an indicator of significant vegetation.

The socioeconomic map identifies areas with permanent and seasonal homes, public facilities, and community cohesion. All commercial and industrial facilities are shown (whether located within or outside route alternatives) to illustrate the potential of the roadway for physically displacing and causing shifts in business and employment.

A tonal chart (see Figure 9) is used to reflect the degree of impact created by each segment alternative according to each construction concept and within the various impact categories. This chart is based on a scoring system documented by actual numbers that appear in the appendix of the report.

The most feasible route segment of the best scoring alternatives was depicted through a computer aerial perspective of that segment area. After the best alternatives were selected by segments, they were synthesized into three primary route locations, represented by the basic concepts of upgrade, bypass, and inland alternatives. Each alternative offers a completely different solution to the major problems associated with North Shore US-61. The three recommended routes are compared on the basis of their respective negative and positive impacts on the environment.

ROUTE SYNTHESIS

In order to arrive at the three final route locations, the best segment alternatives were first combined into 28 different route alternatives. Generally, the lowest-impact segments from one geographic area were directly joined with the lowest-impact segments from an adjacent geographic area to form a complete route alternative. In some combinations, however, the best alternatives had to be connected by an additional segment (see Figure 10).

Each of the 28 route alternatives was categorized according to the upgrade, bypass, or inland location (Figure 8). The route location alternative was then scored for potential negative impacts--the constraints--and for potential positive impacts--the opportunities. The total constraint score was used to rank the alternative within each concept category.

The constraint scores were also translated into an index (see Figure 11) that designated each alter-

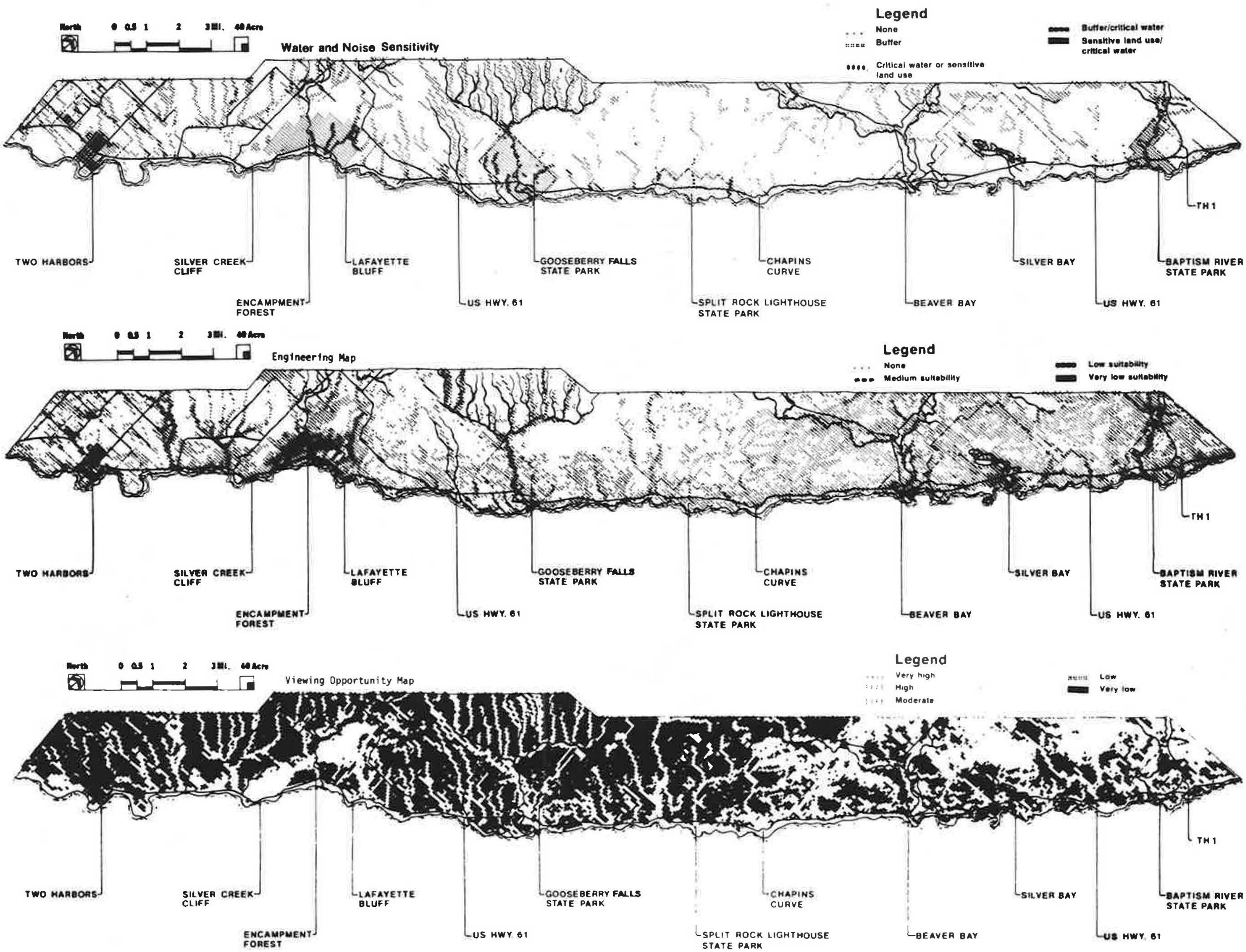


Figure 4. Examples of resource assessment maps.

native as low, medium, or high impact. The index, ranging from 0 to 9, was calculated by dividing the total number of negative impacts by 100. For example, 0 to 3 classifies an alternative in the low-impact range, 4 to 6 in the medium-impact range, and 6 to 9 in the high-impact range.

A route location was then selected from each concept category. The selection was made by weighing the following factors to determine the acceptability of each alternative:

1. The degree of negative impact the alternative would have on the environment (preferably in the low- to medium-impact range);
2. How practical the alternative would be in terms of its alignment and potential construction;
3. How well the alternative represented the concept it was identified with; and
4. How its solution compared with those of other alternatives similar in concept.

The upgrade routes included five possibilities (Figure 11)--alternatives with minor realignments and alternatives with minor realignments and one bypass of a major area. In the bypass category, which included alternatives with two or more separate bypasses or combined bypasses of two problem areas, there were 16 alternatives (see Figure 12). The inland routes for consideration included alternatives with combined bypasses of three or more major problem areas. In this category, six alternatives were considered (see Figure 13).

Figure 5. Water sensitivity diagram showing potential pollution and disturbance.

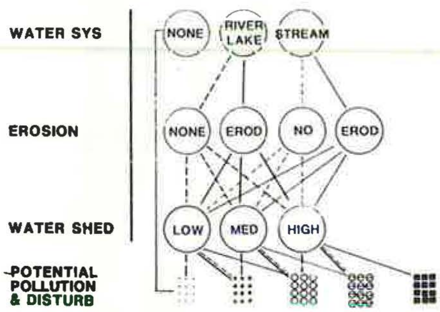
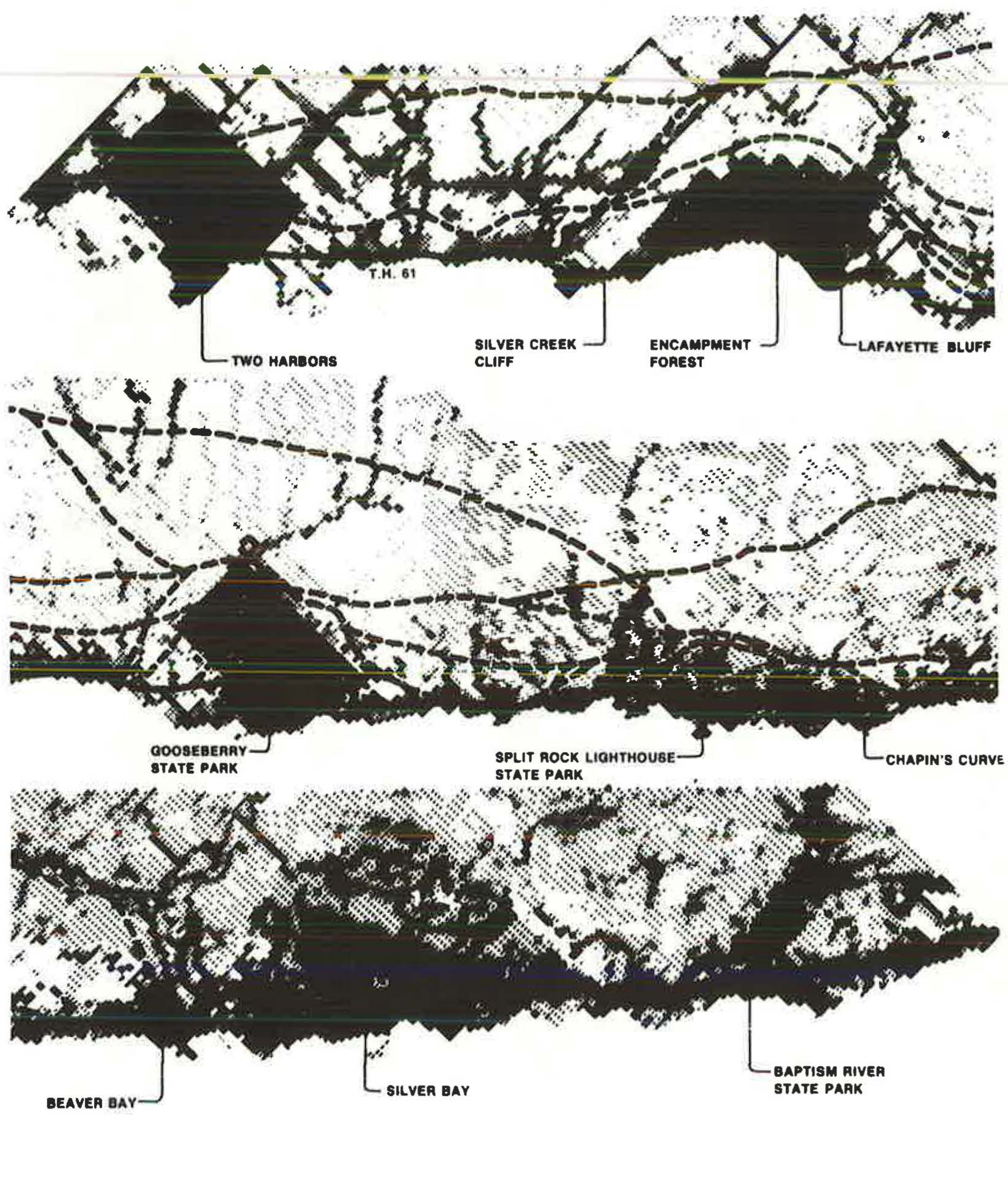


Figure 6. Composite map with 68 proposed route segments.



ALTERNATIVE RECOMMENDATIONS

The final route recommendations for the study include the following route recommendations based on the previous information:

1. Upgrade B--A solution using existing US-61 with new alignment at key problem areas;
2. Bypass B--A solution that follows the upgrade until Gooseberry Falls State Park, where it bypasses this area and Split Rock Lighthouse State Park and Chapins Curve on new alignment;
3. Inland B--A solution on totally new alignment 3 miles inland from Two Harbors to Beaver Bay.

SUMMARY IMPACTS AND COSTS

An upgrade route, a bypass route, and an inland route were selected from the 28 potential alternatives. An aerial perspective of the study area shows the three recommended route locations (see Figure 14). A summary of impacts associated with the three recommended locations is presented in the evaluation chart shown in Figure 15.

The positive impacts of the upgrade route include the more scenic route, the possibility of using staged construction, and little reduction in the business of the commercial establishments. The negative aspects include the major slope impacts. The bypass route has an advantage of going around the Section 4f lands. The negative aspects include the alteration of the slopes and potential impacts on deer habitats. The inland route could be constructed on relatively flat terrain and would allow for the separation of high-mobility and leisure traffic. The disadvantages of the inland route are high impacts on water quality and disturbance of wilderness area. There would also be a reduction in the business of commercial establishments along existing US-61.

The graph in Figure 16 shows the cost estimates for the routes. For the upgrade route, the total cost would be about $27 million. For the bypass route, $30 million would be the base price and adding connector roads and turnback costs to upgrade existing roads to standards would bring the total to $34 million. For the inland route, adding connector roads and turnback costs to improve old highway would total approximately $31 million.

Figure 7. Conceptual framework of three route alternatives.

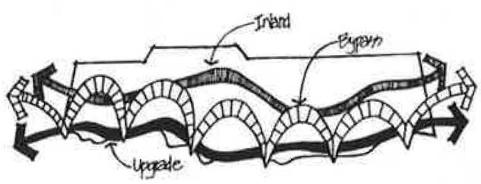


Figure 8. Impact maps identifying wildlife habitat and unique vegetation, engineering considerations, and aesthetics (numbers refer to text of study report).

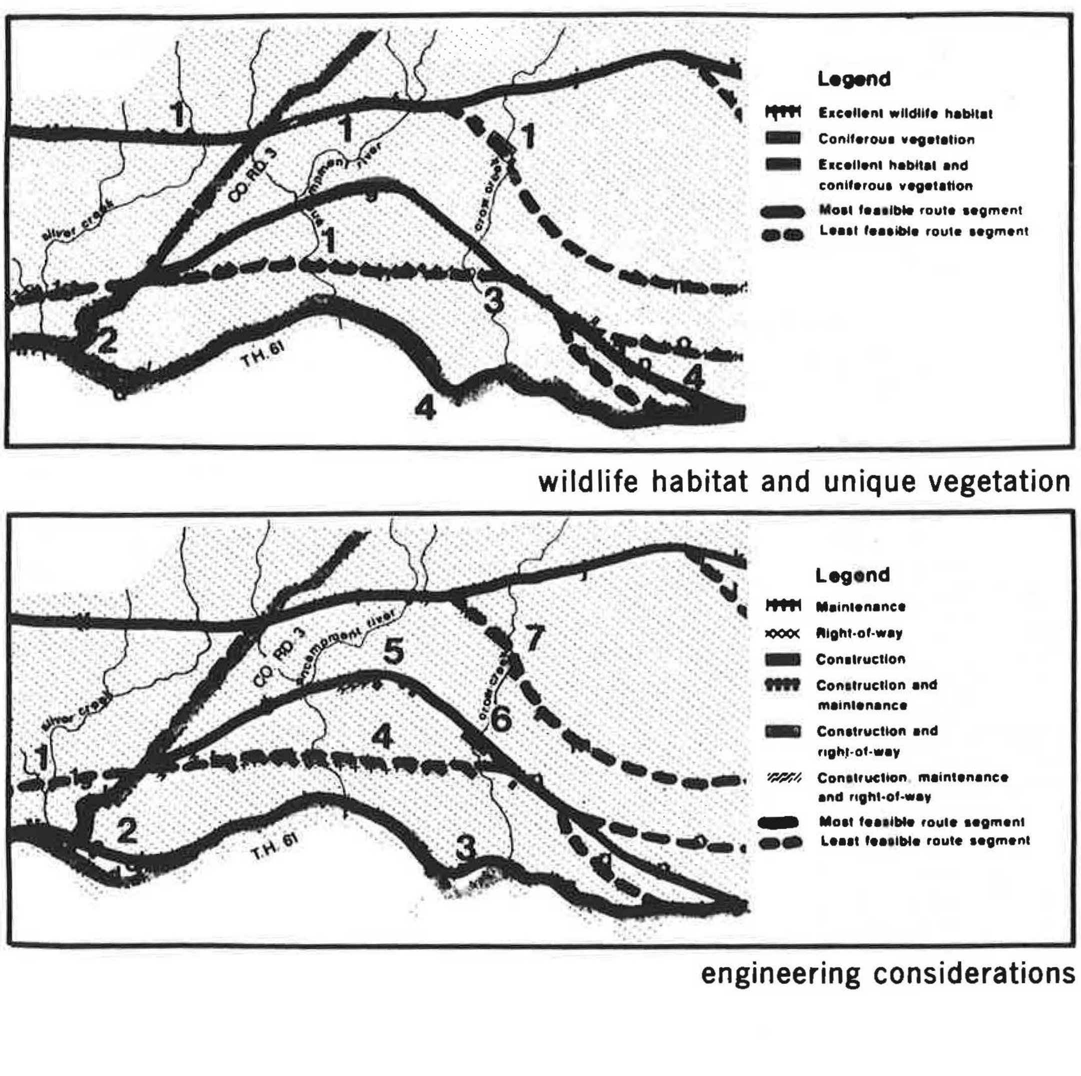


Figure 9. Tonal impact chart.

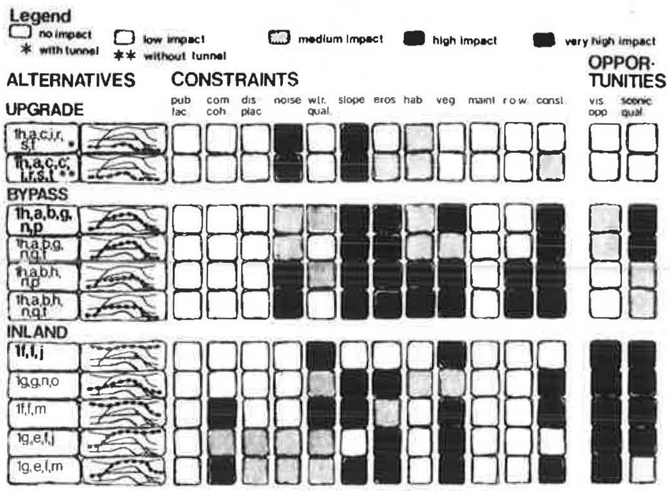
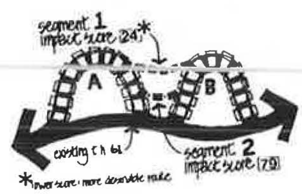


Figure 10. Choice of lowest-scoring connecting segment.



CONCLUSIONS

A goal of this study was to provide a solution to a highway location problem that involved concerns about the natural environment as well as social and economic constraints, practical engineering techniques, and economic feasibility. The solution also had to provide improved highway safety conditions while maintaining the outstanding scenic quality of the area.

When the Minnesota DOT began the North Shore study, there was no agreement about possible alternatives. The Minnesota Department of Natural Resources recommended bypassing the two state parks in the study area, whereas some local citizens and trucking interests preferred a high-speed inland route.

At the scoping meeting on February 26, 1981, it was pointed out that all of the alternatives were scoped on a relative basis and all to the same level. Various representatives of local, state, and federal agencies came to the meeting. The attendees discussed the three recommended concept routes derived through the process and presented their reasons for eliminating some of the recommended routes.

Representatives of the U.S. Fish and Wildlife Service and the Minnesota Department of Natural Resources believed that the inland and bypass alternatives would involve more impacts based on the environmental considerations. As discussions progressed with these two agencies and the Minnesota Pollution Control Agency, their comments led to elimination of the inland and bypass alternatives from consideration for an environmental review process, and the upgrade alternative was recommended. Since that time, FHWA has agreed that an EIS is not

Figure 11. Constraint index scores for upgrade route combinations.

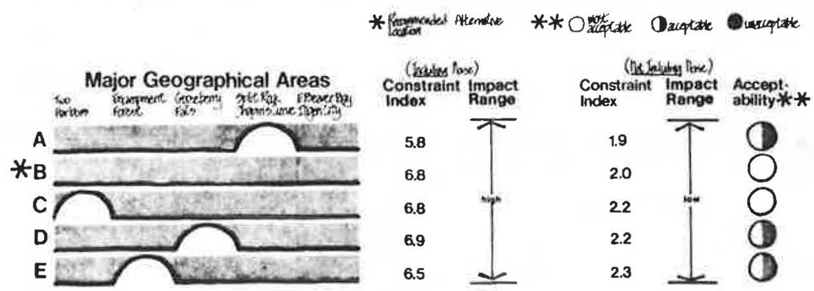


Figure 12. Constraint index scores for bypass route combinations.

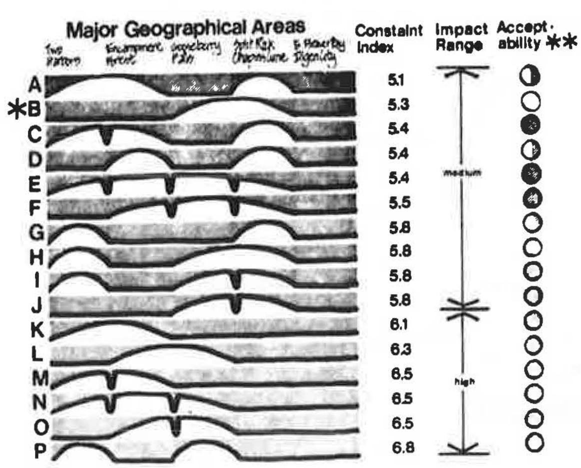


Figure 13. Constraint index scores for inland route combinations.

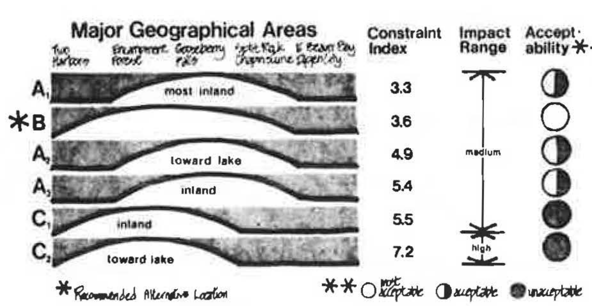


Figure 14. Aerial perspective of study area showing a composite of three recommended route locations.

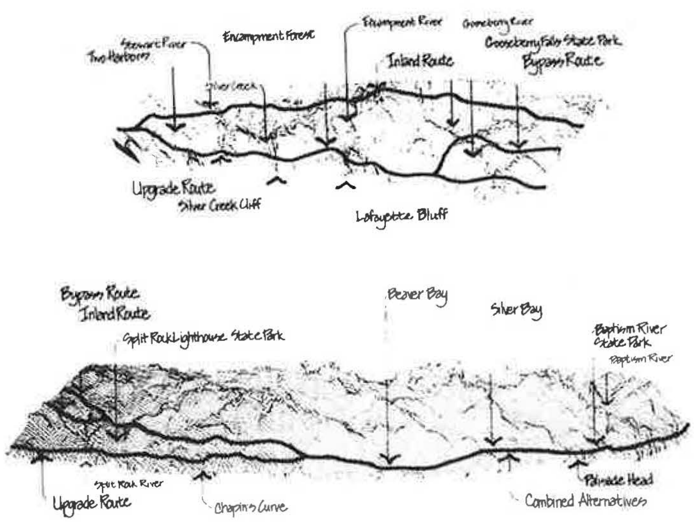


Figure 15. Evaluation chart of impacts for recommended route alternatives.

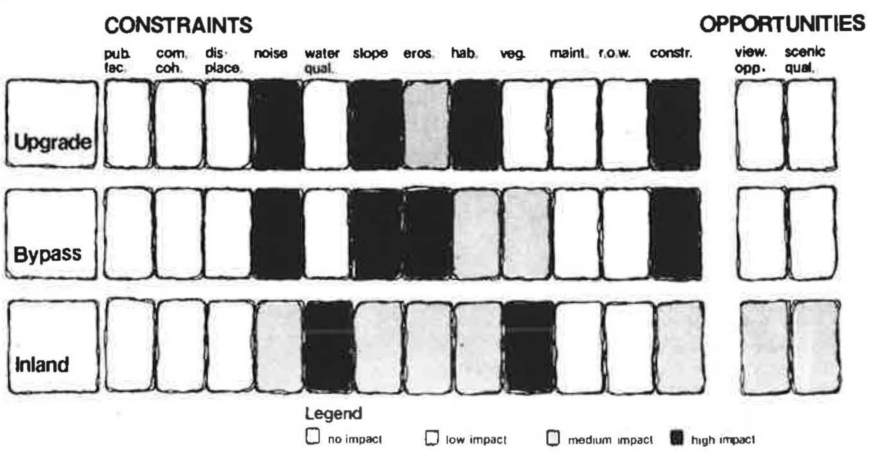
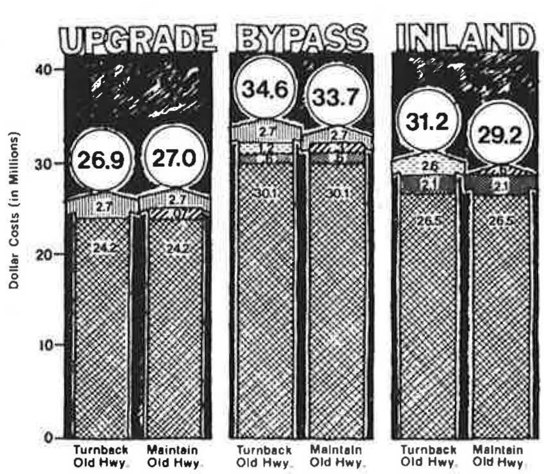


Figure 16. Cost estimate chart.



required for the project. FHWA has instead requested that environmental assessments be prepared for specific improvement areas on the upgrade alternatives or that areas be considered for categorical exclusions.

The scoping process not only shortened the EIS process but also involved the use of several innovative techniques, including the following:

1. The geographic analysis and alternatives evaluation were based on a comprehensive inventory compiled on a 2-2/3-acre cell size in Minnesota.
2. The total length of the study area, 37 miles, is the longest ever considered by the Minnesota DOT in selecting a corridor or route location by using the computer process.
3. Thorough, complete, and systematic interdisciplinary involvement was initiated and continued throughout the process.
4. The graphics and layout were unique and contributed to a more readable report.
5. The study represented the first use by the Minnesota DOT of computer-generated, three-dimensional aerial perspectives to accurately portray route locations.

6. The study was the first Minnesota DOT scoping process that allowed for early elimination of route alternatives and recommended a manageable number of alternatives for the EIS process, thus shortening the EIS process considerably.

The Minnesota DOT was impressed with the scoping technique used in the study, which provided data for decision making in a comprehensive and efficient manner. The phase 2 report was a landmark accomplishment for the Minnesota DOT. The project represents an integration of the computer mapping technique into the transportation planning process.

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Controlling Acidic-Toxic Metal Leachates from Southern Appalachian Construction Slopes: Mitigating Stream Damage

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Highway construction activities in the southern Appalachian Mountains have exposed geological formations that contain pyritic materials (Anakeesta formation), and drainage from slope and fill has caused considerable change in streams that receive these toxic leachates. These drainages, which mobilize high levels of free acidity (low pH) and various toxic metals (aluminum, copper, iron, manganese, and zinc), have destroyed aquatic ecological systems in reaches of mountain streams of the Great Smoky Mountains National Park and tributaries of Citico River and Tellico River drainages in the Cherokee National Forest. In the North River drainage of the Tellico River, mitigation procedures were initiated in 1977 by FHWA to seal exposed Anakeesta road fill of the Tellico-Robbinsville Highway with soil blankets and temporary sodium hydroxide (NaOH) neutralization. A study conducted to evaluate the effectiveness of the controlling technologies used in meeting water-quality objectives is described. Assays of fish population, in-stream fish bioassays, and water-quality assessments were carried out in the watershed during 1978-1979. Initial improvements in stream water quality and biological accommodations occurred in mitigated streams during NaOH treatments and soil blanket installations. However, survivability tests and fish surveys revealed that rainbow trout could not survive in acid streams 6 months after the soil blankets had been installed. Depressed pH and elevated metal concentrations contributed to fish mortalities in these streams.

The scenic Tellico-Robbinsville Highway traverses ridges and peaks of the southern Appalachian Mountains from Tellico Plains, Tennessee, to Santeetlah Gap in the Nantahala National Forest in North Carolina. In 1977 acid drainage originating from recently constructed road-fill areas was contaminating nearby trout streams in the Cherokee National Forest (1). Highly mineralized rock material in the highway embankments was producing acid-toxic metal leachates. Deleterious effects on aquatic biota occurred in McNabb and Hemlock Creeks of the North River watershed and Grassy Branch of the Citico Creek drainage, which were draining the acidic road fill. This problem caused much concern because these streams lie within the Tellico Wildlife Management Area, which is known for its good trout fishing. Furthermore, much of the region had been designated as a potential wilderness area (2).

In early 1978 FHWA began efforts to mitigate the acid drainage problems. The objective of this study was to evaluate the immediate and short-term effectiveness of these measures on aquatic life in the acid-leachate-mitigated streams. In meeting this objective, assays of fish population and in-stream toxicity and water-quality assessments were carried out from April 1978 to July 1979 in the North River drainage basin.

BACKGROUND

Highway construction between Tellico Plains, Tennessee, and Robbinsville, North Carolina, began in 1965. More than a decade later, in 1977, acid drainage from completed sections of the Tellico-Robbinsville Highway altered water quality and reduced the abundance of trout in the adjoining North River and Citico Creek watersheds (1). These watersheds lie within the Tellico Wildlife Management Area, where streams have typically been soft water--slightly acidic drainages supporting an excellent sport fishery of stocked and native trout species. The headwaters of streams affected by Anakeesta leachates--McNabb Creek, Hemlock Creek, and Grassy Branch Creek--are adjacent to highway embankments. These streams exhibited depressed pH values, increased concentrations of sulfates, heavy (toxic) metals, and acidity. Highway embankments containing sulfide-rich pyritic units of the Anakeesta formation were believed to be the sources of acid drainage (1,3).

Neutralization of acid leachates in receiving streams was begun in May 1978 and continued through January 1979. An interim mitigation measure of sodium hydroxide (NaOH) additions was used. A 20 percent solution of NaOH was gravity-metered into the headwaters of affected streams in an attempt to maintain a pH level of approximately 5.8 at the mouths of these streams. Additional remedial actions taken to reduce leachate runoff included asphalt curbing, ditching, and surface drain installations. A lime slurry (approximately 60 tons/acre) was sprayed over selected embankments to reduce migration of acid salts into overlying soil layers to be installed during permanent abatement measures (1).

More permanent mitigation involved sealing exposed Anakeesta material in the road embankments from surface water infiltration. This decreased oxidation-acidification mechanisms and subsequent leachate production. After the addition of lime, approximately 0.6 m of topsoil was placed over exposed rock and seeded with grass. A fiberglass roving layer and then a coating of tar were used to cover the soil to help inhibit erosion. Silt barriers were also placed below soil-covered embankments to reduce inherent siltation problems. Permanent surface sealing of selected embankments was completed by December 1978 (1).

METHODS AND MATERIALS

Hemlock and McNabb Creeks in the North River watershed of the Tellico Wildlife Management Area were sampled for fish populations each quarter from April 1978 to July 1979 by using electrofishing techniques. In conjunction with this assessment, benthic macroinvertebrate communities were characterized monthly. Results of these assessments have been summarized elsewhere (4-6).

Thirty hatchery-reared rainbow trout (Salmo gairdneri) were used at each of 10 stations in 4-day, in-stream bioassays conducted simultaneously with fish population studies to evaluate the suitability of mitigated streams for sustaining fish life. Routine water-quality measurements were taken monthly on site at 13 stations during biological and physical-chemical assessments, and water samples collected from each were placed on ice and returned to the laboratory for more detailed analysis (7). Metal concentrations were analyzed by using atomic absorption spectrophotometry.

RESULTS AND DISCUSSION

Studies of water quality and fish populations and in-stream fish bioassays were carried out from April 29, 1978, through July 1, 1979, to evaluate the effectiveness of acid drainage abatement measures in mitigated streams. The results were separated into three sampling phases:

1. April 28 through May 6, 1978, the period before mitigation efforts by FHWA;
2. June through December 1978, designated the mitigation period, during which acid-receiving streams were affected by both NaOH additions and soil blanket construction over exposed Anakeesta road-fill materials within stream watersheds; and
3. January through July 1979, designated the postmitigation period (installation of road-fill soil blankets and termination of NaOH treatments had been completed before the beginning of this final phase of study).

It should be emphasized that the designations "mitigation period" and "postmitigation period" were chosen for the sake of clarity in discussion. Because installations of soil blankets over exposed Anakeesta road-fill materials were considered permanent measures, there was no definitive period of mitigation.

WATER-QUALITY ANALYSIS

Physicochemical water-quality measurements were taken on April 29 and May 6, 1978, at 13 North River sampling stations before acid mitigation measures began. The results were similar to results obtained by FHWA (1), given in Table 1: depressed pH and alkalinity and increased water hardness and conductivity levels occurred in Anakeesta leachate-receiving streams. The pH varied from 4.0 to 4.8 in McNabb and Hemlock Creeks, from 5.0 to 5.4 in North River below these streams, and from 5.5 to 6.7 at upstream reference stations. The highest acidity values were recorded at affected stream stations, but acidity values were variable; the Sugar Cove reference site (station 1) showed higher acidity (31.3 mg/L as $CaCO_3$) than stations 8, 10, and 11 (22.0 to 22.8 mg/L as $CaCO_3$) in McNabb and Hemlock Creeks. The ranges of dissolved oxygen levels (9.4 to 9.9 ppm) and water temperatures (10.5° to 11.0°C) in acid-receiving streams were comparable to those for reference stations.

Analyses indicated that the water quality of streams that received acid drainage was altered as far downstream as station 13 on the North River, just before its confluence with the Tellico River. This station was located approximately 5.4 and 7.4 km downstream from the two sources of Anakeesta leachates at the headwaters of Hemlock and McNabb Creeks. Water-quality conditions in the North River drainage during the fall of 1977 indicate that these streams were being affected as far downstream as the Tellico River, immediately below the North River (1).

Herricks and Cairns (8) state that the capacity of a stream or river to assimilate acid mine drainage (similar to the highway acid drainage in the study) depends on several factors, most important of which are stream flow (dilution capacity) and total alkalinity (neutralization capacity). In addition to the amount and mineralogical composition of exposed Anakeesta in the highway embankments, the intensity of the acid stress found in McNabb and Hemlock Creeks might be attributed to a very limited buffering capacity in these streams. North River streams typically have low alkalinity, often less than 5 mg/L as $CaCO_3$.

Dilution of the acid Anakeesta drainages in the North River was thought to be the primary factor affecting the concentration of the leachate materials below McNabb and Hemlock Creeks. Maas (9) states that the primary mechanism of recovery from Anakeesta pyrite-related stream contamination is dilution by unaffected side streams and groundwater. Poorer water quality occurred at station 12 (which was affected by both McNabb and Hemlock Creeks) than at upstream station 9, which was immediately below McNabb. Some improvement in water quality was observed several kilometers downstream at station 13 in the North River.

Although dilution and neutralization mechanisms undoubtedly play important roles in regulating concentrations of leachate materials, on the Tellico River flow was observed to be an important regulating factor in mobilizing aluminum. Specifically, as the kinetic energy was reduced in the flow at or near the isoelectric pH, aluminum complexes precipitated as thick, whitish blankets covering stream substrates.

Mitigation Period

Water samples were collected at 13 stream stations from July through December 1978 during NaOH treatments and soil blanket installations over exposed Anakeesta road-fill areas. Improved water quality at McNabb Creek sampling stations was reflected in pH values, which ranged from 6.0 to 7.2 during additions of NaOH. These pH levels were comparable to reference stream values. A reasonable improvement was seen at Hemlock Creek sampling sites, which exhibited pH values between 5.5 and 6.7. However, a

Table 1. Premitigation summary of FHWA water-quality analyses at 14 stream sampling sites from August 15 through October 14, 1977.

		Acidity, Alkalinity, and Hardness (mg/L as $CaCO_3$)				Metals (mg/L)			
Sampling Size	pH	Acidity to pH 8.3	Total Alkalinity	Total Hardness	Conductivity (μmhos/cm)	Sulfate	Aluminum	Manganese	Iron
McNabb									
Headwater	3.7	386.0	0.2	728.0	1353	1070.0	64.00	34.80	2.69
Upstream	4.1	68.5	0.2	128.0	380	194.0	11.00	12.00	0.04
Station 8	4.7	24.4	0.2	56.6	305	75.0	3.50	1.95	0.04
Station 7	4.7	18.6	0.2	47.2	140	62.0	2.52	1.62	0.01
Hemlock									
Headwater	3.9	82.0	0.0	–	1970	–	31.10[a]	8.50[a]	0.07[a]
Upstream	3.3	37.0	0.2	65.0	270	94.0	7.00	5.60	0.03
Station 11	3.4	29.4	0.2	54.6	257	98.0	7.00	1.98	0.10
Station 10	3.5	25.3	0.2	49.0	220	72.0	3.74	1.70	0.26
North River									
Station 6	6.3	2.8	8.6	8.6	24	2.0	0.33	0.01	0.18
Station 9	6.4	1.9	6.3	13.6	32	9.5	0.36	0.20	0.31
Station 12	6.1	1.9	5.2	16.5	41	14.1	0.33	0.26	0.17
Station 13	5.3	2.8	5.3	16.3	41	14.1	0.36	0.20	0.17
Tellico River									
Upstream	6.1	2.8	4.6	5.0	11	1.6	0.40	0.02	0.09
Downstream	5.9	4.2	5.0	11.8	24	9.3	0.33	0.13	0.19

[a]Values measured May 11, 1978.

paired t-test indicated that pH values at both stations 10 and 11 on Hemlock Creek were significantly lower ($p < 0.01$) than at all other stations except station 4 on Laurel Branch, a reference stream. Although mean pH at Laurel Branch was 0.1 to 0.3 pH units lower than mean pH values at other reference sites, no significant differences ($p < 0.01$) in pH were found between any of the reference stream stations.

Laurel Branch could possibly have been receiving low levels of Anakeesta leachates from the road cut that terminated several hundred meters into the watershed or from natural outcroppings of pyritic material that might occur in the drainage. However, the degree of acidity in Laurel Branch was minimal regardless of hydrogen ion sources, and the biological communities in this stream were characteristic of clean-water reference systems throughout the entire study period. The pH values in the North River below acid-mitigated streams were similar to pH values at upstream North River stations.

Longitudinal pH profiles taken in mitigated streams during NaOH treatments indicate that pH neutralization (pH to 7.0) did not occur before a point approximately 2 km downstream of the metering stations (3). Caustic pH values as high as 13 occurred immediately below treatment stations. Thus, improvements in water quality observed at downstream sampling stations were not expected in the headwaters of McNabb and Hemlock Creeks due to excessively high pH levels.

Total alkalinity values measured at the two Hemlock Creek sampling stations ranged from 0.1 to 7.0 mg/L as $CaCO_3$ during this period. Alkalinity values at these stream sites were significantly lower ($p < 0.01$) than at all other stream stations except station 4. At station 4 in Laurel Branch, alkalinity varied from 3.2 to 12.3 mg/L. These values were not statistically different from those at other reference stations, which ranged from 2.0 to 22.8 mg/L. Only minor differences were found between alkalinity levels for McNabb Creek and reference stream stations.

Acidity was variable at all North River sampling stations from September through December 1978, but the highest acidity values were frequently found at reference streams. Additions of NaOH from May through December 1978 probably reduced the level of acidity in receiving streams during these months by reacting with mineral salts and acids in solution.

Measurements of water hardness were also variable from July through December 1978. The highest values were typically found at mitigated stream stations during this period, particularly in Hemlock Creek, which exhibited the highest hardness levels. A peak level of 70 mg/L (as $CaCO_3$) was measured in Hemlock Creek in December 1978.

Conductivity (μmhos/cm) remained high at all mitigated stream stations, in comparison with reference streams, from May through December 1978, apparently as a result of NaOH neutralization measures. Conductivity (or specific conductance), which is a measure of the total amount of ionized materials in the water (10), is a sensitive indicator of Anakeesta leachate contamination in Tellico River streams (1). Conductivity levels generally decreased downstream as a function of increased stream flow and dilution in NaOH-treated streams.

The occurrence of dense precipitates was noted at Hemlock Creek sampling stations from July through December 1978. The yellowish-brown metallic precipitate was also present in McNabb Creek, and the water in McNabb and Hemlock Creeks had a brownish cast during the July 1981 sampling period. This brown coloration was probably due to suspended metallic precipitates.

Wilmouth and Kennedy (11) point out that although increasing pH can remove certain elements (aluminum, iron, and manganese) from solution, many flocs or precipitates that form, especially iron precipitates, are lightweight and tend to remain suspended rather than settle to the bottom. This mobilization appears to be a function of available alkalinity and flow characteristics. The water in Hemlock Creek also had a murky-white coloration during later samplings, particularly during August and December 1978. Because FHWA water-quality data showed relatively high aluminum concentrations at the mouth of Hemlock Creek, where confluent flow dynamics become altered, the whitish color may have been at least partly due to aluminum hydroxide suspension.

In December, manganese levels for McNabb Creek and reference sites showed little variation; however, concentrations in Hemlock Creek approached 0.45 mg/L, which was comparatively high. The aluminum concentration in McNabb Creek was 0.30 mg/L. It reached 1.90 mg/L at the upstream site on Hemlock and 0.65 mg/L at the downstream station during the month of December. Iron and copper concentrations were consistently low at all 13 stream stations. Elevated concentrations of zinc were found in affected streams, the highest value being 0.45 mg/L at the mouth of Hemlock Creek in December 1978.

