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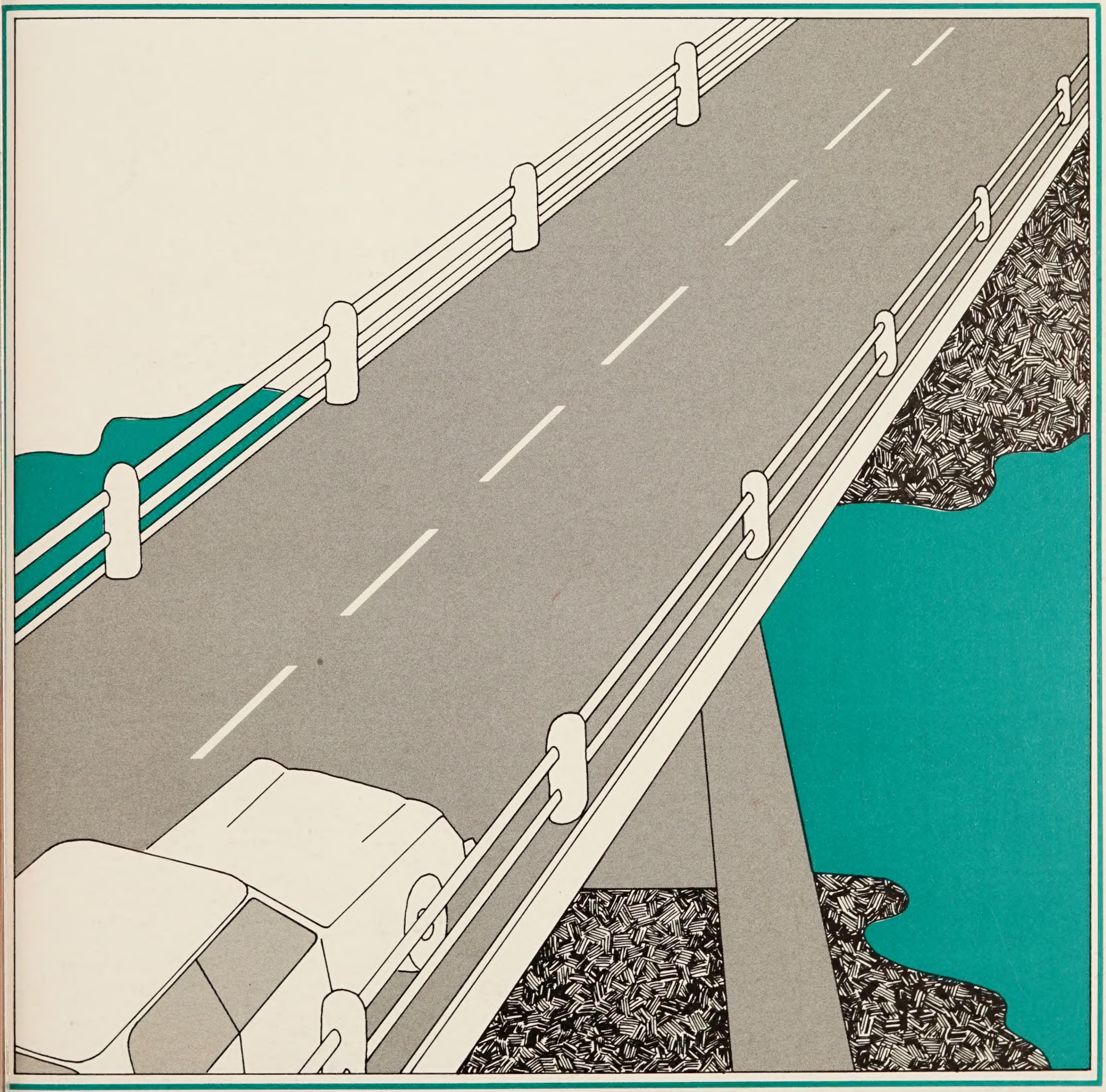

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COVER:
Stream channel aggradation and degradation can impact highway crossings.

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Stream Channel Degradation and Aggradation[1]

by
Stephen A. Gilje

Streambed elevation is not static. However, if the streambed elevation is excessively lowered for long distances, the stream is said to have degraded. When a stream fills with sediment, it has aggraded. The hydraulic consequences of these events are profound. Aggradation and degradation have increased flood risk at thousands of stream crossings nationwide.

This article focuses on the magnitude and causes of aggradation and degradation and the regional extent of the phenomenon. Specific highway hydraulic problems are identified in the article. Methods for recognizing these problems are provided, as are measures for controlling them.

Introduction

Degradation and aggradation are caused by major changes in the river environment that increase or decrease a stream's ability to transport material, or provide significant increases or decreases in material available for transport.

Degradation is not local or general scour. Local scour is caused by flow obstruction. The obstruction causes flow acceleration to satisfy flow continuity. An increase in local flow velocity increases shear stress and the ability to remove material, resulting in a scour hole. During floods, stream flow depths and velocities increase as does local scour.

[1] This article draws from information collected under contract with the Federal Highway Administration and reported in "Stream Channel Degradation and Aggradation: Causes and Consequences to Highways," Report No. FHWA/RD–80/038, *Federal Highway Administration,* Washington, D.C., June 1980.

General scour is also flow phenomenon directly dependent on flow depth, velocity, and stream geometry. General scour is caused by a flow constriction. Where a stream reach has a controlled width less than that upstream, flow velocity is accelerated to pass the discharge through the constricted area. With constriction, flow acceleration is spread over a greater streambed area, hence general scour.

Degradation results when the ability of an alluvial stream to transport material exceeds the amount of material normally supplied for transport. When this condition exists in a stream for an extended time, the deficit of transportable material causes accelerated erosion of the bed and banks. Because the streambed offers a larger contact surface against flow, erosion concentrates there resulting in progressive downcutting.

Degradation is not a local flow phenomenon. It depends on sediment/transport relationships not local hydraulics. Degradation will occur if normal stream sediment discharge is decreased and bed erosion is not prevented by armoring or outcrops of resistant materials exposed in the channel. Degradation may be progressive or sporadic. Although severe degradation is often related to floods, stream degradation is most directly related to sediment availability and transport within the river system and not flow characteristics. A stream undergoing progressive degradation during normal flows could experience filling during a flood if the flood increased overland erosion and out-of-channel sediment availability.

Aggradation is not local filling in the same sense that degradation is not scour. Aggradation occurs when more sediment is supplied to a stream than the stream flow is capable of transporting. This results either from increasing normal sediment supply or decreasing flow velocity below what is needed to carry available bedload. Aggradation is common in the ponded areas upstream of a dam and where sediment input has been greatly increased, such as at construction sites. It does not, however, pose an engineering problem until it is excessive. In extreme situations stream channels have been choked with sediment, and continuing flow and sediment emerge into the floodplain seeking a new channel.

The extent of the stream gradation problem in the United States is demonstrated by a 1978 Federal Highway Administration (FHWA) study on countermeasures for hydraulic problems at bridges. (*1*)[2] Data from 224 sites experiencing various hydraulic problems were assembled and carefully analyzed. Thirty-nine of these sites (17.4 percent) experienced streambed aggradation or degradation.

Stream stability can be evaluated clearly only by considering the whole river system. This is especially true for gradation changes because they are caused by a complex interaction of watershed characteristics.

It is important to distinguish between the effects of local variances in bed elevation, the results of scour and fill, and the effects of general shifts in the entire stream regime—a gradation change. The effectiveness of countermeasures for hydraulic hazards directly depends on accurate early evaluation of the hydraulic forces responsible.

Countermeasures effective for crossing protection against scour and fill have proven ineffective against degradation and aggradation. Unfortunately, observing a streambed depth change at one location frequently will not permit easy characterization of the hydraulic situation. A local change can appear the same as a gradation change that may affect an entire stream system. In the worst case, a general gradation change combines with local effects (degradation and scour) at a crossing to produce a bed elevation change greater than that produced by either hydraulic situation acting alone. The relative effects of scour and fill or gradation change must be determined to properly evaluate and subsequently select mitigative measures.

Aggradation and Degradation Case Studies

This article is drawn from information collected under contract by the FHWA Office of Research. FHWA regions were requested to submit candidate sites where degradation or aggradation was suspected. The usefulness and validity of the FHWA stream degradation study depended a great deal on detailed evaluation of specific aggradation/degradation experiences. Over 200 sites were provided. In addition to these sites, another 75 were provided from the literature. From these 275 sites, 110 were selected by the contract

[2] Italic numbers in parentheses identify references on page 52.

manager together with the contractor for further documentation. These sites were selected to:

- Aid in determining causes of gradation changes.
- Evaluate kinds of highway problems.
- Establish the significance of human activity on river gradation.
- Provide a regional description of degrading and aggrading streams.
- Evaluate and establish guidelines for recognition of gradation problems.
- Evaluate mitigative measures.

Regionalization of Gradation Problems

When planning the study, it was thought that gradation problems would occur in regional clusters, each grouped in areas of similar cause. That intuition was based on an understanding of physiography and a belief that many gradation changes were fostered by natural situations. Because human interference with natural processes proved to be the prevalent cause of degradation and aggradation, an understanding of watershed activities proved more important than regional considerations. Nevertheless, certain areas of the country did contribute an above average share of case histories.

Figure 1 illustrates the location of the case histories. Also identified on the figure are concentrations of sediment in major U.S. streams. With few exceptions the sites with gradation problems lie in areas of the United States with high sediment yields. The majority of these areas are along or west of the Mississippi River. The western and central portion of the country from the western borders of Arizona and Montana to the Mississippi River seems particularly prone to gradation problems. A fair number of sites are located in northwestern California.

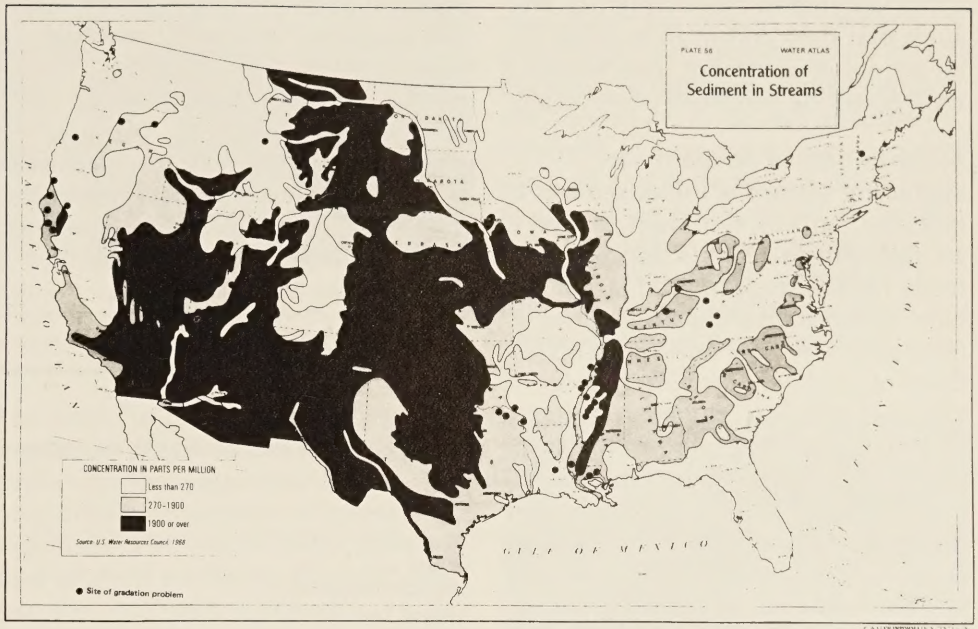

Figure 1.—Location of case histories and concentrations of sediment in streams.

The correlation between sites with gradation problems and streams with high sediment loads is sensible. High sediment loads imply easily erodible soil and streams with sand beds. Such streams are more likely to change course or shape than streams flowing in bedrock channels or channels with very large-sized sediment. Streams carrying a high percentage of bedload also are prone to shoaling and bed erosion whenever normal sediment recharge or flow discharge varies. Figure 1 shows that few case histories are located in the Appalachian Mountains or in the mountainous regions of the West.

Highway Problems Related to Gradation Changes

The highway crossing situation most often associated with aggradation is reduction of flow area resulting in possible flow over the roadway or bridge deck during floods. The bridge can be swept away because of an increase in horizontal force and turning moment. Aggradation at bridge crossings also results in high maintenance costs. When aggradation in the vicinity of a highway crossing is excessive, it becomes necessary to alleviate the problem temporarily by excavation.

If the watershed characteristics are such that they continue to provide an excess of sediment, maintenance efforts will be required periodically.

Another hazard posed by an aggrading stream is increased bank erosion. Surplus sediment carried by an aggrading stream is deposited more rapidly in point and lateral bars. Continued deposition increases sinuosity and redirects flow into actively eroding banks. Channel avulsion, the most dramatic and disruptive aggradation hazard, occurs when the stream channel is completely filled with sediment and the flow seeks an entirely different course. This commonly occurs on alluvial fans.

Highway crossing problems caused by degradation are exposures of bridge footings, piling, and abutments. Degradation also causes undermining of revetment and other countermeasures, results in instability of channel banks, and promotes debris problems. (*2*) Degradation alters crossing conditions, thus increasing hydraulic hazards which under original conditions pose no problem. For example, local scour may be considered adequately under original design conditions but could cause bridge failure when degradation has subsequently lowered the general stream channel. In such a situation it is common to blame the failure on local scour; however, general stream degradation is the true culprit.

There are about three serious highway crossing degradation problems for every serious highway crossing aggradation problem. Degradation problems are more readily apparent and more severe because usually there is much less tolerance for removal of material from beneath a structure's foundation than there is for a lessening of freeboard. Also, highway crossings usually constrict a stream. Such constriction would be the last place affected by aggradation but the first place affected by degradation.

Causes of Gradation Changes

Gradation changes are caused either naturally or by human activities. Human activities are related to 80 percent of the gradation changes analyzed in the FHWA study. In fact, it was difficult to isolate incidences where gradation changes were caused by natural factors alone and

Table 1.—Significance of natural factors on gradation changes

Causes	Number of cases
Debris	5
Meander migration	5
Alluvial fans	4
Braiding	2
Recurrent flooding	1
Armoring	1
Delta growth	1
Highly erodible bed or banks	1

Table 2.—Significance of human activities on gradation changes

Activity	Causes	Number of cases
Channel alteration	Channelization	41
	Longitudinal constriction	11
	Mainstream grade change spread to tributaries	10
	Structures other than dams	8
	Dredging	3
	Clearing debris	1
		74
Damming and reservoir regulation	Clear water releases	10
	Backwater	5
	High sustained regulated flows	3
	Low sustained regulated flows	2
	Dam removal	1
	High controlled irrigation releases	1
		22
Streambed mining	Sand and gravel	16
	Borrow pit	3
		19
Land use changes	Urbanization	4
	Agriculture	3
	Strip mining	2
	Logging	2
		11
Construction activities		3

were unaffected by human activities. Table 1 lists the natural causes of gradation problems with the number of cases isolated for each cause.

Human activity in watersheds has profoundly impacted stream stability. The human activities most responsible for gradation changes are found by analyzing the data base and are given in table 2.

Identifying Degradation or Aggradation

Figures 2–4 illustrate extreme cases of degradation and aggradation. When a situation progresses to that depicted in the figures, it is easy to establish that a problem exists. It would be better if problems were recognized before any appreciable change in bed elevation occurs, especially when planning a new crossing. Fortunately, such early warning is possible to achieve.

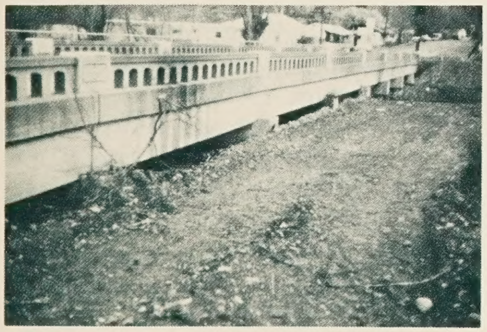

Figure 2.—Excessive aggradation requiring maintenance, which is currently underway.