Metal concentrations (as well as other water-quality parameters) measured by FHWA (12) at the headwaters of McNabb and Hemlock Creeks (above NaOH metering stations) appeared to be directly related to the amount of monthly rainfall. This relationship was most noticeable in December 1978 after a sharp increase in precipitation. Monthly precipitation increased from 7.3 cm in November to 15.7 cm in December 1978. An increase in acid-toxic metal contamination at the headwaters of McNabb and Hemlock Creeks in December was evident from water-quality data collected by FHWA. For example, from November to December 1978 at the headwater site on McNabb Creek, the aluminum concentration increased from 0.07 to 17.80 mg/L, manganese increased from 2.20 to 15.50 mg/L, and iron increased from 0.09 to 2.02 mg/L (12).

There appeared to be no correlation between fluctuations in water-quality parameters at the headwaters of McNabb and Hemlock Creeks and changes that occurred at the respective mouths of these two streams from May through December 1978 (12). Variability in metal concentrations, pH, and other water-quality parameters at the mouths of these two acid-mitigated streams was probably due to changes in stream discharge and the effectiveness of the NaOH treatment at any given time. FHWA water-quality data (12) showed the following metal concentrations at the headwaters and downstream sites of McNabb and Hemlock Creeks:

Site	Metal	Range (mg/L)
Headwaters	Aluminum	0.10-31.10
	Manganese	2.20-23.80
	Iron	0.01-9.90
Downstream	Aluminum	0.05-0.83
	Manganese	0.01-0.54
	Iron	0.01-0.90

Water-quality analyses during the mitigation period from May through December 1978 showed that NaOH stream neutralization measures improved water quality in downstream areas of McNabb and Hemlock Creeks by increasing the pH and precipitating potentially toxic metals. The effectiveness of NaOH additions varied somewhat for different months. This was probably due to fluctuations in stream flow and problems encountered with NaOH metering during the first few months of treatment. The physicochemical water quality of Hemlock Creek improved less than that of McNabb Creek and remained degraded in comparison with that of reference streams.

Postmitigation Period

A considerable change in the water quality of treated streams occurred after the termination of NaOH treatments in January 1979 and the completion of permanent soil blankets over exposed Anakeesta road-fill materials. The lack of buffering capacity in McNabb and Hemlock Creeks was reflected in the alkalinity values from January through March 1979, which were often measured as negative values (see Table 2). The pH fluctuated between 4.6 and 5.9 in McNabb and Hemlock Creeks during this period, and the values were usually from 1 to 2 pH units lower than at reference stations.

Because a decrease of 1 pH unit represents a tenfold increase in hydrogen ion concentration, these differences are substantial. Paired t-tests indicated that pH and total alkalinity values in McNabb and Hemlock Creeks were significantly lower ($p < 0.01$) than values obtained from reference streams. Values of pH at affected North River stream stations were consistently lower than those at upstream reference sites, but the pH was usually above 6.0. The pH of Laurel Branch was significantly lower ($p < 0.01$) than that of other reference streams that had similar pH and alkalinity levels.

Acidity, water hardness, and conductivity values were greater for streams receiving Anakeesta leachate than at upstream reference stations from January through March 1979 (Table 2). Although acidity values were greatest in McNabb and Hemlock Creeks during this period, the difference between affected and control streams varied for different sampling dates. Conductivity and water hardness were consistently high in leachate-receiving streams, and values obtained from McNabb and Hemlock Creeks were significantly greater ($p < 0.01$) than values obtained at reference stations.

Elevated concentrations of aluminum, manganese, and zinc occurred in the acid-receiving streams from March through June 1979 (Tables 2 and 3). Except for the February samples, levels of iron were usually low at all sampling stations. Differences in iron concentrations between reference and acid-affected stations varied for different sampling dates. No detectable levels of copper were measured from stream samples collected in April 1979. Concentrations of aluminum and manganese in McNabb and Hemlock Creeks ranged from 0.10 to 1.98 mg/L for aluminum and 0.09 to 0.54 mg/L for manganese during this period.

Poorer water quality in the mitigated streams revealed that acid-toxic metal leachates from pyritic Anakeesta road-fill materials continued to enter McNabb and Hemlock Creeks even after the soil blankets had been installed. Seasonal increases in rainfall during this period probably affected stream acid-metal leachate concentrations through the following mechanisms:

1. Increased flushing of pyritic oxidation products from the road embankments and sediment traps [during dry periods or periods of light rainfall, pyritic materials oxidize and hydrolyze to produce large quantities of acid and sulfate compounds that may be flushed off in a slug discharge during high-intensity storms (8)];
2. Increased dilution of the acid-toxic metal leachates due to increased stream discharge; and
3. Metallic precipitates that had settled on stream bottoms during NaOH treatments and were probably resuspended during scouring associated with high flow in these steeply sloped mountain streams.

Leaching of the pyritic Anakeesta material during this period could also have been affected by increasing pyritic oxidation rates associated with rising temperatures, high infiltration rates, and little vegetative covering (such as in winter and early spring) (8).

Additional time may be required for embankment stabilization and sealing processes (for example, established vegetative cover) to occur in mitigated Anakeesta road-fill areas of the Tellico-Robbinsville Highway. Further improvements in water quality can be anticipated after residual metal floc accumulations in the stream are flushed out.

Table 2. Water-quality parameters of North River drainages determined monthly from April 1978 through March 1979.

	McNabb Creek			Hemlock Creek		
Item	Premitigation (4/5/78)	Mitigation (7/12/78)	Postmitigation (1/3/79)	Premitigation (4/5/78)	Mitigation (7/12/78)	Postmitigation (1/3/79)
pH						
Mean	4.8	6.4	4.8	4.6	5.7	4.9
Range		6.2-7.2	4.6-5.1		5.5-6.2	4.7-5.0
Conductivity (μmhos/cm)						
Mean	60	88	66	58	91	61
Range		58-135	52-80		57-140	51-70
Acidity (mg/L as $CaCO_3$)						
Mean	19.8	8.1	13.0	22.8	10.2	12.0
Range		1.3-16.0	10.0-18.0		0.8-19.9	8.0-17.0
Alkalinity (mg/L as $CaCO_3$)						
Mean	2.0	7.3	-0.2	2.4	2.5	0
Range		4.0-11.4	-0.5-0.3		-0.1-4.6	-0.1-0
Hardness (mg/L as $CaCO_3$)						
Mean	26	20	24	24	19	25
Range		10-30	12-32		11-30	24-28
Selected metals (mg/L)						
Aluminum						
Mean		0.2[a]	1.4		1.1[a]	1.8
Range		0.2-0.3	0.9-1.7		0.3-1.9	1.7-2.0
Manganese						
Mean		0.2[a]	0.5		0.8[a]	0.3
Range		0.1-0.3	0.4-0.5		0.2-0.5	0.2-0.4
Iron						
Mean		0.1[a]	0.2		0.1[a]	0.2
Range		0-0.1	0-0.4		0.1-0.2	0.1-0.3

[a]Measured November-December 1978.

Table 3. Means and ranges of toxic metal concentration measured at 13 sampling stations in North River drainage from December 2, 1978, through June 28, 1979.

		Metal Concentration (mg/L)							
		Aluminum		Manganese		Iron		Zinc	
Stream	Sampling Station	Mean	Range	Mean	Range	Mean	Range	Mean	Range
Reference	1	0.06	ND-0.21	0.05	ND-0.25	0.05	ND-0.19	0.01	ND-0.07
	2	0.11	ND-0.31	0.04	ND-0.25	0.09	0.01-0.35	0.02	ND-0.10
	3	0.07	ND-0.25	0.06	ND-0.25	0.20	ND-1.40	0.02	ND-0.12
	4	0.11	ND-0.37	0.09	ND-0.40	0.07	ND-0.25	0.03	ND-0.12
	5	0.09	0.01-0.27	0.05	ND-0.25	0.11	0.03-0.40	0.02	ND-0.09
	6[a]	0.13	0.02-0.33	0.10	ND-0.44	0.13	0.03-0.34	0.04	ND-0.18
McNabb Creek	7	0.73	0.27-1.55	0.31	0.01-0.48	0.12	0.04-0.55	0.14	0.03-0.39
	8	0.80	0.17-1.67	0.35	0.06-0.54	0.08	0.01-0.40	0.09	ND-0.24
	9[b]	0.18	0.06-0.31	0.07	0.03-0.24	0.12	0.05-0.35	0.05	0.01-0.18
Hemlock Creek	10	0.56	0.10-1.35	0.25	0.09-0.44	0.14	0.02-0.60	0.16	0.02-0.48
	11	1.07	0.25-1.98	0.31	0.21-0.54	0.12	ND-0.33	0.17	0.03-0.32
	12[c]	0.27	0.05-0.70	0.11	0.04-0.24	0.11	0.05-0.21	0.08	ND-0.32
North River (recovery)	13	0.16	0.02-0.45	0.08	0.03-0.24	0.11	0.04-0.21	0.06	ND-0.22

[a]North River above McNabb confluence. [b]North River below McNabb confluence. [c]North River below Hemlock confluence.

FISH ASSESSMENT

Fish could not tolerate conditions in the streams that received Anakeesta drainages from the road-fill areas. Unpublished surveys by the U.S. Forest Service from June 6 through August 17, 1977, revealed that there were no fish in McNabb and Hemlock Creeks. North River fish populations were also depressed below the mouth of McNabb Creek in comparison with upstream sampling sites.

In later April 1978, before mitigation was initiated, a 4-day in-stream bioassay on rainbow trout was carried out in streams of North River drainage. At stations located at the mouths of McNabb and Hemlock Creeks, all test fish died after less than 24 hr of exposure. Although no rainbow trout died in the North River below McNabb Creek (station 9), 35 percent mortality was observed after 24-hr exposure at station 12 downstream of Hemlock Creek. Only 2 out of 80 rainbow trout died at reference stations during the test. Heavy rainfall (2.9 cm) that occurred during the 4-day bioassay may have flushed high levels of acid and toxic metal compounds into receiving streams. Acid drainage slugs caused by heavy or extended rainfall are a common occurrence in acid-mine drainages (8).

Although the acutely toxic conditions in McNabb and Hemlock Creeks were probably due to a combination of water-quality factors, low pH probably contributed to the death of fish during the in-stream bioassays. Specifically, the pH was 4.4 and 4.0 at the mouths of McNabb and Hemlock Creeks on the first day of the test (April 29, 1978). A pH of 3.7 was measured at the mouths of both streams by FHWA on May 3, 1978, the last day of the experiment (12). Most laboratory data show that a pH level below 5.0

is lethal to fish (13). An extensive survey of Pennsylvania streams polluted by acid-mine drainage showed that no fish were present in waters where the pH was below 4.5 (14). Although there are data that indicate that some fish can survive a pH as low as 4.0, the productivity of aquatic ecosystems is considerably reduced below a pH of 5.0 (13). For example, Menendez (15) reported that in laboratory tests sublethal pH levels (below 6.5) reduced egg hatchability and the growth of young brook trout.

Toxic metals evidently contributed to the lethal effects on rainbow trout in streams receiving Anakeesta drainage. As discussed earlier in the water-quality analysis, elevated concentrations of aluminum, manganese, and zinc (in comparison with control streams) were found in McNabb and Hemlock Creeks during the study period. Many fish examined during the quarterly in-stream bioassays showed obvious signs of gill hyperplasia (a swollen, congested condition). Toxic metal poisoning at acute levels has been shown to cause this symptom (16-19).

Aluminum (17,20,21) and zinc (16,18,19) have been found to be lethal to fish at rather low concentrations, particularly in poorly buffered soft waters. Freeman and Everhart (17) suggest that the safe concentration of either dissolved or suspended aluminum for rainbow trout is well below 0.5 mg/L. Chapman (16) has listed the results of toxicity tests in which lethal zinc concentrations for rainbow trout ranged from 0.24 mg/L (at a pH of 7.2) to 0.85 mg/L (at a pH of 7.1). McKee and Wolf (22) suggest that 1 mg/L of ionic manganese has no deleterious effects on fish. However, the toxicity of manganese (as well as other metals) varies for different species of fish. Aluminum and manganese concentrations measured by FHWA in McNabb and Hemlock Creeks in August 1977 (before mitigation began) greatly exceeded levels reported to be toxic to fish (Table 1).

The toxicity of aqueous metals to fish is modified by many water-quality factors, including water hardness, dissolved oxygen, temperature, pH, and the presence of other metals (16). Although metals are generally more toxic to fish in soft water (which is characteristic of streams in the study area), the toxic nature of metals will vary greatly under different water-quality conditions, particularly chelating organic substances.

Mitigation Period

Mitigated streams showed a general improvement in physicochemical water quality during the period when NaOH neutralization measures were used (Table 2). The positive trends were supported by results of in-stream bioassays done during August and December 1978. Rainbow trout mortalities ranged from 0 to 45 percent after 96 hr of exposure at mitigated sites in August 1978, whereas before mitigation there had been a 100 percent mortality rate within 24 hr. Losses occurred at all stations during the August test, which suggests that the trout had been under stress before testing. Stress due to handling and temperature was apparent while the fish were being transported to study sites during the hot summer conditions. Insufficient acclimation to stream waters may also have contributed to fish mortality in August. Because high mortality (85 percent) was observed at station 6 (a reference station), it is unreasonable to assume that the 45 percent fish mortality in Hemlock Creek was due solely to lethal water-quality conditions.

Only one rainbow trout died at station 7 in Hemlock Creek during the December 1978 bioassay, and no fish died in the other acid-receiving streams during that month's tests. The 89 percent loss observed after 96 hr at station 5 in the North River (a reference station) could only be explained by poor positioning of the test chambers within the stream because no fish mortalities were observed at the other reference sites.

Fish population samples collected in August and December 1978 substantiated these results, revealing a subtle migration of rainbow trout and creek chubs back into the lower reaches of McNabb and Hemlock Creeks. Schools of small fish (possibly creek chubs) were also seen near the mouths of these mitigated streams during invertebrate sampling in August, September, and November 1978. Good populations of rainbow trout were found in reference streams during the quantitative fish collections. Blacknose dace, creek chub, and northern hog sucker were also taken from reference sites on Laurel Branch. Brook trout was the only additional species found in each of the Sugar Cove samples in August and December 1978. Although the number of fish collected in McNabb and Hemlock Creeks was low compared with reference streams, these results indicated that fish were moving back into mitigated drainages.

Because of excessively high pH levels in the headwaters of McNabb and Hemlock Creeks (below NaOH treatment stations), biological accommodations seen at downstream stations during this period were not expected in upstream areas of these NaOH-mitigated streams. Laboratory data have revealed that a pH range of 9 to 10 was harmful to some species of fish and a pH greater than 10 was lethal to all other test species (13). Witschi and Ziebell (23) also reported that a pH of 9.5 to 10.0 was acutely lethal to rainbow trout that had been acclimated to a pH of 7.2. Thus, it is reasonable to assume that fish could not tolerate the high pH conditions in the upper reaches of McNabb and Hemlock Creeks during the NaOH additions.

Postmitigation Period

After the completion of more permanent surface sealing of road embankments and the subsequent termination of NaOH treatments in January 1979, the physicochemical water quality of mitigated streams degraded in comparison with that of reference streams. In-stream bioassays were done in March and June 1979 to evaluate the initial effectiveness of permanent mitigation measures. In these 96-hr tests, acutely lethal conditions (100 percent fish mortalities) were observed at downstream sites on McNabb and Hemlock Creeks. Only a few rainbow trout died at reference and North River stations downstream of the mitigated streams in both the March and June 1979 studies. During these test periods, no fish were collected by electrofishing on McNabb and Hemlock Creeks. Quantitative fish samples collected from reference streams in March and June 1979 were comparable to those collected in earlier sampling efforts.

The pH ranged from 4.6 to 5.6 at McNabb and Hemlock Creek sampling stations from January through June 1979. Concentrations of manganese and aluminum approached 0.6 and 2.0 mg/L, respectively, in these streams. These studies reveal that toxic materials were still entering streams 6 months after completion of the highway embankment soil blankets. Rainbow trout and other native species of fish could not survive mitigated stream conditions where there was a toxic combination of substances. As suggested earlier in the discussion of water quality, an improvement in stream water quality may be expected over time once embankment stabilization occurs in the mitigated road-fill areas of the Tellico-Robbinsville Highway.

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REFERENCES

1. J.L. Blackburn and B.S. Bergendahl. Report on the Geological and Water Quality Study: Tellico-Robbinsville Highway, Station 804+85± to Station 959+10±. Region 15, FHWA, Arlington, Va., Rept. 1, 1977.
2. U.S. Forest Service. RARE II (Roadless Area Review and Evaluation): Summary--Final Environmental Statement. Forest Service, U.S. Department of Agriculture, FS 324, 1979.
3. J.S. Arway, W.S. Porak, and E.L. Morgan. Ecological Monitoring of Mitigation Procedures Used for Acid Heavy Metal Leachates in Southeastern Tennessee. Proc., National Workshop for Mitigation Losses of Fish and Wildlife Habitat, Colorado State Univ. Press, Fort Collins, 1979, pp. 649-652.
4. W.F. Porak. The Effects of Acid Drainage Mitigation upon Fish, Benthic Macroinvertebrates, and Water Quality in Streams of the Cherokee National Forest, Tennessee. Tennessee Technological Univ., Cookeville, M.S. thesis, 1982.
5. J.A. Arway. Fish and Benthic Macroinvertebrate Community Response to Mitigation Efforts Designed to Control Acid Leachates in the Southern Appalachian Mountains. Tennessee Technological Univ., Cookeville, M.S. thesis, 1983.
6. E.L. Morgan, W.F. Porak, and J.A. Arway. The Citico-Tellico Drainage Basins Biological Assessment. FHWA, Draft Final Rept. FHWA-RD-FLH-1-1-(8), Vol. 9, Parts 1 and 2, 1983.
7. Standard Methods for the Examination of Water and Wastewater, 14th ed. American Public Health Assn., New York, 1976.
8. E.E. Herricks and J. Cairns, Jr. Rehabilitation of Streams Receiving Acid Mine Drainage. Water Resources Research Center, Virginia Polytechnic Institute and State Univ., Blacksburg, Bull. 66, n.d.
9. R.P. Maas. Characterization of Water Chemistry of Anakeesta Pyrite Affected Streams in the Great Smoky Mountains National Park. Western Carolina Univ., Cullowhee, M.S. thesis, 1976.
10. G.K. Reid and R.D. Wood. Ecology of Inland Waters and Estuaries, 2nd ed. D. Van Nostrand Co., New York, 1976.
11. R.C. Wilmouth and J.L. Kennedy. Combination Limestone-Lime Treatment of Acid Mine Drainage. Industrial Environmental Research Laboratory, U.S. Environmental Protection Agency, Cincinnati, Ohio, 1976.
12. Tellico Plains-Robbinsville Road Water Quality Data (1978-79). Region 15, FHWA, Arlington, Va., n.d.
13. European Inland Fisheries Advisory Commission. Water Quality Criteria for European Freshwater Fish: Extreme pH Values and Inland Fisheries. Water Research, 1969, Vol. 3, pp. 593-611.
14. E.L. Cooper and C.C. Wagner. The Effects of Acid Mine Drainage on Fish Populations: Part 2. In Fish and Food Organisms in Acid Mine Waters of Pennsylvania, U.S. Environmental Protection Agency, Ecology Research Series EPA-R3-73-032, 1973, pp. 73-124.
15. R. Menendez. Chronic Effects of Reduced pH on Brook Trout (Salvelinus fontinalis). Journal of the Fisheries Research Board of Canada, Vol. 33, No. 1, 1975, pp. 118-123.
16. G.A. Chapman. Effects of Heavy Metals on Fish. In Heavy Metals in the Environment, Water Resources Research Institute, Oregon State Univ., Corvallis, EMN WR 016.73, 1973, pp. 141-162.
17. R.A. Freeman and H.W. Everhart. Toxicity of Aluminum Hydroxide Complexes in Neutral and Basic Media to Rainbow Trout. Trans., American Fisheries Society, 1971, Vol. 4, pp. 644-658.
18. R. Lloyd. Factors That Affect the Tolerance of Fish to Heavy Metal Poisoning. In Biological Problems in Water Pollution, 3rd Seminar, Public Health Service, Cincinnati, Ohio, U.S. Department of Health, Education, and Welfare, Publ. 999-WP-25, 1965.
19. J.F. Skidmore. Toxicity of Zinc Compounds to Aquatic Animals, with Special Reference to Fish. Quarterly Review of Biology, Vol. 39, No. 3, 1964, pp. 227-248.
20. W.D. Burrows. Aquatic Aluminum: Chemistry, Toxicology, and Environmental Prevalence. CRC Critical Reviews in Environmental Control, 1977, pp. 167-216.
21. C.L. Schofield and J.R. Trojnar. Aluminum Toxicity to Brook Trout (Salvelinus fontinalis) in Acidified Waters. In Polluted Rain (T.Y. Toribara, M.W. Miller, and P.E. Morrow, eds.), Plenum Press, New York, 1980, pp. 341-362.
22. J.E. McKee and H.W. Wolf. Water Quality Criteria, 2nd ed. California State Water Resources Control Board, Sacramento, Publ. A-3, 1963.
23. W.A. Witschi and C.D. Ziebell. Evaluation of pH Shock on Hatchery-Reared Rainbow Trout. Progressive Fish Culturist, Vol. 41, No. 1, 1979, pp. 3-5.

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Highway Impacts on Wetlands: Assessment, Mitigation, and Enhancement Measures

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The conservation of wetland acreage in the United States, wherever and whenever practicable, is a national policy objective. This had led to an increased awareness of the need for making wise land use decisions, especially when modification of the natural environment is anticipated. Federal agencies are required to avoid construction in wetlands whenever there is a practicable alternative. However, often there is no practicable alternative. It is important, therefore, to understand the functions, values, and ecological interrelationships of wetland systems so that an appropriate mitigation plan can be developed. General wetland types and their basic functions and values are identified, and highway construction impacts, impact assessment, and mitigation and enhancement procedures are discussed. Special emphasis is given to the reconstruction of wetlands affected by highway construction.

Executive Order 11990, Protection of Wetlands, sets forth a national policy that requires avoiding to the extent possible the long- and short-term adverse impacts associated with the destruction or modification of wetlands. Over the past decade there has developed an increasing awareness of the need for making wise land use decisions to reduce or eliminate adverse modification of the natural environment, including wetlands. Estimates indicate that nearly half of the 120 million acres of wetlands inventoried in the 1950s has already been lost (1). This loss has come largely from the alteration and destruction of wetlands through artificial draining, dredging, and filling. Although there has been a decrease in the percentage of remaining wetlands being lost annually, the subject remains one of concern.

Federal agencies are required to avoid construction in wetlands whenever there is a practicable alternative. This policy applies to any project located in or having an effect on wetlands. FHWA is committed to this policy during the planning, construction, and operation of highway facilities and projects.

DEFINITION OF WETLANDS

An awareness of local wetland statutes and ordinances and their corresponding definitions is extremely important in the environmental impact analysis of proposed highway projects. There is no single, indisputable definition of wetlands because of a high degree of diversity characterized by the continual gradation between dry and wet environments and because reasons and needs for defining wetlands vary.

The definition most commonly accepted by the U.S. Department of Transportation is that of the U.S. Army Corps of Engineers. The Corps of Engineers defines wetlands as areas inundated or saturated by surface water 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 (33 CFR § 323.2c). Consequently, many types of land can be considered. Under the Corps of Engineers definition, wetlands generally include areas such as swamps, marshes, and bogs.

WETLAND TYPES

A swamp is a type of wetland that is often waterlogged in winter and early spring but may be quite dry in the summer. Swamps are characterized by a predominance of woody plants. Swamp vegetation includes willow, oak, maple, gum, alder, and cypress. Swamps usually develop in wet upland depressions, at the edges of lakes and ponds, and along the borders or floodplains of streams and rivers (2).

Marshes can be either saltwater or freshwater. Salt marshes stretch in an almost continuous chain of undulating grasses along the Atlantic Coast and the Gulf of Mexico and account for less than 10 percent of total U.S. wetlands (2). Salt marshes also occur sporadically along the West Coast. Salt marshes are inundated daily by tides, and vegetation consists of salt tolerant plants such as cord grass and marsh hay.

Freshwater marshes account for more than 90 percent of total U.S. wetlands (2). Freshwater marshes may occur inland or adjacent to the coast in low-lying depressions and are most often covered with shallow water. Marshes may be fed by groundwater, surface springs, streams, rainwater, runoff from the surrounding terrain, or all of these. Marsh vegetation is usually characterized by soft-stemmed plants. Vegetation consists of grasses, sedges, waterlilies, reeds, and arrowheads.

The bog is a freshwater wetland most common in the northern and north-central states. Bogs often form in glaciated depressions in forested regions. A bog has very restricted drainage and therefore has almost no inflow or outflow. For this reason, dead organic matter accumulates as peat in layers that are often 40 ft or more in depth (2). Vegetation is characterized by acid-tolerant plants and includes cranberries, blueberries, sedges, and insectivorous plants.

The important point to remember about wetlands is that the dominant factor is saturation with water, which determines the nature of soil development and the types of plant communities that live in the soil or on its surface. Thus, soil types and species of vegetation are the most important physical indicators of wetlands (1).

VALUE OF WETLANDS

Wetland systems serve many functions and provide many benefits. Wetlands provide the vegetative material that is the base for many aquatic and terrestrial food chains. Moreover, vegetative production in wetland systems can be considerable because these aquatic environments act as nutrient traps. Aquatic vegetation can assimilate these nutrients and produce tremendous quantities of plant material. The rates of gross primary productivity in certain types of wetlands are among the highest recorded for any natural systems. Consequently, the potential for supporting large plant and animal populations of diverse species is also high.

Wetlands also provide a vital breeding, feeding, and nursery habitat for many species of waterfowl, fur-bearing mammals, and fishes. The dependence of such species on wetlands at some time in their life cycle is of great economic importance. Many wetlands such as marshes and swamps often act as highly effective flood and erosion buffers. The expanses of shallow water and associated vegetation can slow the velocity of flood water and thus can successfully reduce shoreline and river bank erosion.

Wetlands can also improve water quality through the assimilation of nutrients. This assimilation is accomplished through the filtering capacity of dense stands of wetland vegetation, which provide an effective means of removing suspended solids from polluted waters. In addition, depending on local geologic formations, impoundments of water can allow the slow percolation necessary to replenish underground supplies.

The value of wetlands to recreation is obvious. Many Americans visit wetland areas to observe birds and wildlife. Others enjoy recreational fishing. As noted earlier, many species of sport and commercial fish are dependent on wetlands as sources of food and as spawning areas. Equally important are the benefits of wetlands for environmental research. Wetlands provide natural laboratories where the researchers can view firsthand the many relationships vital to an understanding of ecological systems.

No discussion of the value of wetlands is complete without mention of aesthetics. Although it is difficult to measure, the unique aesthetic value of wetlands is often reason enough for conservation efforts. Some unusual types of wetlands are outstanding from a visual standpoint (1).

CLASSIFICATION OF WETLANDS

The types of wetlands found throughout North America vary widely in terms of vegetation, hydrology, water chemistry, soil, and other characteristics. Many attempts have been made to classify wetlands on the basis of one or more of these characteristics. However, each classification scheme has special advantages in terms of the particular use for which it was devised, and each has serious disadvantages when used outside its own special context. The need for wetland classification has grown out of a need to understand and describe the characteristics of all types of land and to provide uniformity in concepts and terminology for wise and effective management of wetland ecosystems. The U.S. Fish and Wildlife Service has provided the most comprehensive and complete description of classifying wetland systems (3). The primary objective of this classification is to impose boundaries on natural ecosystems for the purposes of inventory, evaluation, and management. Wetland and deepwater habitats are defined separately because the term wetland traditionally has not included deep permanent water. Deepwater habitats are permanently flooded lands where surface water is often deep and is the principal medium within which the dominant organisms live. Both wetlands and deepwater habitats must be considered in an ecological approach to classification.

WETLAND ECOLOGY

Although the characteristics of wetlands differ from system to system, there are basic ecological relationships that are generally pertinent to all wetlands: sunlight energy is transformed into the chemical energy of plants (primary producers), which is transferred to consumers (animals) and further transferred to decomposers (bacteria, fungi, and so on). These sequential transformations of energy constitute a grazing food chain. Organic matter or detritus is also used directly as food by consumers and decomposers, which constitutes a detritus food chain. The decomposition of plant material and other matter results in the release of nutrients that are used by plants and animals. Both grazing and detritus food chains are affected by mechanical energy of tides and waves, horizontal and vertical currents, and diffusion processes that affect the flow of minerals to plants and thereby allow for photosynthesis to occur (4).

HIGHWAY IMPACT ASSESSMENT

It is evident from the preceding discussion that highways can affect the ecological values of wetlands in a number of ways. Highway impacts on a single biotic (living) or abiotic (nonliving) component may affect the dynamics of an entire wetland. These factors are so complex and diverse that impact assessment can become extremely difficult. Although impact assessment has been done for many years, no single guideline for conducting impact assessment has been universally accepted. This is especially true in the area of ecological impacts. Impact assessment continues to be both an art and a science, and judgment is generally required.

A key approach to impact assessment is the ecosystem concept. This concept requires the integration of individual biotic and abiotic components of terrestrial and aquatic systems into a dynamic system (4).

Impact assessment for wetlands should include four major elements (4):

1. Evaluation of the dynamic interrelationships among biotic and abiotic components of the wetland,
2. The specific manner in which highway development can affect each of these dynamic interrelationships,
3. Alternative means of mitigating adverse effects and enhancing desirable effects, and
4. The potential for undesirable secondary impacts from all improvement measures.

The ecological impact assessment process can be divided into three key steps: description of the project, ecological studies, and impact assessments. A description of the project should be provided in as much detail as possible. Such details as grade, alignment, cut and fill, and crossings of water resources should be described. Ecological studies should include an evaluation of the biotic and abiotic factors. The prediction of impacts is the most important step in the process. This step is an integrative procedure in which ecosystem concepts, all environmental data (including abiotic and biotic data), and engineering data must be integrated. Impact assessments should include predictions of the probability, the magnitude, and the time frame of the impacts. Impact assessments must also include the consideration of alternatives and means of providing or enhancing positive impacts (5).

In evaluating the impact of the proposed project on wetlands, the following questions should also be addressed: What is the importance of the affected wetland? What is the significance of the impact on the wetland? The evaluation of importance should consider such factors as the primary function of the wetland and the relative importance of that function. The significance of the highway impact should focus on how the project affects the stability and quality of the wetland. This evaluation should consider the short- and long-term effects on the wetland, the significance of any loss of flood-control capacity, erosion control potential, water pollution abatement capacity, and the value of the wetlands as wildlife habitat. Knowing the importance of the wetland involved and the significance of the impact, the state highway agency and FHWA will be in a better position to determine what mitigation efforts are necessary and the possibilities for enhancement.

MITIGATION AND ENHANCEMENT

There are two fundamental approaches to the mitigation of highway impacts on wetlands. The first approach is to plan or design highways to avoid or minimize the probable occurrence of potential impacts. This approach lies at the heart of Executive Order 11990. The second approach stems from the fact that some impact is often unavoidable regardless of the care and creativity applied during the planning, design, and construction of a highway. Mitigation in these instances may take the form of attempting to reconstruct the basic ecological features that were disturbed by the construction of the facility. Such mitigation may include the restoration of the original hydrologic systems and the replacement of destroyed species of plants with those same species (6).

Mitigation may also take the form of creating alternative ecosystems that offer environmental values equivalent to or more desirable than those of the affected system. It may be possible to create new wetlands in one area as a substitute for areas destroyed or diminished elsewhere. Borrow pits may be located and designed so as to create new wetland habitat. There are many such opportunities for creative design both on and off the immediate right-of-way (6).

In a similar manner, practices and design features may be adapted to enhance or create positive impacts that may result from the highway project. Enhancement is the improvement of a wetland resource so that its values also increase. Some of these practices might include using borrow pits as a sports fishery resource, diversifying wetlands by increasing the mixture and diversity of wetland habitats, and increasing edge effects by the use of islands. Design features might include improving fisheries through the use of culverts for migration, increasing the area or size of a wetland, and developing new wetlands where none existed before. The enhancement concept offers highway personnel an opportunity to innovatively incorporate improvements in wetland environments in their highway projects (6). Because of the variety of potential impacts on wetlands, it is vital that early phases of project development be guided by a careful evaluation of these impacts and the various measures for mitigating them.

In some instances it may be possible to locate alignments that will actually improve the condition of an existing tributary by interrupting potentially toxic wastes that currently flow directly into the tributary. In some instances, it may be more cost effective to span wetlands on structure instead of filling. This will minimize significant adverse impacts on hydraulic flow and bottom substrate. If the decision is made to span wetlands, the alignment should be located so as to minimize impacts on sunlight penetration of underlying waters and thus on photosynthesis (4). Because structures are usually a costly item, it is essential that the impact analysis objectively justify the added expense.

Compensating unavoidable wetland losses is a central component of a mitigation plan. The determination of adequate compensation is largely subjective and involves the consideration of various mitigation and enhancement measures, including restoration, replacement, development, and diversification. A frequently considered compensation measure for highway projects involves wetland establishment. Possibilities for wetland establishment will vary according to geographic area. No general formula can be provided, but some techniques and considerations can be briefly examined.

Wetland establishment is accomplished through wetland construction. The two principal criteria that must be applied in the selection of land for wetland establishment are (a) that the land have low fish and wildlife resource value in its present state and (b) that an adequate water supply be available for connection to ensure successful wetland development (7).

Dredged spoil disposal areas are often successfully used for wetland replacement. Disposal sites for inactive dredged material that are of poor wildlife value should be considered for wetland replacement sites. The disposal materials may be acceptable for highway construction purposes, and the excavated area may qualify as a suitable location for wetland establishment. Moreover, with proper preparation the replacement location can rapidly become established after the installation of wetland plant material.

This process of creating new wetlands generally involves altering existing habitats. It is desirable that the wetlands created be of greater value to fish and wildlife than the habitats that were altered. The replaced wetland need not necessarily be of the same type as that which was lost. A different type of wetland may often provide improvements for fish and wildlife habitat or for the control of water quality, flooding, and shore erosion.

In considering the type of wetland to be replaced, thought should be given to providing a habitat that will lead to an enhancement of the wetland function. Priority should be given to types that establish rapidly, render the most important functions, and are not easily transformed into upland habitat (7).

Wooded wetlands (swamps) cannot be established rapidly because trees require years to mature. High-elevation wetlands or intermittently flooded wetlands have little value to fish and wildlife and are likely to evolve into uplands. Consequently, the best types of replacement wetlands are those that are periodically inundated by tides or permanently flooded by shallow water. Regularly flooded and permanent wetlands have the greatest longevity and the greatest value to fisheries and water-quality control.

When wetlands are replaced in an area dominated by a single wetland type, consideration should be given to establishing a wetland of a different type. Introducing new wetlands of a different type will provide diversification, which may increase the wetlands' overall value.

The junction of two types of habitat often creates a zone with a more diverse biological community than either habitat taken alone. This is known as the edge effect (7). Consequently, a wetland replacement location that offers the opportunity to develop the greatest lineal footage of new edge should be explored. For example, stabilizing an unvegetated shore through wetland establishment provides erosion control and a productive biological edge to upland areas.

The most important factor in wetland establishment is creating the proper elevation. This is accomplished by making a thorough topographic survey of the site. The vegetative composition of nearby wetlands should also be correlated with their topography. This information will be useful in designing final grades and associated vegetative zones.

In the establishment of wetlands and in enhancement measures that involve revegetation, it is important to get the designated plants growing and exhibiting maximum ground coverage and productivity as quickly as possible. In addition, leaving a graded site unvegetated will promote its instability, and grades altered by erosion may not support the designated plants. The necessary plant material may be available from a wetland plant nursery or they may have to be collected from the wild.

In transplanting material from existing natural wetlands, a checkerboard pattern of excavation is recommended to avoid the disruption of single large areas of wetland. However, the preferred method of obtaining materials is through nurseries experienced in handling wetland plants.

Wetland establishment by seeding is the most economical approach, but its success is the least predictable. Seeds must be planted to subsurface depths of generally no greater than 1 in. Seeds that are surface sown with or without mulch wash away during times of high water. Seeds are also subject to uncontrollable factors such as temperature, turbidity, salinity, and siltation, which, if adverse, will lead to reduced productivity.

The most successful as well as the most expensive method of wetland establishment is transplanting nursery-cultivated, peat-potted nursery stock. Peat-potted, nursery-stock aquatic plants can be produced economically outside or in a greenhouse in water-filled compartments. If the physical conditions of a site permit, plants can be transplanted mechanically; otherwise, transplanting must be done by hand. During the period of establishment, litter and debris deposits should be removed from the site, and all transplants lost and bare-seeded areas should be replanted. Litter and debris deposits that adversely affect wetlands might demolish transplants or seedlings unless removed expeditiously.

If there are large populations of wildlife or livestock near a new wetland site, the site may have to be protected during the period of establishment by using enclosures.

Factors that have been found to limit the success of wetland establishment projects are improper final grade, improper wetland species, restricted tidal flow to the site or inadequate water level, improper timing of incorporation of the specified plant materials, erosion, depredation by wildlife and livestock, development of a salt stress zone, and litter deposition and accumulation. All but the last two of these factors can be traced to imperfections in project design, specifications, execution, or inspection (7). The final grade of a site and the plant species assigned to the various elevation zones will dictate the ultimate success of a project.

SUMMARY

In summary, wetlands as defined here are characterized by the periodic or permanent presence of water in a sufficient amount to make it the dominant factor in determining the nature of soil development and the types of plant communities living in it or on its surface. There are many types of wetlands and a variety of classification schemes, depending on the wetland definition that is used. The benefit of classification is to provide uniformity in concepts and terminology for wise and effective management of ecological systems.

Most wetlands are important to fish, shellfish, waterfowl, and other wildlife, although specific values differ. Wetlands also perform functions that are vital to the dynamics of natural ecosystems. Wetlands provide a highly productive habitat for a diverse group of species and beneficial functions in flood control, erosion control, pollution control, and water recharge.

There are many complex ecological relationships associated with wetland systems. It is essential that these relationships be considered in highway planning projects so that highway impacts on wetlands can be accurately determined and analyzed.

Highway construction in wetlands is regulated by federal and state laws. However, the unavoidable adverse impact on wetlands by a proposed highway construction activity may be permitted provided that such construction is judged to be in the public interest and an acceptable plan to mitigate wetland losses is implemented. A central component of the mitigation plan is compensating or offsetting unavoidable wetland losses. The objectives are basically of two kinds: the mitigation of adverse impacts and the enhancement of wetlands.

Highway professionals realize that accurate assessment of impacts on wetlands and appropriate mitigation and enhancement measures are integral to sound highway planning, design, and operation. It is hoped that this trend will continue and that highway professionals will use their planning, engineering, and management skills to implement highway projects that are compatible with natural ecosystems in general and wetlands in particular.

REFERENCES

1. Water-Related Activities of Highway Projects. Office of Environmental Policy, FHWA, 1980, pp. III-1 to III-36.
2. Our Nation's Wetlands: An Interagency Task Force Report. Council on Environmental Quality, Washington, D.C., 1978.
3. L.M. Cowardin, F.C. Golet, and E.T. LaRoe. Classification of Wetlands and Deepwater Habitats of the United States. Fish and Wildlife Service, U.S. Department of the Interior, 1979.
4. Highways and Wetlands: Volume 2--Impact Assessment, Mitigation and Enhancement Measures. FHWA, 1980.
5. Highways and Ecology: Impact Assessment and Mitigation. FHWA, Rept. FHWA-RWE-OEP-78-2, 1978.
6. P.W. Shuldiner and D.F. Cope. Ecological Effects of Highway Fills on Wetlands: Examples from the Field. Transportation Research Record 736, 1979, pp. 29-37.
7. Highways and Wetlands: Compensating Wetland Losses. FHWA, 1982.

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Evaluation of Artificial Wetlands in North Dakota: Recommendations for Future Design and Construction

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Eighteen artificial wetlands were evaluated in North Dakota in 1979 and 1980. Several recommendations are presented for artificial-wetland design and construction based on both the results of the 2-year study and the habitat requirements of breeding waterfowl. It is recommended that (a) a series of wetlands of varying sizes be built rather than one large wetland; (b) multisided, variable-depth wetlands be excavated instead of rectangular, uniformly deep borrow pits; (c) bank gradients not exceed a 10:1 to 20:1 slope; (d) nesting islands be constructed in the center of the wetland when possible; (e) adjacent upland areas be seeded with dense cover to support nesting and reduce erosion and runoff; (f) if necessary, wetlands be sealed with a clay layer to retain water; and (g) topsoil be set aside and respread to provide an adequate substrate for the growth of aquatic plants. Some suggestions for future research on artificial wetlands are also presented.

The value of wetlands as wildlife habitat and in pollution and flood control has been widely recognized (1-4). There is no need to expound the many benefits of wetland protection, enhancement, or replacement here. In accordance with U.S. Department of Transportation order 5660.1A, FHWA is committed to the protection, preservation, and enhancement of the nation's wetlands to the fullest extent practicable during the planning, construction, and operation of highway facilities and projects. FHWA procedural guidelines specify that system planners should assign preferences for highway projects to corridors that have the least number of interconnected wetlands and wetland-related resources (5).

Because of the complex nature of the wetland ecosystem, avoidance is the best method of conforming to stated FHWA goals. However, this is not always possible, and wetlands continue to disappear at an alarming rate because of changes in land use. Therefore, alternative amelioration measures must be considered, including replacement of natural habitat with constructed wetlands.

The need for fill for raising road grades and building interchanges during road construction provides an opportunity to design and construct artificial wetlands. Such was the case during addition of a second roadway to Interstate 29 in North Dakota in 1975. Unlike traditional borrow pits, which are rectangular and uniformly deep [≈2 m (6 ft)], the I-29 ponds were designed as potential marsh habitat (6). The North Dakota State Highway Department (NDHD) also constructed artificial wetlands along I-94 and US-2 in 1976 and 1977. Along I-29 alone, 44 ha (109 acres) of wetlands was excavated.

During 1979 and 1980, 13 artificial wetlands along I-29, 3 along US-2, and 2 along I-94 were monitored. Comparisons were made of waterfowl and marsh-bird use and macroinvertebrate and aquatic-plant diversities and densities of artificial wetlands versus nearby natural-basin wetlands (7). Waterfowl use of artificial wetlands was only slightly less than that observed on natural wetlands and that reported in previous wetland studies. The value of artificial wetlands as wildlife habitat has been reported on at previous TRB committee meetings, and more detailed information is available elsewhere (7).

Although use of the artificial wetlands by waterfowl was closely monitored, the diversity of nongame bird species and the densities of some of these species were also recorded. The following recommendations for artificial-wetland design and construction are based on the results of our 1979-1980 fieldwork and our knowledge of the habitat requirements of target wildlife species.

GENERAL CONSIDERATIONS

Waterfowl are migratory species; they depend on specific kinds of aquatic habitat at different times of the year and during different phases of their reproductive cycle (8,9). Therefore, the design of the artificial wetland depends partly on the latitude at which it is constructed. Much of North Dakota is located within the prairie pothole region, that area of the United States and southern Canada that covers approximately 10 percent of the land area but produces more than 50 percent of the annual duck population (10). Although the specific recommendations in this paper pertain to artificial wetlands designed to support nesting waterfowl, it is assumed that nongame species will also benefit.

SPECIFIC DESIGN FEATURES

Size

The size of natural-wetland basins is highly variable. The artificial wetlands examined in this study ranged in size from 0.2 to 2.7 ha (0.5-7 acres). Thus, no single size is recommended. Instead, size can be determined by construction requirements for fill provided that guidelines on depth, bank gradient, and detention time are considered. Excavation of a 0.4-ha (1-acre) wetland provides 5,700 m^3 (7,500 yd^3) of fill; 19,000 to 23,000 m^3 (25,000 to 30,000 yd^3) of fill was obtained for each of the three 1.6-ha (4-acre) artificial wetlands constructed at the Lincoln Interchange on I-29.

Wetland size is positively correlated with wetland permanence; i.e., smaller wetlands [<0.6 ha (1.6 acres)] tend to be temporary or seasonal whereas larger ones [0.7 to ≥40.5 ha (1.7 to ≥100 acres)] are semipermanent, although there is a high degree of overlap in size among permanence classes (11,12). Again, different classes of wetlands vary in importance during different phases of the breeding cycle.

The importance of semipermanent ponds, where ducks and geese congregate in autumn, is obvious to hunters; however, temporary and seasonal shallow wetlands, which tend to dry out by mid to late spring, also serve a vital function in supporting waterfowl populations (13). In early spring hens depend on high levels of protein and calcium to lay viable eggs; aquatic invertebrates in shallow wetlands provide those nutrients. Thus, to maximize waterfowl productivity, artificial wetlands of varying sizes should be excavated. As an alternative, a series of small satellite ponds could be excavated around a large, central artificial wetland. One satellite-wetland design used successfully at lake Agassiz Refuge, Minnesota, is the donut: soil is excavated and bulldozed into the middle of a circle to create a moat with a central island. Such wetlands provide a food source, a nesting site, and access to larger wetlands for brood rearing.

Shape

Wetland shape is important because waterfowl breeding pairs need isolated areas for courtship, feeding, and other activities. The amount of shoreline that each pair requires is related to the ability of one pair to see other pairs, which in turn is related to wetland shape. Artificial wetlands along I-29 were of three shapes: triangular, rectangular, and hexagonal (see Figures 1 to 3). Waterfowl pair density was significantly higher on irregular-polygon-shaped (hexagonal) wetlands than on triangular or rectangular ones in both 1979 and 1980 [Mann-Whitney U = 0.0005 for 1979 and 0.047 for 1980 (14)]. As with size, decisions regarding wetland shape must be made with due consideration of recommendations on bank gradient and soils, which are discussed in the following paragraphs.

Depth and Bank Gradient

Several processes are affected by wetland depth and bank gradient: water permanence, invasion and survival of aquatic vegetation, and habitat selection and feeding by waterfowl. The relationship between depth and water permanence was referred to in the previous discussion on wetland size. Periodic drought in North Dakota ensures that all but the deepest wetlands will dry out occasionally. Naturally fluctuating water levels help control aquatic-plant density, which precludes the need for intensive management and water-level manipulation.

Wetland depth has a marked effect on aquatic-plant density. By planning artificial wetlands with varying depth, the ratio of emergent vegetation to open water can be controlled. Emergent species, such as cattail (*Typha* spp.) and bulrush (*Scirpus* spp.), usually do not sprout and grow in water deeper than 60 cm (24 in.) (15-17). Studies have indicated that species density and diversity are greatest on wetlands with a 50:50 ratio between total emergent vegetation (both shoreline and center vegetation) and open water (18).

Bank gradients with a 10:1 to 20:1 slope will allow emergent vegetation to establish itself, which will further isolate shoreline habitat of waterfowl pairs. Gently sloping shorelines also provide feeding habitat for many species of shorebirds (7).

In contrast to emergent species, pondweeds (such as *Potamogeton* spp.) invade new wetlands to depths in excess of 60 cm, depending on bottom soils and water transparency (17,19). Waterfowl feed on these plants, so their accessibility to ducks is important. Plants in deep water might still provide food for diving ducks, but dabbling ducks, which tip over to feed, would not be able to reach them. In any case, maximum depth should not exceed 1 m (3 to 4 ft).

Plans can also include a central island in the wetland that provides a safe nesting site. One problem associated with artificial-wetland construction, particularly when the wetlands enhance rather than replace habitat, is that they attract predators from surrounding areas. Safe nesting sites can reduce high levels of predation. Several small man-made islands in prairie wetlands were studied by Johnson et al. (20). Islands were constructed in dry wetlands by using rocks and soil from the marsh bottom. Construction costs averaged $50 per island, and construction time for islands that ranged in size from 0.0002 to 0.01 ha (0.0005 to 0.02 acres) was 1 to 2 hr. Densities of mallards (*Anas platyrhynchos*) on islands averaged 135 nests/ha compared with 0.03 nests/ha on adjacent upland habitat (20). Predators can be excluded from the islands successfully except during periods of low water.

Vegetation

The artificial wetlands studied in North Dakota were not seeded with aquatic vegetation. Chance invasion occurred within 1 year of construction (6). However, after 5 years plant diversity remained lower in artificial wetlands than in nearby natural-basin wetlands. It appears that the conditions for plant invasion and colonization are more important than actual seeding within the wetlands (7).

In contrast, grading and seeding areas adjacent to wetlands with dense nesting cover are important to provide adequate upland habitat. Recommendations

Figure 1. Triangular-shaped artificial wetland with surface area of approximately 1.48 ha (3.7 acres).

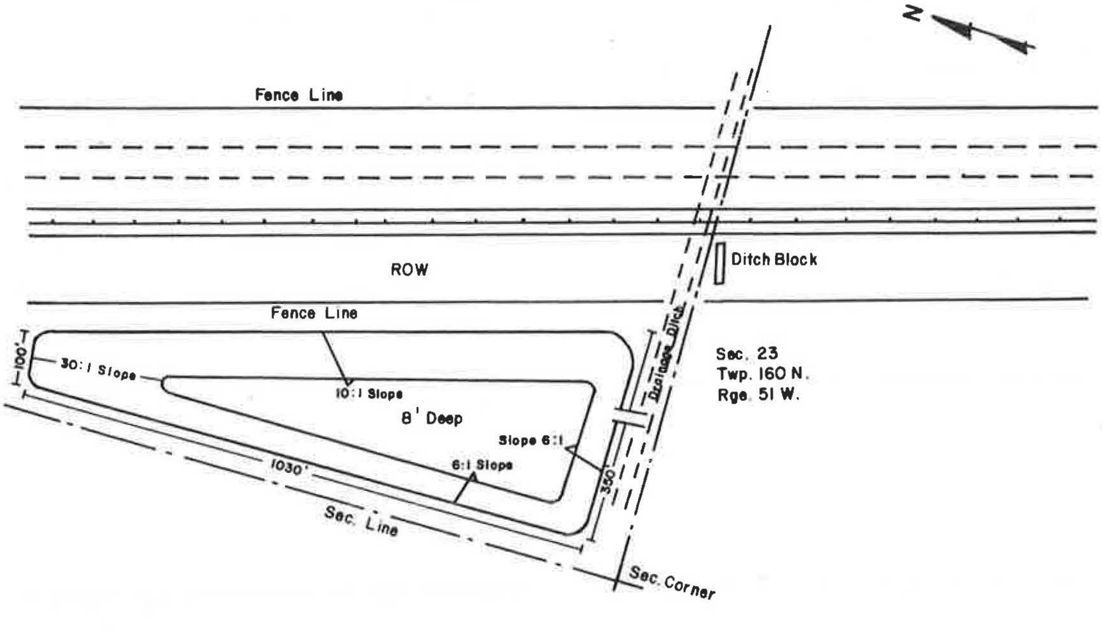


Figure 2. Rectangular artificial wetlands with mean surface area of approximately 0.6 ha (1.54 acres).

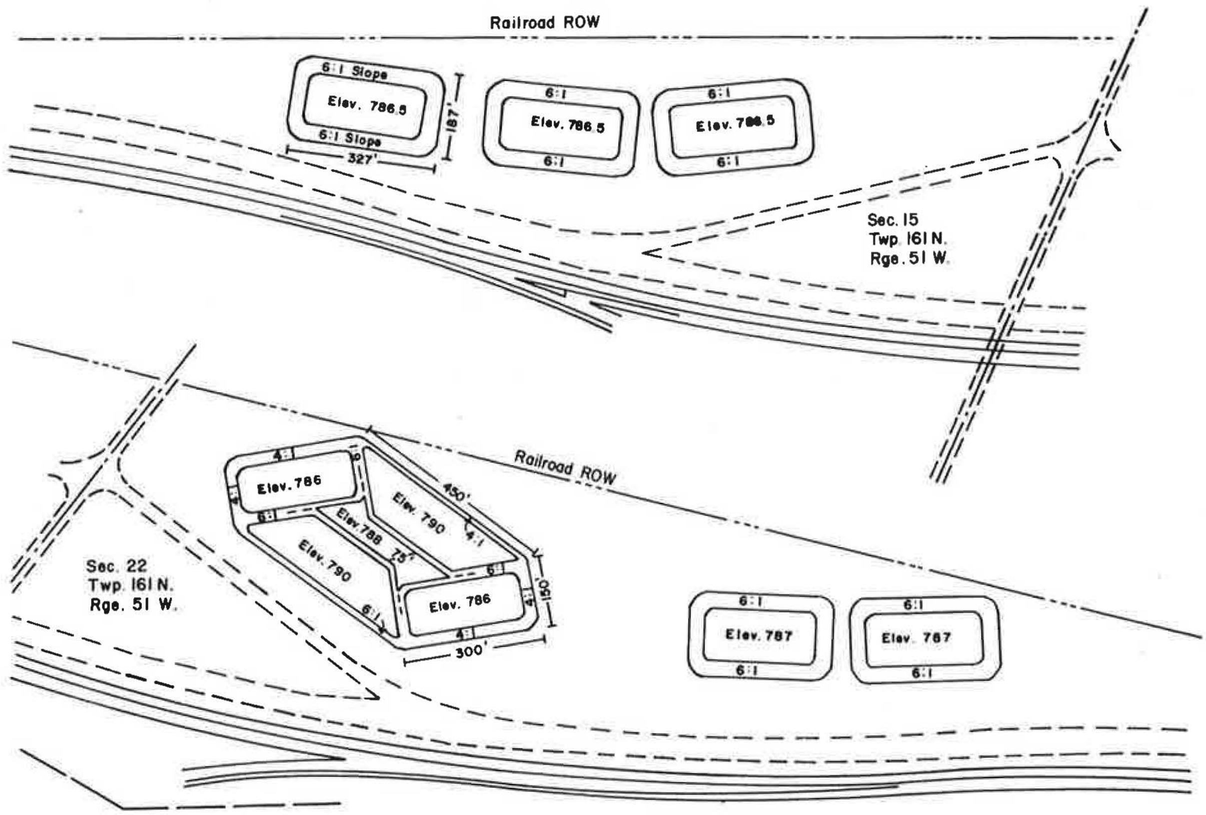
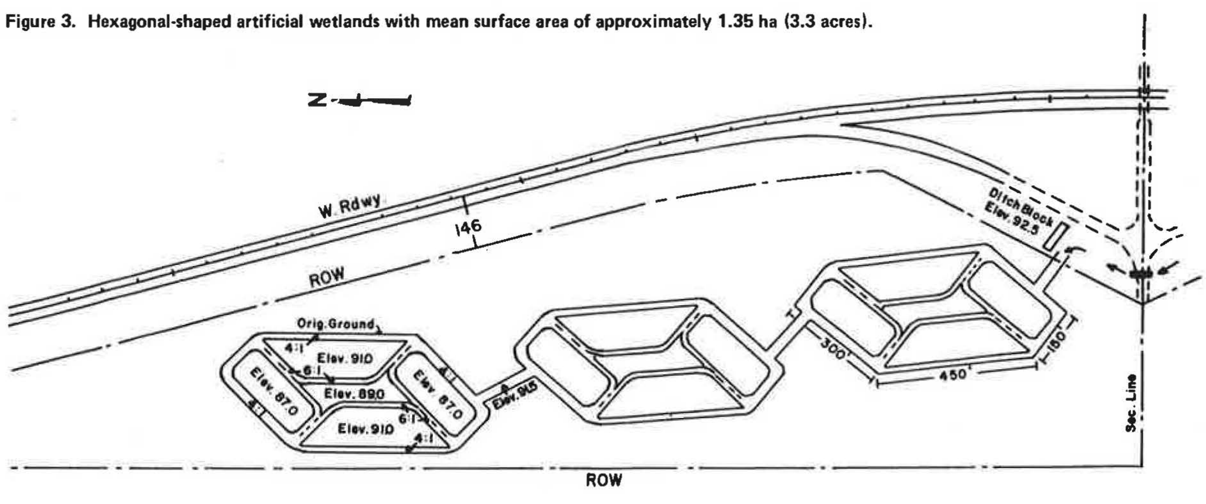


Figure 3. Hexagonal-shaped artificial wetlands with mean surface area of approximately 1.35 ha (3.3 acres).



for seeding mixtures and land use management are readily available in the literature (21-24).

Soils

Soils play two crucial roles in wetland development: holding water in the wetland and providing an adequate substrate for plant growth. After excavation most wetland bottoms must be sealed with a clay (bentonite) layer. Deep, sandy-bottom artificial wetlands along US-2 dried out much earlier in the season than expected (7).

Topsoil removed during excavation should be set aside and respread in the wetland. In North Dakota both plant and macroinvertebrate densities were lower in artificial wetlands with clay bottoms than in those respread with topsoil (7). There are some problems associated with stockpiling muck (substrate material removed from natural wetlands). If this material is to be used in the new artificial wetland, it should be stockpiled in upland areas, not in other wetlands. Muck should be respread within 4 weeks to avoid deterioration from freezing, desiccation, and decomposition. If only a limited amount of topsoil is available, other organic matter, such as hay, should be added to stimulate development of a detritus food chain (25).

Water Quality and Hydrology

Densities and diversities of aquatic plants and macroinvertebrates can be affected by water clarity, pH, and salinity (total ionized material). Clarity becomes increasingly important with depth; turbidity prevents sunlight from reaching aquatic plants growing in deep water. Suspended matter also depletes the amount of oxygen available to macroinvertebrates and can physically abrade invertebrates. If the surrounding upland areas are properly seeded, runoff will be reduced and turbidity controlled. Artificial wetlands examined in North Dakota were clear to depths of 1 to 2 m (3 to 7 ft) (7).

The pH of the water will certainly have an impact on the type of vegetation that the wetland will support. The species composition of an acid bog differs markedly from that of an alkaline wetland. Water samples collected from artificial wetlands in North Dakota ranged in pH from ≈8 to 10. Although pH was not a limiting factor for wetlands excavated near highways in North Dakota, low pH and the presence of heavy metals in the soils have been problems in wetlands constructed on surface-mined land. To overcome this problem, attempts have been made to seal reclamation wetlands with clay and even with plastic liners to prevent leaching of acid soil and deleterious substances.

Total ionized material in water (measured as electrical conductance) has effects on plant species composition similar to those of pH. Electrical conductance is an indicator of wetland salinity (25, 26). In states where large amounts of salt are used to control ice on roads, artificial wetlands close to highways may be affected by runoff (27). Salinity may increase to levels that are toxic to plant and invertebrate life. Salt runoff is an important consideration in designing a water-inflow system for an artificial wetland. Many artificial wetlands along North Dakota highways were constructed on excess parcels of land near interchanges. A channel was excavated from the wetland to the right-of-way. A ditch block was placed in the right-of-way at the head of the channel to shunt water collected in the right-of-way to the wetlands. Acceptable velocity within the channel was determined to be approximately 0.37 m/sec (1.2 ft/sec). By the Manning equation,

$$V = (1.49/n)\, R^{2/3}\, S^{1/2} \qquad (1)$$

where

- n = empirical roughness = 0.07,
- R = hydraulic radius of the channel = 2 x 10 ft, and
- S = estimated slope = 0.002 (0.2-ft drop/100 ft).

Higher scour-producing velocities in channels or culverts could be decreased by energy-dissipating designs or structures (i.e., multiple culverts or walls to spread flow). If water volume is too low, dilution may not be sufficient to prevent high salt concentration (27). Because salt is not used on North Dakota highways, this problem was not encountered in this research.

The ditch block and channel system, together with a high water table and normal precipitation, ensured adequate water levels in the North Dakota artificial wetlands. Only in 1980, a year of severe drought, did the wetlands dry out early in the season. The respective detention times for hexagonal, rectangular, and triangular artificial wetlands were estimated to be 13, 17, and 18 days, based on respective total volumes of 273,121, 363,518, and 297,267 ft^3 and an estimated daily flow rate of 21,600 ft^3 (annual precipitation = 18 in.).

Several methods are available for maintaining preferred water levels in wetlands in areas where natural hydrologic conditions are less than ideal. Detailed plans of various types of artificial wetlands that include devices for water-level manipulation have been prepared by the Atlantic Waterfowl Council Committee on Habitat Management and Development (28). Extensive postconstruction management of artificial wetlands will increase their cost, however, and therefore cannot be included in the plans unless sufficient funds and personnel are available.

COST FACTORS

It is not uncommon for the costs of managing intensively developed waterfowl areas to exceed $10 per acre per year, particularly in diked marshes with elaborate programs of water-level manipulation (9). Actual cost figures for construction of multisided, variable-depth artificial wetlands are not available; however, the cost probably exceeds that of traditional borrow pits because additional fine grading is required. In addition, earthmoving equipment does not have the maneuverability that is necessary and special machinery must sometimes be used.

Although setting aside topsoil should not increase construction costs, respreading it might because respreading is a time-consuming task, particularly when the bottom of the borrow area is wet, as it often is.

If nesting islands, as described previously, are constructed at the time of initial wetland construction, they can probably be completed for less than the $50 per island estimate given by Johnson et al. (20).

As indicated, there has been little or no maintenance cost for the North Dakota wetlands studied. For wetlands that require more intensive postconstruction management, costs could be defrayed by leasing the wetlands to a wildlife or sports club. State regulations may prohibit hunting near major highways and within 0.5 km (0.25 mile) of an inhabited dwelling. Therefore, hunting laws should be considered before site selection if plans include using the wetlands for this purpose.

Regardless of whether the artificial wetlands benefit hunters, they usually do enhance the aesthetic value of the highway for drivers (29).

FUTURE RESEARCH NEEDS

Research on artificial wetlands in North Dakota continues. Phase 2, which began in 1981, should provide additional information on plant and invertebrate abundance in new wetlands. The purpose of phase 1, on which this paper is based, was to attempt to evaluate constructed ponds as a means of replacing natural-wetland habitat affected by highway projects. Of course, it is not possible to do that unless one has determined the value of what has been replaced. Therefore, what is needed is a long-term study of a natural wetland before highway construction and, later, an evaluation of the artificial wetland that replaced it. Only then can we determine how well habitat can be reconstructed.

Finally, many of the design features recommended here were not included in the North Dakota artificial wetlands; rather, these recommendations are made in response to shortcomings perceived in those wetlands. Therefore, a series of state-of-the-art artificial wetlands should be constructed and studied intensively by using accepted wetland evaluation techniques. A study of several artificial wetlands in which one or more design features per wetland have been manipulated may yield information on wetland ecosystems that is not available now.

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REFERENCES

1. Council on Environmental Quality. Our Nation's Wetlands: An Interagency Task Force Report. U.S. Government Printing Office, 1978.
2. U.S. General Accounting Office. Better Understanding of Wetland Benefits Will Help Water Bank and Other Federal Programs Achieve Wetland Preservation Objectives. U.S. Government Printing Office, 1979.
3. C.B. Davis, J.L. Baker, A.G. van der Valk, and C.E. Beer. Prairie Marshes as Traps for Nitrogen and Phosphorous in Agricultural Runoff. In Selected Proceedings of the Midwest Conference on Wetland Values and Management (B. Richardson, ed.), Minnesota Water Planning Board, Minneapolis, 1981, pp. 153-164.
4. G.L. Oberts. Impact of Wetlands on Watershed Water Quality. In Selected Proceedings of the Midwest Conference on Wetland Values and Management (B. Richardson, ed.), Minnesota Water Planning Board, Minneapolis, 1981, pp. 213-226.
5. P.A. Erickson and G. Camougis. Interim Procedural Guidelines: Volume I--Highways and Wetlands. FHWA, Implementation Package FHWA-IP-80-11, 1980.
6. D. Nilson. Roadside Management and Wetland Development Along North Dakota's Highways. North Dakota Outdoors, Vol. 40, No. 1, 1977, pp. 23-25.
7. J.A. Rossiter and R.D. Crawford. Evaluation of Constructed Ponds as a Means of Replacing Natural Wetland Habitat Affected by Highway Projects in North Dakota. FHWA, Rept. FHWA-ND-RD-(2)-79A, 1981.
8. F.H. Kortright. The Ducks, Geese, and Swans of North America. American Wildlife Institute, Washington, D.C., 1943.
9. A.T. Cringan. Waterfowl Areas: Development and Management. In A Manual of Wildlife Conservation (R.D. Teague, ed.), Wildlife Society, Washington, D.C., 1971, pp. 82-84.
10. A.G. Smith, J.H. Stoudt, and J.B. Gollop. Prairie Potholes and Marshes. In Waterfowl Tomorrow (J.P. Linduska, ed.), U.S. Government Printing Office, 1964, pp. 39-50.
11. R.E. Steward and H.A. Kantrud. Classification of Natural Ponds and Lakes in the Glaciated Prairie Region. U.S. Fish and Wildlife Service, Resources Publ. 92, 1971.
12. L.M. Cowardin, D.S. Gilmer, and L.M. Mechlin. Characteristics of Central North Dakota Wetlands Determined from Sample Aerial Photographs and Ground Study. Wildlife Society, Bull. 9, 1981, pp. 280-288.
13. G.L. Krapu. The Importance of Shallow Wetlands to Waterfowl Production. North Dakota Outdoors, Vol. 44, No. 5, 1981, pp. 2-4.
14. R.R. Sokal and F.J. Rohlf. Biometry. W.H. Freeman, San Francisco, 1969.
15. F.C. Bellrose. Duck Food Plants of the Illinois River Valley. Illinois Survey Bull. 21, No. 8, 1941, pp. 237-280.
16. T.F. Hall, W.T. Penfound, and A.D. Hess. Water Level Relationships of Plants in the Tennessee Valley. Journal of the Tennessee Academy of Science, Vol. 21, 1946, pp. 18-59.
17. C.W. Dane. Succession of Aquatic Plants in Small Artificial Marshes in New York State. New York Fish and Game Journal, Vol. 6, 1954, pp. 57-76.
18. M.W. Weller and L.H. Fredrickson. Avian Ecology of a Managed Glacial Marsh. Living Bird, Vol. 12, 1974, pp. 269-291.
19. D.J. Lathwell, H.J. Mulligan, and D.R. Bouldin. Chemical Proper Ties and Plant Growth in Twenty Artificial Wildlife Marshes. New York Fish and Game Journal, Vol. 16, 1969, pp. 158-183.
20. R.F. Johnson, Jr., R.O. Woodward, and L.M. Kirsch. Waterfowl Nesting on Small Man-Made Islands in Prairie Wetlands. Wildlife Society, Bull. 6, 1978, pp. 240-243.
21. G.F. Martz. Effects of Nesting Cover Removal on Breeding Puddle Ducks. Journal of Wildlife Management, Vol. 31, 1967, pp. 236-245.
22. R.B. Oetting and J.F. Cassel. Waterfowl Nesting in Interstate Highway Rights-of-Way in North Dakota. Journal of Wildlife Managment, Vol. 35, 1971, pp. 774-781.
23. H.F. Duebbert and H.A. Kantrud. Upland Duck Nesting Relative to Land Use and Predator Reduction. Journal of Wildlife Management, Vol. 38, 1974, pp. 257-265.
24. L.D. Voorhees and J.F. Cassel. Management of Highway Rights-of-Way for Waterfowl in North Dakota. TRB, Transportation Research Record 674, 1978, pp. 43-47.
25. A.F. Linde and M. Morehouse. To Feed Wildlife . . . Fertilize Game Foods? Wisconsin Conservation Bull. 31, No. 6, 1966, pp. 16-17.
26. T.J. Dwyer. Waterfowl Breeding Habitat in Agricultural and Non-Agricultural Land in Manitoba. Journal of Wildlife Management, Vol. 34, 1970, pp. 130-136.
27. R.E. Hanes, L.W. Zelazny, and R.E. Blaser. Effects of De-Icing Salts on Water Quality and Biota. NCHRP, Rept. 91, 1970.
28. Atlantic Waterfowl Council Committee on Habitat Management and Development. An Illustrated Small Marsh Construction Manual Based on Standard Designs. Vermont Fish and Game Service, Montpelier, 1959.
29. C. Carlozzi. Enhancement of Ecological and Aesthetic Values of Water Associated with Interstate Highways. Water Resources Research Center, Univ. of Massachusetts, Amherst, 1971.

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Impact of Bridging on Floodplains

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Three bridge sites in central Florida (I-4 and Padgett Creek, US-17-92 and Shingle Creek, and US-192 and Shingle Creek) were selected to study (a) the feasibility of treating highway runoff by using floodplains beneath these bridges and (b) the impact of bridging on biological diversity and productivity. Soil and plant samples were collected from bridge areas affected by highway runoff as well as control areas upstream and downstream of the bridge areas. These samples were analyzed for extractable heavy metals (lead, zinc, iron, copper, nickel, chromium, and cadmium) to study the capacity of floodplain areas to retain these metals. Soil samples collected from the bridge areas contained significantly higher concentrations of metals, particularly lead, than the control areas. Metals were fixed in the soil and could be released to the surrounding body of water by highly acidic environments, organic chelating agents, and erosion processes. Plants were also surveyed to determine dominant species, changes in diversity, and productivity at the bridge and control areas. In addition, bioassay experiments were designed to examine the potential impact on algal production of mixing highway bridge runoff and water from adjacent streams. At all sites surveyed there were more individuals of common plant species in bridge areas than in control areas. Bridge areas, however, appeared to be dominated by fewer species than control areas.

Currently, there are increasing legal and environmental concerns that must be considered in building highway bridges and fills on floodplains. Federal, state, and local authorities will intercede in any land use decisions that adversely affect these regulated areas.

Wanielista et al. (1) concluded a study for the Florida Department of Transportation (FDOT) that indicated that most of the metals from stormwater runoff were being retained in the soils adjacent to highway bridges. Yousef et al. (2) have presented data showing that heavy-metal concentrations, particularly zinc (Zn), lead (Pb), chromium (Cr), nickel (Ni), copper (Cu), iron (Fe), and cadmium (Cd), were higher in bottom sediments and biota collected from Lake Ivanhoe beneath bridges equipped with scupper drains than in samples collected beneath bridges without scupper drains. From these studies, it is conjectured that soils under bridges (floodplains) are possible sinks of metals and macronutrients. Consequently, floodplains may be used to remove heavy metals and nutrients from bridge runoff provided there is no sign of adverse effect on the productivity and diversity of associated plant communities.

A research study was initiated to examine the water, soil, and plant life of floodplains adjacent to highway bridge sites and the impact on them of highway bridge runoff. The specific objectives of the study were

1. To analyze soil samples collected from floodplains beneath selected highway bridges (samples were collected from areas exposed directly or indirectly to bridge runoff for comparison with samples collected from control areas, and retention and release of heavy metals were determined to investigate the magnitude and extent of heavy-metal accumulation in floodplains);
2. To survey existing plant communities in floodplains surrounding bridge sites and to examine changes in the productivity and diversity of plants; and
3. To evaluate the toxicity of bridge runoff on algal biomass by means of laboratory studies.

This paper summarizes the study findings, which are presented in detail elsewhere (3).

SAMPLING SITES

Bridge sites located at I-4 and Padgett Creek, US-17-92 and Shingle Creek, and US-192 and Shingle Creek in central Florida were selected for the study. The average daily traffic at these sites during 1981 was 28,584 (I-4), 7,900 (US-17-92), and 24,142 (US-192). Soils and plants from different areas surrounding the bridge sites were collected for analysis (see Figures 1, 2, and 3).

Figure 1. Sample sites and plant identification plots at I-4 and Padgett Creek.

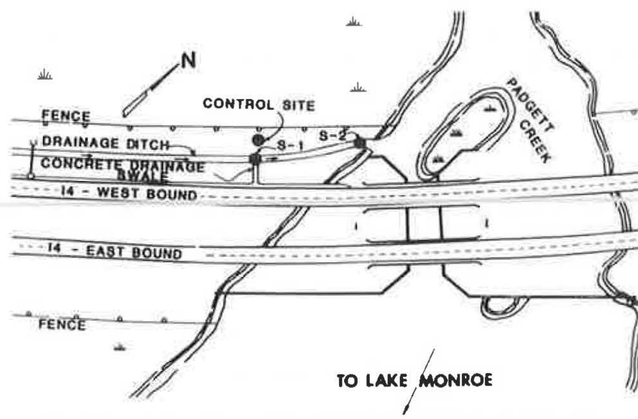
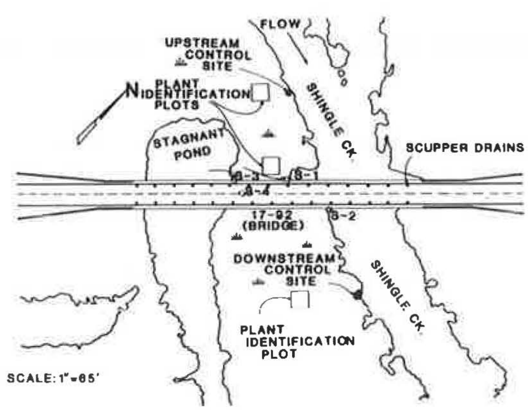


Figure 2. Sample sites and plant identification plots at US-17-92 and Shingle Creek.



SOIL ANALYSIS

The physical characteristics of soil samples taken from the top 1 in. of soil revealed that the study sites were similar (see Table 1). However, the organic content in soils from I-4 was higher than in soils from US-17-92 or US-192. The major fractions of these soils were silt and fine sand. It is realized that floodplain soils are typical of geographic areas covered with sediment and debris deposited during high flood periods.

U.S. Department of Agriculture (USDA) soil surveys state that the I-4 site is bluff sand clay loam and that the US-17-92 and US-192 sites are fine sand. Loam is defined as a soil material that is 7 to 27 percent clay particles, 28 to 50 percent silt particles, and less than 52 percent sand particles. On the other hand, sandy soils contain less than 10 percent clay and 85 percent or more sand particles. Sediment deposits on floodplains during high water appear to increase the clay and organic fractions in the topsoil unless erosion occurs in some areas due to steep slopes and high water velocity.

METAL UPTAKE AND RELEASE BY SOIL

Heavy metals were extracted by six normal nitric acid and analyzed for Pb, Cd, Zn, Cu, Fe, Ni, and Cr by using a plasma spectrometer. The results showed that soils from floodplain areas that were beneath bridges and were directly exposed to highway runoff contained significantly higher concentrations of heavy metals than soils from control areas. For example, the average concentrations of Pb, Cd, Zn, Cu, and Fe in soils from bridge areas at I-4 and Padgett Creek were significantly higher than concentrations in soils from control areas (the t-test probability was 95 percent or better) (see Table 2).

There were higher concentrations of all metals tested in soils from the I-4 site than in soils from the US-17-92 and US-192 sites. This may have resulted from the higher average daily traffic and organic content at the I-4 site. In addition, the USDA surveys suggest that the I-4 site may have higher clay content, in addition to higher organic content, than the US-17-92 and US-192 sites. Clay minerals have selective affinity for heavy metals and can absorb large amounts of organics (4). They may be used as possible sinks for nutrients, toxic metals, and organic contaminants (5).

By averaging the metal content in bridge areas and control areas for each study site, it was concluded that lead was significantly higher in bridge areas than in control areas at all sites (>95 percent probability). Lead is an indication of vehicle-related contaminants. The average lead uptake, in micrograms per gram of oven-dried soil, ranged from 121 to 886 at the I-4 site, 5.8 to 186 at the US-17-92 site, and 17.2 to 37.7 at the US-192 site.

Laboratory studies showed that lead uptake by composite soil samples from all sites was increased by increasing the pH from 5.0 to 9.0. Soils experience a decrease in pH values, dissolved oxygen content, and redox potential due to flooding. The redox potential before flooding may be greater than +200 mV, but it decreases to between 0 and -200 mV during flooding (6). These conditions exist during anaerobic periods in the sediments. Therefore, there may be less absorption of heavy metals or a release of pollutants during anaerobic periods when environmental conditions become more favorable for desorption of these contaminants.

As a result of batch experiments in the laboratory, it was concluded that distilled water, highway runoff water, and calcium chloride solutions do little if anything to make heavy metals that have become incorporated with soil in floodplains soluble again. These metals appeared to be tightly fixed to the soils and were partly released under acidic environment by HCl or HNO_3 solutions. In addition, extractant solutions of oxalic acid, diethylene-tri-amine-penta-acetic acid (DTPA), and ammonium acetate were tested to determine metal release by reducing environments, organic complexing agents, and ion exchange reactions. Oxalic acid and DTPA solutions

Figure 3. Sample sites and plant identification plots at US-192 and Shingle Creek.

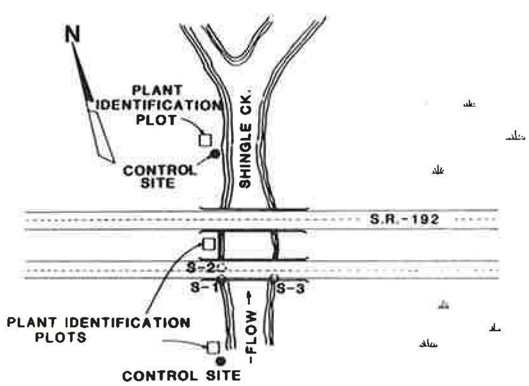


Table 1. Average soil characteristics for samples collected from floodplains beneath bridges.

	Moisture (%)		Loss on Ignition (%)		Particle Size Distribution	
Bridge Site	Bridge Areas	Control Areas	Bridge Areas	Control Areas	Effective Size (mm)	Uniform Coefficient
I-4 and Padgett Creek	33.0	26.2	6.2	11.1	0.10	2.4
US-17-92 and Shingle Creek	21.6	29.2	3.2	4.8	0.08	2.5
US-192 and Shingle Creek	17.3	24.6	3.2	6.6	0.08	2.7

Table 2. Significance of difference between heavy-metal concentrations in soil samples collected from I-4 and Padgett Creek.

	No. of Observations		Concentration of Metals (μg/g oven-dried soil)				
			Control		Bridge		
Metal	Control	Bridge	Mean	SD	Mean	SD	t-Test Probability
Pb	7	13	121.0	82.5	886.0	960.0	98.6
Cd	7	14	0.12	0.31	1.0	1.0	99.3
Zn	7	14	90.4	65.6	292.0	254.0	99.7
Cu	7	14	7.3	4.8	30.4	26.7	99.3
Fe	6	12	3318.0	1250.0	5746.0	3472.0	95.3
Ni	5	10	8.8	3.8	10.8	6.9	50.8
Cr	7	14	16.8	5.5	15.6	8.4	31.2

produced the greatest release of metals (see Table 3). It appears that heavy metals in the floodplain soils will be released to the adjacent body of water by reducing environments, organic complexing, and erosion. Humic substances (humic and fulvic acids) are the most abundant organic compounds in soils and sediments (7). These acids combine with different divalent cations such as Pb, Cu, Ni, Cr, Cd, Fe, and Zn, and the solubilities of these complexes are pH-dependent. Reuter (8) has reported that water-soluble substances such as fulvic acids mobilize pollutant metals but that humic acids in the soil can form complexes and immobilize metals entering the soil. Thus, these metals may become unavailable for biological metabolism.

METAL UPTAKE BY PLANTS

Plants in floodplains die and decay, and their constituents are retained by the soil or flushed into adjoining streams in either particulate or dissolved form. Plants are also consumed by insects, birds, and small animals. It is therefore important to evaluate the extent of heavy-metal uptake by a given plant that grows in the floodplain area. Plants accumulate heavy metals through airborne particulates, highway runoff, and uptake from sediments. Lagerwerff (9, p. 583) has reported that plants grown 200 m from a busy highway could have more than 40 percent Pb, Cd, and Zn in their leaves due to areal contamination.

During this study, plant samples were composited (leaf and stem) and were thoroughly rinsed with distilled water to minimize airborne particulate contamination. It is reasonable to assume that most of the heavy metals detected in these plants are taken up from sediments or highway runoff. In addition, differences between metal content in plants collected from bridge areas and control areas are mainly due to bridge runoff and accumulation of metals in the soil, assuming that the areal deposition is similar in all areas and was minimized by washing the plants before analysis.

Data on the uptake of heavy metals by selected plants from floodplains beneath bridges in the three study sites are given in Table 4. It is interesting to note that the lead concentrations in all species of plants collected from bridge areas affected by highway drainage were higher than lead concentrations in similar species collected from control areas. Lead concentration in elderberry plants at the I-4 site averaged 30 μg/g for bridge areas and 19 μg/g for control areas. The lead content of panic grass at the US-17-92 site averaged 26 μg/g for bridge areas and only 8 μg/g for control areas. At the US-192 site the lead content of panic grass averaged 24 μg/g for bridge areas and 7.8 μg/g for control areas.

The increase in Pb concentration in plants adjacent to bridge areas suggests the lead is a useful indicator of contamination from vehicle-related activities. Lead is not a required nutrient for plant growth and consistently averaged higher in plants from bridge areas than in plants from control areas at all sites studied.

Plants collected throughout this study showed the following average ranges of metal concentration:

Metal	Avg Concentration (μg/g)
Cd	0.04-0.32
Zn	60-103
Cu	5.1-10.5
Fe	91-244
Pb	7.8-34
Ni	2.1-5.6
Cr	1.3-3.7

Most of the plants collected from the US-17-92 control areas were dog fennel (*Euphatorium compositifolium*), which showed the highest average Cu content among all plants tested. Of course, different plants exhibit different affinities for the various trace metals available to them. Dog fennel appeared to concentrate more copper than panic grass or elderberry seedlings.

Table 3. Lead extractions from composite soil samples.