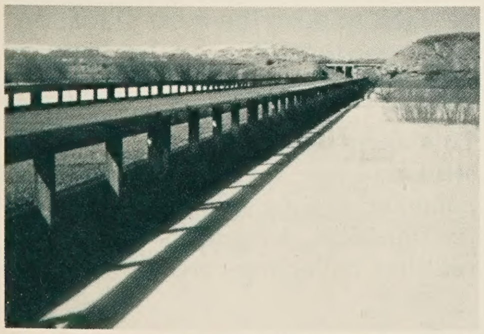

Figure 3.—Extreme case of aggradation that is easily observed.

Figure 4.—Extreme case of degradation resulting in bank erosion (both banks), slumping, and debris problems.

Methods for recognition of a gradation problem can be grouped under three categories—regional awareness, awareness of impacting activities, and simple data collection and analysis methods. Anyone can use the methods, although individuals with training in hydraulics and geomorphology will have a much keener sense of the magnitude of the problem, especially in evaluating the impacts of watershed activities.

Regional awareness

Regional awareness is the easiest and the logical first step in identifying gradation problems. Figure 1 can be used as a guide to problem potential nationwide. On a regional level, past stream crossing experiences should never be overlooked. A significant elevation change in one streambed in an area is a clear warning that other streams in the area may be undergoing change. Similitude is an excellent geomorphic tool. It should be used as a first step to determine if additional information is necessary. At the individual stream site, evaluation must be extended both upstream and downstream; each stream reach is only a segment of the entire stream system.

If any segment of a stream system is degrading or aggrading, this change may affect the stream elsewhere, its tributaries, and streams to which it is tributary. Gradation changes within the stream basin do not necessarily diminish in severity as they progress through the system. It is often easier to recognize degradation on small tributaries than on large streams. Incisement (vertical cutting) at junctures is a good clue to main channel degradation (fig. 5).

Figure 5.—Degradation on small tributary is more easily identified than on main stream.

Awareness of impacting activities

Remaining aware of activities that change sediment/transport relationships in the watershed also is useful in anticipating gradation changes. Research shows that almost all *severe* gradation changes were directly caused or influenced by human attempts to change an aspect of the river's natural morphology. Some of these human impacts are considerable, for example, construction of a dam. Others may be more subtle, such as stream channelization or changes in land use. Table 2 lists activities that cause gradation changes and the number of case studies evaluated. Identifying activities that impact sediment/transport relationships requires a broad understanding of river dynamics. The search for impacting activities must *not* be limited to areas close to the site. The magnitude of the impact in many cases is more important than direct proximity to the crossing. Gradation changes have progressed many kilometres on small streams and hundreds of kilometres on major rivers. When more than one activity is impacting the stream, evaluation is complicated, especially when attempting to determine the degree each action increases or decreases the effects of other activities.

Future land uses within the stream system should be anticipated by consulting organizations likely to cause impacts to the stream system (such as the Corps of Engineers, Bureau of Reclamation, and utility companies) and those that control or monitor activities that could affect streambed stability (such as the

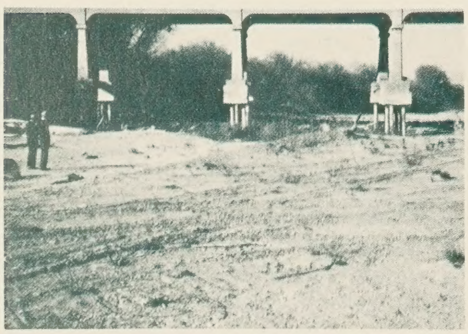

Figure 6.—Degradation caused by gravel mining upstream results in loss of foundation.

Environmental Protection Agency, the Corps of Engineers, and local governments). Impacts by industry and private landowners are more difficult to anticipate; however, past use is often a good guide to potential future use. For example, if a stream has been used for gravel mining, it may again be mined (fig. 6).

Data collection and analysis

Simple data collection and analysis methods for identifying whether a degradation or aggradation problem exists are commonly, but not necessarily, a final check to affirm inferences drawn from regional awareness and awareness of impacting activities.

The most direct method for evaluation of a gradation change is measurement from a fixed object (a bridge deck) to the streambed. Progressive and continual lowering or rising trends in bed elevations over an extended time indicate a grade change. The major drawback of this simple method is that it is time consuming. Original bridge design plans often note depth from bridge deck to streambed at time of construction. A quick field measurement when compared with this existing data is useful to establish if a change has occurred.

Engineers should be careful to avoid measuring local effects. Some streams have streambeds that fluctuate naturally. Also, measurements of bed elevations resulting from high flows must not be compared with those resulting from a dry period. The longstanding average bed elevation is the representative value desired for comparisons. Therefore, repeated measurements from different positions across the stream will yield the best results. The kind of stream and experience of the evaluator will dictate the care required for measurement and the usefulness of the data collected.

If historical data on streambed elevation at the crossing are not available or if the data appear unreliable, stream flow data available at gaging stations of the U.S. Geological Survey and the Corps of Engineers should be used. Shifts in rating curves that relate stage to discharge are often good indicators of changes in streambed elevation.

Sediment load data collected at a gaging station are useful for indicating gradation trends. Any long term change in sediment load signals an imbalance in the energy regime of a stream system. Such imbalances lead to lateral erosion, degradation, and aggradation.

Methods for Determining the Extent of Streambed Degradation or Aggradation

Methods for determining the extent of gradation changes are focused on sediment/transport relationships. These methods range from simple mathematical statements on general geomorphic principles to complex iterative computer models that solve differential flow and sediment transport equations.

The most widely known geomorphic relationship is based on the concept of equilibrium. (*3*) Stream equilibrium occurs when the product of water flow (Q) and channel slope (S) balance with sediment discharge (Q_s) and size (D_{50}). The equation $Q \cdot S \sim Q_s \cdot D_{50}$ describes a stable situation where no aggradation or degradation will occur. Any change in a component will effect a corresponding change in another component. This simple relation can be used to describe most gradation situations but will not provide a quantitative evaluation.

To make a quantitative evaluation, stream power must be evaluated and balanced against sediment available for transport. A rigorous analysis requires collecting considerable field data and having it analyzed by hydraulics experts. Computations are complicated and must be repeated until a balanced situation is defined. Although this kind of evaluation has been undertaken for some highway crossings, it is too expensive for most applications. Methods that simplify sediment/transport relationships and generalize stream behavior are available for use by highway engineers. (*4*)

Countermeasures for Gradation Problems

A gradation control scheme must be based on a clear understanding of the stream system and all hydraulic hazards posed at crossing sites. Control schemes are most effective on small streams. Attempts to control sediment transport problems are less likely to be effective as stream size increases.

The following list summarizes possible countermeasures at a crossing experiencing degradation. (*4*)

- The most successful technique for halting degradation on small- to medium-sized streams is the use of check dams or drop structures (fig. 7).
- The use of channel linings alone has not provided a successful countermeasure against degradation problems.
- On mild abutment fill slopes, properly keyed rock riprap provides sufficient protection against slumping. On steeper slopes, concrete paving has proved successful except where internal slope failures could occur.

Figure 7.—Series of check dams to mitigate a serious degradation problem.

- Combination of bulkheads and riprap revetment have been used to successfully protect abutments where streambanks are characterized by steep cuts against mild abutment fill slopes.
- Riprap does not provide an adequate degree of protection against general channel degradation.
- Deeper foundations at piers and pile bents provide successful pier protection.
- Jacketing piers with steel casings or sheet pile also has proved successful where expected degradation does not extend below the original foundation.
- The most economical solution to degradation problems at new crossing sites on small- to medium-sized rivers is to minimize the number of piers in the flow channel and provide adequate foundation and abutment depths.

Following is a list of conclusions regarding aggradation counter-measures. (*4*)

- Channelization may seem to be a logical countermeasure but even extensive channelization projects are generally unsuccessful at alleviating *long term* general aggradation.
- Where the aggradation is from a *temporary* source or occurs on small channels where the problem is limited in magnitude, sediment removal may be cost effective.
- At crossings with severe problems, such as on alluvial fans, the construction of a debris basin in combination with controlled sand and gravel mining may prove effective.

Summary and Conclusions

Stream channel degradation and aggradation are long term adjustments in stream elevation that depend on the energy regime of the river system and its consequent sediment/transport potential.

Gradation changes are possible on any river that flows on an alluvial bed. They are most prevalent on rivers in areas with a high sediment yield or where human activities have caused major changes in sediment/transport relationships within the watershed. Research on gradation changes indicates that streambed elevation should be inspected for changes periodically, especially in areas prone to problems.

For the majority of streams, degradation and aggradation do not pose significant problems. But in instances where they do, the engineer must be alerted to the possible high magnitude of the problem. Often the symptoms of gradation changes are all that is immediately obvious at the crossing. These symptoms, whether bank erosion, local scour, waterborne debris accumulation, or berming, can be mistakenly viewed as acting independently and not the result of a pervasive stream instability situation. Treatment of only these symptoms without awareness of the pervasive stream instability problem can result in costly maintenance and misapplied countermeasures. In instances where a gradation change is recognized, proper crossing design and countermeasures can be effective.

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(2) S. A. Gilje, "Debris Problems in the River Environment," *Public Roads,* vol. 42, No. 4, March 1979, pp. 136–141.

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(4) S. A. Brown, R. S. McQuivey, and T. N. Keefer, "Stream Channel Aggradation and Degradation: Analysis of Potential Impacts to Highway Crossings," Report No. FHWA/RD–80/159, *Federal Highway Administration,* Washington, D.C., March 1981.

Available Literature

Additional information concerning stream stability and highways is available in the following publications:

"Highways in the River Environment, Training and Design Manual," *Federal Highway Administration,* Washington, D.C., May 1975. For sale by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Price $5.25; Catalog No. TD 2.8:H53/2).

"Countermeasures for Hydraulic Problems, Vol. I, Analysis and Assessment," Report No. FHWA–RD–78–162, *Federal Highway Administration,* Washington, D.C., September 1978.

"Methods for Assessment of Stream-Related Hazards to Highways and Bridges," Report No. FHWA/RD–80/160, *Federal Highway Administration,* Washington, D.C., March 1981.

"Stability of Relocated Stream Channels," Report No. FHWA/RD–80/158, *Federal Highway Administration,* Washington, D.C., March 1981.

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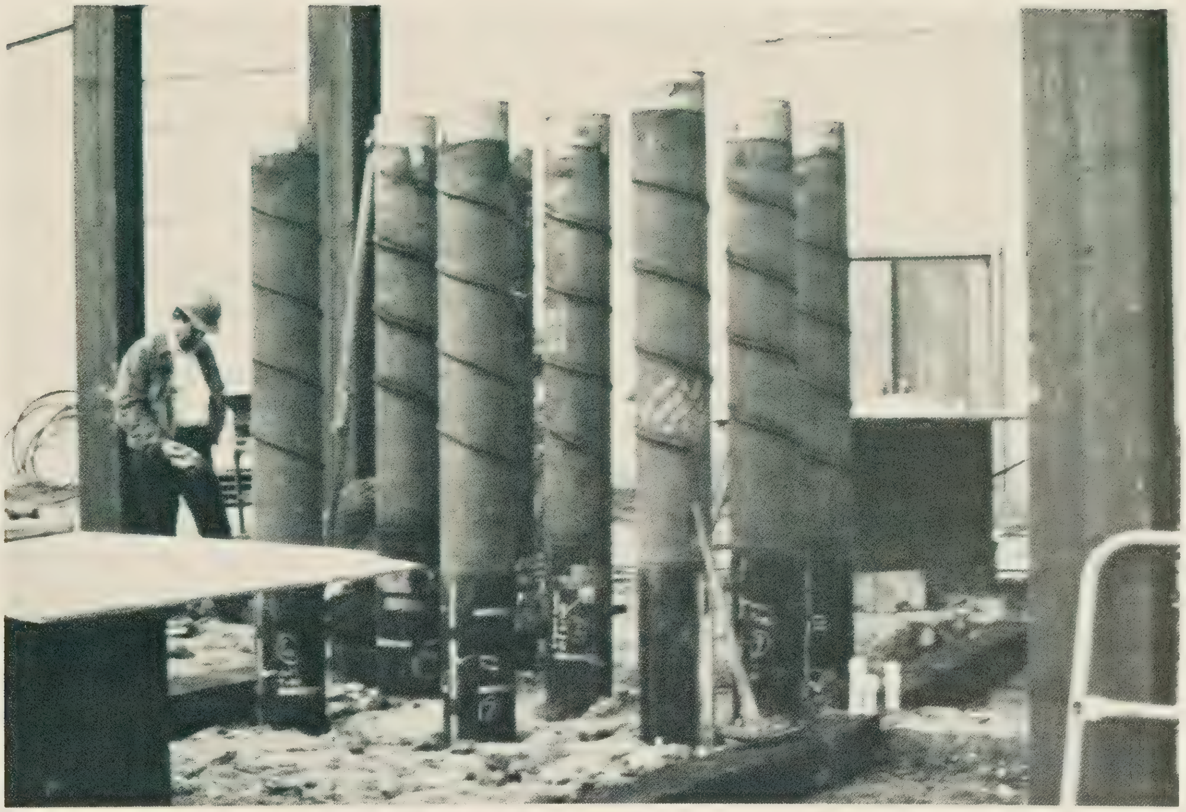

Design Inferences for Pile Groups

by
Michael W. O'Neill and Andrew G. Heydinger

Full-Scale Testing and Mathematical Modeling of Pile Groups in the Design Process

This article is concerned with applying studies of pile group behavior to the foundation design process by addressing where full-scale pile group tests and mathematical modeling of pile groups fit into the overall design process. Either formally or intuitively, engineers follow six basic and distinct steps in designing foundations (fig. 1). (*1*)[1]

In the first step, a preliminary design often is developed by prescribing pile lengths, spacing, and materials predicated directly on classification or index tests, which may include simple strength tests such as the Standard Penetration Test (SPT) or unconfined compression tests, and a knowledge of site stratigraphy and load conditions when the behavior of similar pile groups in like deposits is well known. When specific details of the soil or loading conditions lie outside the engineer's experience, results of soil classifications may be applied by means of empirical correlations to arrive at fundamental parameters (step 2), such as ultimate unit side shear and unit end bearing values for a particular pile type, from which the preliminary design then may be made. Unusual considerations, including special requirements on structural deformations, may occasionally make it prudent to consider special tests (step 3), such as in situ soil tests, to better estimate the deformability of the soil.

Occasionally, predicting capacity and settlement of a pile group becomes so complex or lies so far outside the experience of the designer that full-scale load tests are

[1]Italic numbers in parentheses identify references on page 61.

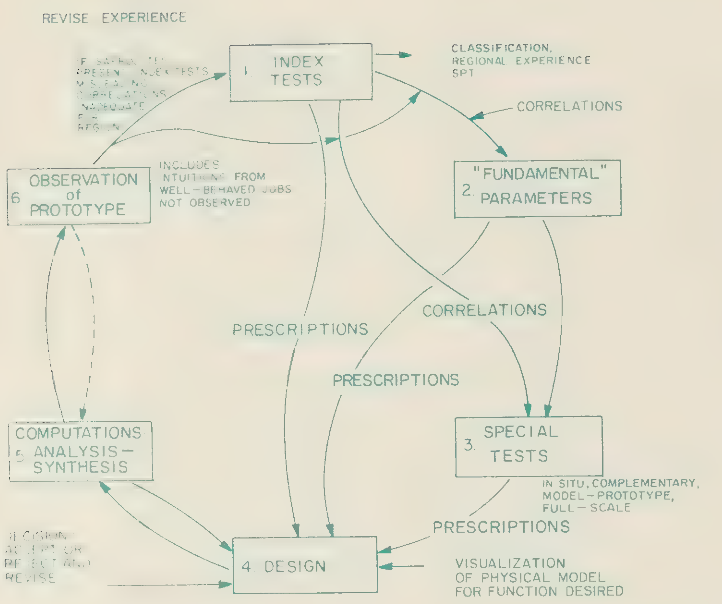

Figure 1.—The "design-experience" cycle. (1)

warranted as an integral part of the design process. A hypothetical example of such a case would be using load tests to determine the potential economic savings inherent in using groups of drilled piers in a sand formation where drilled pier groups have not been used before. Once such special tests have been conducted, the preliminary design can be completed.