Extraction Solution	I-4 and Padgett Creek		US-17-92 and Shingle Creek		US-192 and Shingle Creek	
	Amount (μg/g soil)	Percent	Amount (μg/g soil)	Percent	Amount (μg/g soil)	Percent
1 normal NaAc	18	3	14	11	3	10
0.05 molar DTPA	62	10	65	52	49	100
0.15 molar oxalic acid	130	21	35	28	19	61
0.1 normal HCL	28	5	250	100	2	100
0.1 normal $CaCl_2$	-[a]	0	2	2	-[a]	0

[a]Below detection limit.

Table 4. Uptake of heavy metals by selected types of plants.

Site	Plant	Sampling Area	Average Concentration (μg/g oven-dry soil)							No. of Observations
			Zd	Zn	Cu	Fe	Pb	Ni	Cr	
I-4	Elderberry	Bridge	0.12	60	9.2	142	30	2.7	2.7	14
	Elderberry	Control	0.07	57	9.3	122	19	3.4	3.4	7
	Shepherd's needle	Control	0.19	80	11.4	262	34	11.2	4.3	4
US-17-92	Panic grass	Bridge	0.10	69	5.9	268	26	2.9	2.0	13
	Panic grass	Control	0.34	49	2.3	84	8	2.8	1.2	8
	Dog fennel	Control	0.26	143	20	171	24	3.1	3.5	7
US-192	Panic grass	Bridge	0.09	58	8.0	147	24	0.1	1.8	10
	Panic grass	Control	0.04	71	5.1	91	7.8	2.5	1.3	8
	Dog fennel	Bridge	0.36	132	17.1	119	24.8	1.6	2.1	6
	Sea myrtle		0.28	123	13.9	88	33	1.2	1.3	3

PLANT DIVERSITY

Plants in the control areas and bridge areas of the study sites were surveyed three times between October 1981 and January 1982 (3) in an attempt to detect the effects of bridges on biological productivity and plant diversity. It was of great interest to researchers to isolate any differences between plants in bridge areas and plants in control areas. This was not an easy task because it was impossible to survey all plants in the area and some plants exhibited significant seasonal fluctuations.

Counts of plant species at bridge areas and control areas near I-4 and Padgett Creek showed that there were five dominant plant species in these study areas. The bridge areas appeared to contain greater amounts of pokeberry weed (*Phytolacca americana*), willow (*Salix carolineana*), elderberry shrub (*Sambuccus simpsonii*), and begger ticks weed (*Bidens pilosa*) than the control areas. For example, pokeberry occurred 9 times at the bridge area and 3 times in the upstream control area and did not occur at all in the downstream control area. Elderberry occurred 96 times at the bridge area, 16 times in the upstream control, and 12 times in the downstream control during October 1981. On the other hand, the common reed (*Phragmites australis*) was present in the control areas and absent from the bridge areas during the months of October and November 1981.

Similarly, the dominant plant species at the bridge areas and control areas near US-17-92 and Shingle Creek were counted periodically in 1981-1982. The results showed seven dominant species. Only three species were present in the drainage area of the bridge runoff at US-17-92 and Shingle Creek during October and November 1981: dog fennel, sea myrtle (*Baccharis halimifolia*), and panic grass (*Panicum* sp.). However, the dog fennel disappeared during January 1982. In each case where there was a plant common to both the bridge and control areas, the number was significantly higher in the bridge area. Dog fennel occurred 350 times in the bridge area, 10 times in the upstream control area, and 15 times in the downstream control area (October 1981).

Ten species of existing plans existed during the October 1981 through January 1982 period at the control areas and bridge areas of US-192 and Shingle Creek. Only 4, 7, and 8 species were present in the bridge area, the upstream control area, and the downstream control area, respectively. During the month of January, all plants except sea myrtle disappeared from the bridge area. In addition, where there was a plant common to both bridge and control areas, the numbers were significantly higher in the bridge area. In October 1981 dog fennel occurred 15 times in the bridge area, did not occur in the upstream control area, and occurred 10 times in the downstream control area.

PLANT PRODUCTION

A survey of existing plants in the study areas suggested that there were higher numbers of individual plants of any given species in the bridge area than in the control area of each site. Values for the standing crop of panic grass at the US-17-92 and US-192 sites from November 1981 through March 1982 are given below:

	Avg Standing Crop (g/m^2)	
Site	Bridge Areas	Control Areas
US-17-92	654-927	96-231
US-192	788-1010	132-278

The average crop of panic grass was 821, 182, and 185 g/m^2 at bridge areas and upstream and downstream control areas, respectively. The standing crop of panic grass in the bridge areas averaged 4.5 times the grass crop in the control areas.

It appears that there may have been higher plant production in the bridge areas than in the control and undisturbed areas. This is supported by algal bioassay studies in the laboratory. The bioassay experiments, which used various ratios of bridge stormwater runoff and water from adjacent receiving water bodies, have shown that stormwater runoff has the potential to increase considerably the production of unialgal culture of *Selenastrum capricornutum*.

Stormwater runoff, even in concentrations as small as 1 percent added to 99 percent (by volume) of adjacent stream water, can stimulate productivity. The relative magnitude of this increase is a function of the quality of the stormwater. The quality of highway runoff is a function of traffic volume, antecedent dry period, type of pavement, surrounding land use, and other factors. Stormwater runoff that occurs at a selected site after long antecedent dry periods contains elevated levels of organic compounds and heavy metals that can become toxic to receiving water bodies when mixed in high ratios. During periods of frequent rainstorms stormwater is considerably diluted and may be received by adjacent streams without causing sizable changes in algal production or apparent detrimental effects (3).

CONCLUSIONS AND RECOMMENDATIONS

Three bridge sites in central Florida were selected to study (a) the feasibility of using floodplains beneath highway bridges to treat runoff and (b) the impact of bridges on biological diversity and productivity. The following conclusions were reached:

1. The concentrations of several heavy metals extracted from soil samples at the bridge areas were significantly higher than concentrations of similar metals extracted from the control areas upstream and downstream of each bridge site. Lead in particular was significantly higher: there was more than 95 percent t-test probability at all sites tested. It was apparent that lead is a major indicator of soil contamination by highway runoff.
2. Most of the extractable heavy metals were found in the highest amounts in soils from I-4 and Padgett Creek, where the average daily traffic count and the percentage of organic and clay content were the highest of all sites tested.
3. The existing levels of lead in soils from the study sites were far below the lead uptake capacity determined by slurry tests from laboratory experiments. Lead uptake was increased by increasing pH values from 5 to 9 and may have resulted from precipitation and physical and chemical interactions.
4. Heavy metals retained by floodplain soils can be released back to solution by lowering pH values, reducing environments, and soluble-chelating organic complexes. Heavy metals transported from floodplains to adjacent water bodies probably result from anaerobic conditions during flooding, the presence of humic substances, and soil erosion due to high water flow.
5. A fraction of heavy metals retained by floodplain soils is available for biological uptake by plants and other forms of life. Different plants showed different affinities for heavy metals. However, lead was significantly higher in plants collected from bridge areas than in plants from control areas.
6. Plant communities at bridge areas and control areas were generally similar. Native and estab-

lished trees such as oak and cypress may have been removed from bridge areas during the construction phase, which resulted in the invasion of bridge areas by fugitive plants.

7. Bridge areas appeared to be dominated by fewer species than control areas. They were invaded by grasses such as panic grass, elderberry, willow, pokeberry weed, beggar ticks weed, dog fennel, and sea myrtle.

8. There were more individuals of any common plant species in the bridge areas than in the control areas of all sites surveyed. In addition, survey results for the US-17-92 and US-192 sites show that the production of panic grass was much higher in the bridge areas than in the control areas.

9. The algal bioassay experiments have shown that bridge runoff has the potential to increase algal production in nearby streams considerably. Stormwater runoff that occurs after long antecedent dry periods contains elevated levels of heavy metals and organic compounds that can become toxic to receiving water bodies when mixed in high concentrations.

10. The use of floodplain systems in treating bridge runoff appeared to be feasible, and there were no noticeable detrimental effects in the study areas.

Serious consideration should be given to diverting highway bridge runoff to adjacent floodplains. The following recommendations are made:

1. Develop design criteria for floodplains that receive highway runoff;
2. Develop floodplain management techniques that will maximize contact time, prevent erosion, and fix heavy metals tightly in the soil;
3. Minimize periods of anaerobic environment at the floodplains and the release of humic substances by decayed vegetation and plants; and
4. Avoid disturbance of native plants as much as possible in bridge areas to maintain well-established plant communities.

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REFERENCES

1. M.P. Wanielista, R.N. Gennaro, and I.W. Johnson. Shallow-Water Roadside Ditches for Stormwater Purification. Florida Department of Transportation, Tallahasee, Final Rept., March 1978.
2. Y.A. Yousef, M.P. Wanielista, H.H. Harper, and J.E. Christopher. Management of Drainage Systems from Highway Bridges for Pollution Control. Presented at the 61st Annual Meeting, TRB, 1982.
3. Y.A. Yousef, et al. Effects of Bridging on Biological Productivity and Diversity in the Floodplain. Florida Department of Transportation, Tallahassee, Final Rept., July 1982.
4. U. Forostner and G.T.W. Wittman. Metal Pollution in the Aquatic Environment. Springer-Verlag, New York, 1979.
5. R.G. Bader. Some Experimental Studies With Organic Compounds and Minerals. Symposium on the Environmental Chemistry of Marine Sediments. Univ. of Rhode Island, Kingston, Occasional Publ. 1, 1962, pp. 42-47.
6. K.R. Reddy and W.H. Patrick. Effects of Alternate Aerobic and Anaerobic Conditions on Redox Potential, Organic Matter Decomposition and Nitrogen Loss in a Flooded Soil. Soil Biology and Biochemistry, Vol. 7, 1975, pp. 87-94.
7. R.A. Khalid, R.P. Gambrell, M.G. Verloo, and W.H. Patrick, Jr. Transformation of Heavy Metals and Plant Nutrients in Dredged Sediments as Affected by Oxidation-Reduction Potential and pH: Volume 1--Literature Review. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Rept. D-77-4, 1977.
8. J.H. Reuter. Chemical and Spectroscopic Characterization of Human Substances Derived from River Swamps in the Floodplains of Southeastern U.S. Coastal Plain. Environmental Resources Center, Georgia Institute of Technology, Atlanta, ERC 06-08, 1980.
9. J.V. Lagerwerff and A.W. Specht. Contamination of Roadside Soil and Vegetation with Cadmium, Nickel, Lead and Zinc. Environmental Science and Technology, Vol. 4, No. 7, July 1970.

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Guide for Assessing Water-Quality Impacts of Highway Operations and Maintenance

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A 5-year effort to characterize highway runoff in Washington State resulted in the accumulation of data from more than 500 storms at nine locations and the development of a guide for assessing aquatic impacts of operating highways. The data were used to construct a simple model that expresses cumulative pollutant loadings as functions of highway segment length, average runoff coefficient, and vehicles traveling during storm periods. To assess pollutant loadings and concentrations in runoff from an individual storm, cumulative distributions were analyzed to determine the probability of specific loading and concentration values being exceeded in a given case. Bioassay studies of highway runoff indicated toxicity to aquatic life when heavy deposition of metals from high traffic volumes (>10,000 vehicles per day) or high concentrations of metals in rainfall caused concentrations in runoff to exceed lethal levels. Draining highway runoff through grass channels 200 to 300 ft long greatly reduced concentrations of solids and metals and the consequent toxic effects. The impact assessment guide incorporates these results in a stepwise procedure for use by highway designers and environmental impact analysts in the Pacific Northwest. The guide is organized in three analysis levels, ranging from a rapid screening method intended to identify those cases with a low probability of extensive impacts (level 1) to a detailed evaluation focusing on impact mitigation (level 3). It presents methods for assessing the water-quality impacts of winter maintenance and special problems in addition to the effects of runoff from routinely operating highways.

Comprehensive studies of the characteristics, transport, and environmental effects of runoff from operating highways, as a specific component of general storm drainage, are few in number. Several investigations, however, have thoroughly treated some aspect of the subject. Early work was concerned primarily with deposition of contaminants on urban streets (1-3). This theme was also the subject of more recent work (4-6). Sylvester and DeWalle (7) and Soderlund and Lehtinen (8) were among the first to derive pollutant mass loadings for highways, an effort supplemented by the Municipality of Metropolitan Seattle (9,10). Shaheen (11) advanced the development of pollutant loading information by vacuuming and flushing Washington, D.C., area highways and statistically relating mass loadings to traffic density. Several researchers in the United Kingdom have also been active in characterizing the water quality of highway runoff (12-14).

The Envirex Division of Rexnord Corporation and the California Department of Transportation conducted fairly comprehensive highway runoff studies at multiple sites over periods of several years. The Envirex investigation (15,16) covered five sites in different parts of the country to represent different climatic and traffic conditions. It concluded with the development of a deposition model for predicting the accumulation of pollutants in the periods before storms and a washoff model for forecasting contaminant removal in the runoff. The California study (17) was concerned primarily with thoroughly characterizing the physical, chemical, and biological constituents in California highway runoff. A later report of this work (18) also dealt with the effects of runoff through bioassays in which algae were exposed to highway drainage.

Thus, the literature reflects attention to the origin, characteristics, transport, effects, and modeling of highway runoff. Missing from the research record, however, is assimilation of these various results for the purposes of assessing environmental impact and applying mitigative measures.

The Washington State Department of Transportation (DOT) sponsored a 5-year research effort encompassing all of the elements that determine the nature of highway runoff water quality and its effect on water bodies that receive highway drainage. Preliminary investigation (19) indicated that approaches that were valid elsewhere often were not applicable in the Pacific Northwest because of the unique features of its climate. Therefore, the research was directed at developing and verifying methods that could be used reliably in the region. The ultimate objective of the program was to construct models and protocols that would provide convenient means of analyzing the impacts of operating highways on water resources. The products of the research effort were incorporated in a guide (20) for conducting water-quality impact assessments of highway operations and maintenance. The guide is described in detail in this paper and the supporting research project and its results are summarized.

SUMMARY OF RESEARCH PROGRAM

Experimental Design

One of the first issues faced in the research program was whether to base monitoring on discrete samples collected throughout storms or on composites representing entire storms. Sampling equipment was developed (21,22) to collect composite samples from a storm economically, and the decision was made to sacrifice the better characterization of the pollutographs of a relatively few storms (discrete samples) in favor of composite data from many storm events. This equipment has been used to sample approximately 550 storms at nine locations in Washington State.

The major elements of the sampling system are a calibrated flow splitter and a composite sample collector. Flow splitters contain a series of parallel vertical dividers aligned with the flow that separate the runoff into successively smaller portions. Flow splitters were installed on slopes to ensure supercritical flow and uniform distribution of runoff over the base. The flow splitters were sized to capture a set proportion of the design storm for each site, typically about 1 to 2 percent.

Monitoring sites were selected to represent the range of conditions on Washington State highways. Table 1 gives the highway and environmental char-

Table 1. Summary of characteristics of highway runoff monitoring sites.

Category	Characteristic
General climate	Marine lowlands, Cascade Mountains, arid central basin, dry eastern uplands
Average annual precipitation	7 to >100 in.
Setting	General urban, urban residential, rural open and forested, rural agricultural
Highway type	Limited access, arterial, at-grade, bridge sections
Pavement	Concrete, asphalt, sulfur-extended asphalt
Average daily traffic (one direction)	2,000 to 53,000

acteristics covered among the nine sites, which represented the wide variation in climate and land use type in the state as well as the major highway configurations.

The data collected at each site included continuous, automatic traffic counts and, for each storm, precipitation and runoff volumes. Samples were transported to the laboratory directly from local sites or by parcel mail from remote locations and preserved until analysis. Analyses were performed for total suspended solids (TSS), three metals (lead, zinc, and copper), nutrients (total phosphorus, total Kjeldahl nitrogen, and nitrate plus nitrite-nitrogen), and general measures of organic constituents (chemical oxygen demand and total organic carbon). Sampling and analytic procedures have been reported elsewhere (23-26). One study demonstrated no measurable degradation of the constituents of interest during the sample transport period (23). Measured concentrations and flow volumes were used to estimate pollutant loadings in units of mass per unit of highway length. Tests demonstrated that comparable loading estimates resulted from samples from the composite tank and composites made from discrete samples collected simultaneously by an automatic sampler (21,22).

Pollutant Transport Mechanisms

It was observed during the research that the major sources of pollutants in highway runoff are deposition from vehicles, transport from surrounding lands, pavement wear, accidental spills, and certain maintenance procedures. The data demonstrated that pollutants deposited by vehicles primarily resulted from the spray-washing of material adhering to the undercarriages of vehicles as they travel on wet roadways during the extended rainy periods that occur in western Washington (23,24).

Mechanisms that tend to remove pollutants from highways include hydrologic and vehicular scrubbing, maintenance, and natural and traffic-generated winds. The eruption of Mount St. Helens midway in the project provided an opportunity to observe directly the result of traffic-generated winds. This mechanism is considered to be of major importance in pollutant removal (23,24). In the Pacific Northwest, the transport of highway pollutants appears to be more a function of kinetic energy provided by moving vehicles than by the low-intensity rainfall.

Pollutant Loading Analysis

As the data base developed, an investigation was conducted of the associations among pollutant loadings and a number of site and storm characteristics, including volume, duration, and intensity of precipitation, antecedent dry period, total traffic, and vehicles traveling during storm periods. The analysis that exhibited the most consistent pattern for the various sampling sites and contaminants monitored was an analysis of cumulative pollutant mass per unit of highway length versus cumulative traffic volume during storms (23,24). Figure 1 shows the results for one station. The relationship exhibited a step function form in which the steps were associated with occurrences of winter sanding or, on a few occasions, volcanic eruptions. The fall and spring periods were characterized by linear relationships. When similar plots were observed for all sampling sites, it was seen that the slopes of the lines differed among sites and between winter and other seasons at each site.

Because of these apparent differences, it was natural to hypothesize that site runoff coefficients should have a major influence on the cumulative pollutant mass loading entering the runoff. When this variable was introduced, TSS runoff rates at the various stations fell into two distinct groups: western Washington sites with relatively low rates and eastern Washington sites with substantially higher rates. The elevated rates at arid eastern Washington locations resulted from deposition of loose soils on roadways by relatively high and continuous winds. The relationships can be expressed by a simple linear equation in which TSS loading is proportional to the product of cumulative vehicles during storms (VDS) and runoff coefficient (RC) (23,24):

$$\text{TSS loading} = (K)\,(\text{VDS})\,(\text{RC}) \tag{1}$$

Because TSS loading is directly proportional to runoff flow rate, the constant of proportionality (K), which is the TSS runoff rate at a runoff coefficient of 1, can be established as follows (25,26):

$$K_{(RC=1)} - K_{(RC=n)}/n \tag{2}$$

The mean constant (± one standard error) for western Washington locations was 6.4 ± 0.8 lb/highway mile/1000 VDS (27). By observing runoff after large, intense storms that thoroughly cleaned highway surfaces, it was found that the constant fell to approximately 3 lb/highway mile/1000 VDS, which represents the direct contribution of vehicles alone and excludes import from adjacent land uses (27). The reduction in the loading factor is short-lived, and one or more dry days restore enough solids to return it to approximately the mean value. K for eastern Washington locations was estimated, on the basis of considerably fewer data than were available for western Washington, to be 26 ± 2 lb/highway mile/1000 VDS (27).

Other pollutants generally exhibited a relationship to cumulative VDS similar to that of TSS. Again, a linear relationship during the spring and fall was evident, broken by steps coincident with sanding. The similarities of form among the plots suggested that the various contaminants were transported primarily in combination with the solids and that their loadings could be estimated as proportions of TSS loadings (25,26):

$$\text{(Specific pollutant loading)}\ (P)\ \text{(TSS loading)} \tag{3}$$

The coefficients of proportionality (P) are analogous to potency factors sometimes used in SWMM, STORM, and other models (28). The coefficients derived from the data given in Table 2 can be taken as constants at any Washington State location for organics and nutrients or as linear functions of average daily traffic (ADT) for heavy metals (27).

These equations were developed on the basis of cumulative measures and are thus applicable to assessing total loadings over a time span that includes a number of storms (monthly or annually). However, they are deficient in predicting loadings for individual storms with precision (27). It is hypothesized that there is a cyclic accumulation and washoff of pollutants and that the buildup is associated with small storms that wash the undersides of vehicles. Larger storms then periodically wash the road surface. Therefore, loadings in a given locale are highly variable over a short time but are more constant over the longer term when normalized for traffic and runoff coefficient. In regions such as the Pacific Northwest, where many storms do not have sufficient intensity to clean highways thoroughly, accurate assessment of individual storm loading with a deterministic model is problematic. As discussed in the next section, analyzing loading probability

Figure 1. Cumulative TSS versus cumulative traffic volumes during storms for WA-520 site.

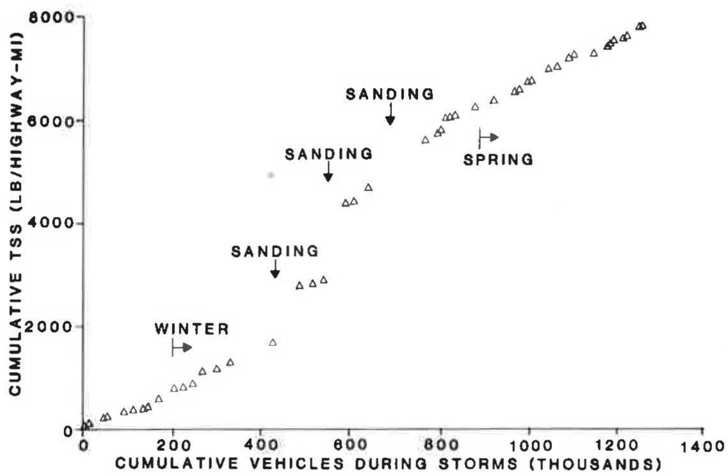


Table 2. Expressions of specific pollutant ratios recommended for use with Washington State highway runoff model.

Pollutant	Expression	R^2	Specifications
Volatile suspended solids	$P_{VSS} = 0.2$	--	All sites
Chemical oxygen demand	$P_{COD} = 0.4$	--	All sites
Lead	$P_{Pb} = 1.5 \times 10^{-4} + (8.7 \times 10^{-8})$ (ADT)	0.978	Western Washington sites
	$P_{Pb} = 5.3 \times 10^{-4} + (2.8 \times 10^{-8})$ (ADT)	0.996	Eastern Washington sites
Zinc	$P_{Zn} = 1.5 \times 10^{-4} + (3.0 \times 10^{-8})$ (ADT)	0.864	Western Washington sites
	$P_{Zn} = 2.0 \times 10^{-4} + (3.2 \times 10^{-7})$ (ADT)	0.932	Eastern Washington sites
Copper	$P_{Cu} = 7.9 \times 10^{-5} + (2.7 \times 10^{-9})$ (ADT)	0.739	All sites
Total Kjeldahl nitrogen	$P_{TKN} = 2.7 \times 10^{-3}$	--	Western Washington sites
	$P_{TKN} = 1.2 \times 10^{-3}$	--	Eastern Washington sites
Nitrate plus nitrite - nitrogen	$P_{NO_3+NO_2\text{-}N} = 2.0 \times 10^{-3}$	--	All sites
Total phosphorus	$P_{TP} = 2.1 \times 10^{-3}$	--	All sites

distributions proved a successful approach to this problem.

Analysis of Individual Event Concentrations and Loadings

In addition to assessing cumulative impacts by means of the loading equations, it is necessary to evaluate acute effects on receiving waters. To do so, individual event concentrations and loadings must be expressed. Storm runoff pollutant concentrations are highly variable from site to site, from storm to storm, and even within storms. As previously noted, loadings also have considerable spatial and temporal variability. When faced with similar data collected in the Nationwide Urban Runoff Program (NURP), the U.S. Environmental Protection Agency (EPA) (29) determined that the data were lognormally distributed and that the distributions could be analyzed to determine the probability of specific concentrations being exceeded in any storm.

The same analysis was used with the Washington State data on highway runoff water quality. The data were aggregated into eastern and western Washington and high- and low-traffic groupings and plotted as probabilities of a given value being exceeded in any storm versus the logarithm of the concentration or loading exceeded (27). The concentrations used were those in the composite samples, which represented the event mean values. These plots were essentially linear and demonstrated the lognormality of the data. Figure 2 shows an example of the most useful form of these charts, in which each pollutant concentration and loading case was graphed separately and curves were added to represent reductions of contaminants by set amounts as a result of removal by treatment methods, dilution by receiving waters, or a combination of the two. Where available, established water-quality criteria were shown to provide a basis for judging effect and assessing impact.

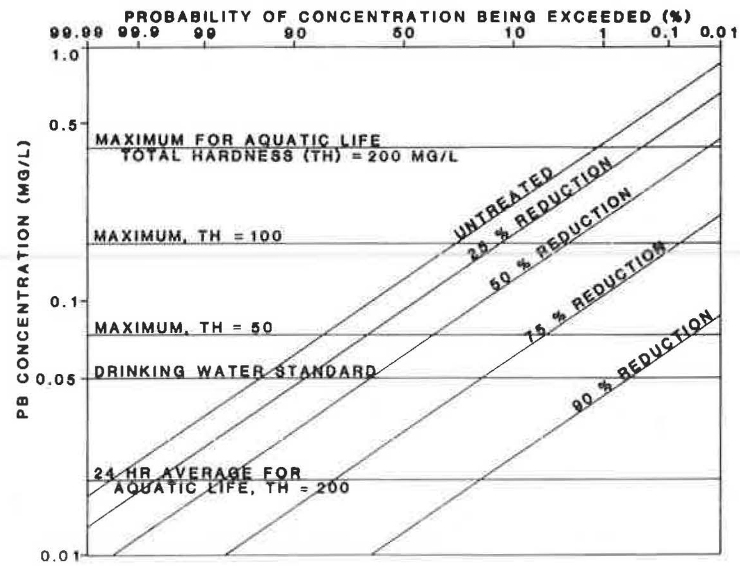
Special Studies

Several special studies were performed within the research program to document particular aspects of highway runoff water quality. Most relevant were the special studies involving trace organic and heavy-metal contaminants, the toxicity of highway runoff to aquatic life, and the effectiveness of vegetated drainage courses in reducing pollutant concentrations and their toxicities.

The investigation of trace organics demonstrated that toxic organic contaminants, if detectable at all, were primarily adsorbed on solid particles (30). An important consequence of this finding is that removing TSS from runoff also reduces organics as well as other pollutants associated with the solids. The absence of an impact due to TSS is currently regarded as a valid indication that organics likewise would not impair the receiving water. The toxicity demonstrated in certain highway runoff samples was thought to be caused chiefly by dissolved metals (27,31).

Toxicity bioassays were performed on a green alga, a cladoceran zooplankter, and rainbow trout fry by using runoff from highways representing a variety of conditions (27,31). Toxicity was absent in the case of highways transporting less than 10,000 ADT in a single direction. The highways rep-

Figure 2. Lead concentration versus exceedence probability distribution for low traffic volumes in western Washington.



resented in these tests had a range of vehicle types and operating modes. Toxic responses were particularly pronounced when specimens were exposed to runoff from a high-volume freeway (approximately 50,000 ADT in one direction) where the local rainfall contained elevated zinc concentrations. Runoff was less toxic for a case in which a highway had a similar traffic volume but the rainfall was lower in zinc. Toxicities to algae (27) and rainbow trout fry (30) were greatly reduced when the runoff was channeled through a grass-lined ditch 250 ft long.

Further investigation documented the effectiveness of vegetated channels in removing solids and other pollutants associated with them from highway runoff. A 200-ft channel was found to be capable of decreasing total suspended solids, chemical oxygen demand, and total lead by approximately 80 percent. More soluble metals, such as zinc and copper, were reduced by approximately 60 percent (32). An unvegetated portion of the channel did not provide filtering action. Residues deposited in such channels were easily entrained by subsequent runoff.

On the basis of these special studies it was concluded that highway location and design should ensure protection of receiving water biota by providing adequate dilution of contamination in runoff or treatment by means of drainage through vegetation, especially when lanes carrying more than 10,000 vehicles/day are involved. Subsequent maintenance practices should promote this protection by minimizing the use of fine sands and road salts in winter operations and maintaining drainage channels in vegetation.

IMPACT ASSESSMENT METHODOLOGY

Scope

The findings of the Washington State DOT/University of Washington research project on highway runoff water quality and relevant material from the stormwater runoff literature were incorporated in a guide for assessing water quality and aquatic ecological impacts of operating highways (20). The assessment approach is responsive to guidelines issued by EPA (33) and is specifically oriented toward the operating requirements and conditions of Washington State highways.

The potential effects of highway operations on water quality are

1. Increases in peak flows in receiving streams;
2. Erosion and sediment transport into receiving waters;
3. Degradation of the quality of receiving water bodies, possible impairment of their beneficial uses, and harm to aquatic biota due to drainage of contaminants incidentally deposited on the roadway; and
4. Effects on receiving water as a result of maintenance procedures.

The guide provides techniques for assessing each of these impacts and recommends strategies for reducing the severity of the identified impacts. It also treats special problems in water-quality impact assessment.

General Approach

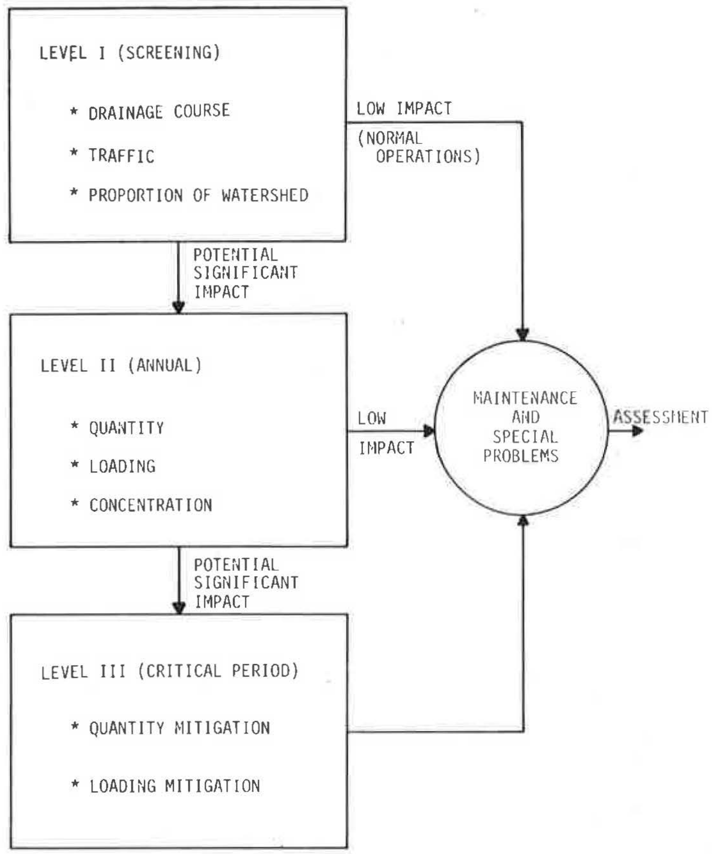
The guide is generally organized as follows:

1. Data needed for preparation of a water-quality assessment (based on EPA guidelines and the particular need for applying the methods developed in the guide),
2. Procedures for assessing the impacts of normal highway operations on water quantity and quality, and
3. Treatment of impacts related to maintenance and special problem areas.

The procedure for normal operations is organized in three levels of analysis, which increase in precision, detail, comprehensiveness, and the quantity of effort required. Maintenance issues and special problems considered are winter sanding and deicing; pesticide applications; construction of woodwaste fills, which is fairly common in the Pacific Northwest; accident spills; and problems related to groundwater. In the case of groundwater, specific reference is made to draft EPA guidelines for assessing the impacts of highways that impinge on designated sole-source aquifers (34).

Figure 3 shows a flowchart of the three levels of detail involved in the general assessment procedure. The first level of assessment merely screens a high-

Figure 3. Overall assessment procedure.



way area in terms of its proportion of the total watershed, the type of runoff drainage system, and the projected traffic volume. Highways that have low ADT, occupy a small percentage of the watershed, or discharge runoff through a vegetated drainage course may be declared to create minimal impact in normal operation. The user is then directed into an analysis of potential impacts from periodic maintenance procedures, accidental spills, and other special problem areas.

If the first level of analysis indicates a potential for a significant impact, the guide prescribes a level 2 analysis based on annual predictions. If necessary, a level 3 analysis, based on monthly projections, can be done to analyze the extent of aquatic impact more thoroughly. Level 2 and level 3 analyses focus on defining the hydrology of receiving water and pollutant loading contributions from the various land uses in the watershed in some detail. The impact of the highway is evaluated in the context of the total burden created by all activities in the watershed.

The second level of analysis guides the user through assessments of peak stream discharge and pollutant loading increases expected to result from the presence of the highway as well as an estimate of the frequency with which given contaminant concentrations would be exceeded. Decision criteria are presented in each instance to judge whether level 3 analysis is warranted. The loading model developed from the research data is the basis for the loading assessment; the probability charts prepared from the data are used in frequency analysis.

The level 3 procedure emphasizes more thorough analysis of the hydrologic and water-quality conditions indicated at level 2 to be of potential significance. Consideration of mitigation measures for demonstrated problems is recommended as well as repeated analysis to forecast their effectiveness. As in level 1, at the conclusion of levels 2 and 3 the analyst is directed to a consideration of special problems.

Levels of Analysis

Level 1

Figure 3 outlines the sequence of steps involved in level 1. In more detail, these steps are as follows:

1. If all runoff discharges by way of a vegetated drainage course at least 200 ft long, go to step 3; otherwise, proceed to step 2.
2. If projected ADT is less than 10,000, proceed to step 3; otherwise, perform level 2 analysis.
3. Determine the total area of the watershed located upstream from the highway runoff discharge point. If there are multiple discharge points, base the determination on the one located farthest downstream.
4. Determine the total area of impervious roadway surface that contributes runoff to the receiving water.
5. If the ratio of impervious roadway surface to total watershed area is less than 0.01, declare no impact from ordinary runoff and proceed to step 6. Otherwise, perform level 2 analysis.
6. Analyze impacts associated with the particular maintenance practices anticipated or any special problem areas.

Each decision criterion has a basis in the research results. The minimum length of vegetated channel is the length identified (32) as reliably providing 60 to 80 percent reduction of major pollutants in highway runoff. The traffic criterion is that below which no toxic effects appeared in bioassays (31). As for the ratio of highway surface to watershed area, it is assumed that the runoff is diluted in the receiving stream in approximately the same ratio. Highway runoff can contain concentrations of toxicants comparable to LC_{50}'s (concentrations lethal to 50 percent of the organisms in an acute bioassay) (31). A common means of protecting aquatic life is to limit receiving water concentrations to 0.01 x LC_{50}. In addition, investigation of the relationship between pollutant concentration and exceedence probability distributions (Figure 2) indicates that a dilution ratio of about 100:1 is generally required to ensure only a slight probability (<0.1 percent) that established water-quality criteria will be exceeded. With a high dilution ratio of ordinary runoff and either low traffic volume or drainage over a vegetated drainage course, it can be stated with some assurance that impact would be insignificant and a more detailed analysis can be avoided.

Level 2

The level 2 assessment of runoff quantity is based on procedures from general practice and the literature because no hydrologic modeling was performed as part of the research project. In its present form, the guide recommends estimating the design for the 25-year-recurrence-interval storm according to the rational method used by the Washington State DOT (35). The procedure can be modified to use a more advanced technique such as the unit hydrograph or a computerized model.

The highway runoff rate should be compared with receiving-stream peak discharge for the same design storm condition. This peak discharge can be established through analysis of the gauging record, if a sufficient one exists, by using an extreme-value (type 1) distribution (36) or, where there is no adequate gauging record, a U.S. Geological Survey procedure developed for the state of Washington (37). An appropriate hydrologic model can also be used to determine design peak stream flow as a percentage increase under the design conditions. If this increase exceeds a permitted amount or is judged to be excessive, the user is directed to level 3 to design detention facilities.

The flowchart shown in Figure 4 provides a guide to the level 2 annual pollutant loading assessment. Fundamentally, it requires that the analyst compare anticipated highway runoff pollutant loadings with loadings already present in the receiving water. These preexisting loadings can be established either by using data on stream water quality and flow, where these are sufficient, or from the land use characteristics of the watershed. The procedure encompasses discharges to standing bodies of water (lakes and wetlands) as well as streams. Its rather arbitrary decision criterion for determining whether further analysis is recommended is a loading increase of more than 10 percent for any pollutant deposited in the receiving water as a result of the highway.

In applying this component of the procedure, TSS loading is first estimated according to the relationship presented in Equation 1. Use of the equation requires an estimate of the annual VDS, which may be proportioned from ADT projections by

$$\text{VDS/year} = \text{ADT}\ [\text{wet hours per year}/(24\ \text{hr/day})] \tag{4}$$

Meteorological records are not routinely kept for the mean duration of precipitation. In the case of Washington State, however, a 16-year record of such data was compiled by the Pacific Northwest River Basin Commission for 32 locations (38). Other data were obtained from the National Climatic Center in Asheville, North Carolina. The record is sufficient to establish satisfactorily mean wet hours per year at almost any location of interest. The data also permitted the derivation of a linear regression equation by which hours of precipitation per year can be calculated as a function of mean annual precipitation. The equation has a high coefficient of determination (0.990) because rainfall intensities are generally light throughout Washington State, although total quantities vary radically.

The TSS loading estimated from Equation 1 should be modified to reflect any runoff treatment provided. The guide presents approximate pollutant reduction capacities of various lengths of vegetated channel derived from the research results. The estimation of highway contaminant loadings is completed by applying Equation 3 to the estimation of chemical oxygen demand, metals, and nutrients from the expected TSS loading and the specific pollutant ratios.

Preexisting pollutant loadings in the runoff receiving water may be estimated from stream water quality and flow data, if they are adequate, or from published loadings from the various land uses in the watershed for lakes and wetlands and inadequately documented stream cases. Provided that consistent units are maintained, the annual pollutant loading in the stream can be estimated from the product of the average discharge and the mean contaminant concentration. For standing water or where hydrologic and water-quality data are lacking, export resulting from general types of land use can be estimated from information taken from the literature and tabulated in the guide and added to known point source loadings to obtain total loadings. Using these data is substantially less satisfactory than using stream records because of the evident dispersion created by combining results from many locations collected by widely differing procedures.

Level 2 gives an additional eutrophication assessment procedure to be used when the receiving water is a lake. This procedure is based on phosphorus loading criteria presented by Vollenweider and Dillon (39).

The final component of level 2 is an assessment of pollutant concentrations based on individual storm events. The key to conducting this analysis is to use an appropriate graph of the type illustrated in Figure 2. The first step is to estimate pollutant reduction factors due to treatment and dilution. Treatment factors can be estimated for vegetated channel drainage according to the guidelines derived from the research or from the performance characteristics of detention or other control devices. The mean dilution ratios (DR) can be approximated as follows:

For streams,

$$DR = Q/(Q + \bar{Q}_s) \tag{5}$$

For lakes or wetlands,

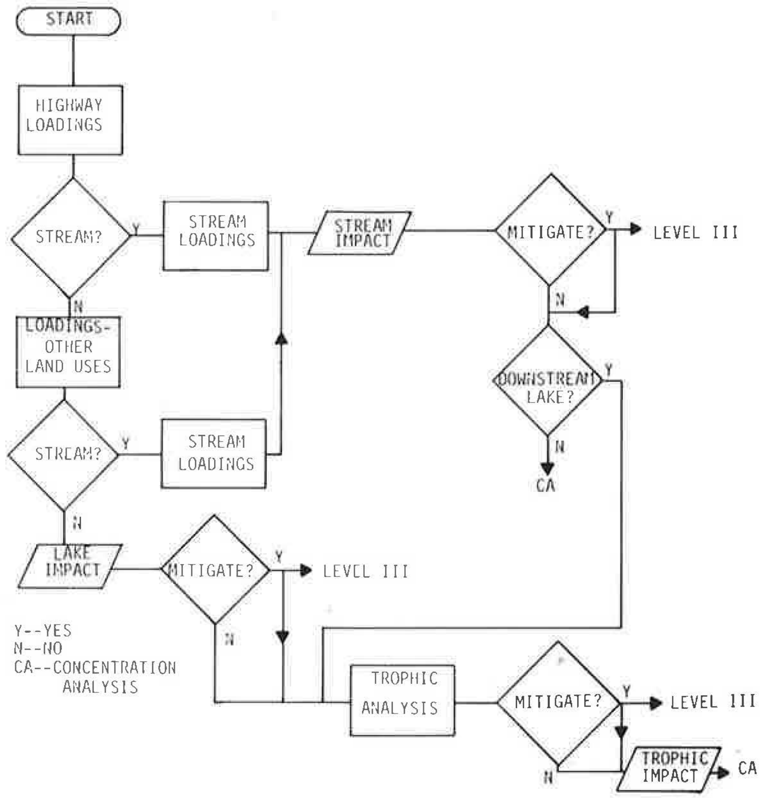
$$DR = QT/(QT + V) \tag{6}$$

where

Q = design highway runoff flow rate,
$\bar{Q}_s$ = average stream discharge,
T = design storm runoff duration, and
V = lake or wetland water volume.

It is recommended that Q be estimated by using

Figure 4. Flowchart of level 2 procedure for pollutant loading assessment.



the rational method or an alternative procedure, where intensity (I) is the average for the 25-year-recurrence-interval, 24-hr-duration storm as given by an isopluvial map of the National Oceanic and Atmospheric Administration (40). The overall pollutant reduction factor is then the product of the decimal-fraction efficiency due to treatment and the dilution ratio.

As an example of the use of the probability distribution charts, consider the lead concentration for the western Washington low-traffic-volume case illustrated in Figure 2. In waters with a total hardness of 50 mg/L as $CaCO_3$, the log probability plot predicts an 83 percent probability of the maximum concentration for aquatic life protection being exceeded if highway runoff is untreated and undiluted; i.e., about 5 of every 6 storms would cause a lead violation. If, however, a 90 percent reduction in lead could be achieved through treatment or dilution or both, the probability of the water-quality criterion being exceeded would drop to 0.04 percent, or approximately 1 storm in 2,500 would result in a violation. As with the loading assessment, the analyst must apply judgment to decide whether to proceed to a level 3 analysis of further mitigative action.

Level 3

Level 3 has an arrangement parallel to that of level 2 in that it comprises assessments of runoff quantity, accumulated pollutant loadings, and individual event occurrences. The particular procedure would be applied, however, only for the specific problem(s) identified as potentially significant in the level 2 analysis.

Level 3 differs from the previous level in several ways. The quantity assessment emphasizes design, or redesign, of detention facilities, using customary highway design procedures, to prevent excessive increases in stream peak flow. The loading assessment is for the monthly period in which the most hours of rainfall occurred rather than annual as in level 2, and a more detailed definition of land use is used. Otherwise, the loading analysis is identical to the level 2 procedure.

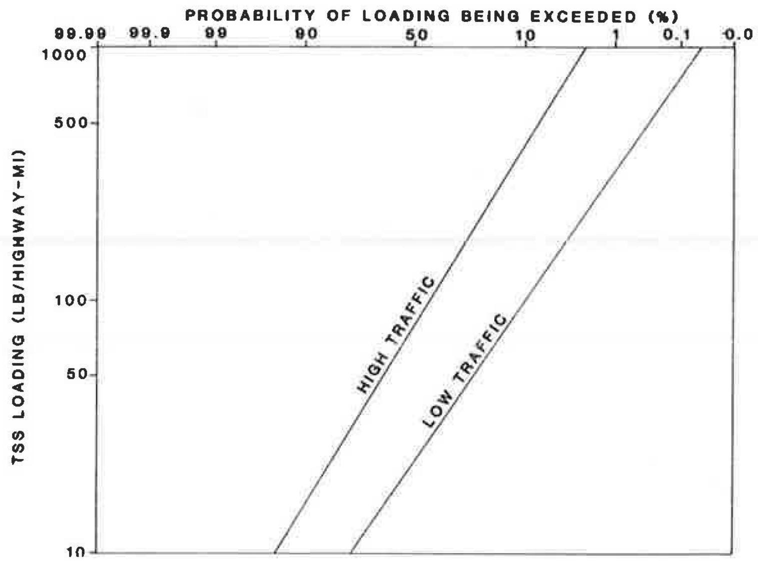
The individual event assessment is directed at water-quality impact mitigation and provides a basis for the design of control facilities. That basis is shown in Figure 5 in the form of a TSS loading-probability distribution for western Washington (an analogous plot exists for eastern Washington). The analyst may select a design probability--e.g., the loading exceeded in only 10 percent of the storms--to use in selecting and sizing the control device. Pollutants other than TSS may be brought into the analysis through use of the multipliers given in Table 2.

Assessment of Impacts Associated with Maintenance Practices and Special Problem Areas

The assessment methodology described applies only to the aquatic impacts associated with ordinary runoff events on normally operating highways. Periodic and extraordinary phenomena must be analyzed separately. Included in this category are winter sanding and deicing, pesticide application, construction practices that create continuing effects on surface waters, and accidental spills. The assessment document provides guidance in these areas although without the specificity made possible by the large data base underlying the routine evaluation.

The various pollutant loadings are augmented by the presence of sand. Winter data demonstrated that sanding contributed a major portion of TSS seasonally, the amount varying with the sand application rate and other sources of solids. The results were

Figure 5. TSS loading versus exceedence probability distribution for high and low traffic volumes in western Washington.



insufficient to model the proportion of applied sand entering the runoff. Thus, currently it is necessary to estimate the proportion roughly on the basis of sand characteristics, plowing, and sweeping.

The research demonstrated that the ratios of other pollutants to the TSS associated with sanding were equal to those given in Table 2 for the high-traffic-volume sites. With lower traffic volume, pollutant deposition failed to saturate the sand particles and the ratios were substantially lower on a cumulative basis (41). It is recommended, therefore, that the loadings of other pollutants be established according to the procedures given for levels 2 and 3 when ADT is projected to exceed 10,000. With lower traffic volume, the assessment should reflect the elevated TSS loading due to sanding but should not augment the loadings of other pollutants in proportion to the TSS resulting from sanding.

Deicing impacts were not specifically investigated in the Washington State research. The guide does provide a procedure drawn from work in Massachusetts (42) for estimating sodium and chloride loadings and concentrations from prevailing application rates.

A comprehensive study of leachates from woodwaste fill sections was undertaken during the research (43). The guide presents an aquatic impact assessment protocol for that regionally important problem. Pesticide applications and accidental spills are covered qualitatively.

SUMMARY AND CONCLUSIONS

A procedure for assessing highway runoff water quality has been formulated on the basis of a large data base that represents conditions prevalent in the state of Washington. The procedure consists of a component for predicting total pollutant loadings over an extended time period and a series of charts with which the impacts of individual events can be assessed probabilistically. These elements, as well as protocols for assessing the impacts of runoff quantity and special problems, have been assembled in a guidebook for evaluating aquatic impacts due to highway operations and maintenance. It is believed that the specific research findings and the proposed impact assessment procedures apply throughout the Pacific Northwest and that the techniques used to monitor storms and analyze the data are more generally applicable.

A significant finding of the research was that highway runoff in Washington State contains lower levels of contamination than general urban drainage and, in normal runoff events, does not have a substantial impact on the quality of receiving water. For highways that carry large traffic volumes, approximately 50,000 ADT in a single direction, a high impact potential exists. That potential can be greatly reduced by avoiding direct drainage to receiving waters by way of piping or bare channels and by avoiding circumstances in which the highway runoff constitutes a substantial fraction of the receiving water flow.

Several aspects of the problem discussed in this paper stand out as areas requiring further research. The assessment guide emphasizes mitigation where impact potential is high and provides design bases for its execution. However, the state of the art is not sufficiently advanced to make full use of that information. The performance of detention basins, vegetated channels, and other control methods must be investigated over gradients in various conditions to add the necessary background for complete engineering design. The contribution of winter sanding was shown to be of considerable importance in determining instantaneous pollutant concentrations and long-term loadings. Additional research is needed to quantify those contributions more thoroughly and to incorporate them in the assessment procedure. Finally, comprehensive procedures for assessing the impact of highway runoff should be developed for other regions. Existing data sets should be reviewed with this goal in mind, and necessary additional data collection should be prescribed.

The result of the Washington State research was that limited problem areas were identified and thus a basis was provided for reducing mitigation costs overall and applying resources to those cases most in need of attention. It is possible to achieve these savings nationwide while providing environmental protection where the need is greatest.

ACKNOWLEDGMENT

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REFERENCES

1. American Public Works Association. Water Pollution Aspects of Urban Runoff. Federal Water Pollution Control Administration, 1969.
2. J.D. Sartor and G.B. Boyd. Water Pollution Aspects of Street Surface Contaminants. U.S. Environmental Protection Agency, 1972.
3. R.E. Pitt and G. Amy. Toxic Materials Analysis of Street Surface Contaminants. U.S. Environmental Protection Agency, 1973.
4. T.L. Miller, R.R. Rinella, S.W. McKenzie, and J. Parmenter. Analysis of Street Sweepings, Portland, Oregon. U.S. Geological Survey, Open-File Rept., 1977.
5. W.G. Wilber and J.V. Hunter. Distribution of Metals in Street Sweepings, Stormwater Solids and Urban Aquatic Sediments. Journal of Water Pollution Control Federation, Vol. 51, No. 12, 1979, pp. 2810-2822.
6. P.K. Hopke, R.E. Lamb, and D.F.S. Natusch. Multi-Element Characterization of Urban Roadway Dust. Environmental Science and Technology, Vol. 14, No. 2, 1980, pp. 164-172.
7. R.O. Sylvester and F.B. DeWalle. Character and Significance of Highway Runoff Waters. Washington State Highway Commission, Olympia, 1972.
8. G. Soderlund and H. Lehtinen. Comparisons of Discharges from Urban Storm Water Runoff, Mixed Storm Overflow and Treated Sewage. Proc., 6th International Conference on Water Pollution Research, Pergamon Press, Ltd., Elmsford, N.Y., 1972.
9. Municipality of Metropolitan Seattle. Freeway Runoff from the I-90 Corridor. Washington State Highway Commission, Olympia, 1973.
10. Municipality of Metropolitan Seattle. Water Quality Investigations Along SR-90: Phase IV. Washington State Highway Commission, Olympia, 1973.
11. D.G. Shaheen. Contributions of Urban Roadway Usage to Water Pollution. U.S. Environmental Protection Agency, 1975.
12. G. Hedley and J.C. Lockley. Quality of Water Discharged from an Urban Motorway. Water Pollution Control, Vol. 74, No. 6, 1975, pp. 659-674.
13. D.P.H. Laxen and R.M. Harrison. The Highway as a Source of Water Pollution: An Appraisal with the Heavy Metal Lead. Water Research, Vol. 11, No. 1, 1977, pp. 1-11.
14. W. Pope, M.J.D. Graham, R.J. Young, and R. Perry. Urban Runoff from a Road Surface: A Water Quality Study. Progress in Water Technology, Vol. 10, No. 5/6, 1978, pp. 533-543.
15. T.L. Meinholz, N.P. Kobriger, M.K. Gupta, and R.W. Agnew. Predictive Procedure for Determining Pollutant Characteristics in Highway Runoff: Volume III--Final Report. FHWA, 1978.
16. N.P. Kobriger, T.L. Meinholz, M.K. Gupta, and R.W. Agnew. Constituents of Highway Runoff: Volume III--Predictive Procedure for Determining Pollutant Characteristics in Highway Runoff. FHWA, 1981.
17. R.B. Howell. Water Pollution Aspects of Particles Which Collect on Highway Surfaces. California Department of Transportation, Sacramento, Rept. FHWA-CA-TL-78-22, 1978.
18. G.L. Winters and J.L. Gidley. Effects of Roadway Runoff on Algae. FHWA, Rept. FHWA-CA-TL-80-24, 1980.
19. R.R. Horner et al. Highway Runoff Monitoring: The Initial Year. Washington State Department of Transportation, Olympia, 1979.
20. R.R. Horner and B.W. Mar. Guide for Water Quality Impact Assessment of Highway Operations and Maintenance. Washington State Department of Transportation, Olympia, 1982.
21. D.L. Clark and B.W. Mar. Composite Sampling of Highway Runoff, Year 2, August 1979. Washington State Department of Transportation, Olympia, 1980.
22. D.L. Clark, R.L. Asplund, J.F. Ferguson, and B.W. Mar. Composite Sampling of Highway Runoff. Journal of the Environmental Engineering Division, ASCE, Vol. 107, No. EE5, 1981, pp. 1067-1081.
23. R.L. Asplund, J.F Ferguson, and B.W. Mar. Characterization of Highway Runoff in Washington State. Washington State Department of Transportation, Olympia, 1980.
24. R.L. Asplund, J.F. Ferguson, and B.W. Mar. Characterization of Highway Runoff in Washington State. Journal of the Environmental Engineering Division, ASCE, Vol. 108, No. EE2, 1982, pp. 391-404.
25. T.W. Chui, B.W. Mar, and R.R. Horner. Highway Runoff in Washington State: Model Validation and Statistical Analysis. Washington State Department of Transportation, Olympia, 1981.
26. T.W. Chui, B.W. Mar, and R.R. Horner. A Pollutant Loading Model for Highway Runoff. Journal of the Environmental Engineering Division, ASCE, Vol. 108, No. EE6, 1982, pp. 1193-1210.
27. L.M. Little. Model Calibration and Probability Distributions for Washington State Highway Runoff Water Quality. Department of Civil Engineering, Univ. of Washington, Seattle, M.S.C.E. thesis, 1982.
28. S.W. Zison. Sediment-Pollutant Relationships in Runoff from Selected Agricultural Suburban and Urban Watershed. U.S. Environmental Protection Agency Rept. EPA-600-3-80-22, 1980.
29. Preliminary Results of the Nationwide Urban Runoff Program. Water Planning Division, U.S. Environmental Protection Agency, Vol. 1, 1982.
30. K.R. Zawlocki, J.R. Ferguson, and B.W. Mar. A Survey of Trace Organics in Highway Runoff in Seattle, Washington. Washington State Department of Transportation, Olympia, 1981.
31. G.J. Portele, B.W. Mar, R.R. Horner, and E.B. Welch. Effects of Seattle Area Highway Stormwater Runoff on Aquatic Biota. Washington State Department of Transportation, Olympia, 1982.
32. T.S. Wang, D.E. Spyridakis, B.W. Mar, and R.R. Horner. Transport, Deposition and Control of Heavy Metals in Highway Runoff. Washington State Department of Transportation, Olympia, 1982.
33. Guidelines for Review of Environmental Impact Statements: Volume I--Highway Projects. Office of Federal Activities, U.S. Environmental Protection Agency, draft, 1968.
34. Curran Associates, Inc. Guidelines for Review of EIS's Subject to Section 1424 (e) of the Safe Drinking Water Act. U.S. Environmental Protection Agency, draft, 1977.
35. Highway Hydraulic Manual. Washington State Department of Transportation, Olympia, 1972.

36. R.K. Linsley, Jr., M.A. Kohler, and J.L.H. Paulhus. Hydrology for Engineers, 2nd ed. McGraw-Hill, New York, 1975, pp. 245-349.
37. J.E. Cummans, M.R. Collings, and E.G. Nassar. Magnitude and Frequency of Floods in Washington. U.S. Geological Survey, Tacoma, Wash., Open-File Rept. 74-336, 1975.
38. Climatological Handbook for the Columbia River Basin. Pacific Northwest River Basin Commission, Vancouver, Wash., Vol. 3, Part B, 1968.
39. R.A. Vollenweider and P.J. Dillon. The Application of the Phosphorous Loading Concept to Eutrophication Research. Center for the Environment, National Research Council of Canada, Burlington, Ontario, Rept. 13690, 1974.
40. Atlas 2: Volume IX--Washington. National Oceanic and Atmospheric Administration, U.S. Department of Commerce, 1973.
41. R.L. Asplund. Characterization of Highway Stormwater Runoff in Washington State. Department of Civil Engineering, Univ. of Washington, Seattle, M.S.E. thesis, 1980.
42. L.R. Frost, Jr., S.J. Pollock, and R.F. Wakelee. Hydrologic Effects of Highway Deicing Chemicals in Massachusetts: Executive Summary. U.S. Geological Survey, Boston, Open-File Rept. 81-210, 1981.
43. K.H. Vause, J.F. Ferguson, and B.W. Mar. Water Quality Impacts Associated with Leachates from Highway Woodwaste Embankments. Washington State Department of Transportation, Olympia, 1980.

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Soil Erosion Study of Exposed Highway Construction Slopes and Roadways

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The quantities of sediment produced from construction slopes and roadways are determined, and a methodology to assist in the determination of these quantities is presented. During the study a portable rainulator was fabricated and applied to collect water runoff and soil erosion data from forest logging roads in northern California. The data collection program was conducted on 10 representative soils in the study area and included testing of cut slopes, fill slopes, road surfaces, and undisturbed sites above the roadway. The data were analyzed and used as input to a simple mathematical model. Additional input parameters were estimated and the model was calibrated. The mathematical model was found to reproduce accurately measured values of water and sediment yield from the roadways. After the mathematical model was calibrated, a procedural guide and an interactive program were developed. Both can be used to assist the forest planner in determining the sediment produced from different roadway geometries and in assessing roadway design alternatives.

Erosion resulting from the construction and use of highways is a problem that continues to plague highway planners and designers. The sediment generated during and after construction is often excessive unless proper erosion control measures are taken. These measures, which should be incorporated in every roadway design, vary in nature from vegetative to structural methods for controlling erosion. In selecting the optimum roadway design, it is necessary to estimate the erosion that will be generated as a result of the design. Hence, methods or procedures for determining these erosion quantities are essential to the selection process.

A recent study conducted by Colorado State University for Region 5 of the U.S. Forest Service (1) culminated in the development of two methods for estimating water and sediment yield from roadways of different designs. The study included (a) selection of a mathematical model to simulate the erosion processes, (b) a field data collection program to provide input to the mathematical model, (c) refinement of the mathematical model based on the collected data, and (d) generation of a procedural guide and an interactive program. The procedural guide provides the field practitioner with a simple and useful method of estimating water and sediment yields for use in road design and environmental impact analysis. The interactive program provides the same capability but eliminates the time-consuming hand calculations required by the procedural guide.

MODEL SELECTION, COMPONENTS, AND DATA INPUT

Predicting water and sediment yields that result from highway construction and use is a complex problem. Solutions to this problem depend on the accurate estimation of a wide array of variables and usually require the use of a mathematically based model. Regression models based on limited field data are unable to cope with this problem on a widespread basis because (a) they are restricted by the data from which they are developed, (b) they assume that the physical environment is both time- and space-invariant, and (c) they tend to group controlling processes into a few coefficients. This grouping decreases the usefulness of the model in examining the effects of individual processes on water and sediment yield from the road.

Physical process models, on the other hand, represent the system being modeled by decomposing it into its respective components, thus avoiding the lumping of processes or parameters. By simulating selected phenomena through separate components, each process can be individually analyzed and refined or altered to meet the needs of the user. As each process component is upgraded, the model becomes more representative of the physical system. The use of component process models also allows the input of variables that have physical significance to the user and the field situation. In addition, because these models are formulated according to physical processes, they are applicable to areas where the governing natural phenomena are the same. Consequently, physical process models are becoming more widely used in assessing ecosystem responses. In some cases, however, such models are as complex and difficult to understand and to use as the process system they simulate.

The limitations of regression models and the user restrictions imposed by more complex physical process models led to the development of simplified physical process models. Model simplification can reduce complexity and, if the simplification maintains the basic physical processes, there will be no significant loss in accuracy. This is the basis for the simplified physical process model selected for this study.

The simplified model was developed by Simons, Li, and Ward (2) to aid in assessing sediment yield from sites subject to erosion. It most readily conformed to the requirements of the study and was selected for its widespread applicability to roadways and construction sites. The physical processes considered in the simplified model include interception losses, infiltration, determination of water yield, and determination of sediment yield by comparing sediment transport capacity with sediment supply. The determination of the sediment supplied during a storm involves consideration of erosion by raindrop splash and overland flow.

Application of the simplified physical process model required a knowledge of key input data, including geometry, soil characteristics, vegetative data, overland flow resistance parameters, size distribution of sediment, and rainfall data. Most input data were available through the field data collection program, and other input parameters were estimated by using established guidelines (3).

FIELD DATA COLLECTION PROGRAM

Before the mathematical model could be used to estimate soil loss and rainfall runoff from specific locations, an on-site field data collection program was required. Critical to the field data collection program was the generation of specific rainfall conditions created by a portable rainfall simulation system (rainulator). Once fabricated, the rainulator could be used to collect the data needed to analyze rainfall and sediment variables associated with roadway erosion.

The data collection program was conducted during two consecutive field seasons and included the testing of two typical roadway designs (in-slope and out-slope). The out-slope roadway design normally consisted of cut slope, road surface, and fill slope (see Figure 1). The in-slope roadway design included a ditch between the cut slope and the road surface (see Figure 2). On two separate occasions, the watershed area above the cut slope was also tested. In some of the soils tested, the slumping of cut slope materials into the ditch provided a constant source of sediment for transport by ditch runoff. Although the contribution of sediment from the ditch was not separately measured or evaluated, it was taken into consideration and measured as part of the sediment contributed from the cut slope.

Reliable measurement of the water and sediment discharge from the field plots was important to the success of the study. Parshall, HS, and cutthroat flumes were considered for measuring water discharge. Important considerations included accurate low-flow measurement, easy field installation, and durability. After field tests were conducted, HS flumes were selected and mounted with mechanical water-level recorders to measure water runoff. Sediment yield was measured by taking grab samples of the flume discharge at 2-min intervals. The grab samples were later analyzed to determine sediment concentration.

Preparing a site for data collection involved defining the boundaries of the test area, performing a location survey to establish the geometric characteristics of the test area, and then installing the rainulator system and the measuring flumes. The boundaries of the test area were defined with berms and isolation and drainage trenches. Isolation trenches and berms were used to prevent the runoff from the areas adjacent to the site from entering the test area. Drainage trenches were used to collect the runoff occurring within the test area. Whenever possible, the rainulator system was positioned to allow simultaneous testing of three of the four subsites (watershed, cut slope, road surface, and fill slope) with trenches defining the boundaries.

Wind conditions were the primary criteria used to determine whether or not a test run would be conducted. A steady wind of more than 5 to 7 mph or the occurrence of wind gusts would disrupt the uniform rainfall distribution over the test area. If the wind conditions were favorable, samples were taken before the test run. These samples included topsoil swept with a whisk broom from three 2x2-ft areas within each subsite and soil cores for moisture measurements taken just before the test run. The topsoil samples were used to determine the material available for transport by the runoff. Moisture samples taken from three areas within each subsite were used to evaluate antecedent moisture conditions. Drainage trenches were flushed with water to the point of saturation before a run to minimize infiltration losses in the trench. This also removed loose soil and organic material from the trenches. A soil binder was also used to prevent additional

Figure 1. Out-slope road design.

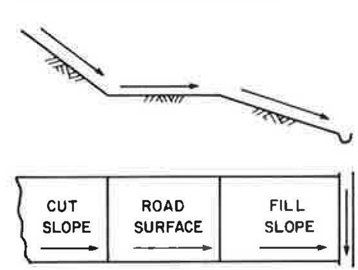
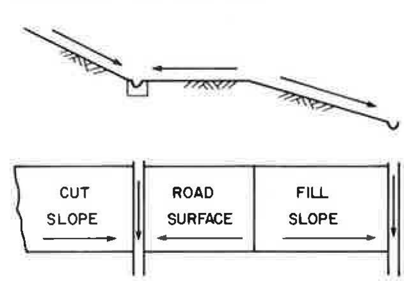


Figure 2. In-slope road design.



erosion from occurring due to the channelization of runoff within the trench system.

When sufficient water was available for a 20-min test run and all prerun samples were obtained, the test run began and the data were collected. Discharge samples were taken at 2-min intervals in accordance with the procedure previously described. After the rainfall ended, bottles were sealed and labeled, charts were collected, topsoil and moisture samples were taken, and rain gage readings were tabulated. A detailed discussion of the field data collection program, including analysis of the collected data and the tabulated results, is provided in the report by Simons, Li, and Anderson (1).

RESULTS

The analysis of the collected data provided the input needed to apply the simplified mathematical model. Application of the model indicated whether the collected data were physically realistic and whether the measured results could be reproduced. Information on specific input parameters was obtained by analyzing the data base and was then used for model calibration.

The results of the model application are shown in Figures 3 to 6. Figures 3 and 4 show the correlation of the predicted and measured water yields for the two consecutive field seasons. The correlation of the predicted and measured sediment yields is shown in Figures 5 and 6. Good agreement is shown in Figures 3 and 4 because rainfall and runoff were used to verify the soil infiltration parameters. Figures 5 and 6 for sediment yield show that small yields are more difficult to reproduce because of the larger relative magnitude of measurement errors and the inherent variability in the mechanics of erosion. These results do indicate, however, that road sediment yields can be modeled by using rainulator data and that the model realistically reproduces the measured values.

APPLICATIONS

After it was verified that the mathematical model could reproduce the measured values, it became the primary tool in producing a quantitative procedural guide and an interactive program capable of estimating soil loss and runoff volumes from timber access

Figure 3. Measured versus predicted water yield: first field season.

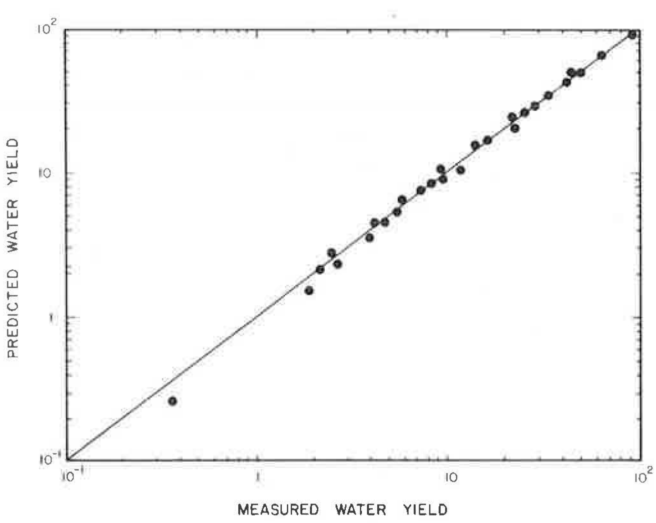


Figure 4. Measured versus predicted water yield: second field season.

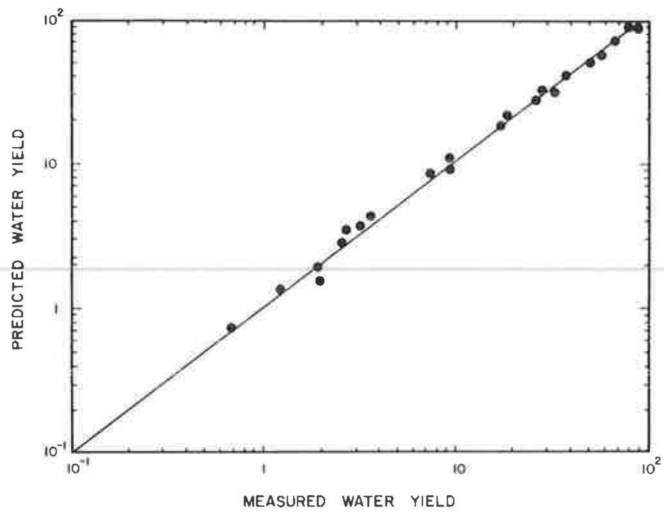


Figure 5. Measured versus predicted sediment yield: first field season.

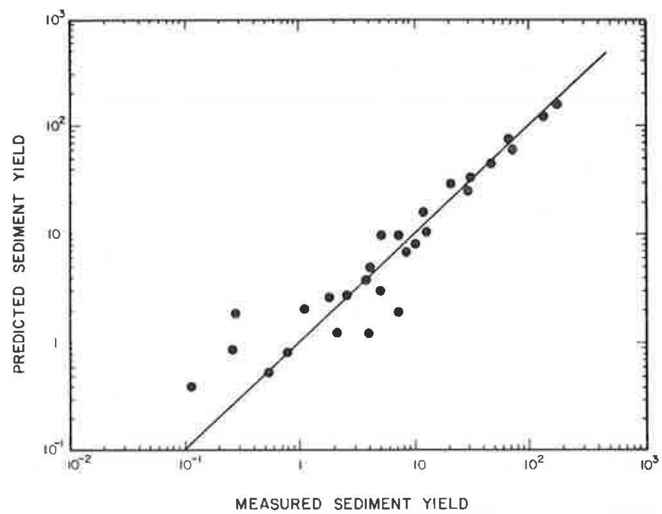
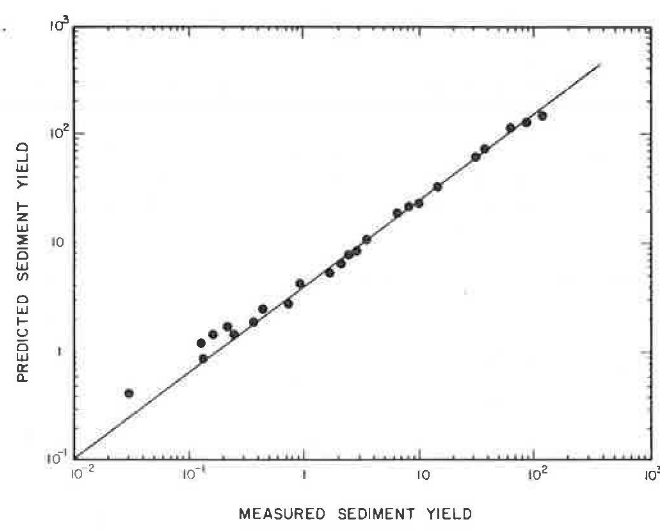


Figure 6. Measured versus predicted sediment yield: second field season.



roads. Both products are able to assess the effects of changing input conditions and land use practices and can subsequently assist in the selection of the optimum roadway design alternative.

In producing the procedural guide and the interactive program, a correction for the difference in the erosion index (EI) between a simulated rainstorm and a natural rainstorm was applied. The average value of the EI for the portable rainulator system was determined to be 40 percent of that for natural rainfall. Research has indicated a directly proportional relationship between the EI and soil loss (4,5). Further research indicated that when simulated rainfall EI values are made equal to natural rainfall EI values by an approximate straight-line adjustment, soil loss values measured under simulated rainfall can be adjusted accordingly (6). In the procedural guide, the adjustment to account for the difference in the values for the EI was made to the sediment supplied by raindrop splash because this quantity is directly related to the magnitude of the EI. By using the reciprocal of 40 percent, all values obtained for material detached by raindrop splash are increased by a factor of 2.5.

Procedural Guide

The procedural guide generated by the mathematical model consists of series of design graphs and has been documented in a separate report by Simons, Li, and Anderson (7). The graphs relate such variables as rainfall intensity, storm duration, infiltration rate, sediment size, ground cover conditions, road gradient, inclination of cut and fill slopes, and water and sediment discharge. As an aid to the forest planner, the procedural guide provides an effective means of determining the sediment discharge from various roadway design alternatives and is especially suited for use by field practitioners.

The governing factors considered in the procedural guide were determined by a sensitivity analysis that involved use of the road sediment model and consultation with personnel from U.S. Forest Service Region 5. The factors considered are rainfall intensity, storm duration, ponding time for surface water, infiltration rate, soil detachment rate, sediment size, ground cover conditions, road gradient, inclination of cut and fill slopes, and sediment and water discharge. The ranges of the key design factors considered in the procedural guide are given in Table 1.

Changing ground cover conditions included gravel pavement on the roads and sparse and dense grass or vegetation on the cut or fill slope. Also incorporated in the development of the procedural guide were 10 representative soil types from Region 5: cobble-stony clay loam, Dubakella gravelly loam, Boomer gravelly loam, Nuens very gravelly loam, Nuens-Sheetiron very gravelly loam, Cagwin loamy sand, Chaix sandy loam, Windy sandy loam, Josephine gravelly loam, and Musick loam.

The series of graphs generated for the procedural guide includes graphs for determining ponding time, rainfall excess rates, and potential sediment transport capacity. Figures such as Figure 7 were generated, showing the ponding time from which surface runoff begins on each cut, road, and fill section for each of the 10 soils. Additional figures, such as Figure 8, provided rainfall excess rates resulting from different rainfall intensities and storm durations of 15, 30, and 60 min. Together, Figures 7 and 8 can be used to make a quick estimate of the volume of rainfall excess and the corresponding water yields. The complete procedural guide includes six figures for determining ponding time and 18 figures for determining rainfall excess rates.

Figures for determining the overall sediment transport capacity were generated next. The figures developed encompassed eight sediment sizes; varied cut, road, and fill slopes; bare soil and gravel pavement as the road surface; and dense and sparse vegetation on the cut and fill slopes. Figure 9 shows an example of the generated relationship between sediment transport capacity and water discharge for a bare soil road surface and a sediment size of 0.10 mm. For the bare soil road surface, 8 figures encompassing the eight sediment sizes were developed. A total of 64 figures were generated to aid in determining the overall sediment transport capacity for a variety of cut, road, and fill slope conditions.

Table 1. Range of key design factors in the procedural guide.

Design Factor	Value
Roadbed gradient	0.01 to 0.20
Slopes (horizontal to vertical)	
Cut	0.5:1 to 2:1
Fill	1:1 to 3:1
Rainfall intensity (in./hr)	1 to 16
Sediment size for determination of transport rate (mm)	
Clay and silt	0.01
Very fine sand	0.1
Fine sand	0.2
Medium sand	0.35
Coarse sand	0.75
Very coarse sand	1.5
Very fine gravel	2.8
Fine gravel	5.5

Figure 7. Ponding time versus rainfall intensity for road surface.

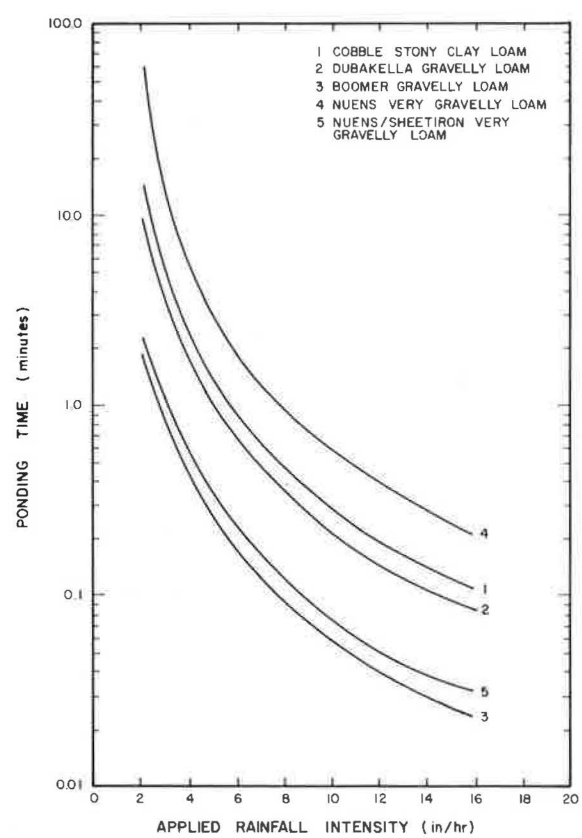


Figure 8. Rainfall excess rate versus rainfall intensity for storm duration of 30 min for road surface.

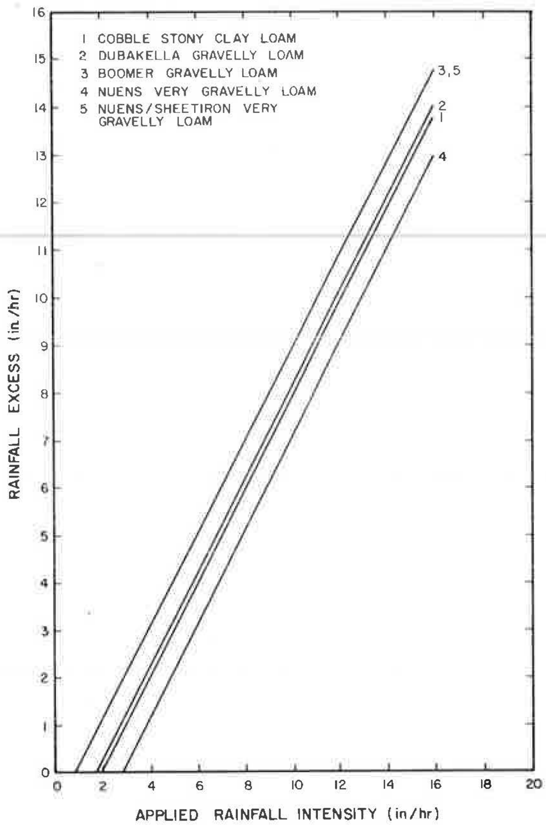
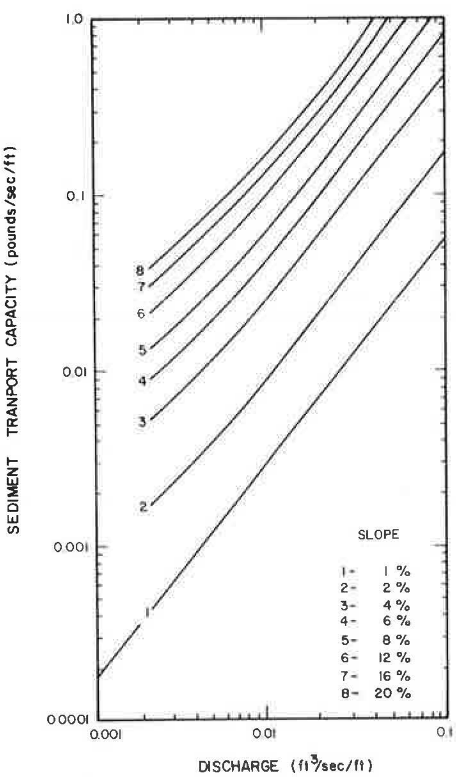


Figure 9. Sediment transport capacity versus water discharge for bare soil road surface and sediment size of 0.10 mm.



With the aid of the figures, the total potential sediment transport capacity can be determined. The sediment yield can then be approximated by comparing the total sediment transport capacity and the overall sediment availability or supply during the storm.

The supply comes from two mechanisms: detachment by raindrop splash and detachment by overland flow. Detachment by raindrop splash can be formulated as a simple power function of rainfall intensity (8):

$$V_r = a_1\, i^2\, LWT\,(1-\phi)\, A_b \tag{1}$$

where

V_r = nonporous volume of material detached by raindrop splash,
i = rainfall intensity,
T = storm duration,
a_1 = empirically determined constant describing erodibility of the soil,
A = area reduction factor,
ϕ = soil porosity,
L = length, and
W = width.

The variable A_b represents the fraction of unprotected or bare soil in the area and is given as

$$A_b = 1 - C_g - C_c + (C_g C_c) \tag{2}$$

where

C_g = ground cover,
C_c = canopy cover, and
$C_g C_c$ = areas of cover overlap.

As mentioned previously, the raindrop splash detachment volume for natural rainfall must be adjusted by a factor of 2.5 to allow for the difference in EI values exhibited by simulated and natural rainfall. With this factor, the volume of material detached by raindrop splash for natural rainfall becomes

$$V_r = V_r \times 2.5 \tag{3}$$

Sediment supply by overland flow detachment is determined by

$$V_f = D_f\,(V_t - V_r) \tag{4}$$

where

V_f = volume of soil detached by overland flow,
D_f = flow detachment coefficient, and
V_t = volume of potential transport determined from the generated figures.

If $V_t < V_r$, there is no overland flow detachment because the transport rate is limited by the transport capacity and V_f is equal to zero. The total available sediment supply is

$$V_a = V_r + V_f \tag{5}$$

Sediment yield is controlled by either supply or capacity. If supply is greater than capacity, capacity controls, and vice versa. As particle size changes, so do capacity and supply. Therefore, supply and capacity must be compared for each particle size. The individual capacity is determined by

$$V_{t_i} = V_t\, P_i \tag{6}$$

where

P_i = percentage of each sediment size,
Ψ_{t_i} = individual demand or capacity for the particle size, and
Ψ_t = total transport capacity determined from the figures.

The available supply is

$$\Psi_{a_i} = \Psi_a P_i \qquad (7)$$

where Ψ_{a_i} is the available supply for the particle size. Values of Ψ_{a_i} and Ψ_{t_i} can now be compared. If Ψ_{t_i} is greater than Ψ_{a_i}, supply controls; if Ψ_{a_i} is greater than Ψ_{t_i}, capacity controls; or

$$\Psi_{y_i} = \Psi_{a_i} \quad \text{if } \Psi_{t_i} > \Psi_{a_i} \qquad (8)$$

and

$$\Psi_{y_i} = \Psi_{t_i} \quad \text{if } \Psi_{t_i} < \Psi_{a_i} \qquad (9)$$

where Ψ_{y_i} is the volume yield for the particle size fraction. The total yield will then be

$$Y_s = \gamma_s \sum_{i=1}^{N} \Psi_{y_i} \qquad (10)$$

where Y_s is the sediment yield by weight.

The procedures for using the procedural guide and estimating water and sediment yields are illustrated in nine example problems presented by Simons, Li, and Anderson (7). Each example provides a detailed step-by-step procedure for determining water and sediment yield. In the interest of brevity, an example is not included in this paper.

Interactive Road Sediment Program

The interactive road sediment program resulted from a reformulation of the mathematical model. It is capable of producing the same results as the procedural guide and has the added advantage of executing all time-consuming calculations and procedures quickly and efficiently. The interactive computer program has the obvious limitation of computer accessibility and requires that the user have a basic understanding of interactive computer operating procedures.