Rarely can a preliminary design for a pile group be finalized without some kind of analysis. Even where full-scale tests have been conducted, exact inservice conditions may not have been modeled. Such analysis is simple initially, involving computation of pile-head loads (using assumptions that allow principles of simple statics to be invoked), deformations (based perhaps on parametric solutions derived from elasticity theory (*2*) or from a sophisticated mathematical model (*3*)), and, perhaps, cap stresses. Inadequacies of the preliminary design discovered in the analysis phase (including economic inadequacies) require modifying the preliminary design. This subprocess proceeds iteratively until the preliminary design becomes a probable design (step 4). At this stage, sophisticated mathematical models (usually in the algorithmic form of computer programs) may be used if the probable design involves "unusual" conditions. Such conditions may involve very long flexible piles, complex combined loadings, dynamic loadings, the need for variable pile lengths, or three-dimensional pile geometry. An appropriate model might compute the deformations of the group under a prescribed system of loads, distribution of loads against the cap (from which the cap can be designed), stresses along the piles, and group capacity. At this stage a computational procedure commonly is applied to the predictions of long term behavior, notably settlement or possible changes in pile stresses from downdrag (step 5). If the piles are to be driven, computations also are applied to predictions of the piles' driveability and anticipated driving systems. If the probable design is unacceptable based on predicted performance and constructability, it is rejected and a new probable design is developed and subjected to mathematical modeling. This subprocess also proceeds iteratively until an acceptable design is found.

It should be mentioned that most mathematical models for predicting the performance of pile groups are deterministic. Because the soil properties, structural material properties, loading, and environmental effects can never be modeled exactly, judgment is essential in applying the results of mathematical modeling to the design of pile groups.

Once the final design is constructed, the prototype's performance should be observed (step 6). The results of such observations serve two main purposes. First, on a major project, accurate and timely observations of the constructability and initial performance of foundation elements constructed early in the project may permit adjustments to the design of later elements where necessary. (*4*) Second, accurate and appropriate observations of behavior, whether made on prototypes or during full-scale tests, are the only positive means of completing the design cycle because the cycle is intrinsically tied to experience of the individual engineer and the profession as a whole. Here the role of applied research involving full-scale testing becomes evident because such research can enhance the profession's experience in a way that is seldom matched on a production project. That is, it is possible to investigate failure mechanisms, to examine the relative influence of various soil properties, and, perhaps most importantly, to measure accurately.

To generalize behavior patterns from reported full-scale test data, the parameters that control such behavior must be identified. The aspects of pile group behavior most commonly of interest are efficiency (capacity of pile group/N times the capacity of an isolated control pile, where N = number of piles in the group) and settlement ratio (the ratio of settlement of the pile group to the settlement of an isolated control pile when the load on the group divided by the number of piles in the group is one-half the failure load of the control pile). The evaluation of both efficiency and settlement ratio requires evaluation of control pile and pile group capacity, or failure load. (*5–7*)[2]

FHWA Stiff Clay Pile Group Study—Houston Site

To evaluate methodology for analyzing and predicting the behavior of full-scale pile groups, the Federal Highway Administration (FHWA) sponsored a series of load tests of an instrumented full-sized group of closed-ended steel pipe piles in the stiff, fissured Beaumont and Montgomery clay formations in Houston, Tex. (*8*) The arrangement of the test site is shown in figure 2 and general stratigraphic conditions are shown in figure 3. Instrumentation consisted of calibrated strain gage circuits to measure loads and load transfer patterns, pile and soil piezometers to measure pore pressure buildup and dissipation, soil telltales to measure vertical movements in the soil, and total pressure cells to measure total normal stresses at the pile-soil interfaces. Settlements of the various piles and cap movements were measured by dial gages suspended from a reference system. Backup load transfer data were obtained with a mechanical telltale system; backup load data were obtained with electronic load cells; and backup translation data were obtained through survey methods.

The piles were loaded by jacking against a reaction frame that was secured at two points by concrete anchors whose bases were about 19 m (63 ft) below the tips of the piles. The piles were arrayed in a 3 × 3 group that was capped by a rigid concrete cap suspended above the soil surface. Two control piles (piles No. 1 and No. 11 in fig. 2) were tested essentially concurrently with the group tests, which were conducted to failure three different times after installation to investigate freeze phenomena. Following the nine-pile group tests, piles 4, 6, 8, and 10 (fig. 2) were cut away from the cap, and a fourth test was conducted on the subgroup consisting of the remaining piles. Following that test, the center pile (pile 2) was cut away from the cap, and a fifth test was conducted on the resulting subgroup.

The actual positions and alinements of the piles were measured by survey methods and by pile inclinometers. The 273 mm (10.7 in) diameter piles were nominally vertical with nominal 3 diameter spacings. The mean top-to-tip deviation from vertical alinement was about 150 mm (5.9 in).

[2] M. W. O'Neill, "Field Study of Pile Group Action, Interim Report," March 1979. Copies available from the Materials Division, HRS–21, Federal Highway Administration, Washington, D.C. 20590.

The soil profile in figure 3 shows that there were six identifiable strata within the zone of interest. A major contact (between the Beaumont and Montgomery formations, two conformable Pleistocene terraces) existed at a depth of 8 m (26 ft). Standard penetration data are shown. Strength, compressibility, and stress-strain data were obtained for the site soils by numerous field and laboratory procedures, including the static

Figure 2.—Pile layout and ground instrumentation plan for FHWA Houston tests.

Figure 3.—General soil conditions at FHWA Houston test site.

cone penetrometer, self-boring pressuremeter, crosshole seismic, normalized CU triaxial, residual direct shear, and one-dimensional consolidation tests. (*8*)

The principal findings of the study in terms of efficiencies, settlement ratios, and load distribution among piles are presented in tables 1 and 2. The efficiencies for the nine-pile group and subgroup tests were essentially 1 (when the cap weight was included as applied load), the settlement ratios were 1.2 to 1.6 depending on the size of the loaded group, and the load distribution among the piles was relatively uniform. These findings appear to be the result of three principal factors:

1. The soil at the test site had a recognizable secondary structure consisting of closely spaced slickensides above a depth of 7.9 m (26 ft) and sand partings below that depth. This in situ soil structure permitted rapid dissipation of excess pore water pressures generated by driving the piles. Rapid pore pressure dissipation also may have resulted because the piles were driven from the center outward in a serpentine pattern so that excess pressures from the interior pile were not "trapped" by exterior piles. Pore pressures at the time the first load test was conducted were essentially hydrostatic.

2. The soils at the test site were insensitive.

Table 1.—Summary of gross test results for FHWA Houston tests

Test[1]	Failure mode	Plunging failure load	Settlement at 50 percent of plunging load	Efficiency	Settlement ratio at 50 percent of plunging load
		k[2]	*inches*		
SP 1 (15 days)	Plunging	168	0.068		
SP 11 (15 days)	Plunging	133	0.048		
9–Pile (20 days)	Plunging of individual piles with tipping of group	1,332	0.094	0.99	1.62
SP 1 (78 days)	Plunging	187	0.080		
SP 11 (78 days)	Plunging	170	0.067		
9–Pile (82 days)	Plunging of individual piles	1,532	0.113	0.98	1.54
SP 1 (105 days)	Plunging	177	0.080		
SP 11 (105 days)	Plunging	181	0.077		
9–Pile (110 days)	Plunging of individual piles	1,541	0.116	0.96	1.48
5–Pile (113 days)	Plunging of individual piles	832	0.096	0.93	1.31
4–Pile (116 days)	Plunging of individual piles	661	0.085	0.92	1.19

[1] Times between pile installation and test shown in parentheses.
[2] Includes cap weight and weight of loading accessories.
1 k(f)=4.45 kN
1 in=25.4 mm

Table 2.—Summary of load distribution for first FHWA Houston test[1]

	Average load per pile (k) at approximately one-half plunging failure load			Average load per pile (k) at plunging failure load		
Pile position	Pile head	Shaft load	Tip load	Pile head	Shaft load	Tip load
Center	59.9	51.8	8.1	153.2	119.3	33.9
Edge	64.0	57.4	6.6	138.9	112.5	26.4
Corner	66.4	58.2	8.2	141.5[2]	113.8	27.7
Control	63.1	60.3	2.8	149.5	127.0	22.5

[1] Shaft and tip loads are apparent loads that do not consider residual stresses produced by installation.
[2] Corner Pile 8 failed before failure of group as a whole.

1 k(f)=4.45 kN

3. The elastic stiffness of the soil in situ, as determined through seismic and pressuremeter tests, increased linearly with depth to the full depth of extensive exploration (19.3 m [63.3 ft]).

The first two factors were important in producing the nearly uniform efficiencies. The last factor was significant in producing low settlement ratios because the stiffness of the soil beneath the pile tips appeared to govern the short term load-settlement behavior.

The design implications of these observations are that efficiencies in small groups in stiff clay with moderate spacing are likely to be near 1 if a strong secondary structure pattern exists (or if the time between installation and first loading is sufficient to allow dissipation of excess pore pressures) and if the soil is insensitive. Furthermore, where the soil also possesses stiffness that increases with depth, settlement ratios will be low and load distribution will be relatively uniform. As a point of reference, purely elastic analyses (assuming the soil to be uniform and infinitely deep) yield settlement ratios of 1.9 (four-pile subgroup) to 3.5 (nine-pile group) compared with the actual measured range of 1.2 to 1.6. (*8*)

Failure of the group and subgroups was found to be caused by punching failure of individual piles and not by "block action." Pile failure was progressive, both along a given pile and within piles in the group. The specific pattern of progressive failure observed in the first test is demonstrated in figure 4. Side shear failure began at both the tops and bottoms of the piles at about 90 percent of the plunging load (Load 6 in fig. 4). Failed zones are indicated by black bars. Complete failure was reached in pile 8, and then in all other piles on the upper and middle rows. This pattern resulted in the average load-settlement curve, which indicates apparent plunging failure because of rotation of the group cap before all piles had failed. End bearing was rather minor in the first group test, amounting to only about 20 percent of the applied load. For all practical purposes, the piles failed when complete failure had occurred between the pile shafts and the soil.

An important design inference then of progressive failure mechanisms in primarily friction pile groups in strain-softening soils, such as the stiff fissured clay at the Houston test site, is that plunging failure under long term sustained loadings is likely to occur at the load level that initiates the progressive failure (about 90 percent of the short term failure load at the Houston test site).

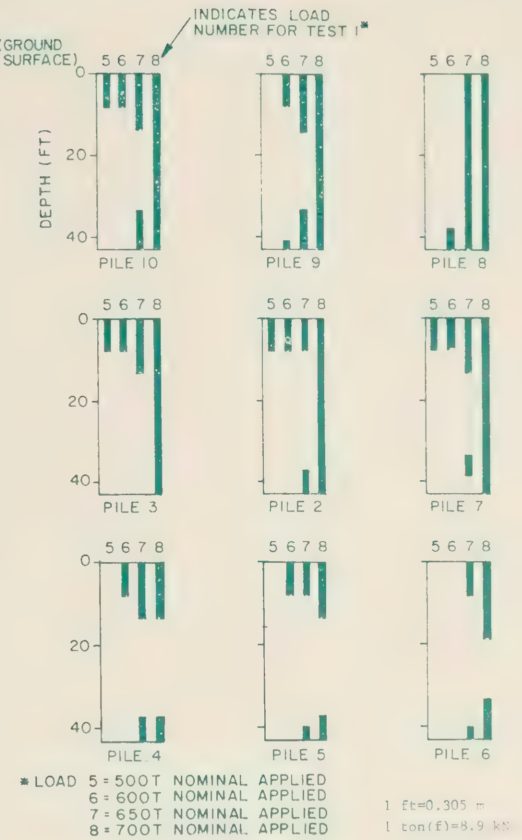

Figure 4.—Progressive failure in group piles at Houston test site.

Therefore, for groups supporting long term sustained loads it may be prudent to account for this effect in factor of safety or probability of failure calculations. It is possible to detect initiation of progressive failure in a short term load test. (*8*) On the other hand, if a significant portion of the design load is transient and is applied only occasionally, it may be unnecessary to consider progressive failure.

Table 1 reveals some increase in group and control pile capacity with time, probably because of increased tip capacity brought about by cyclic loading to failure instead of set up or freeze in the classical sense of response to pore pressure dissipation.

Mathematical Modeling

The design inferences discussed above apply only to small groups of moderately spaced driven displacement piles in stiff soil of the kind found at the Houston test site. Sound engineering judgment must be used before applying these inferences to sites with different soil characteristics or to larger groups or groups with more closely spaced piles. Mathematical modeling is one vehicle for applying that judgment.

An example of a mathematical model available for pile group design is a specific model used in the Houston field study. This "hybrid model"[3] assumes that the various group piles consist of flexible discrete elements that respond to both axial and lateral loading. The piles are supported by nonlinear soil springs that describe unit soil resistance as a function of relative movement between an isolated pile and the surrounding soil. Soil response can be prescribed in both axial and lateral modes of loading, and the hybrid algorithm developed for the Houston study also permits the piles to be battered in any three-dimensional configuration. The hybrid scheme also can model progressive failure caused by eccentric loading.

A simplified schematic of the discrete element model for axial response is shown in figure 5. The unit soil reaction curves are "f–z" curves, which represent unit side shear (f) versus pile deflection (z), and "Q–z" curves, which represent tip resistance (Q) versus tip deflection (z). These curves are developed from load tests on instrumented control piles or from published criteria and are programed into a computer by the user. Figure 6 shows typical nondimensional formulations for f–z and Q–z curves, in which f_{max} and Q_{max} are maximum values, and the z_c values are the shaft or tip deflections at which the maximum values occur. (9)

Figure 7 depicts the general procedure for modeling group action. First, load-settlement, or "mode," curves are generated for every pile in the group based on the unit soil resistance curves and pile properties that are input using the discrete element model shown in figure 5. These mode curves typically are nonlinear. A stiffness

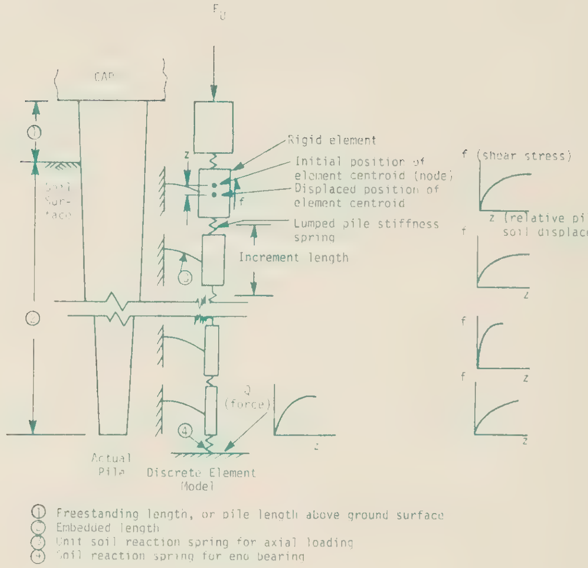

Figure 5.—Discrete element model for axially loaded pile.