The data input for the interactive program was derived directly from the data base used by the calibrated model. One set of data exists for each of the 10 soils tested in the study. It is a special feature and an obvious advantage of the interactive program that the user can quickly edit each data set to allow for changes in slope, geometry, number of rainstorms, rainfall intensity and duration, soil suction pressure, hydraulic conductivity, saturation index, porosity, ground cover, canopy cover, and canopy and ground cover interception values. As in the procedural guide, this feature provides the flexibility needed to model the water and sediment yields from similar soils in the study area while considering changes in roadway design, location, maintenance, and various stabilization and treatment measures. The input values entered interactively by the user to change the data sets should be determined after a thorough investigation of the site to be modeled. In some instances this may require an on-site inspection for an estimation of ground and canopy cover and the collection of soil samples needed to determine the appropriate infiltration parameters.

The interactive program provides as output a listing of the pertinent input parameters, estimated water yield, estimated sediment yield by size fractions, and total sediment yield. Output is generated and displayed for each storm and surface type. A report by Li, Collette, and Anderson (9) documents the use and editing procedure of the interactive program. It also provides examples of application, listings of input and output data, an explanation of the computer language used, and the amount of storage required. The example that follows illustrates program execution, input prompts, and output generation.

Determine the water and sediment yield from a road surface given the following information: length = 500 ft, width = 10 ft, slope = 0.04, ground cover = 0.05, storm intensity = 3 in./hr, storm duration = 1 hr, and soil = Boomer gravelly loam. (The data input values contained in the data base will be edited in this example.) After the appropriate execution command has been issued, the terminal responds with the following.

```
    SELECT THE SOIL TYPE THAT BEST CORRESPONDS TO THE
    SOIL BEING EVALUATED.
         0 :  STOP
         1 :  COBBLE STONY CLAY LOAM
         2 :  DUBAKELLA GRAVELLY LOAM
         3 :  BOOMER GRAVELLY LOAM
         4 :  NUENS VERY GRAVELLY LOAM
         5 :  NUENS/SHEETIRON VERY GRAVELLY LOAM
         6 :  CAGWIN LOAMY SAND
         7 :  CHAIX SANDY LOAM
         8 :  WINDY SANDY LOAM
         9 :  JOSEPHINE GRAVELLY LOAM
        10 :  MUSICK LOAM

    ENTER CORRESPONDING NUMBER FOR SELECTION
?   3
    SELECT SURFACE DESIRED
         0 :  NO SLOPE DESIRED
         1 :  CUT
         2 :  ROAD
         3 :  FILL

    ENTER CORRESPONDING NUMBER FOR SELECTION
?   2

    DO YOU WISH TO EDIT THE ROAD DATA?
    ENTER YES OR NO
?   YES

    DO YOU WISH TO EDIT THE GEOMETRY DATA FOR THE
    SELECTED SURFACE?

    SLOPE LENGTH (FEET)                31.00
    WIDTH          (FEET)              41.39
    SLOPE   (DECIMAL FRACTION)           .13

    ENTER YES OR NO
?   YES
    ENTER SLOPE LENGTH (FT)
?   500
    ENTER WIDTH (FT)
?   10
    ENTER SLOPE (DECIMAL FRACTION)
?   .04

    DO YOU WISH TO EDIT THE RAIN DATA FOR THE
    SELECTED SURFACE?

    NUMBER OF RAIN STORMS =            1
    STORM      DURATION           INTENSITY
               (HOURS)            (INCHES/HOUR)
    1.         .33                2.53

    ENTER YES OR NO
?   YES
```

```
   ENTER NUMBER OF RAIN STORMS
?  1

   DO YOU WISH TO EDIT THE RAINFALL DURATIONS?
   ENTER YES OR NO
?  YES
   ENTER RAINFALL DURATION (HRS) FOR STORM  1
?  1

   DO YOU WISH TO EDIT RAINFALL INTENSITIES?
   ENTER YES OR NO
?  YES
   ENTER RAINFALL INTENSITY (IN/HR) FOR STORM  1
?  3

   DO YOU WISH TO EDIT THE SOIL DATA FOR THE
   SELECTED SURFACE?

   SOIL SUCTION HEAD           (INCHES)         .480
   HYDRAULIC CONDUCTIVITY (INCHES/HOUR)         .600
   SATURATION INDEX   (DECIMAL FRACTION)        .300
   POROSITY           (DECIMAL FRACTION)        .450

   ENTER YES OR NO
?  NO

   DO YOU WISH TO EDIT VEGETATION DATA FOR THE
   SELECTED SURFACE?

   GROUND COVER (DECIMAL FRACTION)              .10
   CANOPY COVER (DECIMAL FRACTION)             0.00
   GROUND COVER INTERCEPTION (INCHES)           .50
   CANOPY COVER INTERCEPTION (INCHES)          0.00

   ENTER YES OR NO
?  NO

   ==================================================

   SOIL TYPE       :  BOOMER GRAVELLY LOAM
   SOIL SURFACE    :  ROAD

             SOIL PARAMETERS

   SOIL SUCTION HEAD (INCHES)                .48
   HYDRAULIC CONDUCTIVITY (INCHES/HOUR)      .60
   SPECIFY GRAVITY                          2.65
   SATURATION INDEX (DECIMAL FRACTION)       .30
   POROSITY (DECIMAL FRACTION)               .45

             VEGETATION PARAMETERS

   GROUND COVER (DECIMAL FRACTION)           .10
   CANOPY COVER (DECIMAL FRACTION)          0.00
   GROUND COVER INTERCEPTION (INCHES)        .05
   CANOPY COVER INTERCEPTION (INCHES)       0.00

             GEOMETRY PARAMETERS

   SLOPE LENGTH (FEET)             500.0
   SLOPE WIDTH  (FEET)              10.0
   SLOPE                              .04

             SOIL SIZE DISTRIBUTION

   GRAIN SIZE (MM)                PERCENT BY WEIGHT
      15.41                              .22
       4.36                              .58
       1.41                              .09
        .71                              .05
        .35                              .01
        .18                              .01
        .10                              .01
        .06                             0.00
        .01                              .03
```

```
               WATER YIELD

   STORM                                      1
   DURATION OF RAINFALL (HOURS)            1.00
   RAINFALL INTENSITY (INCHES/HOUR)        3.00
   TIME TO PONDING (HOURS)                  .01
   RUNOFF VOLUME (CUBIC FEET)            878.44

               SEDIMENT YIELD

   GRAIN SIZE (MM)           SEDIMENT YIELD (POUNDS)
      15.41                        0.00
       4.36                        0.00
       1.41                        0.00
        .71                        7.41
        .35                        5.10
        .18                        7.67
        .10                        7.67
        .06                        0.00
        .01                       23.02

   TOTAL SEDIMENT YIELD (POUNDS) :  50.87
```

SUMMARY AND CONCLUSIONS

The results and methods derived from this study represent significant steps toward the accurate and simplified estimation of water and sediment yields from roadways. The simplifications have been incorporated to enable the relatively untrained individual to estimate these erosion quantities. The two major accomplishments of this study, the procedural guide and the interactive program, have been documented and represent advancements in these areas. This is not to say that other methods of estimating the sediment generated by roadways have not been developed. On the contrary, previous studies have documented procedural guides similar in scope and methodology to the one discussed here. However, they have only been applied qualitatively and have exclusively considered soils consisting of a single sediment size.

The procedural guide produced as a result of this study considers 10 different soils composed of a wide range of sediment sizes. Furthermore, it has been derived from the results of a field data collection program and can be applied quantitatively to estimate the sediment generated by various roadway design alternatives. Even though increasing the number of size fractions makes the computations more tedious and time-consuming for the individual user, the capability of the procedural guide to estimate the sediment yield from complex soils enhances its versatility and applicability and is considered a worthwhile trade-off.

A number of computer programs and mathematical models for determining the sediment yield from roadways have also been developed and are available. They often require a basic knowledge of the modeling processes as well as of the physical significance of the input parameters. The model used in this study is an example of such a model. The data base assembled during this study, however, was incorporated as an integral part of the interactive program, and this factor, coupled with the interactive capability of the program, has eliminated this requirement. Thus, the interactive program is ideally suited for the untrained user to produce results with only a limited knowledge of interactive computer operating procedure. In addition, the interactive capability of the program permits rapid appraisal of changes in road gradients, cross sections, route locations, types of surfacing, and spacing of cross drains in relation to the quantities of sediment produced.

However, the limitations as well as the capabilities of the procedural guide and the interactive program must be recognized and addressed. Both methods are limited regionally to the soils tested and the sites evaluated during the study. Furthermore, the interactive program and the procedural guide are limited to assessing the erosion from relatively simple road geometries. In spite of these limitations, however, it is conceivable that, with an expanded field data collection program, improved methodologies similar to those presented in this paper could be produced for any selected geographic region.

ACKNOWLEDGMENT

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REFERENCES

1. D.B. Simons, R.M. Li, and B.A. Anderson. Soil Erosion and Sedimentation Analysis of Forest Roads in Northern California. Colorado State Univ., Fort Collins, Rept. CER81-82DBS-RML-BAA74, 1982.
2. D.B. Simons, R.M. Li, and T.J. Ward. Simple Procedural Method for Estimating On-Site Soil Erosion. Colorado State Univ., Fort Collins, Rept. CER76-77DBS-RML-TJW38, 1977.
3. D.B. Simons, R.M. Li, T.J. Ward, and K.G. Eggert. Estimation of Input Parameters for Modeling of Water and Sediment Yields. Department of Civil Engineering, Colorado State Univ., Fort Collins, Draft Rept. CER-79DBS-RML-TJW-KGE50, April 1979.
4. W.H. Wischmeier and D.D. Smith. Rainfall Energy and Its Relationship to Soil Loss. Trans., American Geophysical Union, Vol. 39, No. 2, 1958, pp. 285-291.
5. W.H. Wischmeier. Rainfall Erosion Potential. Agricultural Engineering, April 1962.
6. R.A. Young and R.E. Burwell. Prediction of Runoff and Erosion from Natural Rainfall Using a Rainfall Simulator. Proc., Soil Science Society of America, Vol. 36, 1972.
7. D.B. Simons, R.M. Li, and B.A. Anderson. Procedural Guide for Estimating Water and Sediment Yield from Forest Roads. Colorado State Univ., Fort Collins, Rept. CER81-82DBS-RML-BAA4, 1981.
8. L.D. Meyer. Soil Erosion by Water on Upland Areas. In River Mechanics, Volume 2 (H.W. Shen, ed.), Colorado State Univ., Fort Collins, 1971.
9. R.M. Li, D.A. Collette, and B.A. Anderson. Users Guide for SIMAK: Interactive Program for Estimating Soil Erosion from Forest Roads. Colorado State Univ., Fort Collins, Rept. CER81-82RML-DAC-BAA75, 1982.

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Roadside Erosion Causes and Factors: Minnesota Survey Analysis

ROXANNE SULLIVAN AND LAWRENCE E. FOOTE

A roadside erosion survey was conducted along all state, county, and township roads in Minnesota. The locations and estimated volumes of roadside erosion, cross-sectional road designs, roadway ownership, type and causes of erosion, and history of the road (time since construction disturbance) were noted. The total estimated soil loss was 116,203,336 ft^3 at 17,902 sites located along 115,570 miles of roadway. The cross-sectional design that resulted in the most soil loss was the cut-fill design. The fill design had the lowest soil-loss volume. Erosion occurred most often along at-grade roads and least often along fill roads. Volumes and occurrences were slightly more along township than along county roads and much less along state roads. Ditch bottoms were the most common location of erosion on roadsides and water-related erosion was the major type. Although erosion occurred more often along older roads, eroded sites were larger along newer roads. The larger sites were generally caused by (a) inadequate design in areas with rough terrain or poor soils or near waterways and (b) lack of administrative direction and emphasis on establishment of cover and control of unauthorized activities, including farming the right-of-way and use of roadsides as borrow areas or for recreation. Erosion was often associated with drainage from adjacent areas, steep slopes, inadequate design, and lack of administrative direction and emphasis. Corrective measures were recommended, and many counties fully implemented such measures. However, some sites remain uncorrected and others have increased. Lack of funds is the main reason for the absence of corrective measures, particularly on township roads. More construction of roads with a fill cross-sectional design and less of cut-fill roads, especially in rough terrain, should reduce the potential for future erosion.

The potential for erosion is ever present. The process of detachment, transportation, and redeposit of sediment is by far the greatest contributor of pollution to streams and lakes. Sediment in waterways increases turbidity, inhibits photosynthesis, interferes with respiration of aquatic organisms, tends to destroy habitat, and degrades water quality. Sedimentation in culverts, ditches, stream channels, reservoirs, and other conveyance or storage structures decreases capacity and reduces the effectiveness of such structures. The removal of sediment from these structures and public water supplies is costly. Loss of topsoil by erosion reduces vegetation productivity and increases rainfall runoff.

Significant erosion occurs most frequently on agricultural and other disturbed lands (i.e., borrow pits and recreational areas); along stream banks, drainage ditches, and lakeshores; and at or near construction sites, including those of roads, bridges, and other transportation facilities.

Highway construction often drastically disturbs natural soil deposits and formations. It includes removal of vegetation, displacement of topsoil, and alteration of natural slopes and drainages. Sediment eroding along the roadside originates from the roadbed itself, from areas within the roadside right-of-way, and from areas outside the right-of-way. Loss of soil from the roadbed itself or from in-slopes results in structural instability of the roadway and a potential safety hazard for the motoring public. Loss of topsoil along the right-of-way lowers soil fertility, which reduces the ability of slopes and ditches to revegetate and control future erosive action. Erosion that begins on land adjacent to the right-of-way often continues unchecked onto the right-of-way. Such erosion scars the landscape, mars the aesthetic appeal of the area, and reduces the capacity of roadside ditches.

The cost of repair and maintenance for erosion sites is proportional to the quantity of erosion involved. Unless proper measures are taken to prevent and control erosion during construction or reconstruction of roads and during maintenance on existing roads, serious erosion and sedimentation problems often develop. Uncontrolled erosion can lead to the need for structural corrections to the road itself or installation of costly control structures.

A roadside erosion survey was conducted along all state, county, and township roads in Minnesota to identify and document the location and amount of erosion occurring along these roads. Reports of the survey results were prepared and published by the Minnesota Department of Transportation for each of the 87 counties in the state.

In this paper, various general facts, problems, conditions, and relationships concerning roadside erosion in Minnesota are identified and analyzed. Recommendations are made that should have wide application and be of general use for the prevention of roadside erosion through design as well as control and repair. Basic causes and factors associated with the occurrence of roadside erosion are discussed. Greater knowledge and consideration of these causes should make it possible to design and construct or reconstruct roadways that avoid or provide control measures so as to decrease the potential for erosion.

PROCEDURES

Data forms, maps, and procedures used in the survey were distributed at a series of instructional meetings. Then an organizational meeting of participating groups was held in each county to help acquaint personnel with the survey methods. Before starting, the county groups made trial runs and erosion site estimates were checked by actual measurements.

Roadside survey field work was conducted in the spring after snowmelt but before extensive vegetative growth or in the fall after growth had stopped and had been affected by frost. The survey was conducted by two- or three-person teams within each township of each county. Instructions were to report all eroded areas larger than 100 ft^2 in surface area, at least 6 in. deep, or more than 50 ft^3 total volume. The teams recorded the location of each erosion occurrence and its volume on the survey data form (see Figure 1). Data on road design (see Figure 2), roadway ownership, type and cause of erosion, history of the roadway, and control measures needed were also requested. Roadside erosion sites were located on aerial photographs and on new plat books.

On a county-by-county basis, volumes and occurrences of roadside erosion were tabulated according to road design (at-grade, fill, cut, cut-fill, and other); roadway ownership (township, county, state, forest, and private); location of erosion (ditch bottom, in-slope, backslope, adjacent areas, and other as well as combinations thereof); type of erosion (slide, wash, and blowout); cause (disturbance, inadequate design, and other); history of the roadway (3 years or less since construction and more than 3 years since construction); and control needed (till and establish cover; slope, shape, till, and establish cover; or slope, shape, till, and establish cover and structure). County totals were summed for similar statewide totals and averages.

Data on volume and occurrence were then tabulated on a per-mile basis for each category of roadway ownership. These per-mile data normalized erosion for the purpose of comparing counties or roadway ownerships with unequal mileages. The average volume per mile and occurrence rate were calculated for each county. The average volume per occurrence for each road design and ownership category and average volume per occurrence countywide and statewide were calculated.

RESULTS

Erosion on All Roadways

The total estimated volume of erosion reported in the roadside survey was 116,203,336 ft^3 distributed over 17,702 sites located along 115,570 miles of roadway (see Table 1). As the following table indicates, the volume of erosion per county ranged from 20,895 to 23,444,632 ft^3:

	Volume (ft^3)	
Range	Per County	Per Mile
Low	20,894	17
High	23,444,632	27,173
Median	521,681	433

The total statewide volume was 116,203,336 ft^3.

The average volume of soil lost per county was 1,335,728 ft^3; however, only 15 percent of the counties had more than the average estimate (see Figure 3). Specific causes of soil loss in counties with the most severe roadside erosion included lack of vegetative cover, rough terrain, use of roadsides as borrow areas, lack of erosion control during development, recreational activities, lack of past administrative emphasis on erosion control and repair, and lack of funds to correct existing problems.

Occurrences of erosion per county ranged from 9 to 1,015, as the following table indicates (one site every 8.3 miles = 0.12):

	No. of Occurrences	
Range	Per County	Per Mile
Low	9	0.006
High	1,015	0.87
Median	166	0.12

The total number of occurrences for the state was 17,902. The average number of sites per county was 206. Of the counties, 36 percent had more than the average number of sites inventoried (see Figure 4). The major causes of erosion in counties that had the most numerous occurrences included agricultural drainage or drainage from ditches mandated by statute, poor vegetative cover, inadequate road design, and use of roadsides as borrow areas.

Figure 1. Sample data sheet used in gathering information on the survey.

SCSA ROADSIDE EROSION CONTROL SURVEY

Township: Indian
County: Sioux
Page 5 of 5 pages

Survey Members: John Doe
Joe Smith
Jim Jones

SAMPLE

No.	Date	General Location — Record No. on Photo Mark Loc. in Rec. Record Section, & Rd. Designation if given below.	Road Design No.	Location of Erosion: Ditch Bottom	Inslope	Backslope	Adjacent Areas	Other	Dimensions (Ft.): Width	Depth		Vol. Cu. Ft.	Type of Highway	Type of Erosion: Slide (gravity)	Washing (water)	Blow out (wind)	Control Needed: Till Establish Vegetation	Slope Shape Till Establish Vegetation	Structure Slope Shape Till Establish Vegetation	Cause: Disturbance	Inadequate Design	Other	History: 3 Y and Under	Over 3 Y	Note
1	4-20	COUNTY RD. 21 SEC. 23	3			X	X		200	50	.4	4000	C		X		X					X	X		FILLING FROM ADJACENT AREA
2	4-20	GULL LAKE RD. SEC. 23	1	X					120	4	1.5	720	T		X				X		X				STEEP DITCH GRADIENT
3	4-20	TH 168	2		X				80	60	20	9600	S	X			X					X		X	NATURAL SOIL SLIDE
4	4-20	SEC. 24	1				X		60	60	.3	1080	T			X	X			X				X	PIPELINE CONST.
5	4-20	COUNTY RD. 38 SEC. 24	4			X	X		60	14	24	2016	C		X			X		X				X	TRAIL BIKES CLIMBING SLOPES
6	4-20	COUNTY RD. 39 SEC. 26	2	X	X	X		X	300	50	6	9000	C		X			X				X		X	ERODING PUBLIC DRAINAGE DITCH

Figure 2. Identification of roads by cross-sectional design and location of erosion.

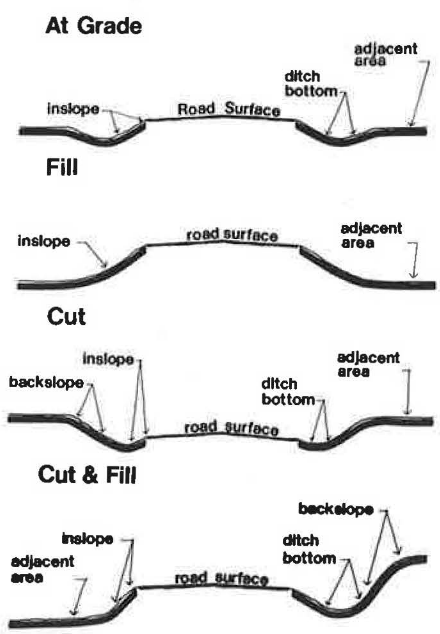


Table 1. Summary of survey results: volume estimates and occurrences of roadside erosion.

Category	No. of Occurrences[a]	Volume (ft^3) Total	Per Occurrence
Road design			
At-grade	7,181	35,751,365	4,934
Fill	1,602	8,155,015	5,091
Cut	4,672	31,484,429	6,739
Cut-fill	4,199	39,470,568	9,400
Other	184	9,502,009	51,641
Ownership			
Township	8,660	45,859,258	5,296
County	7,962	44,522,514	5,591
State	1,080	22,773,360	21,689
Location			
Ditch bottom	11,390		
Inslope	9,223		
Backslope	7,394		
Adjacent areas	9,329		
Other	406		
Type			
Slide (gravity)	3,183		
Wash (water)	15,309		
Blowout (wind)	284		
History			
<3 years[b]	2,217	34,490,843	15,557
>3 years[b]	13,747	88,698,250	6,452
Control needed			
Till and establish cover	4,471		
Till, establish cover, slope, shape cover	10,099		
Till, establish cover, slope, shape, structural control	2,668		

Note: Data not available for volume and volume per occurrence for ditch bottom, inslope, backslope, adjacent areas, and other locations. Data not available for volume and volume per occurrence for slide (gravity), wash (water), and blowout (wind) types.

[a] Because some sites occurred on more than one portion of the cross section and some data were omitted, some totals vary.
[b] Time since construction or reconstruction.

Figure 3. Distribution of total volume ratings of erosion on all roads by county.

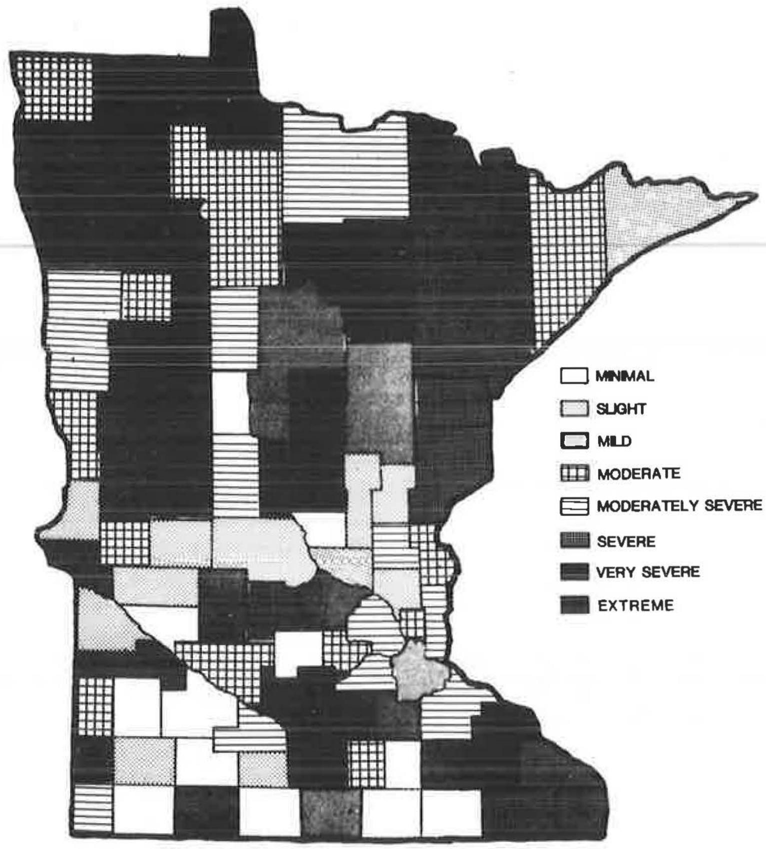
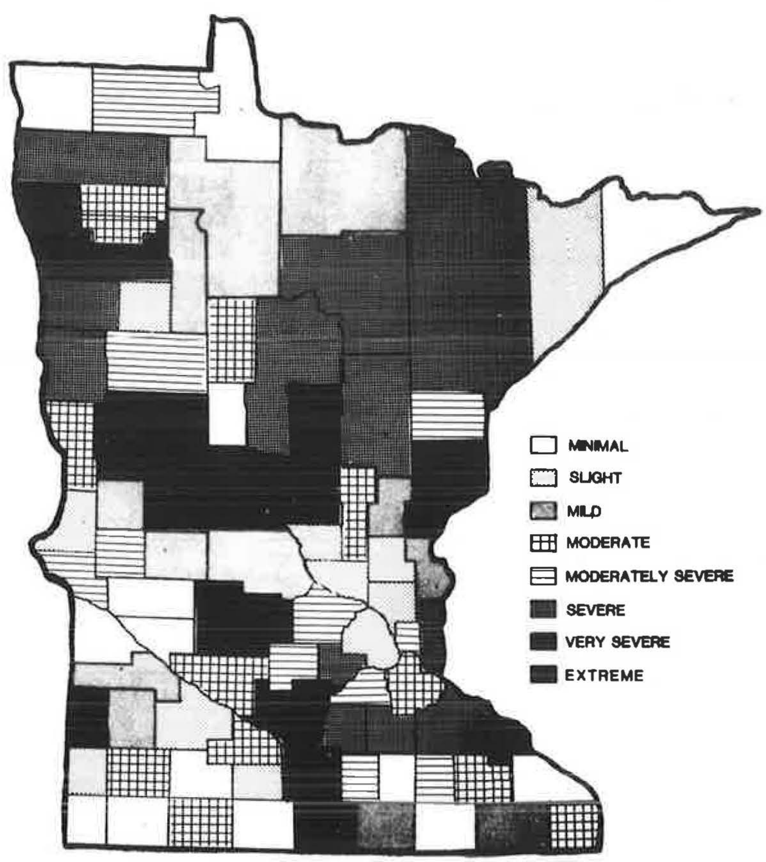


Figure 4. Distribution of total occurrence ratings of erosion on all roads by county.



Mileage per county ranged from 432 to 4,390. The average mileage per county was 1,329 miles. Volume per mile estimates per county ranged from 17 to 27,173 ft^3. The average volume per mile statewide was 1,006 ft^3. Erosion was less severe than the volume estimates indicated in counties that had more miles of road than the average and was more severe in counties that had many fewer miles of road than the average. About 27 percent of the counties had more than the average volume of erosion per mile.

The frequency of erosion sites per county ranged from one site every 167 miles to one site every 1.15 miles. The average frequency of erosion sites statewide was 0.17, or one site every 5.88 miles. Approximately 32 percent of the counties had more than the average number of sites per mile. As with the estimates of volume per mile, counties with many more miles than the average had less frequent occurrences of erosion per mile than indicated by the occurrence data and counties with many fewer miles had more frequent occurrences of erosion per mile than indicated by the occurrence data.

Volumes per occurrence indicate the average size of erosion sites in the various counties:

Range	Avg Volume per Occurrence (ft^3)
Low	230
High	37,006
Median	3,008

The average size of erosion sites ranged from 230 to 37,006 ft^3/county. The statewide average size of a site was 6,492 ft^3. In general, counties that had low volumes and few occurrences had low volumes per occurrence. Some counties had low volume estimates but a high number of sites. In these counties, erosion is more a potential problem than an actual one. A few counties had high volume estimates but a low number of occurrences. In these counties, large erosion sites were generally found in proximity to waterways. Counties in which both volume and number of occurrences were high produced high estimates of volume per occurrence. Causes of erosion at these sites included proximity to rivers and streams, rough terrain, lack of vegetative cover, and lack of past administrative emphasis on prevention, control, and repair of erosion sites.

Erosion According to Road Design

Data on mileage of individual road designs are not available. However, the at-grade design is the most common and thus has the most mileage (any cross section with less than 4 ft of vertical variance up or down was defined as at-grade). The next most commonly constructed roadway cross sections are the fill design and the cut design. The cut-fill roadway design has the least mileage. The cut-fill cross-sectional design had been perceived as the most economical to build in areas of rough terrain. Other cross-sectional designs (as noted in the survey forms) were generally a combination of or a transition between the preceding types.

The greatest volume of erosion was found on roadways built according to the cut-fill design. An estimated 39,470,568 ft^3 of soil was lost along roadways of this design (Table 1). More soil and drainage patterns, both surface and subsurface, are distributed along the roadway during construction with the cut-fill design than with any other design. Soils and rock are exposed during the cut and moved to the lower fill areas, which often produces unevenly consolidated heterogeneous soil mixtures.

In the construction and maintenance of cut-fill-design roads, problems often result from the use of heterogeneous roadbed materials, introduction of less stable materials, and the intersection of groundwater. Other problems relate to altered drainage ways, differential soil consolidation resulting from nonuniform compaction of the fill slope during construction, and related structural instabilities. The potential for erosion appears higher along newly constructed cut-fill-design roads than along roadways of any other age or design.

In one county, Wabasha, 17,650,000 ft^3 of soil had been lost along cut-fill roads (probably an overestimate). Most of the roads in the rough terrain of Wabasha County were constructed as cut-fill roads. Deletion of the data for this county reduces the total volume of soil loss along cut-fill roads to 21 million ft^3. This volume is still significant because the cut-fill design is much less frequently constructed than the at-grade, fill, or cut design.

The total volume of erosion recorded on at-grade-design roads was 35,751,365 ft^3. Early roads in Minnesota were often constructed as at-grade roads. Ditches were generally V-bottomed with steep slopes that tended to slide, slump, and cave. As these early at-grade roads are reconstructed, inadequate ditch designs are corrected and the potential of future erosion should be reduced. However, problems associated with agricultural drainage generally occur on at-grade roads and are difficult to correct. Because many more miles have been constructed with the at-grade design than with other designs, actual erosion on the at-grade road is less severe than the total volume estimate indicates.

The total estimated volume of erosion on cut-design roads was 31,484,429 ft^3. Because the cut design is considered to be less common than either the at-grade or the fill road, the volume of erosion noted for the cut design is more severe than comparison of volumes by design indicates. Deletion of the Wabasha data does not significantly reduce the amount of soil lost along cut-design roads.

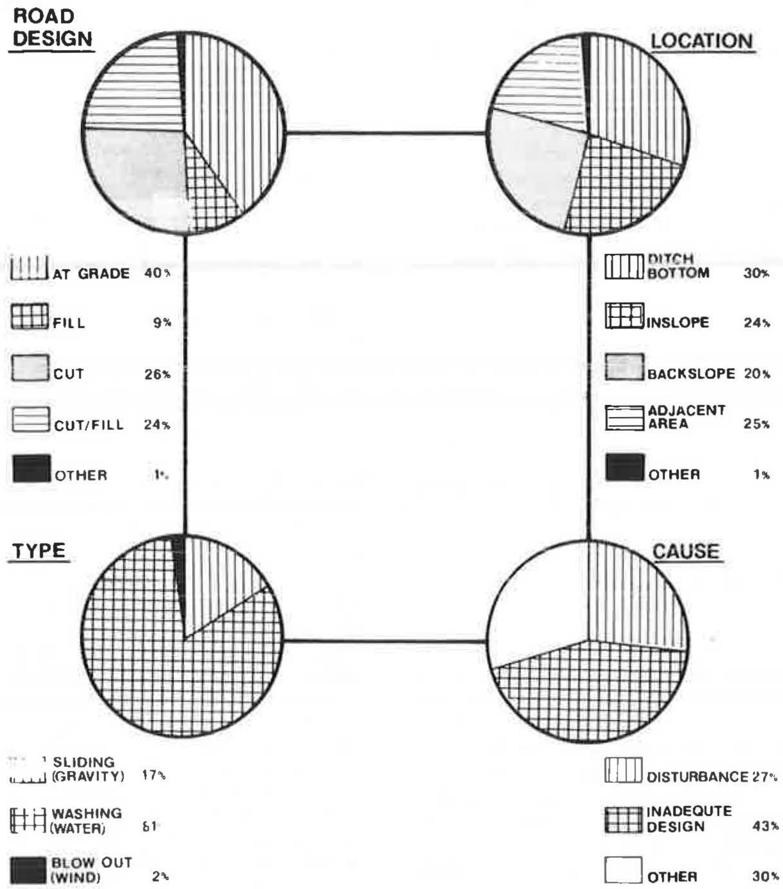
An estimated soil loss of 9,502,009 ft^3 was noted for all other road designs. Considering the infrequency with which other designs was noted (just 184 occurrences), the magnitude of this volume mostly associated with transitions should be emphasized.

The lowest estimated volume of erosion--8,155,015 ft^3--was along the fill-design road. During selection of material to be used as fill, undesirable soils are rejected and only stable, more granular soils are accepted. A reduction in the amount of heterogeneous soils used in constructing the roadbed reduces the potential of erosion. Problems relating to the design and construction of fill roads usually involve the availability of suitable materials and the proximity of these materials to the construction site. Because the fill design is the second most commonly used road design, the low volume of erosion recorded along fill-design roads is important to note.

Erosion occurred most frequently on at-grade roads (Table 1 and Figure 5). A total of 7,181 sites, or 40 percent of all occurrences noted by design, were on at-grade roads. This is expected because there are many more miles of at-grade roads than of any other cross-sectional road design and the sites on at-grade roads are, on the average, the smallest in size.

A total of 4,672 sites, or 26 percent, were along cut-design roads and 4,199 sites, or 24 percent, were along cut-fill roads. There were fewer sites along both cut and cut-fill roads than along at-grade roads. However, cut-fill roads are less common than cut roads and both are much less common than at-grade roads. Because of this, erosion was considered more frequent on cut-fill roads than on cut roads and more frequent on both than on at-grade roads.

Figure 5. Erosion factors according to occurrences of roadside erosion.



Erosion sites along fill roads constituted only 9 percent of the total--1,602 sites. Because fill roads are the second most commonly constructed cross-sectional design type, the infrequent occurrence of erosion along roads of this design is important.

There were 184 occurrences of erosion on roads of other designs. This represented about 1 percent of the total sites, but the average volume per occurrence was high: 51,641 ft^3, the highest of any road design type. Lack of vegetation is the most common cause of erosion at the largest sites. The road design with the second largest average site was the cut-fill design (9,400 ft^3); this was followed by the cut, fill, and at-grade designs (6,739; 5,091; and 4,934 ft^3/site, respectively) (Table 1).

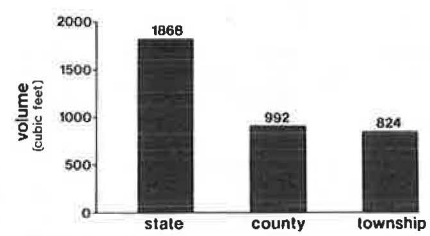
Erosion According to Ownership

On a statewide basis total estimated volume and occurrence of erosion were slightly more along township roads than along county roads. The results estimated 45,859,258 ft^3 of soil, or 41 percent of the total volume, to have been eroded along township roads. An estimated 44,522,514 ft^3, or 39 percent, was recorded along county roads. The total reported along state roads was less: 22,773,360 ft^3, or 20 percent. The number of sites on township roads was 8,660, or 49 percent of the occurrences. Along county roads 7,962, or 45 percent of the sites, were noted. A total of 1,080 sites, or 6 percent, were along state roads.

Minnesota has 55,367 miles of township roads, 44,683 miles of county roads, and 12,189 miles of state roads. The statewide erosion volume per mile was 1,004 ft^3. Because of an extremely large volume of erosion noted on state roads in Wabasha County, 18,337,000 ft^3, the estimated volume per mile by ownership was largest along state roads: 1,868 ft^3. Exclusion of the Wabasha data reduces this estimate for state roads to 326 ft^3/mile. The estimated volumes per mile for township and county roads were 824 and 992 ft^3, respectively (see Figure 6).

The average occurrence per mile by ownership was 0.157, or one site every 6.42 miles. The frequency of erosion along both township and county roads was similar to the statewide average--0.156, or one site every 6.53 miles, along township roads and 0.177, or one site every 5.65 miles, along county roads. The average frequency of erosion along state roads was 0.086, or one site every 11.63 miles (see Figure 7).

Figure 6. Average volume of erosion per mile of roadway by ownership.



Volume per Occurrence

Again, because of the extremely large volume of erosion noted on state roads in Wabasha County, the estimated volume per occurrence was highest along state roads: 21,689 ft³/site (see Figure 8). If the erosion data for Wabasha County are deleted, the estimated volume per occurrence along state roads is 4,274 ft³/site. The average size per site is somewhat more along both county and township roads: 5,591 and 5,296 ft³, respectively. The estimated average volume per occurrence statewide was 6,428 ft³.

Location and Type of Erosion

Each occurrence was analyzed according to location of the erosion on the roadway cross section and to type of erosion. Because erosion occurred in more than one location on the cross section or was of more than one type, the totals according to location and type of erosion differ from the grand total compiled according to occurrence.

The most frequently noted location of erosion was ditch bottoms (11,390 sites, or 30 percent). Loss of soil from adjacent areas and in-slopes was noted somewhat less (9,339 occurrences, or 25 percent, and 9,223 sites, or 24 percent). Erosion from backslopes was less frequent (7,394 sites, or 20 percent). Other areas were identified 406 times (Figure 5).

The type of erosion sited most frequently was washing or water-related. Erosion by water accounted for 15,309 occurrences, or 81.5 percent of the roadside sites. Sliding or gravity-related soil loss accounted for 3,183 occurrences, or 16.8 percent of the sites. Wind-related erosion accounted for 1.7 percent, or 284 occurrences (Figure 5).

History of Roads

In general, erosion tended to occur more frequently on older roads (more than 3 years since construction or reconstruction), but the size of erosion sites was larger along new roads [3 years or less since construction or reconstruction (Table 1)]. Present road standards require more disturbance of the landscape.

Figure 7. Occurrences of erosion per mile of roadway by ownership.

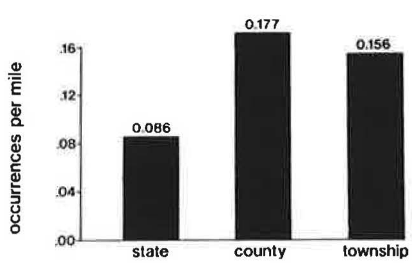
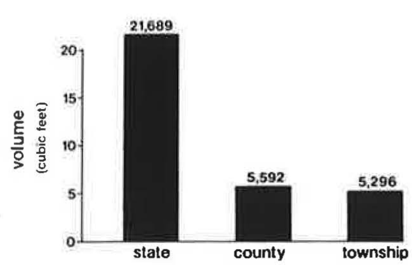


Figure 8. Volume of erosion per occurrence by ownership.



Causes of Erosion

The causes of erosion noted on the data forms were inadequate design, disturbance, or other. The greatest part of the volume--43.7 percent--was attributed to inadequate design of the road. Disturbance was responsible for 26.7 percent, and other causes for 29.5 percent (Figure 5).

In addition to the causes noted on the survey forms, more specific causes were identified under a note or comment column for 24 percent of the total volume. According to the comments, lack of vegetation was the single most important cause of erosion along roadsides. Loss of topsoil from roadsides lowers soil fertility and reduces the ability of slopes and ditches to revegetate and control future erosive action. The second most frequent cause related to agricultural activities, including drainage ditches, unauthorized farming of roadsides, surface drainage or field wash, livestock, and tile outlets. These causes of erosion are often beyond the control of the road authorities.

Other important reasons for erosion identified from the survey notes include steep slopes and rough topography; river fall; improper design of roads, especially near streams; poor soils; use of roadsides as borrow areas; lack of funds for correcting existing erosion problems; and lack of past administrative emphasis on controlling erosion.

Corrective Measures

Three levels of corrective measures were recommended. The first level, tillage and establishment of vegetation, was recommended for 27 percent of the sites (Table 1). This form of control is least costly and easiest to implement. The second level includes the first level plus slope shaping and is more costly, requiring some physical alteration of the roadside slope. Second-level controls were recommended for 58 percent of the sites. Third-level controls are most costly, involving the greatest modification of existing conditions. Third-level measures include secondary controls plus installation of some type of structural correction and were suggested for 15 percent of the sites.

The counties were also asked to identify corrective measures needed to control erosion and corrective measures implemented after completion of the survey and to discuss special problems regarding the control of erosion in each county. According to responses regarding the need for and implementation of corrective measures, the need for structural correction was overestimated by the survey and the importance of vegetative cover was significantly underemphasized. Of the 55 counties that responded regarding implementation, 18 had completed second-level controls, 10 had implemented first-level controls, and 14 had installed structures. Most controls had been completed on county and state roads. Townships did not have the funds to correct many erosion problems, especially those that required the more costly reshaping and structural controls. Lack of funds was identified as the major factor preventing correction of many erosion problems.

CONCLUSIONS

Differences in the amount and occurrence of roadside erosion do exist. The larger sites of roadside erosion in Minnesota are generally caused by

1. Inadequate design of the roads, especially in areas of rough terrain or poor soils and near streams or rivers, and
2. Lack of administrative direction and emphasis, especially in the establishment of vegetative cover, maintenance repair of erosion sites, and control of unauthorized activities along the roadside, such as farming and use of the roadside for borrow or for recreational purposes.

In the design of new or reconstructed roads, a reduction in the construction of cut-fill-design roads and an increase in the construction of fill roads, especially in areas of rough terrain, should significantly reduce the potential of future roadside erosion. Special consideration is needed to control erosion in areas of transition between cross-sectional designs. If vegetative cover can be established during or soon after construction, erosion can be minimized. Technical emphasis and administrative direction are also important in the control of erosion. This emphasis and direction has been identified as the major factor producing differences in erosion between counties with similar topography, soils, climate, and land use.

Frequent occurrences of erosion are often associated with

1. Drainage from ditches mandated by statute, agricultural lands, and private developments;
2. Rough terrain or steep slopes;
3. Inadequate design of roads in areas of rapid changes in topography; and
4. Lack of technical emphasis and administrative direction.

Because road authorities in Minnesota are required by statute to accept natural drainage from adjacent areas into the road ditch system (including drainage ditches), it is difficult for the road authorities to control erosion associated with drainage from adjacent areas. A cooperative effort by all authorities involved will be needed to control this type of erosion problem. Selection of the fill-design road in areas of rough terrain and administrative emphasis on the need for erosion control should reduce the occurrences of erosion along roadsides.

Many corrective measures were recommended, and in many counties these recommendations have been implemented. However, some erosion sites have remained unchecked and other sites have increased in size. Lack of funds is the most frequently identified reason for not repairing or correcting erosion. Townships in particular do not have money available for erosion control. An increase in funds and technical assistance for controlling and preventing erosion is needed on the township level.

The results of this survey have made it possible to

1. Determine the extent and type of roadside erosion,
2. Evaluate the present application of erosion control techniques,
3. Determine the corrective measures required,
4. Determine highway alignment and design deficiencies and the roadway cross sections with the greatest erosion potential,
5. Determine and locate conditions and situations that are apt to result in severe roadside erosion,
6. Locate problem soil types,
7. Locate private roads and other land use contributing to the sedimentation problem, and
8. Note other factors contributing to the roadside erosion problem.

The county surveys, data sheets, and maps have served to help develop township or county programs for additional action on controlling erosion. The results should be reduced sedimentation and pollution, improved highway safety and roadway stability, reduced highway operations and maintenance costs, improved water quality and wildlife habitat, and enhanced aesthetics in the rural landscape.

ACKNOWLEDGMENT

The Minnesota roadside erosion survey resulted from interagency coordination on the state, county, and township levels. It is an excellent example of what can be accomplished through such an effort. The survey was a joint effort of the county engineers, the Agricultural Extension Service, the Minnesota Department of Transportation (DOT), the Soil Conservation Service, and the Soil and Water Conservation Districts. The Minnesota chapter of the Soil Conservation Society of America initiated the survey by establishing a special committee to plan and guide the statewide erosion survey. Lawrence E. Foote was chairman. John Bedish and Maynard Scilley of the State Soil Conservation Service were vice-chairmen. The support of W.S. Ekern (retired) and Gordon Fay of the Minnesota DOT greatly helped in starting the survey. Chapter presidents of the Soil Conservation Society of America continue to support the effort. Appreciation is extended to all who conducted the survey or helped compile or write the county reports.

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Scour at Culvert Outlets in Multibed Materials

STEVEN R. ABT, JAMES F. RUFF, AND CESAR MENDOZA

An investigation of scour at culvert outlets in noncohesive and SC-type cohesive materials is presented. A series of empirical equations was derived to predict the depth, width, length, and volume of scour downstream of culvert outlets under various field and laboratory conditions. The rate of scour was determined for each material. The effects of culvert shape, tailwater depth, and headwall presence were correlated with the ultimate scour dimensions. The area affected by the scour process was quantified as a function of culvert diameter and discharge. General observations on scour-hole formation, growth, and stabilization are reported.

One of the major considerations in the design and construction of a roadway system is the conveyance of tributary drainage through the roadway embankment. As drainage waters are conveyed through the embankment, flow discharges from the culvert and impinges on the material beneath the outlet. The impinging jet lifts the material particles and transports those particles downstream of the impact area. The jet impact area is transformed into an energy dissipator, and a hole is created at the outlet. The eventual result of this scour and erosive process, if it is left unchecked, is degradation of the roadway embankment, degradation of the area beneath and adjacent to the culvert outlet, and aggradation of the channel, land areas, or properties downstream of the outlet. Because of these severe and often costly damages, the study of localized scour is an important step in the evaluation, control, and management of roadway embankment erosion.

The investigation of scour at culvert outlets has continued for several decades. Early studies, beginning with Rouse (1) and Laursen (2), attempted to understand the general nature and principles of scour. Scour was observed to be a function of discharge, time, and material characteristics. A number of investigations have identified the significant characteristics of localized scour at culvert outlets. These studies address the following general topics: the effects of falling jets (3) and the degree of armor plating in gravel bed materials (4); the effect of headwalls on predicting scour in sand bed materials (5); the effects of material shape (6), tailwater (7,8), and jet impingement on riprap materials (9); and the prediction of scour in a cohesive material (10). In most cases emphasis is placed on predicting the extent and dimensions of the degradation processes.

The most notable studies were those of Bohan (7) and Fletcher and Grace (11) of the U.S. Army Engineer Waterways Experiment Station. They formulated a series of empirical equations that predicted the length, depth, width, and volume of scour as a function of discharge, culvert diameter, and time. Bohan and Fletcher and Grace realized the significant effect of tailwater conditions on ultimate scour-hole dimensions. Therefore, their prediction equations were generalized to encompass a spectrum of tailwater conditions. Although several studies followed the investigations of Bohan and Fletcher and Grace, their work has been adopted as the most comprehensive design criteria available for the prediction of scour-hole dimensions.

The ability to predict the magnitude and geometry of localized scour and subsequent deposition at culvert outlets is a useful evaluation tool in the control and management of erosion along roadway embankments. However, because previously developed equations are considered conservative, it is advantageous to investigate localized scour in noncohesive and cohesive materials at culvert outlets.

EXPERIMENTAL FACILITIES AND PROCEDURES

In the investigation of scour at culvert outlets in mixed-bed materials, five materials were used under a variety of conditions. Materials were tested in two facilities with culverts ranging in diameter from 4 to 18 in. (10.2 to 45.7 cm). Test periods ranged from 316 to 1000 min in duration and discharges ranged from 0.11 to 29.13 ft^3/sec (0.003 to 0.83 m^3/sec). A summary of the experimental materials, models, discharges, and test durations is given in Table 1.

Experimental Facilities

Two hydraulic flumes were used for conducting the scour investigation. The initial testing program was conducted in an outdoor concrete flume 100 ft (30.5 m) long, 20 ft (6.1 m) wide, and 8 ft (2.4 m) deep. A smooth pipe (culvert) was horizontally cantilevered through the headwall between the sidewalls and extended approximately 6 ft (1.9 m) into the test section. Bed material was placed adjacent to the pipe invert and leveled with a minimum thickness of 6 ft.

The scope of the study was expanded to incorporate a smaller indoor flume 15 ft (4.5 m) long, 4 ft (1.2 m) wide, and 2 ft (0.6) deep. The indoor facility was a 1:5 Froude scale model of the outdoor flume.

Description of Bed Materials

Four noncohesive materials and one cohesive soil were used as bed materials for the scour investigation. The soil properties of each bed material were obtained and recorded in accordance with procedures outlined in ASTM specifications.

Noncohesive Materials

The four noncohesive materials tested included a uniform sand, a uniform gravel, a graded sand, and a graded gravel. The soil properties of the noncohesive material are summarized in Table 2. The specific gravity of the noncohesive material source was determined to be 2.65.

Cohesive Bed Material

The cohesive material was derived from a residual Colorado expansive clay mixed with a graded sand. The tan-green sandy clay mixture is classified as an SC soil type in accordance with the Unified Soil Classification system. An agricultural analysis further categorized the material as a sandy loam composed of 58 percent sand, 27 percent clay, and 15 percent silts and organic matter. The cohesive material properties are summarized in Table 3.

Test Procedure

The bed was leveled adjacent to the culvert invert elevation. Water was directed from the water source

Table 1. Test program parameters.

Material	d_{50} (mm)	Model (ft)	Pipe Diameter (in.)	Discharge Range (ft^3/sec)	Times of Data Collection (mins)	Tailwater Elevation[a]
Uniform sand	1.86	20	4	0.11- 1.14	31,100,316	0.45
		20	7	1.15- 7.65	31,100,316	0.00
		20	7	1.15- 7.65	31,100,316	0.25
		20	7	1.89- 9.45	31,100,316,1000	0.45
		20	13.5	3.85-19.26	31,100,316,1000	0.45
		20	17.5	7.31-29.23	31,100,316,1000	0.45
		4[b]	4	0.16- 0.91	31,100,316,1000	0.45
		4[c]	4	0.16- 0.91	31,100,316,1000	0.45
		4	4x4	0.25- 1.16	31,100,316	0.45
Graded sand	2.0	4	4	0.18- 0.73	31,100,316	0.45
Uniform gravel	7.62	20	3	1.91- 7.65	31,100,316	0.0
		20	3	1.91- 7.65	31,100,316	0.25
		20	3	1.91- 7.65	31,100,316	0.45
Graded gravel	7.34	20	7	1.91- 7.65	31,100,316	0.45
Cohesive	0.15	20	7	1.91- 7.65	31,100,316,1000	0.45
		20	13.5	3.81-15.23	31,100,316,1000	0.45
		20	16	7.28-29.13	31,100,316,1000	0.45

[a]Water depth as a portion of culvert diameter.
[b]Without headwall.
[c]With headwall.

Table 2. Properties of noncohesive materials.

Soil Type	d_{50} (mm)	Standard Deviation[a]	Unit Weight (lb/ft^3)	Angle of Repose (°)	Fall Velocity (cm/sec)
Uniform sand (medium)	1.86	1.33	93.8	34.8	27.1
Graded sand	2.00	4.38	105.9	31.8	27.3
Uniform gravel	7.62	1.32	94.4	37.3	63.0
Graded gravel	7.34	4.78	117.9	37.3	64.0

[a]Standard deviation (σ) is $(d_{84}/d_{16})^{0.5}$

Table 3. Properties of cohesive materials.

Characteristic	Characteristic Value
Soil type	SC
Texture	Sandy loam
Atterberg limits	
Liquid limit	34
Plastic limit	19
Plastic index	15
Soil composition (%)	
Organic matter	1
Sand	58
Silt	14
Clay	27
pH	7.8
Mean grain size (mm)	0.15
Uniformity coefficient	300
Fall velocity (d_{50}) (ft/sec)	0.08
Cation exchange capacity (meq/100g)	9.0
Soil fabric	Dispersed
Dispersivity of colloid fraction	Nondispersive
Permeability (cm/sec)	6.4×10^{-6}

to a sump located downstream of the material testing basin until the water reached the desired tailwater elevation. The control valve was gradually opened until the discharge reached the desired level. The tailwater elevation was adjusted and maintained above the bed at levels given in Table 1.

In general, scour profiles were taken after predetermined time periods measured from the beginning of each test. The model bed was mapped by using a point gauge.

Discharge Intensity

The Froude number of circular sections ($\overline{V}g^{-0.5}D^{-0.5}$) was modified to $Qg^{-0.5}D^{-2.5}$, where $V = Q/A$ and $A = \pi D^2/4$. This form of the Froude number is referred to as the discharge intensity (DI).

RESULTS AND DISCUSSION

General Observations

Noncohesive Materials

A series of 75 scour tests was conducted, observed, and documented as water discharged from a culvert outlet onto a bed of noncohesive materials. Scour holes were generally similar in geometric configuration and appearance. They were circular in shape at low DI (DI $\leq$ 1.0) and elongated to an oval shape as the DI exceeded 1 (DI > 1.0).

Scour holes were created by a water jet striking a horizontal bed of noncohesive material. The force of the jet and subsequent turbulence lifted and entrained the material particles. Large-diameter materials were transported as bed load along the bottom of the scour hole and along bed downstream of the hole. Smaller materials were entrained by the flow and deposited around the rim of the hole, in the subsequent dune or mound downstream from the hole, or in the material settling basin. The mounds that formed were fan-shaped for DI < 1.0 and became elongated as the DI reached 1 (DI $\geq$ 1.0). The surface of the mounds was flat, paralleling the water surface. The mound height was generally observed to be approximately 0.6 to 0.8 of the tailwater depth.

Cohesive Material

A series of 12 scour holes was observed and documented for the cohesive bed material. The scour holes were generally similar in geometric configuration. Scour holes were circular in shape at low DI (DI $\leq$ 1.0). As the DI increased (DI > 1.0), the holes elongated to an oval shape.

The force of the water jet striking the bed weakened the cohesive bonds of the material and dislodged particles from the bed. The material was then lifted and entrained into the turbulent flow. Large-diameter materials (sands and clods) were transported as bed load along the bottom of the scour cavity and deposited immediately downstream of the jet impact area. A mound subsequently formed downstream of the cavity. The smaller materials (clay and silt particles) were entrained by the flow and trapped in void spaces along the mound or transported to the material settling basin. Each mound was generally flat, less than 0.25D in height, and fan-shaped downstream of the cavity. The mound was primarily composed of large-diameter sands and clods and fine material filling the void spaces.

Considerable deposition of sands and clods was observed around the rim of the hole at the conclusion of each experiment. This apparent armoring effect consistently occurred at the downstream face and along the rim of all scour holes. Limited armoring was observed within the scour hole. Armoring materials could not be supported along the cavity walls due to steep sideslopes and vertical sidewalls. In some cases cantilevering occurred.

Quantitative Results

After the scour tests were completed, an analysis was conducted to correlate the depth, width, length, and volume of scour with materials, culvert, and discharge. Scour-hole depth, width, length, and volume are expressed by the following dimensionless parameters:

Characteristic	Parameter
Depth	d_{sm}/D
Width	W_{sm}/D
Length	L_{sm}/D
Volume	V_{sm}/D^3

The relationships presented here are based on the maximum scour-hole dimensions. Test durations were in accordance with the times given in Table 1.

Graphical representations were compiled correlating the dimensionless depth, width, length, and volume parameters with DI, as shown in Figure 1 for uniform sand (d_{50} = 1.86, σ = 1.33, and t_o = 1000 min). A power regression line was fit through each logarithmic plot, which yielded a series of expressions of the following general form:

$$y = a\,x^b \tag{1}$$

where

y = dependent variable of d_{sm}/D, W_{sm}/D, L_{sm}/D, or V_{sm}/D^3;
a = a constant; and
b = slope of the linearized plot.

From these expressions it was evident that the maximum scour-hole characteristics of depth, width, length, and volume can be correlated with the culvert diameter (D) and culvert discharge (Q). Replacing the independent variable of Equation 1 with the DI yields the following expression for d_{sm}/D or any of the other dimensionless parameters:

$$d_{sm}/D = a\,(Q/g^{1/2}\,D^{5/2})^b \tag{2}$$

The DI relationships yield a conservative estimate of scour-hole dimensions for partly filled culverts (DI < 1.0).

A number of empirical relationships were similarly formulated for the cohesive soil correlating the four maximum dimensionless scour characteristics with the inverted shear number (S_n), as shown in Figure 2. The inverted shear number is

$$S_\eta = \rho\overline{V}^2/\tau_c \tag{3}$$

where the critical shear stress (τ_c) is

$$\tau_c = 0.001\,(S_v + 130)\tan(30 + 1.73\,I_p) \tag{4}$$

The maximum scour-hole depth, width, length, and volume can be correlated with D, average velocity of fluid at the culvert outlet ($\overline{V}$), fluid density (ρ), saturated soil shear stress (S_v), and soil plasticity index (I_p). Replacing the independent variable of Equation 2 with the inverted shear number yields the following equation for any of the four dimensionless parameters:

$$d_{sm}/D = a\,(\rho\overline{V}^2/\tau_c)^b \tag{5}$$

Figure 1. Dimensionless scour-hole parameters versus DI for uniform sand, where d_{50} = 1.86 mm, σ = 1.33, and t_o = 1,000 min.

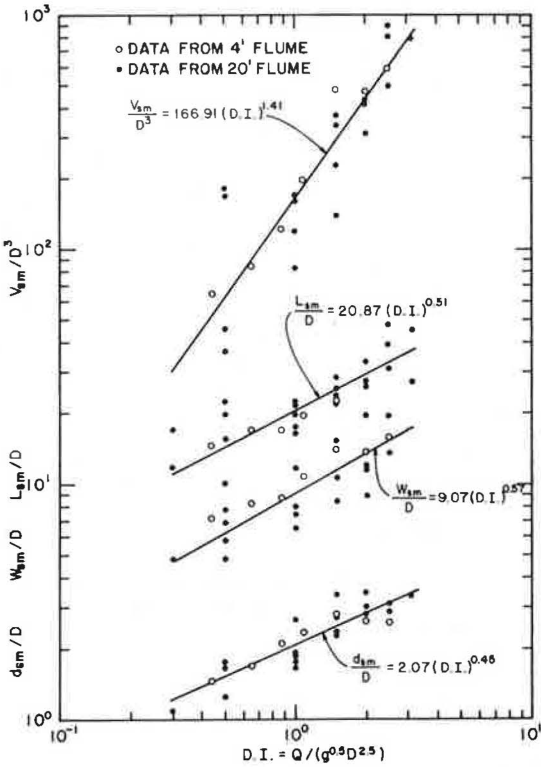
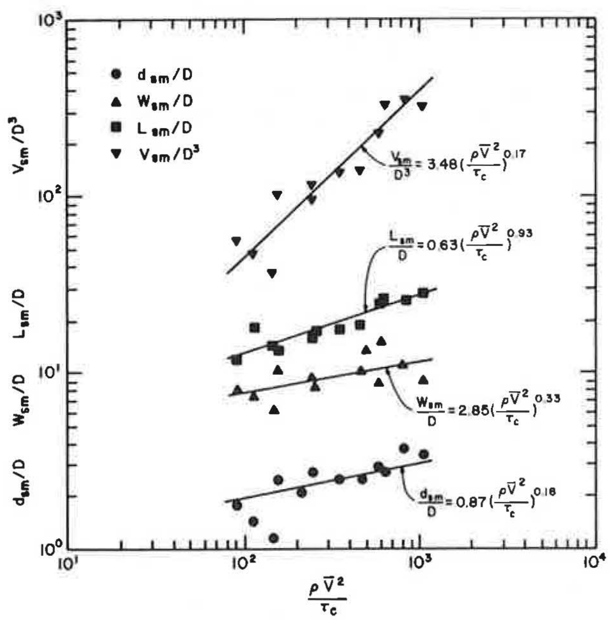


Figure 2. Characteristic dimensions of scour versus inverted shear number.



Time Relationships

Scour-hole measurements were taken at 31, 100, and 316 min for all of the noncohesive materials; a portion of the uniform sand tests and all of the cohesive soil tests extended to 100 min in duration. The observed scour-hole characteristics were evaluated for each time interval. Characteristic values were normalized with reference to the final or maximum values obtained after the appropriate test durations.

After 31 min of testing, a minimum of 80.0 percent of the 316-min scour depth was attained independent of the type of material tested. Furthermore, the 31-min values for scour-hole width, length, and volume averaged 83, 80, and 61 percent of the 316-min values, respectively. The 31-min rate of scour of the cohesive material was close to the average 31-min rate of scour of the noncohesive materials.

A comparison of the 316-min scour-hole dimensions with the 1000-min scour-hole dimensions indicates that, although the duration of scour extended 684 min (216 percent) longer, the scour-hole dimensions increased on an average by 14 percent in depth, 7 percent in width, 16 percent in length, and 46 percent in volume.

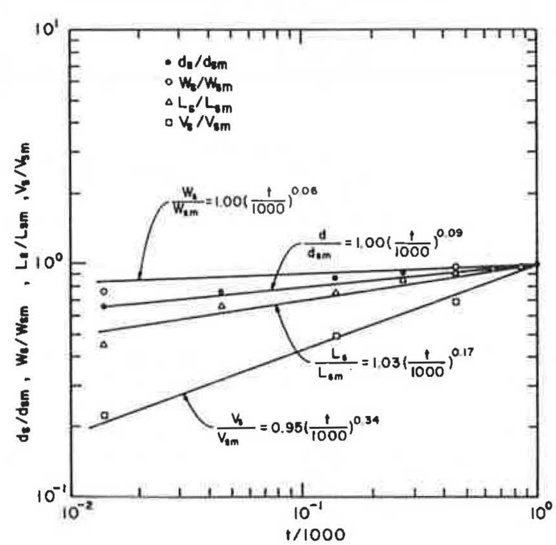
Logarithmic plots of normalized scour-hole characteristics versus normalized time (t/t_o) are shown in Figure 3. In this case, t is any time less than or equal to the duration of scour (t_o). By using the power relationship depicted in Equation 1, a number of regression curves were fit to the data where time is the independent variable.

Scour Relationships

By using the dimensionless parameters and characteristic relationships developed thus far, it is possible to formulate equations that estimate scour-hole dimensions at any finite time between 31 min and the test duration of 316 to 1000 min. Combining Equation 1 with the time expression yields an equation that relates a desired hole characteristic to its maximum value as a function of time. The resulting equation is

$$y = a\,(x)^b * (t/t_o)^c \qquad (6)$$

Figure 3. Normalized scour-hole characteristics versus normalized time for uniform sand.



where

- a = a coefficient;
- b = slope of the desired characteristic curve;
- c = slope of the desired time relationship;
- x = independent variable of $Qg^{-0.5}\,D^{-2.5}$ or $\rho V^2(\tau_c)^{-1}$; and
- y = dependent variable of d_s/D, W_s/D, L_s/D, or V_s/D^3.

Furthermore, because some of the tests were run for a 316-min duration and others were run for 1000 min, the coefficient a can be multiplied by the appropriate time normalization percentages to ensure that all of the materials will have the same divisor for the time parameter.

Table 4 gives a summary of the coefficients and exponents for each material and scour-hole parameter. Exponents b and c are the same as the component regression analyses for each independent variable. The coefficients a are the product of the coefficients from the component regression analyses for each independent variable times the normalization percentages d_{s316}/d_{s1000}, W_{s316}/W_{s1000}, L_{s316}/L_{s1000}, and V_{s316}/V_{s1000} for the runs that extended to 1000 min.

A compilation of maximum scour-hole depths versus DI for four uniform materials and two graded materials is shown in Figure 4.

Tailwater Effects

Tests were run to investigate further the effects of tailwater (TW) depth on scour-hole dimensions. Previous tests by Bohan (7) and Fletcher and Grace (11) of the U.S. Army Corps of Engineers demonstrated that maximum scour occurred when TW was below the culvert center line. Tests were run at TW of zero, 0.25D, and 0.45D to determine where scour tended to be the greatest.

The depth, width, length, and volume of scour were correlated with the DI for each TW elevation as in the plots shown in Figure 5. Little difference in scour is observed between zero TW and 0.25D TW for the sand bed material. As the tailwater was raised from 0.25D to 0.45D, the depth and width of scour decreased while the length and volume of scour increased. Overall, scour-hole dimensions at 0.25D TW were no more than 10 percent greater than tests at 0.45D TW. Little difference was observed in the maximum scour-hole dimensions as the TW varied from zero to 0.45D for DIs of ≥ 1.5.

Effects of Culvert Shape

Test runs were performed to investigate how the shape of a culvert affected scour-hole depth, length, and volume. Only circular and square culverts were considered in this analysis.

Culverts were sized so that the diameter of the circular culvert was equivalent to the length of one side of the square culvert. Because the cross-sectional areas of the two culverts were not identical, it was necessary to compare the results based on parameters other than DI and D.

Analyses were performed by using an equivalent depth parameter (Y_e). The equivalent depth is a characteristic length applicable to culverts of any shape and is expressed as

$$Y_e = (A/2)^{1/2} \qquad (7)$$

where A is the cross-sectional area of flow.

A comparison of equivalent depth with three of the dimensionless scour-hole parameters--d_{sm}/Y_e, L_{sm}/Y_e, and V_{sm}/Y_e^3--is shown in Figure 6. It is observed

Table 4. Equation coefficients and exponents in terms of constant 316-min duration.

Material	d_{50} (mm)	σ	Dependent Variable (y)	Independent Variable (x)	a	b	c
Uniform sand	1.86	1.33	d_s/D	$Qg^{-0.5}D^{2.5}$	1.86	0.45	0.09
			W_s/D	$Qg^{-0.5}D^{2.5}$	8.44	0.57	0.06
			L_s/D	$Qg^{-0.5}D^{2.5}$	18.28	0.51	0.17
			V_s/D^3	$Qg^{-0.5}D^{2.5}$	101.48	1.41	0.34
Graded sand	2.00	4.38	d_s/D	$Qg^{-0.5}D^{2.5}$	1.22	0.82	0.07
			W_s/D	$Qg^{-0.5}D^{2.5}$	7.25	0.76	0.06
			L_s/D	$Qg^{-0.5}D^{2.5}$	12.77	0.41	0.04
			V_s/D^3	$Qg^{-0.5}D^{2.5}$	36.17	2.09	0.19
Uniform gravel	7.62	1.32	d_s/D	$Qg^{-0.5}D^{2.5}$	1.78	0.45	0.04
			W_s/D	$Qg^{-0.5}D^{2.5}$	9.13	0.62	0.08
			L_s/D	$Qg^{-0.5}D^{2.5}$	14.36	0.95	0.12
			V_s/D^3	$Qg^{-0.5}D^{2.5}$	65.91	1.86	0.19
Graded gravel	7.34	4.78	d_s/D	$Qg^{-0.5}D^{2.5}$	1.49	0.50	0.03
			W_s/D	$Qg^{-0.5}D^{2.5}$	8.76	0.89	0.10
			L_s/D	$Qg^{-0.5}D^{2.5}$	13.09	0.62	0.07
			V_s/D^3	$Qg^{-0.5}D^{2.5}$	42.31	2.28	0.17
Cohesive sandy clay		0.15	d_s/D	$Qg^{-0.5}D^{-2.5}$	1.86	0.57	0.10
			W_s/D	$Qg^{-0.5}D^{-2.5}$	8.63	0.35	0.07
			L_s/D	$Qg^{-0.5}D^{-2.5}$	15.30	0.43	0.09
			V_s/D^3	$Qg^{-0.5}D^{-2.5}$	79.73	1.42	0.23
Cohesive sandy clay		0.15	d_s/D^3	$\rho V^2 \tau_c^{-1}$	0.86	0.18	0.10
			W_s/D	$\rho V^2 \tau_c^{-1}$	3.55	0.17	0.07
			L_s/D	$\rho V^2 \tau_c^{-1}$	2.82	0.33	0.09
			V_s/D^3	$\rho V^2 \tau_c^{-1}$	0.62	0.93	0.23

Note: Modified equation: $y = a\,(x)^b \cdot (t/316\ \text{min})^c$, where $t \leqslant 1000$ min and $t \geqslant 31$ min.

Figure 4. d_{sm}/D versus DI for cohesive and noncohesive material (curves adjusted to common time base of 316 min).

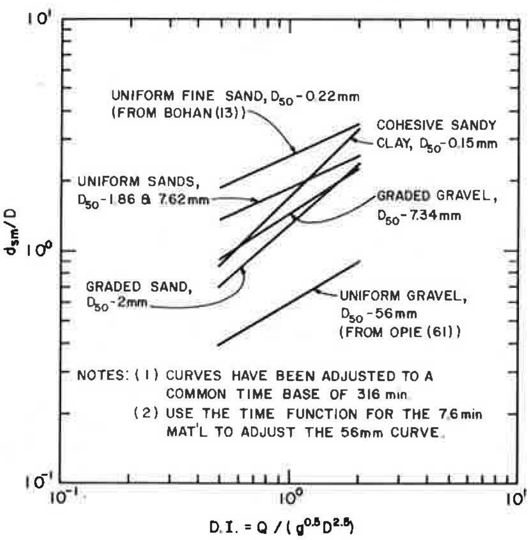
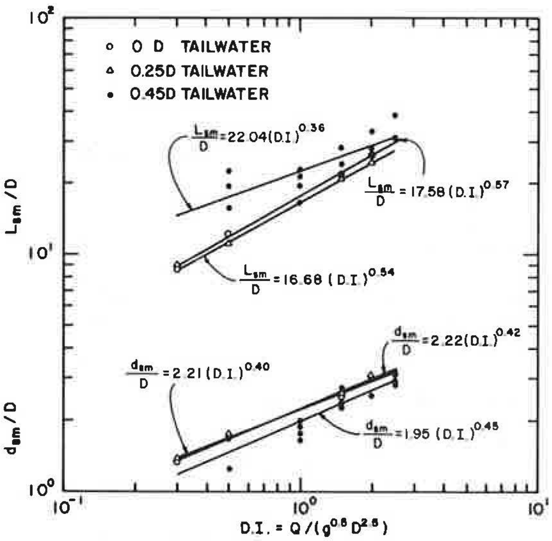


Figure 5. Tailwater comparison of scour-hole depth and length versus DI for uniform sand.



that the dimensions of scour are greater for circular-shaped culverts than for square-shaped culverts of similar characteristic lengths.

The Froude relationship was also selected for analysis where the Froude number (F) is defined as

$$F = \overline{V}/\sqrt{gL} \tag{8}$$

where g is the acceleration of gravity and L is the culvert characteristic length. The hydraulic radius (R_H) was selected as a common-denominator characteristic length and is defined as

$$R_H = A/WP \tag{9}$$

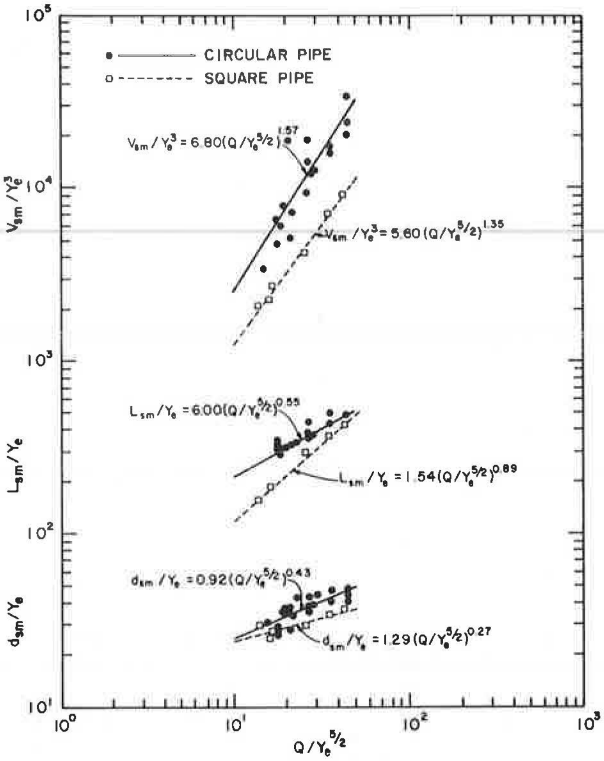
where A is the cross-sectional area of flow and WP is the wetted perimeter.

The Froude number was compared with the dimensionless scour-hole parameters of d_{sm}/R_H, L_{sm}/R_H, and $V_{sm}/R_H{}^3$. Figure 6 shows the dimensions of scour to be greater for circular-shaped culverts than for square-shaped culverts of similar characteristic lengths. Froude numbers varied from 2 to 6.5.

An analysis was performed to evaluate which parameter, equivalent depth, or Froude number more closely predicted scour-hole dimensions when culverts were flowing full. The resulting equations, using the equivalent-depth analysis, yield a conservative estimate of scour-hole dimensions at low DI (DI < 1.0) and tend to underestimate scour-hole dimensions at higher DI (DI $\geq$ 1.5).

It was observed that as the flow discharged from the square outlet the jet dispersed and impacted over a wider area than the more concentrated jet from the circular culvert. Therefore, it is anticipated that the estimated depth of scour will be less for rectangular culverts than that predicted for

Figure 6. Equivalent-depth comparison of circular- and square-shaped culverts for uniform sand.



circular- and square-shaped culverts at identical Froude numbers. In this case, the culvert height is used as the characteristic length.

These test results indicate that circular-shaped culverts yield more conservative scour-hole dimensions than square-shaped culverts of similar characteristic lengths. A Froude number analysis using the hydraulic radius and equivalent-depth analysis can be used to predict scour-hole dimensions adequately at all DIs.

Headwall Effects and Scour Profiles

Tests were performed by placing a headwall adjacent to the culvert outlet in the uniform sand material. The scour-hole dimensions were compared with tests performed under similar conditions without the headwall. The test results for conditions with and without headwalls are shown in Figure 7. The results indicate that little difference exists in the scour-hole dimensions for the two conditions.

Dimensionless profiles of the scour-hole centerline are shown in Figure 8 for conditions with and without headwalls. The profiles indicate that the scour-hole depth and length downstream of the culvert outlet are approximately the same with and without a headwall. The maximum depth of scour was observed at a point between approximately $0.3L_{sm}$ and $0.43L_{sm}$ downstream of the culvert outlet, where L_{sm} is the maximum length of scour. Erosion was observed directly under the culvert outlet to be approximately $0.4d_{sm}$, where d_{sm} is the maximum scour depth for the no-headwall condition. Furthermore, undermining of the culvert often extended $0.2L_{sm}$ into the embankment from the culvert outlet without the headwall.

Figure 8 shows that, if a headwall is installed at the culvert outlet, scour can extend downward adjacent to the headwall to a depth equal to the maximum depth of scour. Therefore, the headwall should extend below the maximum expected depth of scour to prevent the headwall from being undermined.

For the gravel and cohesive material, the dimensionless scour-hole profiles revealed that the maximum depth of scour occurs at a distance ranging from $0.30L_{sm}$ to $0.45L_{sm}$ downstream from the culvert outlet. Scour occurs directly under the culvert outlet to a depth of approximately $0.4d_{sm}$. The sidewall slope of the scour cavity is considerably steeper on the culvert side than it is on the opposite side where the water jet impacts.

Mound Geometry

One component of the local scour process near a culvert outlet is the formation of an aggraded mound downstream of the scour area. The mound dimensions of height (h_m), width (W_m), and length (L_m) were correlated with DI by normalizing mound dimensions in terms of the culvert diameter (D). Logarithmically plotting the mound dimensionless parameters versus the DI yields a series of linearized relationships. Fitting a power regression equation through each set of data yields a general equation of the following form:

$$\text{Maximum mound dimension}/D = (DI)^b \tag{10}$$

where the maximum mound dimension is height, width, or length and a and b are curve coefficients.

Figure 9 shows that after 316 min the height, width, and length of the mound can be correlated with the DI when the tailwater is set at 0.45D. The equation coefficients are as follows:

Mound Dimension	a	b
h_m	0.34	0.73
W_m	21.19	1.05
L_m	15.22	0.90

It is evident that the mound geometry can be described as a function of the culvert hydraulics for a uniformly graded sand material.

Relation Between Mound Geometry and Scour-Hole Dimensions

Relationships correlating the geometric characteristics of the scour hole with corresponding dimensions of the mound were derived. The general equation describing this relationship can be expressed in the following general form:

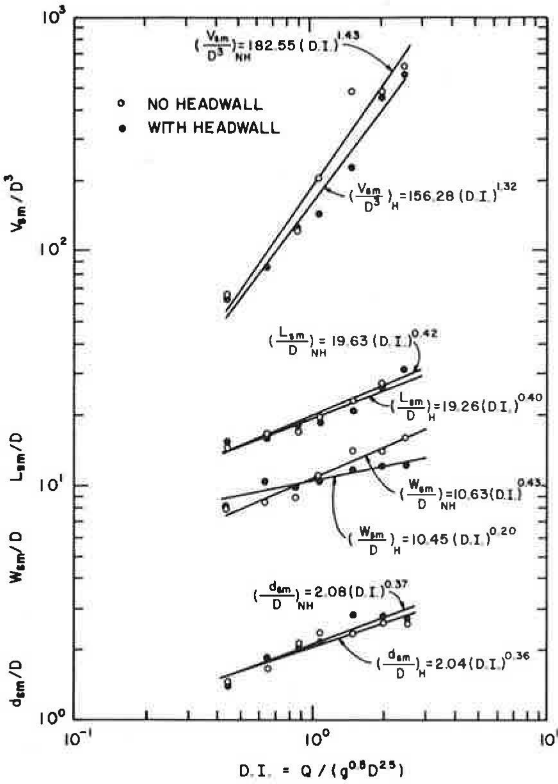
$$\text{Maximum mound dimension}/D = c\,(\text{maximum scour-hole dimension}/D)^d \tag{11}$$

where the maximum scour-hole dimensions are depth, length, and width; the maximum mound dimensions are height, width, and length; and c and d are curve coefficients. The resulting correlation coefficients are summarized below:

Mound Dimension	Scour-Hole Dimension	c	d
Height	Depth	0.10	1.52
Width	Width	0.55	1.47
Length	Length	1.83	0.78

There exists a direct proportionality between mound dimensions and scour-capacity dimensions at the equilibrium stage. For example, the mound height is generally 10 to 15 percent of the scour-hole depth when the culvert is flowing less than full. However, the mound height elevates to approximately 20 percent of the scour-hole depth when the

Figure 7. Comparison of scour-hole characteristics with and without headwall for uniform sand.



culvert flows full. These relationships permit the estimation of the extent of mound deposition due to localized scour in a sand material.

Scour Influence

The scour mechanism was found to affect areas downstream of the scour hole through the deposition of scoured materials. The length of scour influence (SI) is defined as the length of the scour hole (L_{sm}) plus the length of the mound deposition (L_{mm}). The width of SI is the width of the mound deposition. In order to quantify the SI tests were performed in uniform sand to document the extent of mound movement.

Figure 10 shows the relationship found between DI and the maximum length and maximum width of SI expressed in culvert diameters. An analysis indicates that the length of the mound is approximately the same as the maximum length of the scour hole for a culvert flowing full. The length of SI is twice the length of the scour hole measured downstream from the culvert outlet. It was observed that the width of SI ranges from two to three times the scour-hole width for DIs of 1.0 to 3.0. The SI influence relationships apply to culverts flowing full.

The total area affected by the local scour and deposition process, the SI area (A_{SI}), can be estimated as

$$A_{SI} = (L_{sm} + L_{mm}) \times W_{mm} \qquad (12)$$

where W_{mm} is the maximum mound width. When the length and width components in Equation 12 are replaced by the function presented in Figure 10, A_{SI} can be approximated as

$$A_{SI} = 672.4\, D^2\, (DI)^2 \qquad (13)$$

Therefore, A_{SI} can be depicted in a sand material as a function of the hydraulic characteristics.

CONCLUSIONS

Based on the results obtained through the extensive experimental program described in this paper, it was determined that scour-hole depth, width, length, and volume can be predicted for a noncohesive bed material. The rate of scour was also quantified and observed to reach approximately 80 percent of the maximum scour-hole dimensions after the initial 31 min of testing. The maximum scour depth was located at a point approximately 0.3 to 0.4 the maximum length of scour measured downstream of the culvert outlet.

Tests were performed in noncohesive bed material to measure the effectiveness of a headwall placed adjacent to the culvert outlet. The scour-hole dimensions were similar both with and without a headwall. The headwall protected the embankment from undermining as the scour hole extended as far as $0.2L_{sm}$ into the embankment for the no-headwall condition. However, the headwall should extend to a depth greater than the maximum predicted depth of scour to prevent undermining of the culvert and headwall.

The area affected by scour can be predicted for noncohesive bed materials. It was observed that the length of the affected area was approximately twice the length of the scour hole and that the width of the area was approximately two to three times the width of the scour hole for culverts flowing full.

Figure 8. Dimensionless scour-hole profile with and without headwall for uniform sand.

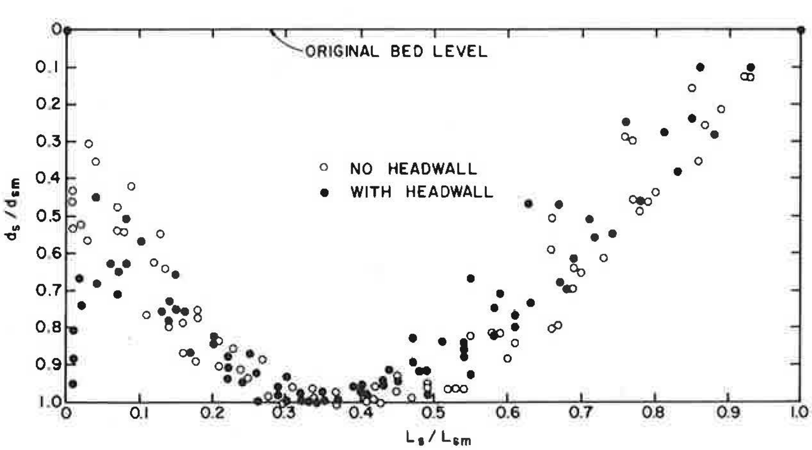


Figure 9. Mound length, width, and height versus DI after 316 min for uniform sand.