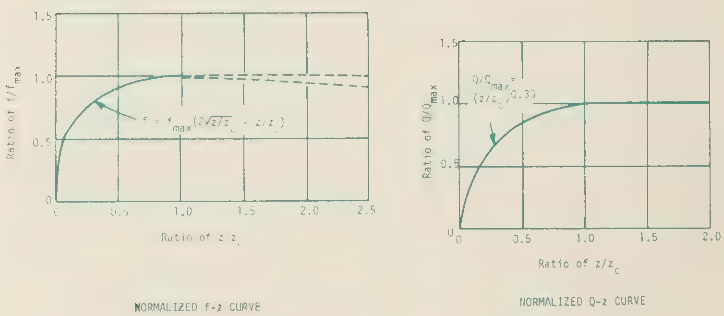

Figure 6.—Typical formulation for f-z and Q-z curves. (9)

matrix is computed from the initial slopes of the mode curves and group geometry. A small load then is applied to the group, pile-head deformations and loads are computed and accumulated, and stiffnesses are adjusted. Another small increment of load is applied, and the process is repeated until the specified load is reached. Then, using the axially loaded pile algorithm with the accumulated pile-head loads as boundary conditions, the distribution of load along every pile is computed. Group effects then are introduced by calculating at every node of every pile the soil deformations caused by distributing the computed load to the soil along every node on every other pile. These calculations apply Mindlin's equations and assume that

[3] M. W. O'Neill, "Field Study of Pile Group Action, Interim Report," March 1979. Copies available from the Materials Division, HRS–21, Federal Highway Administration, Washington, D.C. 20590.

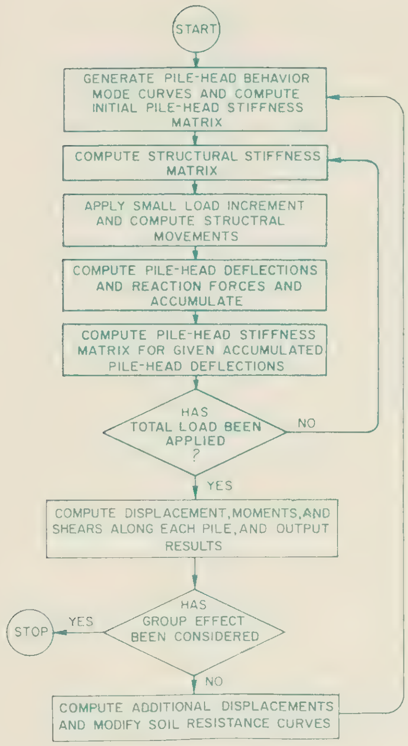

Figure 7.—General flow chart for hybrid model algorithm.

the soil surrounding the piles is elastic. (*8*)[4] Once the additional displacement (caused by loads on other piles) is determined for a given node on a given pile, the f–z or Q–z curve for isolated behavior at that node is modified so that the unit load transfer is compatible with the additional displacements caused by group action,[5] and the entire loading process is repeated. The resulting solution (settlements, pile-head loads, and load transfer patterns) is then an approximation of the expected behavior, provided group action is principally mechanical. Where group action is not mechanical (for example, where installing piles in groups substantially changes the density or stress field in soil immediately adjacent to group piles from that adjacent to isolated piles, such as may occur in deposits of loose sands or sensitive clays), this model may not yield good results unless the unit load transfer curves are adjusted externally for such effects.

The Houston site tests were modeled mathematically using the hybrid model in the form of program PILGP1. The f–z and Q–z curves were developed directly from the initial load tests on the two isolated, instrumented piles. It was found that for the soil at the Houston site, the α method yielded values of f_{max} (fig. 6) that averaged about 10 percent too high when cohesion was measured by unconsolidated, undrained triaxial tests; the λ method yielded values about 20 percent too high; and the effective stress method yielded extreme overestimates of f_{max} in the upper half of the piles but overestimated f_{max} by only about 20 percent below that depth. (*8*) The factor z_c for side resistance varied from layer to layer. As a reasonable representation, z_c can be approximated as 5 mm (0.2 in) in Strata I–III (fig. 3) and 2.5 mm (0.1 in) in Stratum IV. A rational equation for z_c (*8*) yielded approximations of measured z_c when the elastic moduli obtained from in situ pressuremeter tests were used. (Moduli from laboratory tests are too low, and z_c values determined from such moduli are greatly overpredicted.) The apparent Q_{max} value was about five times the product of the shear strength indicated by pressuremeter testing in the soil at the level of the pile tips and the tip area. This apparent Q_{max} value does not include residual loads present at the tips after installation. The z_c value for the pile tips was equal to about 3 percent of the tip diameter.

Figure 8 depicts load-settlement relationships predicted by PILGP1 using unit load transfer curves derived from tests on the control piles and two different values for

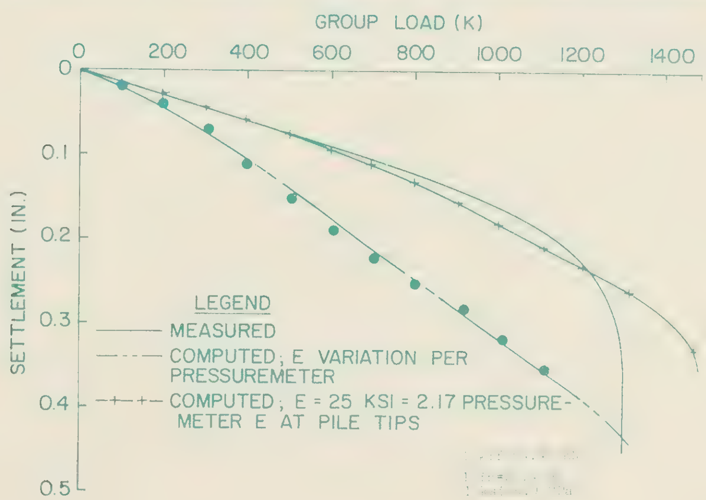

Figure 8.—Predicted and measured pile group load-settlement curves.

[4] Ibid.

[5] Ibid.

group effect modulus, E, for use in Mindlin's equations. Poisson's ratio was established at 0.5. Settlement was overpredicted using pressuremeter E values, presumably because the strain level at which the modulus first could be read on the pressuremeter exceeded the strain level produced by loading in the mass of soil surrounding the group. This resulted in interpreting an equivalent modulus that in the nonlinear soil was too low. Predicted and measured settlement correlated well when E was taken as the average of the seismic crosshole and pressuremeter values at the level of the pile tips (=2.17 E indicated by the pressuremeter). Capacity was slightly overpredicted because the cap weight was not included in the measured load and certain approximations were made in assigning f–z curves to various nodes.

E for group effect calculations can be expressed in terms of the cohesion value measured from unconsolidated, undrained triaxial tests at the Houston site. E/c was approximately 1,200. This factor is consistent with ratios calculated for other very stiff, fissured clay sites. In softer clays, a slightly lower value (about 850) appears appropriate. (*10*) Preliminary studies of a well-instrumented, full-scale group test on tapered piles 11 m (36 ft) long in medium to dense sand (*11*) indicate that the modulus should be varied uniformly for those conditions from about 100 MN/m^2 (14,000 psi) at very low loads to 20 MN/m^2 (2,800 psi) at failure.[6]

[6]"Analyses of Load Tests of Single Pile and Pile Group in Sand Under Axial Loading," thesis by D. H. Aggarwal for Master of Science in Civil Engineering, The University of Texas at Austin, Austin, Tex., 1980.

Conclusions

In addition to its more analytical aspects, the Houston study represents a significant link in the "design-experience" cycle (fig. 1). Preliminary inferences that can be drawn from the test include the following:

- For small friction pile groups in insensitive, overconsolidated clay having a strong secondary structure where pile spacing is in the 3 to 4 diameter range, group efficiency is near 1.

- Settlement ratios, which were in the range of 1.2 to 1.6, are smaller than those computed from elastic theory for infinitely deep uniform soils if soil stiffness increases with depth. The hybrid model can be used to compute settlement accurately if elastic moduli corresponding to an average of seismic and pressuremeter values in the soil at and just below the pile tips are used. This implies that settlement ratio is controlled strongly by the stiffness of soil below the pile tips.

- Group failure in insensitive, overconsolidated clays with strong secondary structure is by failure of individual piles rather than by failure of a block of soil contained within the piles. Block failure should be considered in designing groups only when static computation of the capacity of the block, using the full undrained shear strength (as opposed to the reduced strength used to compute single pile shaft capacities) to obtain the capacity of the soil in side shear around the block, is less than the sum of single pile side shear capacities.

- In soils that are strain softening, piles fail progressively if the load is not applied at the center of reaction. Progressive failure will decrease efficiency and should be considered in designing groups for predominantly sustained loads and for eccentric and inclined loads.

- Loads are distributed relatively uniformly to the cap (table 2) for the soil conditions found at the Houston site.

- The values of various inputs for f_{max}, Q_{max}, z_c, and E appear appropriate for analysis by PILGP1 for small groups of piles in insensitive, overconsolidated clay.

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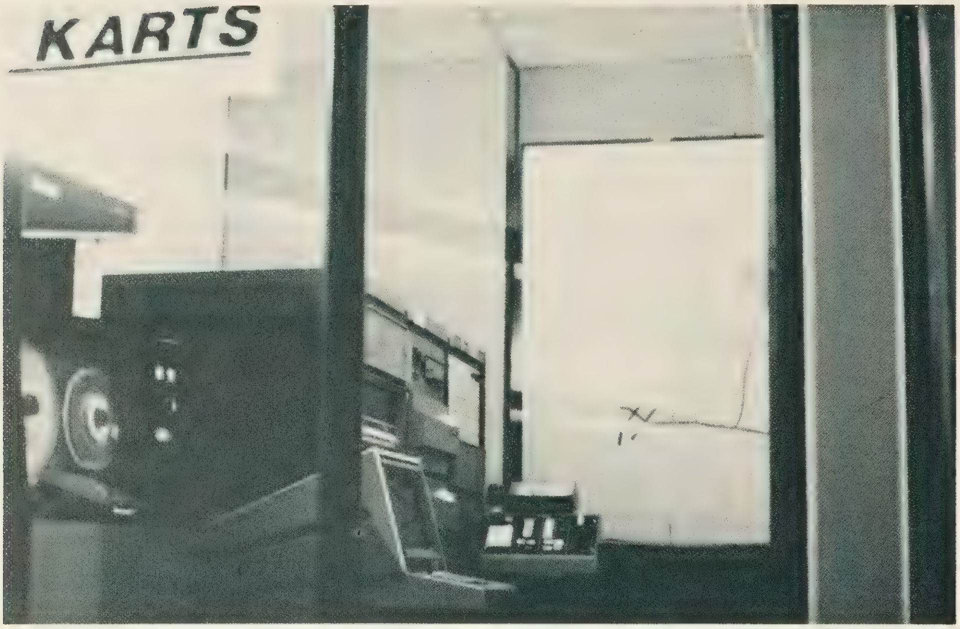
Role of Computer Graphics in Realtime Traffic Control Systems

by
Richard Lee Oaks



This article demonstrates how computer graphics can be used to help the traffic engineer retime traffic signals within a computerized signal system.

System Overview

The City of Kettering is a suburban community in the greater Dayton metropolitan area in the southwest quadrant of Ohio. Kettering has a population of over 60,000 and covers an area of approximately 52 km^2 (20 mi^2). Kettering does not have a central business district but has many shopping centers scattered throughout its area. These business areas are connected by a major arterial system that is basically on a 1.6 km (1.0 mile) grid. The traffic tends to be directional as people travel to and from their jobs in the morning and evening. There are 29 km (18 miles) of roadway with weekday average daily traffic (ADT) volumes in excess of 25,000 vehicles. Kettering has 65 traffic signals; several are major, and one carries almost 70,000 ADT at grade, with four signal phases including two double left turn approaches.

Interstate 75 lies to the west of Kettering, and Expressway Route 35 lies to the north. If the proposed Interstate 675 is built to relieve existing neighboring community capacity problems, most of Kettering still will lie beyond the 5-minute service radii of the interchanges.

To maximize the efficiency of Kettering's existing multilaned roadway system, in 1976 the Federal Government, State of Ohio, and City of Kettering contracted for a computerized traffic signal system called KARTS—Kettering Area Responsive Traffic System.

Using 1976 dollars, the entire system cost just over $1 million. KARTS, located on the first floor of the Kettering Government Center and visible to citizens as they enter the Government Center complex, has a full city map with intersection detail, line printer, card reader, magnetic tape, cathode ray tube (CRT), 64 K central processor unit (CPU), "real time" clock, two disk drives, communication display board, three terminals (ASR 43), hard copy output for the CRT, and an uninterruptible power supply (UPS).

The CPU communicates in a full duplex time division multiplex mode over 40 km (25 miles) of City-owned 22-gage twisted pair, shielded cable to an interface unit (IFU) at each local intersection. The IFU directs the CPU commands to the local intersection controller and then sends back to the CPU the status of the green lights and vehicle detector data.

KARTS operates with over 300 vehicle detectors. Both volume and time-on-loop (occupancy) are recorded in 15-minute increments and stored on disk for 7 days for each loop detector. These data are transferred manually to magnetic tape storage each week for later retrieval as needed.

After the computer was turned over to the City, the original traffic control software was enhanced. The background processing capabilities have been used to process various traffic analysis programs, including a full accident analysis data retrieval system. Recently, a program has been written to calculate splits at major multiphased intersections with overlaps using detector data directly from the computer files. In addition to these background programs, various others have been written including several graphic programs to display detector volume and occupancy data that are collected by KARTS from its 300 detectors.

Graphic Programs

The detectors located on the southbound Woodman Drive approach at the intersection of

Figure 1.—Typical southbound Woodman Drive p.m. peak traffic approaching Patterson Road.

Patterson and Woodman will be used to demonstrate the graphic programs (fig. 1). The two loop detectors are 0.56 m^2 (6 ft^2) and located approximately 91 m (300 ft) before the stop bar. Detector No. 281, the curb lane detector, is in the southbound right through lane, and detector No. 282, the median lane detector, is in the left southbound through lane.

Volume data

The first program was written to display traffic volumes for a 24-hour period in 15-minute increments on the horizontal axis and volume in vehicles on the vertical axis (fig. 2). The maximum scale value of vehicle volume per 15 minutes is an operator input manually selected to match the data. The detector identification number, day of week, and date the plot was made are just below the hour times on the horizontal axis.

For detector No. 281, traffic was well below 25 vehicles per 15 minutes until about 6 a.m. when the traffic started to build to a peak of approximately 125 vehicles at about 6:45 a.m. A sudden unexplained drop bottoms out at about 9 a.m. Continuing, there are fluctuations in the volume level with an evening peak of about 150 vehicles at 5 p.m. and another peak at about 5:30 p.m.

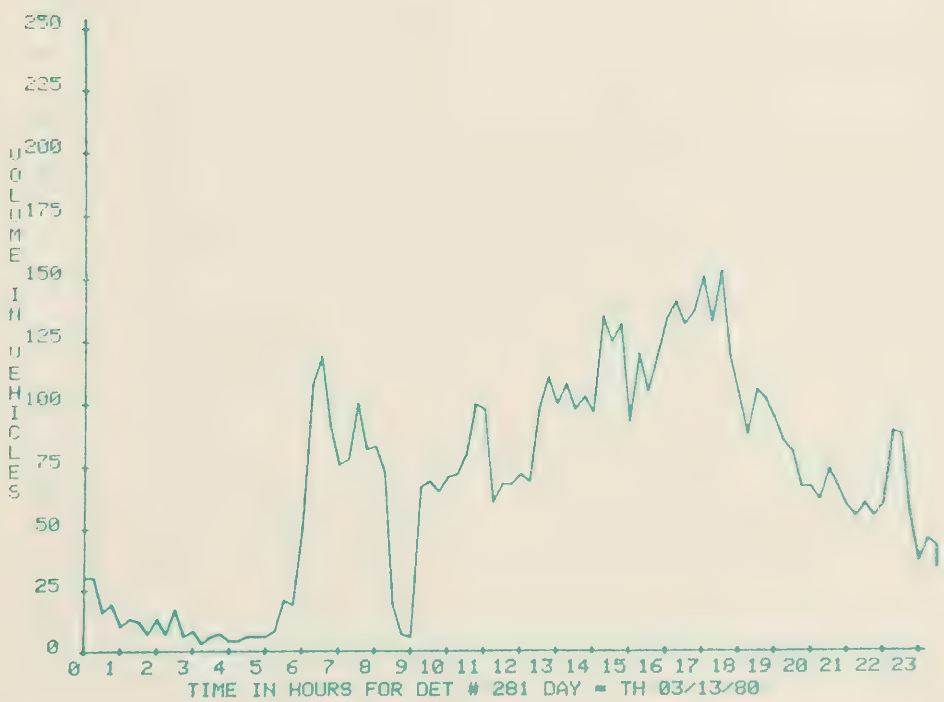


Figure 2.—Traffic volume in 15-minute increments.

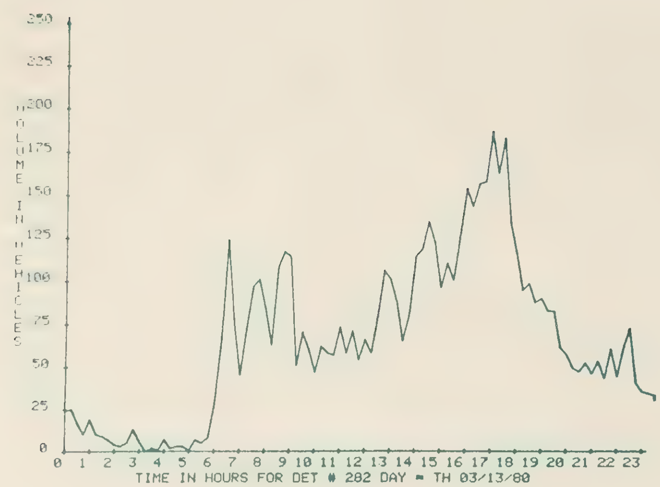


Figure 3.—Traffic volume at approximately 9 a.m. increased because of restriction in adjacent lane.

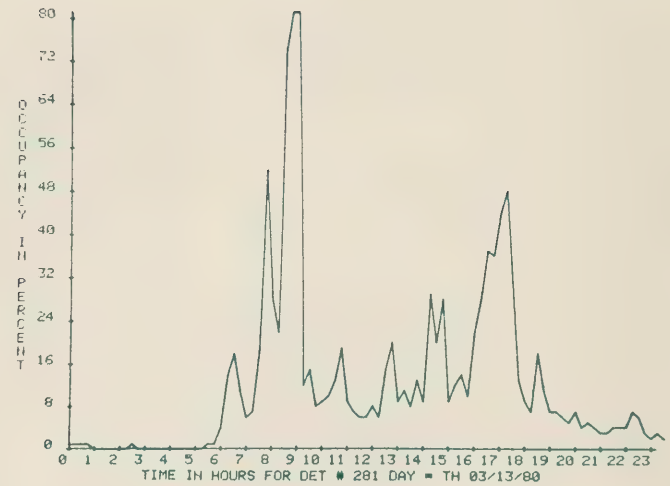


Figure 4.—Traffic occupancy in 15-minute increments—provides an additional dimension to traffic flow.

Figure 3 (detector No. 282) shows the same general volume trends with perhaps slightly higher overall volumes. A careful comparison of the volumes around 9 a.m. indicates that the median lane (fig. 3) carried the volume that appears to be missing in the curb lane (fig. 2).

Occupancy data

The next program was written to display percent occupancy on the vertical axis. Figure 4 plots the occupancy versus time for the curb lane detector No. 281. Occupancy exceeded 80 percent around 9 a.m. It is speculated that a road blockage caused the high occupancy rather than a vehicle loop malfunction because figures 2 and 3 indicate that the volume was transferred to the adjacent lane. Throughout the day, traffic density fluctuated, peaking to approximately 50 percent about 5:30 p.m. and decreasing rapidly at approximately 6:30 p.m.

Volume-occupancy

The above example indicates that volume data alone can be misleading. The occupancy data provide an additional dimension to the traffic flow. Unfortunately, it is difficult to correlate mentally the volume and occupancy data. One solution is to display both the volume and the occupancy data on one graph. Figure 5, a volume versus occupancy graph of detector No. 281 between 7 a.m. and 10 a.m., depicts the severity of the congestion at 8:45 a.m. and 9 a.m. The congestion began at 8:15 a.m. and increased as the volume dropped with a corresponding increase in occupancy. At 9:15 a.m. the restrictor of traffic flow was removed.

During a.m. and p.m. peaks, volume versus occupancy graphs generally depict the classic parabolic shape. It is not unusual to see the result of cycle length/split changes. If traffic continues to build and congestion increases, the point in time when intersection timing needs to be improved can be seen.

Volume divided by occupancy

Figure 6 shows volume divided by occupancy on the vertical axis and time on the horizontal axis. The scale can be selected to match the characteristics of the detector being observed. During the offpeak times when there is no congestion, the graph generally exceeds the selected scale. However, the critical time periods are well within the scale so data can be analyzed and compared.

The peak congestion points match in figures 5 and 6. Also notable in figure 6 is that congestion increased as the data points approach the time axis. The units of volume divided by occupancy reduce to metres per second (feet per second) or speed. Figure 7 illustrates this by plotting speed of vehicles over the loop versus time. The only difference between figures 6 and 7 is the scaling factor to account for the statistical length of the vehicle and the trap length of the vehicle loop detector.

Cumulative Relative Frequency

Recently a program was written to provide a cumulative relative frequency (CRF) plot of detector data. A high graphic correlation was observed for detector No. 281 for 3 days of volume versus time data. Other locations also have been observed to exhibit high correlation for weekdays and week to week. However, like a fingerprint, each location is different.

Although it would be impossible to generate coefficients for a least squares polynomial of the volume versus time plot, it should be relatively easy to generate the coefficients for the CRF. The actual use of such a technique is not fully known, but it is thought that there is potential application to statistically estimate or project an expected traffic volume. This may have use in a traffic responsive or critical intersection control algorithm.

Time-Space Diagrams

The graphic CRT also is being used to draw time-space diagrams. The same information needed to draw a time-space plot manually is required to use this program. Typical information includes distance between intersections, the amount of green time to be displayed to the street, cycle length, bandwidth, and desired speeds. The program operates from a list of codes. By selecting the appropriate code, changes to individual components of the time-space diagram can be made. Basically the operator is stepped through the program so it is difficult to forget something. The program has no intelligence unto itself but when a signal engineer or technician uses the program as a tool, a signal progression can be designed quickly, accurately, and efficiently.

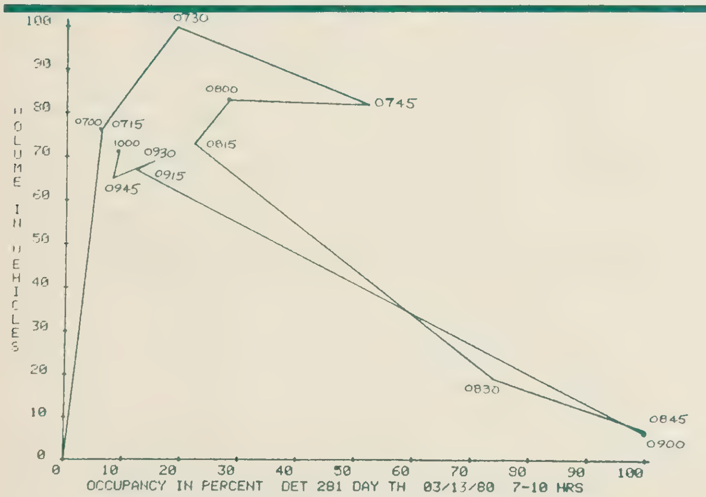


Figure 5.—Volume-occupancy combines two parameters providing more complete representation of traffic stream.

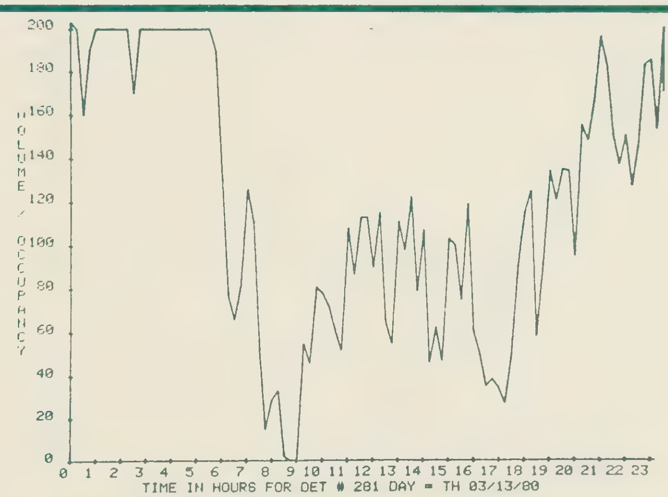


Figure 6.—Volume divided by occupancy provides an index of congestion for a full 24-hour period.

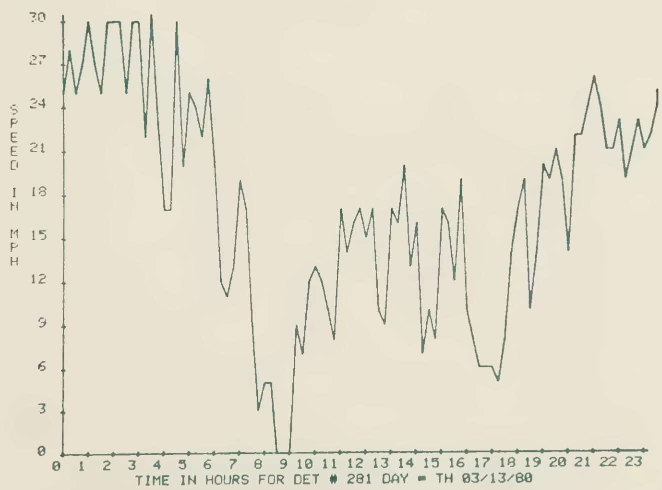


Figure 7.—Scaling factor accounts for statistical length of vehicle and length of loop detector.

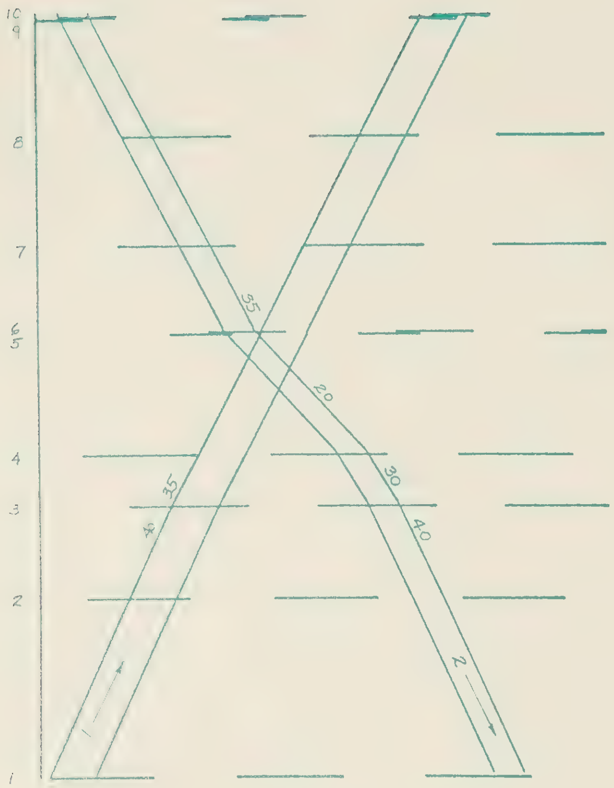


Figure 8.—Example time-space diagram showing lead-lag capability and initial signal relationships.

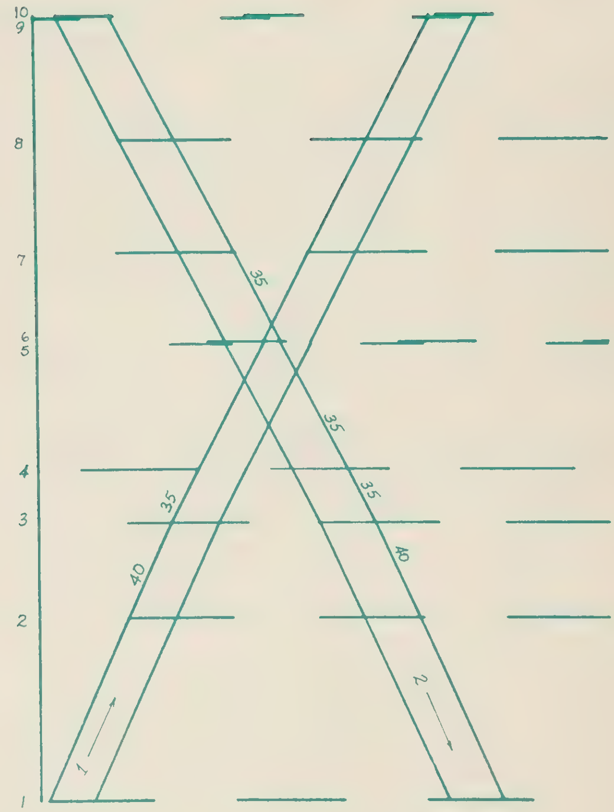


Figure 9.—Modification of initial example. Additional modifications would be required before system could be implemented.

For example, in figure 8 there are eight intersections with the through street greens being displayed by the black bars. Intersection Nos. 5 and 6 and Nos. 9 and 10 are very close together. Describing this distance between intersections as, say 15 m (50 ft), allows the traffic engineer to "trick" the program and work with lead-lag signal displays. In this example, when traveling in direction 2, there is a 56 km/h (35 mph) speed between intersection Nos. 10–6, a 32 km/h (20 mph) speed between intersection Nos. 6–4, a 48 km/h (30 mph) speed between intersection Nos. 4 and 3, and finally, a 64 km/h (40 mph) speed between intersection Nos. 3–1. A narrow green bandwidth in direction 2 has been selected as a beginning.

The first change is to increase the 32 km/h (20 mph) speed between intersection Nos. 6–4 to 56 km/h (35 mph). In Figure 9 note that the green band for direction 2 from intersection Nos. 4–1 has been moved to the left because of the speed change.

Next, the speed between intersection No. 4 and No. 3 is changed from 48 km/h (30 mph) to 56 km/h (35 mph). By increasing the offset value for intersection No. 2, this green can be placed within the progression, and the green band for direction 2 also can be increased. These changes are implemented and the new plot in figure 9 is made.

To see the status of the data values used to draw a graph, the appropriate code, in this case code 14, is entered and all the pertinent data are printed out (fig. 10). Based on the changes made, the updated time-space diagram can be reviewed. Certainly in the real situation further changes and enhancements would be made to this diagram to improve progression for both directions.

Summary

Whether drawing graphs of traffic volume data or working with time-space diagrams, nontechnical individuals can understand computer graphics better than computer printouts with rows and columns of numbers. Many engineers and technicians also find it easier to quickly capture from the graphs information they need to make efficient changes in traffic signal timing. It is also hoped that graphic techniques will lead to new and better ways to have an online traffic control system such as KARTS automatically make optimum decisions for timing changes.