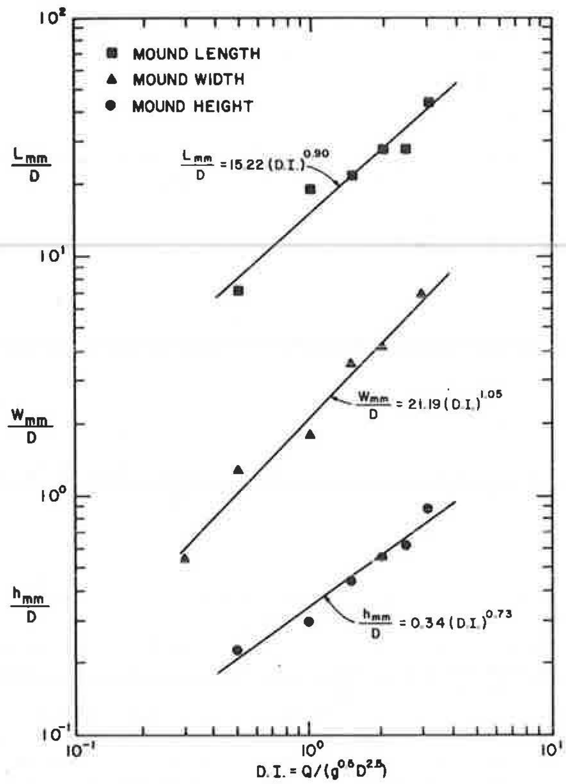
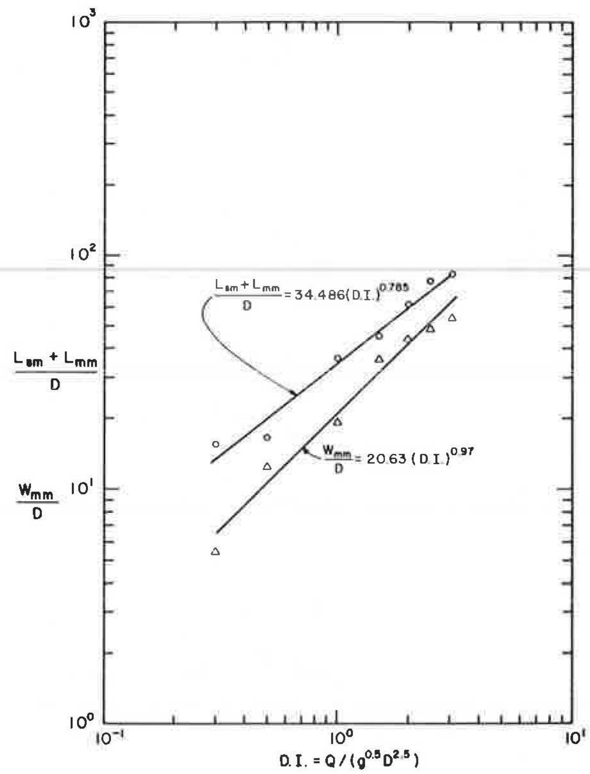


Figure 10. SI versus DI for uniform sand.



It was determined that the dimensions of scour could be predicted in an SC cohesive soil. Depth, width, length, and volume were correlated with soil saturated shear strength and plasticity index, fluid density and velocity, and time. It was observed that 70 percent of the maximum scour-hole dimensions was reached after the initial 31 min of testing. Furthermore, the maximum scour depth was located at a point approximately 0.35 the maximum length of scour measured downstream of the culvert outlet.

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REFERENCES

1. H. Rouse. Criteria for Similarity in the Transportation of Sediment. Proc., 1st Hydraulic Conference, Iowa State Univ., Iowa City, Bull. 20, March 1940, pp. 33-49.
2. E.M. Laursen. Observations on the Nature of Scour. Proc., 5th Hydraulic Conference, June 9-11, 1952.
3. R.A. Thomas. Scour in Gravel Beds. Colorado A&M College, Fort Collins, M.S. thesis, 1953.
4. D.E. Hallmark. Scour at the Base of a Free Overfall. Colorado A&M College, Fort Collins, M.S. thesis, 1955.
5. C. Mendoza. Headwall Influence on Scour at Culvert Outlets. Colorado State Univ., Fort Collins, M.S. thesis, 1980.
6. T.R. Opie. Scour at Culvert Outlets. Colorado State Univ., Fort Collins, M.S. thesis, 1967.
7. J.P. Bohan. Erosion and Riprap Requirements at Culvert and Storm-Drain Outlets. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Rept. H-70-2, 1970.
8. D. Doddiah. Comparison of Scour Caused by Hollow and Solid Jets of Water. Colorado A&M College, Fort Collins, Master's thesis, 1950.
9. M.A. Stevens. Scour in Riprap in Culvert Outlets. Colorado State Univ., Fort Collins, Ph.D. dissertation, 1969.
10. S.R. Abt and J.F. Ruff. Estimating Culvert Scour in Cohesive Material. Journal of the Hydraulics Division, ASCE, Vol. 108, No. HY1, Proc. Paper 16784, Jan. 1982, pp. 25-34.
11. B.P. Fletcher and J.L. Grace, Jr. Practical Guidance for Design of Lined Channel Expansions at Culvert Outlets. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Rept. HPR-1 (70-IH), Oct. 1974.

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Hydraulic Design of Stormwater Pumping Stations: The Effect of Storage

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In the design of stormwater pumping stations, skillful design can reduce both the initial cost and operating costs. The initial cost can be reduced by providing storage to reduce the peak pump rate. Savings can be achieved by reducing the size of the pump, the pump motor, piping, and valves, and substantial savings can accrue if the number of pumps is reduced. Most electrical utilities assess a fixed charge for the electrical capacity that the utility must maintain to provide service. Because horsepower is directly proportional to the pumping rate, any reduction in the pumping rate will be reflected in the fixed charge. By providing storage to reduce the peak pumping rate, operating costs can be considerably reduced. The effect of storage can be evaluated by using a mass curve routing procedure. This design procedure combines three independent components—the inflow hydrograph, the stage-storage relationship, and the stage-discharge relationship. The mass curve routing procedure identifies the amount of storage required to reduce the peak rate of flow to the fixed discharge rate. Design guidance is also provided for estimating the amount of storage required to accomplish a given reduction, and formulas are provided for calculating the volume of storage basins.

In most localities stormwater pumping stations only operate for a relatively short period of time during a year. This means that a substantial capital investment must sit idle for long periods of time. Therefore, the design and operation of stormwater pumping stations provide a most promising opportunity for cost reduction. Potential savings are even more promising in areas where storms are less frequent.

The merits of providing storage to reduce the peak pumping rates of pumping stations have long been recognized by engineers. To control the costs of stormwater projects, engineers are now examining potential savings much more closely. To achieve meaningful cost reductions, savings must be accomplished in both construction cost and maintenance and operations costs.

Initial costs can be reduced by providing storage to reduce the peak pumping rate; this will produce savings in the cost of the pump, the pump motor, and the instrumentation. Additional savings can be achieved by reducing the size of piping and valves. Substantial savings can accrue if the number of pumps is reduced. These savings will be offset by the cost of providing storage, but in many cases a net savings will occur if the storage can be provided at a low cost.

Maintenance and operating costs can be lowered by reducing the fixed electrical charge assessed by most electrical utilities. This charge is basically for the electrical capacity that the utility must maintain to service the pumping station and is usually proportional to the horsepower of the station. Because the horsepower is directly proportional to the pumping rate, any reduction in the pumping rate will be reflected in the fixed electrical charge.

This paper provides design guidance for sizing storage facilities used to reduce peak rates of flow at stormwater pumping stations.

DESIGN PROCEDURE

The merits of using storage to reduce peak flows are illustrated here by using a generalized case because the case of an actual pumping station may be complicated by the varying pumping rates and discontinuities as the pumps turn on and off. This is shown in Figure 1, where the shaded area between the curves represents the volume of stormwater that must be stored to reduce the peak flow rate. Storage exists in natural channels, storm drain systems, constructed basins or forebays, and storage boxes. Engineers must be able to identify and analyze the effect of storage on the discharge rates from the pumping station.

Designers must establish the interrelationship of three separate components:

1. The inflow hydrograph must be determined for the contributing watershed.
2. The volumetric storage capability of the storage facility must be identified.
3. The stage-discharge curve of the pumps must be determined.

Once these three components have been established, a mass curve routing procedure can be used to analyze the problem.

The mass curve routing procedure is based on the assumption that the storage facility acts as a reservoir. Figure 2 helps to illustrate two shortcomings of this assumption. First, the velocity in the pipe is not zero (this shortcoming should be minor because the velocities should be low). Second, the water depth of the reservoir decreases to zero as the water surface approaches the invert of the pipe. During peak flow conditions the pipe will be flowing full and the design conditions shown in Figure 2 will not exist.

The second shortcoming can be eliminated by providing an isolated depressed storage facility that is independent of the storm drain system; however, in most systems it is quite cost effective to use the storm drain system as the storage facility. If a boundary is drawn around the storage facility and

Figure 1. Generalized storage case.

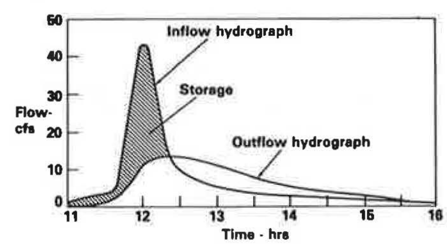
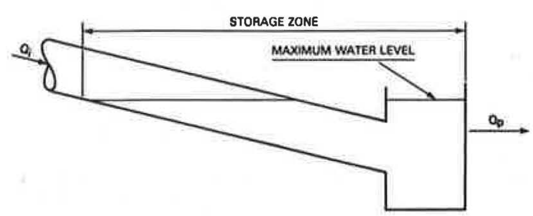


Figure 2. Reservoir routing.



the pumping station so that they act as a unit, the reservoir assumption is valid because the design conditions conform to the general storage equation:

$$(\bar{I} - \bar{O})\ \Delta t = \Delta S \tag{1}$$

where

$\bar{I}$ = average inflow rate,
$\bar{O}$ = average outflow rate,
Δt = time increment, and
ΔS = change in storage.

The inflow rate is defined by the design hydrograph, the outflow rate by the pumping discharge rates, and storage by the stage-storage curve. Because Equation 1 is valid inside the boundary, the reservoir assumption is an adequate representation of design conditions.

An example problem is used to illustrate the development of the routing procedure. The inflow hydrograph used for this example problem is shown in Figure 3.

ESTIMATING REQUIRED STORAGE AND PUMPING RATES

Because of the complex relationship among the variables of pumping rates, storage, and pump on-off settings, a trial-and-error approach is usually necessary for estimating the pumping rates and storage required for a balanced design. A wide range of combinations can be used to produce an adequate design. A desirable goal is to maximize storage capacity so as to minimize pumping capacity.

The relationship between storage and the pump discharge curve for one pump is shown in Figure 4. The area between the curves to the left of the pump-on point represents the storage available in the system below the pump-on elevation, and the remaining area above the pump discharge curve represents the storage volume required above the pump-on elevation.

Some approximation is necessary to produce the first trial design. In one approach, shown in Figure 5, the peak pumping rate is assigned and a horizontal line representing the peak rate is drawn across the top of the hydrograph. The shaded area above the peak pumping rate represents the volume of storage required above the last pump-on elevation. This area is then measured to size the storage facility.

The number of pumps and their respective pumping rates are selected along with the pump on-off settings, and the trial dimensions of the storage basin are assigned to produce the required volume of storage, represented by the shaded area in Figure 5, above the last pump-on elevation.

For the example problem, a peak pumping rate of 14 ft^3/sec was assigned; this can be achieved by using two 7-ft^3/sec pumps. The pumping rate is plotted as a horizontal line and the shaded area is measured to determine the required volume (4,500 ft^3) above the last pump-on elevation.

STAGE-STORAGE RELATIONSHIP

Engineers have a wide range of tools available to them for providing the necessary storage at a pumping station. Natural or constructed earth basins are the most cost effective; however, at most highway pumping stations the stormwater must be stored underground. This can be done by oversizing the storm drain or providing a concrete storage box.

Routing procedures require the development of a stage-storage relationship. This is done by calculating the available volume of water for storage at uniform vertical intervals. Usually, the stage-storage curve is developed by using 0.25- to 0.50-ft intervals.

Two important points should be remembered:

1. The stage-storage curve must be developed for the entire range of elevations encountered.
2. To be cost effective, all of the pipes should be filled at peak conditions.

Natural Earth Basins

Storage provided by irregular natural terrain is calculated by determining the area of horizontal planes associated with the vertical intervals. The areas of adjacent planes are averaged and multiplied by the vertical increment to determine an incremental volume. Starting at the bottom of the basin, the volumes are summed to obtain the stage-storage curve.

Trapezoidal Basins

The stage-storage curve can be developed by using the prismoidal formula; however, because most engineers will want to develop computer programs for determining the volumes, the following equation is more useful:

$$V = LWD + (L + W)Z\,D^2 + (4/3)Z^2\,D^3 \tag{2}$$

where

V = volume of the basin (ft^3),
L = length of the basin at the base (ft),

Figure 3. Design inflow hydrograph.

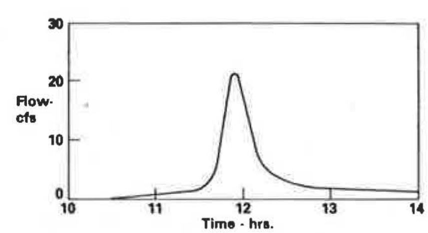


Figure 4. Relation between storage and pump-on.

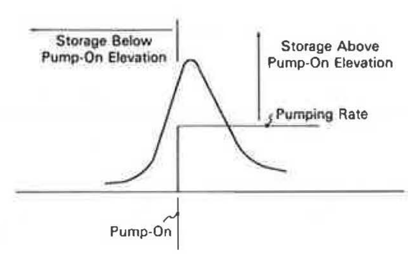
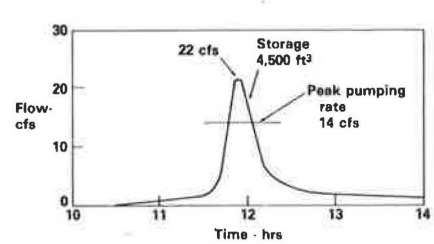


Figure 5. Estimating required storage.



W = width of the basin at the base (ft),
D = depth of the basin (ft), and
Z = side-slope factor, ratio of horizontal to vertical.

The equation, developed in Figure 6, is based on the premise that all of the sloping surfaces have the same slope. If different side slopes are required, the equation can be adjusted. A special case occurs when the basin is square (pyramid) and the equation for calculating the volume can be simplified:

$$V = L^2 D + 2LZD^2 + (4/3)Z^2 D^3 \qquad (3)$$

Storm Drains

Whenever the pump start elevation is above the invert of the storm drain, the storm drain will perform more as a storage basin than as a means of conveyance. By oversizing the storm drain, a true storage basin can be created to provide a meaningful reduction in pumping rates. One length of pipe could be designed to act as a storage basin, or the storage zone could be extended into several lengths of the storm drainage system.

The volumes for establishing the stage-storage curve can be calculated by using the prismoidal formula:

$$V = (L/6)(A_1 + A_2 + 4M) \qquad (4)$$

where

V = volume of water in pipe (ft^3),
L = wetted length of pipe (ft),
A_1 = wetted cross-sectional area of lower end of pipe (ft^2),
A_2 = wetted cross-sectional area of upper end of pipe (ft^2), and
M = wetted cross-sectional area of midsection of pipe (ft^2).

Relative area-depth curves or tables for the particular storm drain shape must be consulted to determine the cross-sectional areas. An FHWA report (1) provides relative area-depth tables for various cross-sectional shapes. Because the invert of the pipe should be sloped to aid in sediment removal, calculating the cross-sectional areas becomes more difficult. If the pipe is circular, a special case exists and the volume can be determined by the ungula of a cone formula (see Figure 7):

$$V = H\,[(2/3\,a^3 \pm cB)/(r \pm c)] \qquad (5)$$

where

H = wetted length of the pipe (ft),
a = one-half of the water surface width (ft),
c = distance from the center of the pipe to the water surface (ft),
B = wetted cross-sectional area of the base (ft^2), and
r = radius of pipe (ft).

When the water surface is above the center of the pipe, + is used; when the water surface is below the center of the pipe, - is used. Relative depth-area values for a circular section are given in Table 1.

Figure 6. Volume of trapezoidal basin.

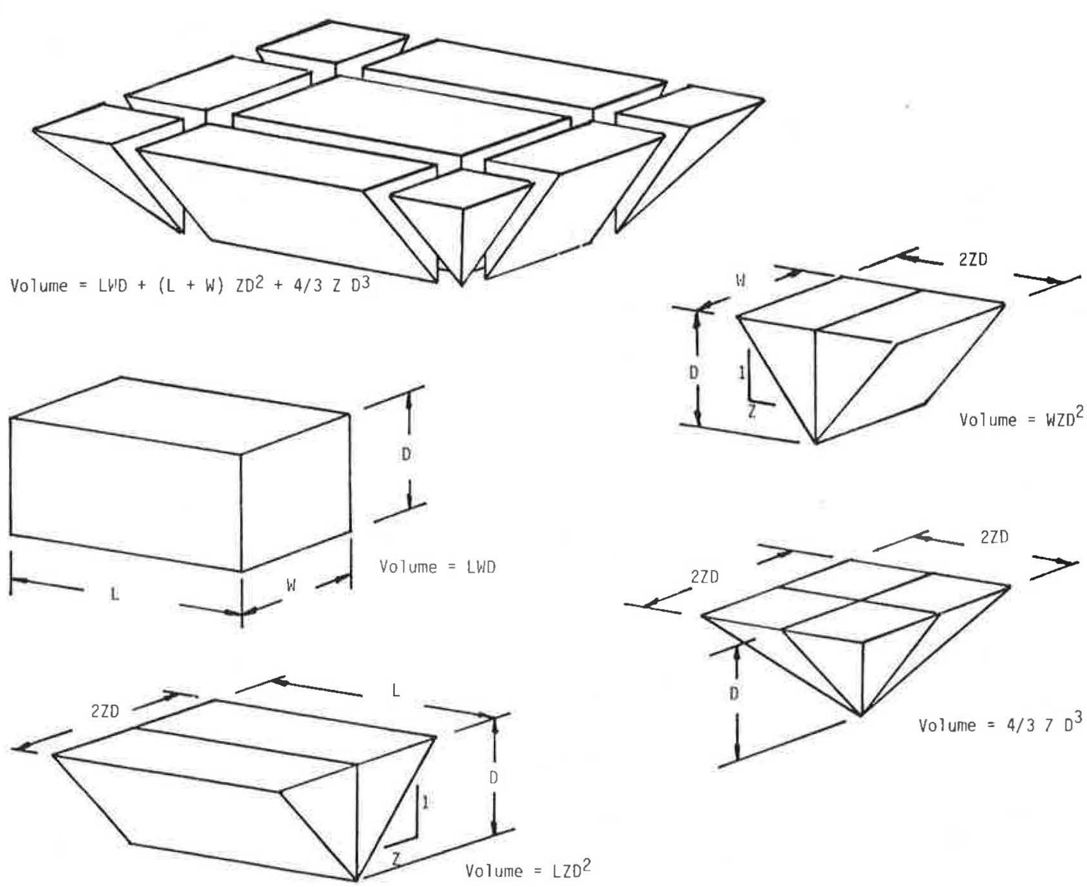
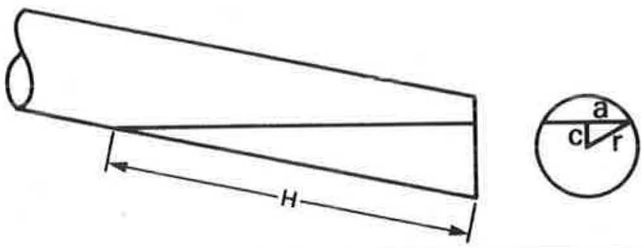


Figure 7. Ungula of a cone.



Storage Boxes

Underground storage boxes would most likely be rectangular reinforced concrete boxes. The volumes at the various stages can be calculated by using a combination of formulas for regular prisms and triangular wedges. Procedures for determining the volume of storage boxes are provided in Figure 8.

EXAMPLE PROBLEM

In the example problem, a 520-ft-long, 48-in.-diameter circular concrete pipe with a 0.40 percent slope is provided as a storage pipe, as shown in Figure 9; a 21- ft-diameter wet well is also provided. The storage volumes for the respective elevations are given in Table 2, and the stage-storage curve is plotted in Figure 10.

Stage-Discharge Relationship

Mass curve routing procedures require that a stage-discharge relationship be established. For the example problem presented here, the following stage-discharge relationship was developed:

	Discharge Rate	Elevation (ft)	
Pump	(ft^3/sec)	Pump Start	Pump Stop
1	7	2.0	0.0
2	7	3.0	1.0

This stage-discharge relationship is based on three design assumptions: (a) pump 1 stops at 0.0-ft elevation, (b) the pumping range = 2 ft, and (c) there is a 1-ft difference in elevation between pump starts.

Figure 11 shows the pumping arrangement. Because pumping station design is basically a trial-and-error approach, this pumping arrangement should be considered a first attempt.

INFLOW MASS CURVE

To obtain an inflow mass curve, the inflow rates at the limits of a time increment are averaged and multiplied by the time increment to obtain an incremental volume. These incremental volumes are then totaled to obtain a cumulative inflow and plotted versus time to create an inflow mass curve. The inflow hydrograph for the example problem (Figure 3) is totaled in Table 3 and plotted in Figure 12 as the inflow mass curve.

MASS CURVE ROUTING

After the three components, the inflow hydrograph, the stage-storage relationship, and the stage-discharge relationships, have been determined, a graphical mass curve routing procedure can be used. In actual design practice, the inflow hydrograph, which is developed by an acceptable hydrologic method, is a fixed design component; however, the storage and pumping discharge rates are variable. The designer may wish to assign a pumping discharge rate based on environmental or downstream capacity considerations. The required storage is then determined by various trials of the routing procedure.

As the stormwater flows into the storage basin, it will accumulate until the first pump-start elevation is reached. The first pump is activated and, if the inflow rate is greater than the pump rate, the stormwater will continue to accumulate until the elevation of the second pump start is reached. As the inflow rate decreases the pumps will shut off at their respective pump-stop elevations.

These conditions are modeled in the mass curve diagram by establishing the point at which the cumulative flow curve has reached the storage volume associated with the first pump-start elevation. This storage volume, 2,025 ft^3 (Figure 10), is represented by the vertical distance between the cumulative flow curve and the base line as shown in Figure 13. A vertical storage line is drawn at this point because it establishes the time at which the first pump starts.

The pump discharge line is drawn from the intersection of the vertical storage line and the base line upward toward the right; the slope of this line is equal to the discharge rate of the pump. The pump discharge curve represents the cumulative discharge from the storage basin, and the vertical distance between the inflow mass curve and the pump discharge curve represents the amount of stormwater stored in the basin.

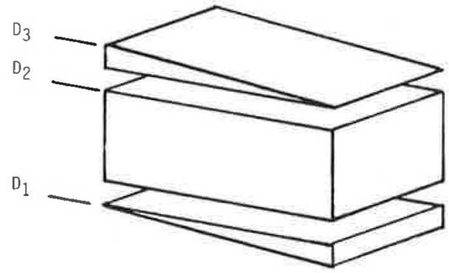
If the rate of inflow is greater than the pump capacity, the inflow mass curve and the pump dis-

Table 1. Relative depth-area values for circular pipes.

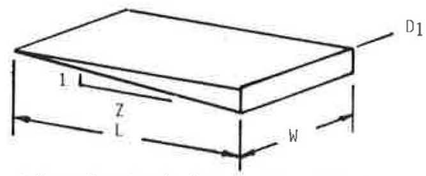
	C									
d/D	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0		0.0013	0.0037	0.0069	0.0105	0.0147	0.0192	0.0242	0.0294	0.0350
0.1	0.0409	0.0470	0.0534	0.0600	0.0668	0.0739	0.0811	0.0885	0.0961	0.1039
0.2	0.1118	0.1199	0.1281	0.1365	0.1449	0.1535	0.1623	0.1711	0.1800	0.1890
0.3	0.1982	0.2074	0.2167	0.2260	0.2355	0.2450	0.2546	0.2642	0.2739	0.2836
0.4	0.2934	0.3032	0.3130	0.3229	0.3328	0.3428	0.3527	0.3627	0.3727	0.3827
0.5	0.393	0.403	0.413	0.423	0.433	0.443	0.453	0.462	0.472	0.482
0.6	0.492	0.502	0.512	0.521	0.531	0.540	0.550	0.559	0.569	0.578
0.7	0.587	0.596	0.605	0.614	0.623	0.632	0.640	0.649	0.657	0.666
0.8	0.674	0.681	0.689	0.697	0.704	0.712	0.719	0.725	0.732	0.738
0.9	0.745	0.750	0.756	0.761	0.766	0.771	0.775	0.779	0.782	0.784
1.0	0.785									

Note: $B = C D^2$, where B = flow area, C = coefficient, D = diameter of the pipe, and d = depth of flow.

Figure 8. Volume of storage box.

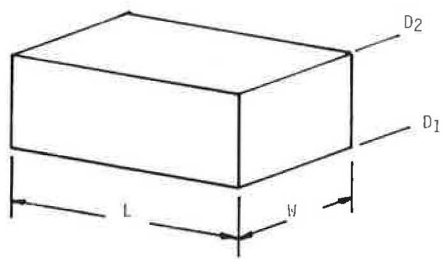


1. Volume for depths less than D_1



$$V_1 = 1/2\ WZ\ D^2$$

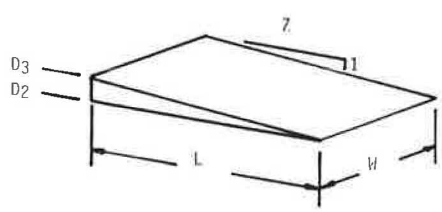
2. Volume for depths between D_1 and D_2.



$$V_2 = LW\ (D - D_1)$$

$$\text{Total } V = V_1 + V_2$$

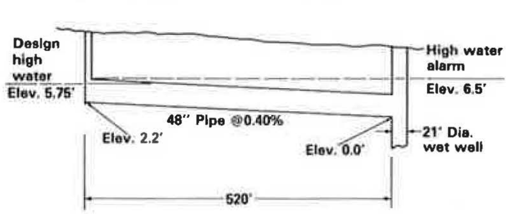
3. Volume for depths between D_2 and D_3.



$$V_3 = 1/2\ WZ\ (D - D_2)^2 + W\ (L - Z\ (D - D_2))\ (D - D_2)$$

$$\text{Total } V = V_1 + V_2 + V_3$$

Figure 9. Storage pipe used in example problem.



charge curve will continue to diverge until the volume of water in storage is equal to the storage associated with the second pump-start elevation (4,226 ft^3). At this point the second pump starts, and the slope of the pump discharge line is increased to equal the combined pumping rate.

The procedure continues until peak storage conditions are reached. At some point on the inflow mass curve, the inflow rate will decrease and the slope of the inflow mass curve will flatten. To determine the maximum amount of storage required, a line is

Table 2. Stage-storage volumes for 48-in.-diameter pipe at 0.40 percent slope with 21-ft diameter wet well.

Elevation (ft)	Volume (ft^3) Pipe	Wet Well	Total
0.0	0	0	0
0.5	45	173	218
1.0	251	346	597
1.5	672	519	1,191
2.0	1,333	692	2,025
2.5	2,213	866	3,079
3.0	3,187	1,039	4,226
3.5	4,168	1,212	5,380
4.0	5,072	1,385	6,457
4.5	5,773	1,559	7,332
5.0	6,230	1,732	7,962
5.5	6,468	1,905	8,373
6.0	6,534	2,078	8,612
6.5	6,534	2,251	8,785
7.0	6,534	2,424	8,958

Figure 10. Stage-storage curve.

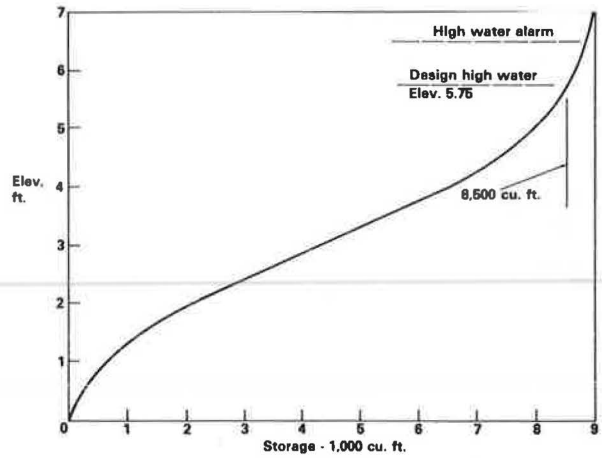
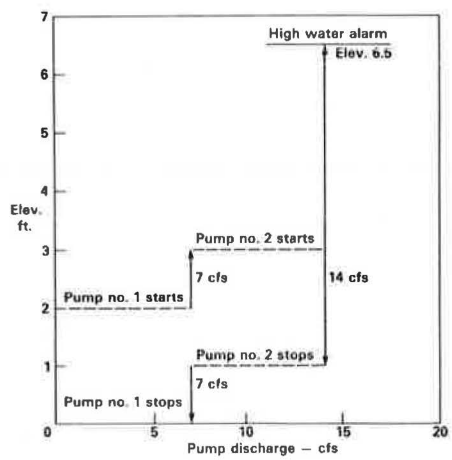


Figure 11. Stage-discharge curve.



drawn parallel to the pump discharge curve tangent to the inflow mass curve as shown in Figure 13. The vertical distance between the lines represents the maximum amount of storage required.

The routing procedure continues until the pump discharge curve intersects the inflow mass curve. At this point the storage basin has been completely emptied and a pumping cycle completed. As the storm recedes, the pumps will cycle to discharge the remaining runoff.

In developing the pump discharge curve, the engineer should remember that the pump performance curve is quite sensitive to changes in head and that the static head will fluctuate as the water level in the storage basin fluctuates. The designer should also recognize that the pump discharge rate represents an average pumping rate.

The storage volumes for the two 7-ft^3/sec pumps described earlier in the example problem are as follows:

	Pump Start		Pump Stop	
Pump	Elevation (ft)	Storage Volume (ft^3)	Elevation (ft)	Storage Volume (ft^3)
1	2	2,025	0	0
2	3	4,226	1	597

Table 3. Development of inflow mass curve.

Time	Inflow (ft^3/sec)	Average Inflow (ft^3/sec)	Incremental Flow (ft^3)	Cumulative Flow (ft^3)
10:30	0			0
		0.05	15	
10:35	0.1			15
		0.15	45	
10:40	0.2			60
		0.25	75	
10:45	0.3			135
		0.35	105	
10:50	0.4			240
		0.45	135	
10:55	0.5			375
		0.55	165	
11:00	0.6			540
		0.65	195	
11:05	0.7			735
		0.75	225	
11:10	0.8			960
		0.85	255	
11:15	0.9			1,215
		0.95	285	
11:20	1.0			1,500
		1.05	315	
11:25	1.1			1,815
		1.15	345	
11:30	1.2			2,160
		1.85	550	
11:35	2.5			2,710
		3.5	1,050	
11:40	4.5			3,760
		8.0	2,400	
11:45	11.5			6,160
		15.2	4,560	
11:50	19.0			10,720
		20.2	6,060	
11:55	21.5			16,780
		19.2	5,760	
12:00	17.0			22,540
		14.5	4,350	
12:05	12.0			26,890
		9.2	2,760	
12:10	6.5			29,650
		5.8	1,740	
12:15	5.0			31,390
		4.5	1,350	
12:20	4.0			32,740
		3.8	1,140	
12:25	3.5			33,880
		3.4	1,020	
12:30	3.3			34,900
		3.0	900	
12:35	2.7			35,800
		2.6	780	
12:40	2.5			36,580
		2.4	720	
12:45	2.3			37,300
		2.2	660	
12:50	2.1			37,960
		2.05	620	
12:55	2.0			38,580
		1.95	580	
13:00	1.9			39,160

Note: Time increment = 300 sec.

Figure 12. Inflow mass curve.

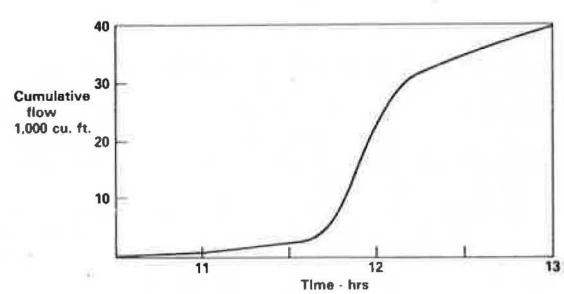
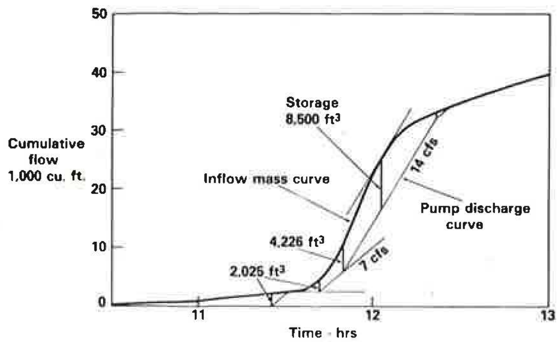


Figure 13. Mass curve routing.



As shown in Figure 13, pump 1 is activated when the cumulative flow fills the storage basin to an elevation of 2.0 ft (2,025 ft^3). The pump discharge curve is drawn from the base line at a rate of 7 ft^3/sec. Because the rate of discharge is greater than the inflow rate, the basin will quickly empty and pump 1 will shut off. The pump discharge curve will be horizontal because there is no pumped discharge until the inflow builds up to the pump 1 start elevation again.

Pump 1 comes on again as the inflow builds up. Because the inflow rate is greater than the discharge rate, the curves will diverge until the available storage (4,226 ft^3) is reached at the pump 2 start elevation. The combined discharge rate is plotted, and a line is drawn parallel to the discharge curve through the point of tangency of the inflow mass curve to determine the maximum amount of storage required, as shown in Figure 13. The vertical distance between the lines represents the maximum amount of storage required (8,500 ft^3).

The peak storage conditions have now been reached and storage decreases. The routing continues until the two curves intersect, at which time the basin will have emptied. Pump 2 will shut off when the storage volume is equal to the volume (597 ft^3) associated with the pump 2 stop elevation (1.0 ft); pump 1 will shut off when the storage pipe has been emptied at the pump 1 stop elevation (0.0 ft). Subsequent inflows will cause the pumps to cycle as the storm flow recedes; for purposes of simplicity this additional cycling is not shown.

The design is adequate because the available storage at the high-water alarm is 8,785 ft^3. High-water design conditions are plotted on the stage-storage curve (Figure 10) for reference.

In the final design, the mass curve routing procedure can be fine-tuned after the pumps have been selected. For example, if two equal pumps are selected, the pumping rate when only one pump is pumping most likely would be greater than half the combined rate. Another refinement can be made for the condition that prevails when all of the pumps have come on line and peak pumping conditions have been reached. The pump discharge curve can be adjusted to reflect changes in the pumping caused by changes in the static head. However, it should be noted that these refinements do not act on the side of safety.

DISCUSSION OF RESULTS

Pumping Design

The designer now has a complete design that allows the problem to be studied in depth. The peak rate of runoff has been reduced from 22 ft^3/sec, the inflow hydrograph peak has been reduced to 14 ft^3/sec, and the maximum pump discharge rate has been reduced by 46.5 percent by providing for 8,500 ft^3 of storage. This is only one possible design option. The designer may wish to reduce the pumping rate further by providing more storage, and additional combinations of pump discharge and storage can be considered.

It is important that the designer visualize what is happening during the peak design period. To aid in this process, the pump discharge curve developed in Figure 13 can be superimposed on the design inflow hydrograph of Figure 3, as shown in Figure 14, to obtain another picture of the routing process. The shaded area between the curves represents stormwater that is going into storage. Again, pump cycling at the end of the storm has been omitted to simplify the illustration.

To complete the design the designer should investigate more frequent storms (storms with a 2- to 10-year recurrence interval) and evaluate pumping cycles during these storms. Less frequent storms (a 100-year recurrence interval) should also be investigated to determine the amount of flooding that will occur.

Stormwater pumping station design is one of the most promising areas in which cost reductions can be achieved by good engineering. Engineers will want to analyze a wide range of storage and pumping rate combinations to obtain the most economical design. To assist in design calculations, a programmable calculator program (2) has been prepared for the routing procedure.

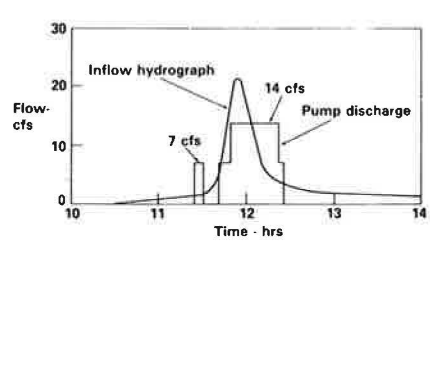
Sediment Problems

The handling of sediment remains a difficult problem in pumping station design. Mechanical engineers prefer that as much sediment as possible be deposited in the storage boxes and the wet well to minimize wear on the pumps, whereas maintenance engineers prefer that the sediment be passed through the system so that the station is as maintenance-free as possible. Although both of these goals may have merit, they are at cross-purposes, and some trade-off must be made.

Because the velocity in the storage pipe is quite low (1 to 2 ft/sec), sediment will tend to settle out in the storage pipe. Some engineers recommend a relatively steep slope of 1 to 2 percent to pass the sediment into the wet well. As a general statement, the steeper the grade the better is the sediment removal; however, the steeper grade may cause the station wet well to be driven deeper into the ground and thus increase its cost. A steep grade may also limit the length of pipe that would otherwise be available for storage.

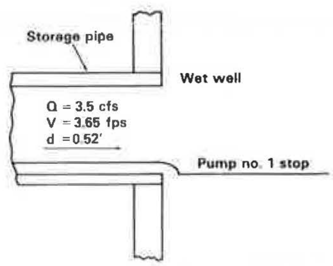
It is difficult to analyze flow and sediment conditions in the storage pipe; one approach would be

Figure 14. Pump discharge.



to investigate the flushing case. Design publications recommend a minimum velocity of 3 ft/sec when the pipes are flowing full; however, in the pumping station case the pumping rates determine the pipe velocity. For the flushing case, it is assumed that all main pumps have stopped and that the inflow rate is half the smallest pumping rate to ensure pump cycling. The slope of the storage pipe is then selected to provide a velocity of at least 3 ft/sec. The flushing case for the example problem is shown in Figure 15.

Figure 15. Flushing conditions for storage pipe.



The selection of the size and slope of the storage pipe is an important element in the trial-and-error design procedure. Table 4 gives design slopes for concrete pipes of various sizes that will produce a velocity of 3 ft/sec when flow is at a depth of 1/8 of the pipe diameter. Although these criteria are quite rigorous, the resulting slopes for the larger pipes are still quite flat. If the storage pipe is depressed or isolated from the upstream storm drainage, a minimum pipe grade of 0.35 percent is suggested to prevent low spots in the pipe.

Table 4. Trial slopes for flushing concrete pipe.

Pipe Size (in.)	Pipe Slope (ft/ft)	Pipe Size (in.)	Pip Slope (ft/ft)
24	0.0083	54	0.0028
27	0.0071	60	0.0024
30	0.0062	66	0.0021
33	0.0054	72	0.0019
36	0.0048	78	0.00172
42	0.0039	84	0.00156
48	0.0033	90	0.00142
		96	0.00130

Note: Velocity of 3 ft/sec, 1/8 full flow, n = 0.013.

In summary, the storage pipe and wet well should be designed to handle sediment; however, the pumping system should be designed to carry sediment-laden stormwater in case sediment removal does not occur in the wet well.

REFERENCES

1. P.N. Zelensky. Computation of Uniform and Non-uniform Flow in Prismatic Conduits. Office of Research and Development, FHWA, 1972.
2. R.H. Baumgardner, S.B. Chase, and P.D. Wlaschin. Hydraulic Design of Stormwater Pumping Stations Using Programmable Calculators. Hydraulics Branch, FHWA, CDS 5, 1982.

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