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```
SPACE   ENTER CHANGE CODE: XX
00  SPACE   ! 00 14

SPACE
SPACE      CYCLE LENGTH = 120
SPACE      INTER  DISTANCE  OFFSET  GREEN
SPACE        1        0        5      68
SPACE        2     3100       58      66
SPACE        3     1600       58      76
SPACE        4      900       28      74
SPACE        5     2100       85      39
SPACE        6       50      109      49
SPACE        7     1500       52      75
SPACE        8     1900       55      70
SPACE        9     2050        0      30
SPACE       10       50       14      37
SPACE      DIRECTION  DISPLACEMENT  BANDWIDTH
SPACE          1            5           30
SPACE          2           14           20
SPACE      INTER  SPEED1  SPEED2
SPACE        1      0       0
SPACE        2     40      40
SPACE        3     40      40
SPACE        4     35      35
SPACE        5     35      35
SPACE        6     99      99
SPACE        7     35      35
SPACE        8     35      35
SPACE        9     35      35
SPACE       10     99      99
SPACE             COMMENTS
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Figure 10.—Data values used to create time-space diagram in figure 9.

Wet Weather Accidents and Pavement Skid Resistance

by
Rudolph R. Hegmon

Introduction

Highway traffic accidents are a major national concern. State highway and transportation departments spend large amounts of their annual budgets on accident reduction programs and on measures for minimizing injuries and damage if accidents do occur. The National Safety Council's 1978 summary of all accidents states that more than 100,000 people were killed, over 10 million were injured, and total accident cost was estimated at $70 billion. Motor vehicle accidents accounted for about 50 percent of the fatalities and accident costs and about 20 percent of the disabling injuries. (*1*)[1]

One of the many different causes for highway accidents is slippery pavements. Opinions differ on the importance of slipperiness relative to other accident causes, but there is no doubt that slippery pavements contribute to accidents. The basic, physical process is simple. All of the forces acting on a vehicle must be resisted by the friction forces between the pavement and tires. Dry pavements and many wet pavements have sufficiently high friction potential to meet the friction demand under all driving conditions. Some pavements, however, are too slippery when wet to provide adequate friction. It is difficult to establish and agree on which friction level is adequate. Most agree, however, that this level varies for different roads and traffic conditions.

Problem Statement

Research on pavement skid resistance, its causes, and its measurement has produced significant results. Skid resistance measurement in the United States is standardized and routinely conducted by at least 44 States. (*2, 3*) The reliability of these measurements has been greatly enhanced since the regional calibration and correlation centers began providing services, including diagnosis of measurement system performance and guidance of system operators. (*4*) Also, many Federal Highway Administration (FHWA) studies have increased the understanding of the effect of the many variables in skid resistance.[2] Basic knowledge for detecting and correcting skid-prone locations is available and is being used extensively. It is extremely difficult, however, to establish a clear relationship between skidding accidents and pavement surface condition. This article will identify the reasons for this, suggest new approaches to the problem, and attempt to stimulate researchers to renewed efforts.

[1] Italic numbers in parentheses identify references on page 73.

[2] "Slippery When Wet," final report of an FHWA research project on skid accident reduction. Not yet published.

Accident-Skid Resistance Relations

Many studies have been conducted to determine how serious the problem of slipperiness is. For example, a Kentucky study cited in an article in the September 1978 issue of *Public Roads,* vol. 42, No. 2, showed that a 25 percent reduction in the friction level could increase the proportion of wet pavement accidents by more than 50 percent. (*5*) Another study concluded that there is a small but statistically significant relationship between skid resistance and wet-pavement accident rate; however, the large variability of accident rates limits the usefulness of the relationship for predicting the accident rate for a given road section in a given year. (*6*)

The unreliable accident data and the characterization of pavement surfaces by a single index, the skid number (SN), are two possible reasons for weak correlations between accidents and low skid resistance. If the many accidents on U.S. highways are separated into categories by accident cause, the statistical base for each category shrinks considerably. Also, because of the lack of uniform reporting guidelines, accident reporting varies from State to State and jurisdiction to jurisdiction. Accident causes are seldom clearcut and may not be recognized by the accident investigator. This is especially true in the case of skidding accidents. Accident investigators have neither the means for measuring the skid resistance at an accident site nor the training to make a subjective judgment on the pavement condition. In some cases when slipperiness is suspected, a skid resistance value may be obtained from the State's files or the State highway and transportation department may dispatch a skid tester to take a measurement. In either case, the reported skid resistance may not be the same as prevailed at the time of the accident because skid resistance may change significantly in a short time. (*7*)

Rarely will two repeat measurements on any given pavement section result in the same SN. Some of this lack of repeatability can be attributed to measurement errors, but the large, observed variations in skid resistance are real and can be attributed to seasonal and short term changes in the pavement surface. (*8*) These short term changes are believed to follow rain and dry spells. The objectives of current research are to explain these seasonal and short term variations and to develop a method for adjusting skid resistance measurements made at any time to an SN corresponding to a particular time. (*9*) If large-scale, in-depth accident investigations were to be made, skid resistance estimates would be needed for every accident in which skidding is suspected to contribute. Current knowledge of the measurement and variations of skid resistance would assure reliable skid resistance data. Such accident investigations, however, would have to run over several years and would be very expensive.

Accidents rarely have a single cause, and it is often difficult to identify skidding accidents. For this reason accidents are frequently classified as dry and wet weather accidents, under the assumption that skidding may contribute to the latter. It is not easy, however, to determine whether a pavement is wet or dry. Is the pavement wet and slippery as soon as the rain starts? Clearly it is not dry immediately after the rain stops. Making this decision is especially difficult for the accident analyst who must use accident information from one source and rainfall information from another. (*10*) The conclusions will depend largely on the interpretation of the data. A recent report by the National Transportation Safety Board concluded: (*11*)

> Data developed by the Safety Board indicate that during 1976 and 1977, 13.5 percent of all fatal accidents occurred on wet pavement, while precipitation occurred only about 3.0 to 3.5 percent of the time nationwide. This indicates that fatal accidents on wet pavement occur 3.9 to 4.5 times more often than might be expected, and that the wet-pavement accident problem should be of concern to all States.

In this study wet time was based on minimum rainfall rates of 0.25 mm/hr (0.01 in/hr). Clearly a different criterion for wet time would have resulted in a different wet-to-dry road accident ratio.

Pavement Surface Characteristics

Because of the many difficulties in relating accident causes to low skid resistance or to wet pavements, other approaches for establishing the safety performance of wet pavements were tried. One such approach is to determine pavement friction demand for specific highway sites, based on traffic speed and kind of driving maneuver at these sites. Such requirements could be established by analysis, as in the design of horizontal highway curves. The design speed for a curve is based on the curvature, the superelevation, and the estimated coefficient of friction. Another method for establishing these requirements is by observing the driving speeds and patterns on typical highway sites. (*12, 13*) It is then possible to estimate the required skid resistance for safely accommodating a large percentile of drivers. This approach did not gain general acceptance because many highways with much less than the "required skid resistance" had below average accident occurrence. So far none of the methods for determining the frictional needs for specific pavement sites and driving maneuvers was completely successful.

Therefore the question could be asked: Can a single value, the SN, sufficiently characterize the safety performance of pavement surfaces? This question is not new and has been addressed before in different ways. One idea was to combine a number of factors, including skid resistance, into a "wet weather safety index" (WWSI), weighting each factor according to its contribution to safety. (*14*) Derivation of a reliable WWSI for any given site requires detailed input data for the site. In this respect it is similar to pavement friction demand, except that it presents a more systematic approach. In both cases the SN must be selected for the local travel speed.

It is well known that wet-pavement skid resistance generally decreases with increasing speed. The test speed is standardized at 65 km/h (40 mph). How then can skid resistance at any other speed be predicted? An obvious answer is to measure at the speed of interest, but this has practical drawbacks. Because skid testing is done in the regular flow of traffic, with slow moving vehicles in the traffic stream, it would be impossible to maintain a constant test speed. Alternately, from measurements at two different speeds it is possible to estimate the skid resistance-speed gradient and thus estimate the skid resistance at any other speed. But to conduct two measurements as a standard procedure is not practical because of the increased cost.

Experience has shown that the decrease of skid resistance with increasing speed depends on pavement texture and is smaller for coarser textures. Pavement texture is a random collection of asperities covering an enormous range of typical dimensions. This range was arbitrarily divided into microtexture and macrotexture. Initially, research concentrated on the effects of macrotexture, its relationship to skid resistance, how to provide adequate macrotexture on bituminous and on portland cement pavements, and how to measure macrotexture. (*15, 16*)

If macrotexture could be measured, the skid resistance-speed gradient could be predicted because coarse texture provides better drainage of the tire-pavement contact patch and thus reduces the loss of skid resistance experienced at higher speeds. Initial efforts to show a texture-gradient correlation failed. The introduction of the percent normalized gradient (PNG) concept, in which the gradient is normalized by the skid resistance at the same speed, resulted in high correlations with skid resistance-speed gradients. (*17–19*)

Meanwhile, research on texture is continuing, focusing primarily on the development of high-speed measurement methods and the quantification of all effects of texture such as skid resistance, noise, wear, and rolling resistance. (*20, 21*)

The presence and contribution of microtexture at the pavement surface were deduced from indirect measurements such as particle size of polishing agents, insolubles in the aggregate, microscopic observations, and scanning electron microscope studies. More recently, direct profiling of the microtexture has been accomplished.[3] From this and the macrotexture profile, a complete texture distribution for any surface can be obtained. The following ranges of interest tentatively have been defined:

- Microtexture—0.01 to 0.50 mm (0.0004 to 0.0200 in). Sharp microtexture breaks through thin water films and produces direct tire-pavement contacts.

- Macrotexture—0.50 to 10.00 mm (0.02 to 0.40 in). Provides drainage of the tire-pavement contact patch and is especially important when the tire treads are worn. Macrotexture also contributes to the friction force through hysteresis losses, however, this contribution has been shown to be small. (*22*)

Characterization of Pavement Surfaces

Because of the success in quantifying microtexture, efforts were made to express skid resistance in terms of the two texture parameters.[4] Initial results confirm that microtexture governs the low speed skid resistance, while macrotexture contributes the speed dependent term. Thus skid resistance now can be tentatively computed from two texture values. This, however, does not improve the relationship of skid resistance to safety or accidents.

[3] "Pavement Surface Texture—Significance and Measurement," Pennsylvania Transportation Institute, *Federal Highway Administration*, Washington, D.C. Report not yet published.

[4] Ibid.

A standard ribbed tire is used for skid testing, but the specifications require that it not be used when it is worn to a groove depth less than 4.2 mm (0.165 in). (*23*) Because this ribbed tire provides adequate drainage, it is not sensitive to the drainage capability of the pavement macrotexture. The skid resistance measured with this tire on dense graded (fine textured) pavements would not predict the low friction potential that such a pavement might have for a car with badly worn tires on a heavily wetted pavement. Bald tires on fine textured, wet pavements may lead to near hydroplaning conditions, although full, dynamic hydroplaning is highly unlikely. (*24*)

In a number of research studies bald tires as well as the standard ribbed tire were used for skid resistance measurements. A study in California on more than 20 concrete pavements showed that the two measurements were highly correlated (r=0.95). (*25*) A study in Virginia on 31 test sites, including bituminous and concrete pavements, compared bald and ribbed tire SN's by grouping the pavements by texture depth. (*26*) In some of these groups the SN's were almost identical, in others they differed significantly. (Similar findings were recently reported and are shown in figures 1 and 2.)[5] Both data sets in the figures are from the same 22 test sites. The 1979 data points in figure 1 cover a range of 49 SN's with the bald tire versus 44 with the ribbed tire. The bald tire skid numbers (SN_B) are consistently lower than the ribbed tire skid numbers (SN_R), but although no value is given, the correlation appears to be good.

[5] "Measurement and Evaluation of Pavement Surface Characteristics," by J. J. Henry. Paper presented on Project 1W at the Federally Coordinated Program Conference, Williamsburg, Va., December 1979.

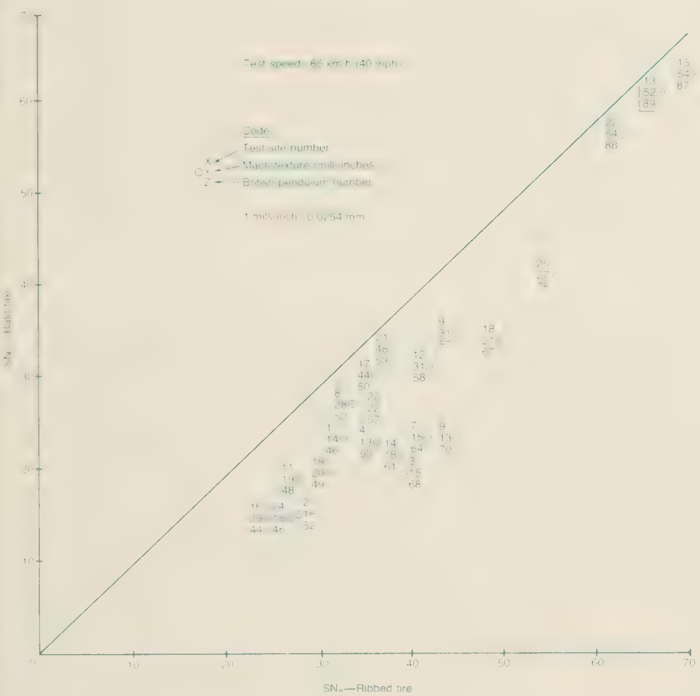

Figure 1.—Skid resistance of 22 test sites, spring 1979.

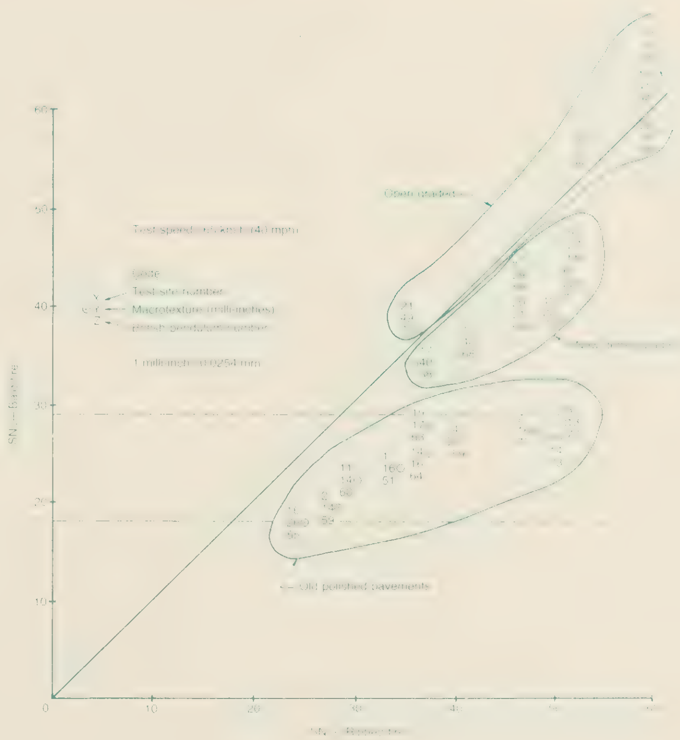

Figure 2.—Skid resistance of 22 test sites, fall 1978.

The data points in figure 2 cover a range of 48 SN_B versus only 35 SN_R. On a few pavements the bald tire gave higher values than the ribbed tire, but most SN_B values are lower than the corresponding SN_R values. The pavements seem to fall into three distinct groups. The open graded and new dense graded pavements have SN's above 35 with both tires and the slopes of the regression lines are about 45 degrees. The third pavement group, identified as old, polished pavements, ranges from 17 to 28 SN_B versus 25 to 49 SN_R, and the slope is much flatter.

If the range of skid numbers is a measure of sensitivity, then the bald tire is more sensitive than the ribbed tire over the full sample of pavements in this study but less sensitive to the old, polished pavement surfaces. Unless accident records or other evidence shows that all of these old, polished pavements are unsafe, use of bald tire data only, because of the lower rating, is not justified nor is use of ribbed tire data only, because it discriminates better within this pavement group.

There is no evidence from safety or accident experience to conclude that SN's measured with the bald tire are better indicators than those measured with the ribbed tire, or vice versa. But because the numbers differ for different pavement types, it can be concluded that the two test tires—ribbed and bald—have different sensitivities to the several pavement surface features.

Needed Research

If both sets of data are used, could one generate a reliable indicator of wet-pavement safety performance? To answer this question the following hypothesis is proposed and needs to be tested in a research study.

The friction potential (FP) of a wet-pavement surface is a function of many variables. It could be calculated by the following regression equation.

$$FP = f(SN_R, SN_B, MTX, mtx, PNG, TD, WD)$$

Where,

SN_R = the standard skid number with the ribbed tire.

SN_B = the skid number with the bald tire.

MTX = the macrotexture.

mtx = the microtexture.

PNG = the percent normalized gradient (= SNG/SN, both at the same speed).

SNG = the skid resistance-speed gradient (= dSN/dV).

TD = the tire tread depth.

WD = the effective water depth.

The next step is to simplify this equation. The pavement should serve safely all vehicles with tire treads above a certain minimum tread depth. Thus TD becomes a constant and can be eliminated from the equation. Further, for a given water depth in skid testing, good correlations have been achieved between macrotexture and the percent normalized gradient. Thus, PNG also is eliminated.

It is difficult to establish the effective water depth. It is a function of rainfall rate, road geometry, rut depth, and pavement macrotexture, which is already in the equation. Thus, a rainfall rate modified by road geometry and rut depth could be substituted for WD. On the other hand, a highway department could assume some design water depth that would represent the most unfavorable condition to be expected at some selected percentile of the time for a particular road. This would make WD another design constant. The friction potential then becomes a function g of four variables, modified by a constant A, which is to be selected for the chosen design values of TD and WD.

$$FP = Ag\,(SN_R, SN_B, MTX, mtx)$$

The remaining four variables are measurable pavement surface parameters. In the suggested research study multiple regression analyses will have to be made to determine the dependence of FP on the four variables in the second equation. These analyses are expected to show that the variables within the parentheses are not independent of each other, allowing the equation to be further simplified. Practical considerations, such as ease of measurement, may help in deciding which of these variables to retain.

Just defining a friction potential does not solve any problem. To establish the desired relationship between skid resistance and accidents, it will be necessary to provide a reliable data base, that is, a large number of pavements on which the four variables can be measured and on which the friction potential can be estimated independently. It is preferable that this estimate be based on accident data. A possible model would give the wet (pavement) accident rate (WAR) as an exponential function of the friction potential.

$$WAR = x(e^{-FP})$$

The most difficult part will be to relate the proposed friction potential to highway safety, that is, establish the value of the unknown x. Unless a good accident data base is developed, surrogate measures may be necessary. Observation and measurement of vehicle maneuvers in emergency situations may provide the required data base. This is not a simple undertaking, but it may be easier and less costly than a large-scale accident study.

Summary

The problems in relating pavement skid resistance to accidents have been reviewed in this article. The failure to agree on minimum skid resistance requirements is not because of a lack of effort. There is no doubt that slippery pavements cause skidding accidents, but the measure of safety is not necessarily a single skid number but rather a complex function of pavement surface characteristics.

However, it should not be interpreted that current understanding of the problem is insufficient for reducing the hazards of slippery roads. State highway and transportation departments have the know-how for providing skid resistant pavements. The locked wheel skid tester is a reliable tool and should continue to be used extensively to rank pavements for skid resistance improvements. A friction potential has been defined and a hypothetical relationship has been suggested in this article. It is hoped that these ideas will stimulate discussion and generate sufficient interest to test the hypothesis in future research studies.

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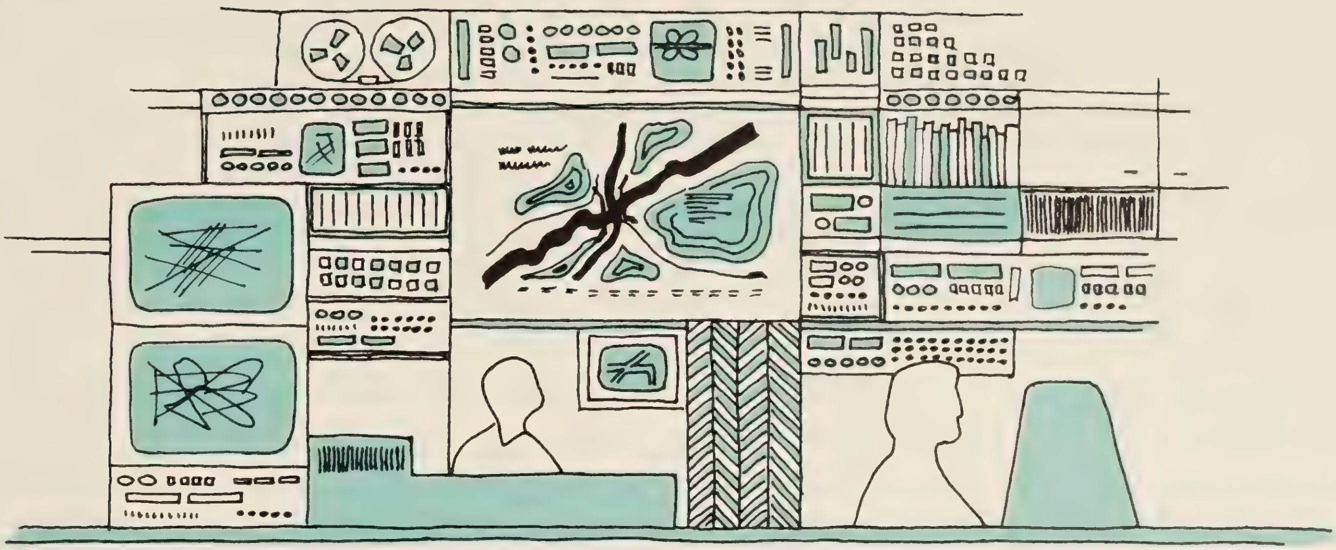


The Future of Traffic Simulation[1]

by
Paul Ross

Introduction

The Federal Highway Administration (FHWA) Office of Research has long been a leader in traffic simulation. NETSIM, probably the most widely used traffic simulation program in the world today, was originally developed under contract to FHWA and has been modified and extended by the Office of Research many times. Many other developments in traffic simulation originated in the Office of Research, where modification of old programs and development of new programs are continuing efforts. This article projects the future of traffic simulation—what features traffic simulation programs will have 20, 30, or even 50 years from now.

The statements in the article are necessarily hypothetical and subjective and will exclude ideas on traffic simulation that are currently being developed or planned. Pilot studies or preliminary research on nearly all of the features described indicates that many of these features will not be incorporated into publicly released traffic simulation programs within the next 5 years.

Trends in Computer Technology

The future of traffic itself will have the most important effect on the future of traffic simulation. Will there be smaller, larger, or even individual vehicles at all in the year 2000? The present conflicting trends in vehicular traffic are difficult to analyze. In this article it will be assumed that traffic will be a collection of individual vehicles and not radically different than it is now. To limit the scope of the discussion it is further assumed that no other changes—except perhaps in computers and computer technology—need to be predicted at this time. What, then, are the likely changes in computers?

[1] This article is based on a presentation at the Transportation Research Board Traffic Simulation Models Conference in Williamsburg, Va., June 3–5, 1981. The ideas presented here are solely those of the author and do not represent the opinion of the Federal Highway Administration (FHWA). In particular the reader is cautioned not to draw any conclusions as to areas of research that FHWA will or will not undertake based on this discussion.

1. Mainframe computers will become bigger, faster, and more expensive. This is a simple extrapolation of the well-known current trend. The price per calculation will continue to decrease, although not as rapidly as it has recently. Because staff costs are essentially negligible in computer calculations, the cost of an individual calculation will remain insensitive to labor costs and will not rise in terms of real dollars. However, because the capital cost of individual mainframe computers will increase as their computing power increases, fewer organizations will be able to afford the most powerful computers.

2. Small computers will become more powerful, and powerful computers will become smaller. Already there are computers with full abilities that are small enough to carry in a briefcase, although they are not yet pocket sized. Offices will replace typewriters with less expensive word processors—very much in the way that pocket calculators have replaced the old mechanical desk calculators. These word processors will have the computational capability to run traffic simulation programs. It may seem these devices might be better thought of as computers that also do word processing, but they are more likely to be identified as office equipment than as laboratory equipment. The more deluxe word processors on the market already can do mathematical calculations, and soon the cost of adding programable scientific calculations will be just a few dollars—the cost of a silicon chip.

3. Public time-sharing services will excel at providing service to small- or medium-sized organizations whose computing requirements fluctuate widely. Clearly it will not be cost effective for a small consulting company to purchase its own computer if it needs negligible computing time except for running a large simulation once a week that requires 1 or 2 hours and a few megawords of random access storage. Such a company would have to turn to a public computer sharing service. It will be in the interest of such a service to use the most powerful computer available to service as many customers as possible simultaneously. Extraordinary demands will probably be satisfied on a batch mode basis.

Organizations with reasonably constant need for computing power will tend to buy their own computer because they will be able to choose one that best meets their individual requirements. These organizations will probably provide real-time operation on small computers or time sharing among several individuals on medium-sized computers simply because human time will be worth more than computer time. Batch mode processing at night or on weekends may be required for large jobs within such an operation.

4. Every computer powerful enough to run a traffic simulation program will have some form of graphic output; graphical devices probably will be cheaper than hard-copy printers. Liquid crystal matrix displays can be made without the complicated moving parts inherent in hard-copy printers. When cathode ray tube or liquid crystal displays become common there will be no reason to restrict the outputs to alphanumeric characters, and most output displays will have full graphic capabilities.

Future Developments in Traffic Simulation

How will the above trends in computer use affect traffic simulation? Following are speculations arranged with the most certain and immediate prospects first and the most speculative and remote ideas last. Indeed, experimental versions of the first three ideas already are in use, but a traffic simulation program with these features is not currently scheduled for development or release.

Simulation programs will make greater use of graphic displays because virtually all computer terminals will have graphic devices as their normal form of output. Incorporating graphics into FHWA simulation programs has been delayed because there is no common graphic language. The language used for computer graphics depends on the make of the terminal; graphic languages are not like FORTRAN, which can be executed on virtually any brand of computer.

At first, simulation outputs will show queue lengths at all the intersections, average number of stops, total delay, and various other forms of output information. Such displays will allow the user to grasp the overall operation of a network more quickly and meaningfully than is possible with present computer printouts.

Graphical output will be followed by graphical displays of the simulation program in operation. Animated operating displays, thought to be a good public relations tool, are certainly useful for explaining the simulation program to non-experts, but they are even more useful to the practitioners themselves. There is no more certain way to find mistakes in the input than to look at how the computer represents the system operation. Left turns coded as right turns or mistakes in signal phasing stand out immediately. Full animation technology has been available for only a few years and is very expensive. However, it is certain to become less expensive and more readily available over the next 20 years.

Traffic simulation programs usually will be interactive. Operators will be able to interrupt the programs during execution and change various parameters. This interactive ability will be a natural outgrowth of the widespread use of graphic terminals. Although it is possible to run a program in batch mode and then later look at the outputs generated by the computer, this is not a convenient or natural way to use computer graphics. With graphic displays it is common to have the computer instruct the display device to draw some complicated picture and then await confirmation that the picture was indeed drawn before proceeding with the next calculation. Consequently, it is a very small step to allow the operator to interrupt and change the program, because the computer usually is waiting for a response from the terminal anyway.

With time-sharing option or dedicated operation on a small computer at least some interactive computing seems inescapable. At a minimum, the program will analyze the input data and inform the operator of obvious errors before the operator leaves the terminal. A simple program could operate this way, but it seems likely that programs soon will ask the operator for input data in plain English and analyze it item by item for obvious errors and consistency with previous data. The operator will be informed of problems before his or her attention has moved on to the next data item.

Until graphic devices become common, this may be all the interactive capability that will be useful. But once the operator can see at a glance from some animated operating display how the entire network is operating, he or she will want to be able to control that operation. Adding interactive abilities during program execution will be natural.

The interactive and graphic display features will lead to online simulation, a service provided to operators of computer traffic control systems. With this feature the operator will, at the touch of a button, start an interactive graphic simulation running. The program will start with initial conditions that are identical to those that are in the real network at the time the button is pushed. If the program runs four or five times as fast as real time, the operator will be able to foresee events in the actual network and possibly test alternative strategies.

Such ability would be useful in many cases; for example, a situation in which an accident completely closes a network link. Even if the control algorithm is able to provide an appropriate response to such a traffic situation, it will be useful to foresee how the traffic disturbances will propagate so that police can be dispatched appropriately and perhaps news media notified of impending congestion at critical locations.

The ultimate stage in online simulation will be a program that runs continuously and checks itself against the real traffic so that the simulation program can adjust itself for changes in the vehicle mix and driver behavior without human intervention.

As users of NETSIM and other microscopic simulation programs know, input preparation and data collection are inordinately tedious and expensive. There is a real need for automatic input—providing accurate geometric data (such as link lengths, grades, and corner radii) and traffic data (volumes, turning movements, and traffic composition) with little or no human intervention.

Aerial photographs projected onto a digitizing tablet would be quite useful for automatic input. Link geometry could be entered quite accurately just by touching origin and destination nodes. Corner radii could be entered if needed. A single aerial photograph is not much use in estimating volumes, but the simulation programs easily could be written to use density (vehicles/kilometre [vehicles/mile]) instead. Input that starts from cars at specific locations throughout the network would have the additional advantage that no initialization period would be needed before the simulation results were valid. A majority of the input now required for simulation programs could be entered just by touching points on a digitizing tablet. Although this would require substantially different forms of data input processing, the basic operation principles of these programs would not be affected. The technology to do all this is available now.

An operator must point out the nodes, cars, trucks, and corner radii to the computer in the above method. However, technology could be adapted to a completely automatic input system. SCAN —Sensor for Control of Arterials and Networks—is an experimental television-based detector system in which the computer identifies the images of moving vehicles and tracks them over space and time. This technology could be adapted so that aerial motion pictures could be analyzed, and virtually all the simulation input could be assimilated into the computer without human intervention. The SCAN detector could pick up the network geometry, volumes, and turning movements automatically. In effect, all one would have to do would be to show the computer a movie of the network operation, and the computer would be able to simulate the entire operation.

Finally, all these features will be integrated to produce fully microscopic citywide simulations. One will be able to show the computer an aerial motion picture and the computer will identify all the fixed objects and all the moving objects. The computer will be able to classify the moving objects automatically as small, medium, or large automobiles, trucks, transit buses, schoolbuses, or fixed rail transit. It will even, if so ordered, identify all the pedestrians. After identifying these objects it will deduce the origin-destination table for each class of moving object. The program will deduce the acceleration/deceleration curves for each object class and the distribution of headways for "followers."

In short, the computer program will be able to assimilate all the information that it needs to run a complete simulation of everything that moves in a whole city—all the statistical distributions and all the adjustable parameters. It will measure these quantities and not merely assume characteristics measured in some other city.

The graphic output from this citywide simulation will be extremely lifelike. Computer techniques already exist to identify and manipulate elements of pictures while maintaining photographic realism. The simulation program will identify the fixed background photograph and continuously maintain it on the output display. The simulation program also will have identified which photographic elements represent cars, buses, trucks, trains, and pedestrians. The output will have these photographic elements superimposed on the fixed background picture and moving in lifelike ways. When the simulation program generates a new vehicle, it will represent the vehicle in the output with a photographic element chosen at random from those photographic elements identified as being members of the same vehicle class.

None of this is particularly visionary. The techniques to accomplish all of these things already exist, although in cumbersome and expensive experimental forms. As long as there are university, industrial, and government research programs there is no doubt that the techniques will continue to be researched. Whether or not any of these techniques become common depends on government regulation, economic climate, and public concern. However, some items will not become popular although they are known to be feasible. There will never be enough market demand to make them viable products.

A traffic simulation language probably will never become popular. General simulation languages such as SIMSCRIPT and Q-GERT serve a real need for those who have to simulate unique operations, but there are not enough people working in traffic analysis to support a specific traffic simulation language. Moreover, traffic situations do not vary so much that they all cannot be encompassed in a single program. It is hard to define the dividing line between a program and a language. A very general and flexible program could be regarded as a language by some. Certainly traffic simulations will become more general and flexible, but the effort to keep them easy to use will maintain their identity as programs and not languages.

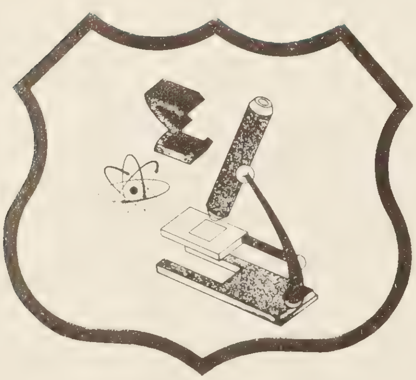
Finally, the best simulation programs will not incorporate optimization. Of course, the optimization programs that are already in use do incorporate some form of simulation or evaluation. However, there are reasons they are not the most accurate forms of evaluation.

The most efficient forms of mathematical programing, such as linear programing, require that the system model have certain mathematical simplifications. Linear programing, for example, requires that the model be piecewise linear and the region of feasible solutions be convex. Other techniques require other restrictions. The optimization methods that use only the model output and make no assumptions about the form of the model ("hill climbing" or other gradient methods) are inherently inefficient and cannot guarantee a global optimum. A microscopic simulation program cannot be used with an efficient and accurate optimization technique, making it useless to use the best evaluation models. Equal accuracy can be achieved by using simple, but good, models and quick and accurate optimizations. However, future signal optimizations probably will be done automatically online.

Paul Ross is a highway research engineer in the Traffic Systems Division, Office of Research, Federal Highway Administration. His fields of interest include traffic flow theory and simulation. Dr. Ross has published other research in traffic theory, simulation, and truck operations. He is a member of the Institute of Transportation Engineers, Operations Research Society of America, and the Transportation Research Board.

Recent Research Reports You Should Know About

The following are brief descriptions of selected reports recently published by the Office of Research, Federal Highway Administration, which includes the Structures and Applied Mechanics Division, Materials Division, Traffic Systems Division, and Environmental Division. The reports are available from the address noted at the end of each description.



Laboratory Evaluation of Sulphlex–233: Binder Properties and Mix Design, Report No. FHWA/RD–80/146

by FHWA Materials Division

FHWA's Office of Research conducted a study on the mix design, physiochemical behavior, and safe handling procedures of Sulphlex–233 binder material that is being used for experimental construction.

The Marshall stability and flow criteria are satisfied for Sulphlex–233 mixtures that have binder contents with essentially equal volume and equal weight replacement of asphalt; however, the air void criterion is satisfied only at the equal volume replacement value. With diabase and limestone aggregates, Sulphlex–233 mixtures were extremely sensitive to water damage. Immersion-compression specimens containing diabase aggregate disintegrated during immersion at 60° C (140° F). Similar specimens containing limestone aggregate maintained their integrity at 60° C (140° F) but showed a severe strength loss exceeding 90 percent. Anti-strip agents, such as tall oil pitch, commonly used in asphalt mixtures will reduce significantly this sensitivity to water damage.

Bulk Sulphlex–233 binder stored at ambient temperature loses penetration with time. The loss is more severe for material treated with anti-strip agents. Sulphlex–233 is soluble in bromoform and 1,1,2,2,-tetrabromoethane. A minimum retained penetration value of 50 is recommended for Sulphlex–233 after the thin-film aging test. Future development of Sulphlex formulations will strive for softer binders to yield mixtures with lower stiffnesses.

The report is available from the Materials Division, HRS–23, Federal Highway Administration, Washington, D.C. 20590.

Development of a Rapid-Set Epoxy Adhesive for Traffic Markers, Report No. FHWA/RD–80/194

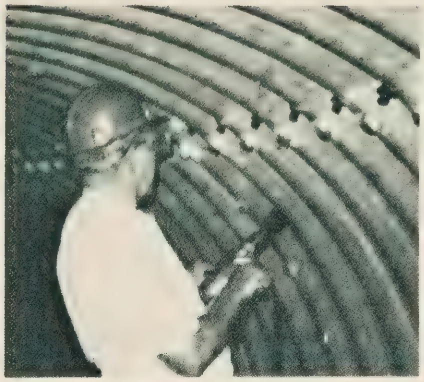
by FHWA Materials Division

A new generically defined, rapid-set epoxy adhesive for bonding traffic markers to roadway surfaces has been developed, field tested, and produced in semicommercial quantities. Adhesive 118–AF has significantly faster setting times and lower viscosity than current rapid-set materials. It is a two-component system designed for machine mixing and dispensing on a one-to-one volume basis. It has a pot life of about 4 minutes at 25° C (77° F), can be used at lower temperatures than conventional materials, can tolerate some dampness in the concrete, and is free of asbestos as a thixotropic agent.

Adhesive 118–AF uses a polymercaptan as the hardener. It differs from existing rapid-setting systems in that all of the cure results from the fast reaction of a mercaptan group with an epoxy group.

The asbestos version, Adhesive 118, was compared in a field test with the California rapid-set adhesive and a second experimental adhesive. Raised ceramic and plastic retroreflective markers were placed on a portland cement concrete highway with high traffic density. Adhesive 118 had the best overall handling, cure rate, and marker retention rate during the first year of service. After reformulation, in which the asbestos thixotrope was replaced with fibrillated high-density polyethylene, production was increased and 1 325 L (350 gal) of satisfactory adhesive was produced for evaluation by various State highway agencies.

The report is available from the Materials Division, HRS–23, Federal Highway Administration, Washington, D.C. 20590. In conjunction with this report, a materials and performance specification and users guide, Report No. FHWA/RD–80/032, has been prepared and also is available from the Materials Division.



Evaluation of Highway Culvert Coating Performance, Report No. FHWA/RD–80/059

by FHWA Structures and Applied Mechanics Division

The life expectancy of rigid and flexible pipe culvert in highway drainage installations is determined partly by the ability of the pipe material to resist deterioration from corrosive and abrasive elements in the culvert environment. Measures to extend culvert service life include coatings, linings, cathodic protection, increased metal thickness, and careful material selection. This report presents the results of research to evaluate the performance of various coatings for highway culverts and recommended methods of extending culvert service life.

Technical literature was reviewed, interviews were conducted with cognizant State highway personnel throughout the United States, and culverts were field inspected at 82 locations in 9 States. It was determined that although culvert coatings generally are effective where runoff does not include abrasive debris or high percentages of soluble salts, any pipe culvert material or coating may have reduced durability in a particular adverse situation.

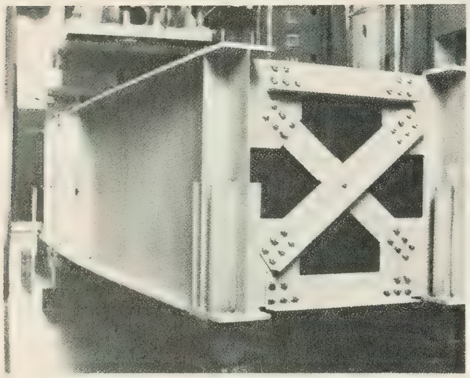
Coatings used on culverts are either metallic or barrier. Metallic coatings provide a corrosion- and abrasion-resistant surface and cathodic protection for the base metal by acting as the sacrificial anode in a galvanic cell. Barrier coatings prevent direct contact between the metal or concrete pipe material to be protected and a hostile pipe environment. Culvert invert protection against abrasion and corrosion also can be provided by asphalt paving, vitrified clay linings, or fiberglass linings.

Protective features evaluated at various culvert installations included asphalt, asbestos-bonded asphalt, asphalt paving, coal tar laminate, polyethylene, polyvinyl chloride, vinyl plastisol, aluminized steel, aluminum-zinc, and epoxy-coated concrete. Other available materials that may be used to extend pipe culvert service life include urethanes, neoprene, fusion-bonded coatings, ceramics, and metallized coatings.

Structural aids that may increase the service life of pipe culverts in abrasive conditions include the use of runoff stilling basins and energy dissipators. Use of greater pipe wall thicknesses to compensate for corrosion and abrasion losses is another method of increasing the life expectancy of culverts if the degree of expected corrosion and abrasion can be determined for the site involved. Severe corrosive or abrasive conditions can limit the applicability of this method.

The report presents a proposed evaluation program and recommendations for possible improvements of existing culvert specifications.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 81 115461).



Fatigue of Curved Steel Bridge Elements—Design Recommendations for Fatigue of Curved Plate Girder and Box Girder Bridges, Report No. FHWA-RD-79-138

by FHWA Structures and Applied Mechanics Division

Horizontally curved steel plate girder and box girder highway bridges are esthetic and economical to construct. This report presents fatigue design recommendations for horizontally curved welded steel highway bridge members. These recommendations were developed from findings described in seven related reports, each of which covers a specific aspect of the research.

In bridges with horizontally curved members, using lateral bracing between curved plate girders or internal bracing and stiffening of curved box girders to resist torsion may result in structural connection details that can be highly sensitive to fatigue under repetitive loading. The laboratory investigation of large-scale curved specimens included the following:

- Analysis, design, and fatigue testing (with 2,000,000 cycles of load application) of five horizontally curved steel plate girder assemblies and three horizontally curved steel box girders.

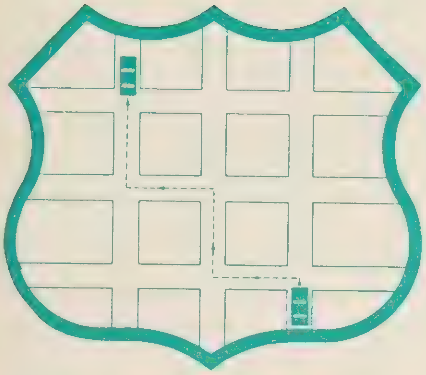
- Analytical studies of the influence of various design and construction parameters on fatigue life.

- Loading to failure (after fatigue testing) of three of the curved plate girder assemblies and two of the curved box girders (composite reinforced concrete slabs were cast on two of the three curved plate girder assemblies and on both curved box girders).

- The development of design recommendations suitable for inclusion in the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges.

The curved plate girder assemblies and the curved box girders contained 142 weld details, which provided 284 possible fatigue crack locations. Fatigue cracks observed at 107 of these locations included 61 through-thickness cracks. The fatigue resistance at 48 of these 61 locations was equal to or greater than that stipulated for the respective categories of straight girders in the current AASHTO Specifications. In each of the 13 other locations, a lower than specified fatigue resistance was attributed to unsatisfactory weld quality, steel flaws, or stress and deflection conditions that were not considered in design. Analysis of the fatigue test results showed that the welded steel fatigue design provisions for straight girders in the current AASHTO Bridge Specifications are satisfactory for designing horizontally curved welded steel plate girders and box girders as long as stresses are evaluated adequately for the curved structures.

The report is available from the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 81 115115). Other reports in this series also available from NTIS include the following: Analysis and Design of Plate Girder and Box Girder Test Assemblies, Report No. FHWA-RD-79-131 (PB 81 116899); Stress Concentration, Stress Range Gradient, and Principal Stress Effects on Fatigue Life, Report No. FHWA-RD-79-132 (PB 80 187495); Fatigue Tests of Curved Plate Girder Assemblies, Report No. FHWA-RD-79-133 (PB 80 187503); Fatigue Tests of Curved Box Girders, Report No. FHWA-RD-79-134 (PB 80 187511); Effect of Heat Curving on the Fatigue Strength of Plate Girders, Report No. FHWA-RD-79-135 (PB 81 116907); Effect of Internal Diaphragms on the Fatigue Strength of Curved Box Girders, Report No. FHWA-RD-79-136 (PB 81 116915); and Ultimate Strength Tests of Curved Plate and Box Girders, Report No. FHWA-RD-79-137 (PB 81 116923).



Estimation of an Origin-Destination Trip Table Based on Observed Link Volumes and Turning Movements, Report Nos. FHWA/RD-80/034-037

by FHWA Traffic Systems Division

The LINKOD system estimates a trip table based on observed link volumes and turning movements. This trip table can be used as an input to traffic assignment and simulation models. LINKOD is designed for small area analysis where all link volumes are known.

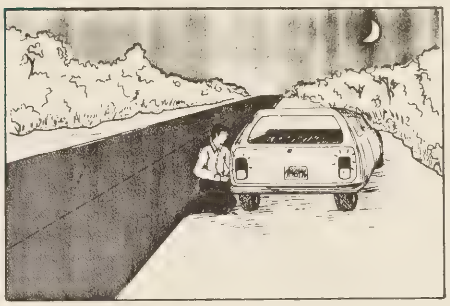
The LINKOD system is documented in four volumes. Volume 1, **Technical Report,** presents a description of the theories used in the model and a description of test problems that were run. Volume 2, **Users Manual,** describes how to use the program, and Volume 3, **Program Manual,** contains the subroutine documentation. The fourth volume is the **Executive Summary.**

Limited copies of the reports are available from the Traffic Systems Division, HRS-31, Federal Highway Administration, Washington, D.C. 20590.

Safety Aspects of Using Vehicle Hazard Warning Lights, Report No. FHWA/RD-80/102

by FHWA Traffic Systems Division

This report describes a series of experiments that was conducted to examine the effectiveness of four-way flashers for both disabled vehicles and slow-moving vehicles. Experiments were performed at two- and four-lane locations both during the day and at night.



The slow-moving vehicle tests involved a staged slow-moving car or tractor-trailer that entered the traffic stream. The overtaking vehicles were observed. The disabled vehicle tests evaluated red and amber four-way flashers as well as flares and reflectorized triangles. Motorists approached the slow-moving and disabled vehicles more cautiously (slowed down sooner) and passed more carefully when four-way flashers were on.

Limited copies of the report are available from the Traffic Systems Division, HRS-33, Federal Highway Administration, Washington, D.C. 20590.

Analyses, Experimental Studies, and Evaluations of Control Measures for Air Flow and Air Quality On and Near Highways, Volumes I-IV, Report Nos. FHWA/RD-81/051-054

by FHWA Environmental Division

These reports describe the effects of vehicular exhaust emissions, highway geometric factors, and traffic flow conditions on air quality near uniform highways for quasi-steady traffic. Findings are based on analyses, extensive measurements at three highway sites, and wind tunnel tests validated by field tests.

Volume I, **Experimental Studies, Analyses, and Model Development,** discusses the problem rationale, approaches to evaluations and measurements for three highway sites, and observations of hundreds of wind tunnel tests. Heavy reliance was placed on tracer gases. A partial analysis of experimental results is presented. An empirical dispersion model, ROADMAP, was developed and verified.

Volume II, **User Guidelines and Application Notes for Estimating Air Quality for Alternative Roadway Configurations,** discusses basic aspects of air pollution dispersion including related windspeed, direction and turbulence, site geometry, and

traffic effects. An assessment methodology presented includes vehicle emission estimates, ROADMAP and other model uses, and appropriate applications of measured results for various highway conditions. Examples of roadway air pollution patterns related to road geometry, traffic, and wind conditions are discussed. Guidance for air quality management by roadway location, design, and operation is given.

Volume III, **Executive Summary,** briefly indicates the study objectives and scope, experimental program and results, and significant findings more completely discussed in Volumes I and II.

Volume IV, **Users Manual for FHWA Data Base and Retrieval Programs,** provides a summary of the experimental data and pertinent information for these data. Directions and examples are given for retrieval of the experimental data and related parameters from six computer magnetic tapes that store the actual data. The data base has been formulated to aid the understanding of the interactive roles of roadway site conditions, traffic, and meteorological effects.

Limited copies of the reports are available from the Environmental Division, HRS–42, Federal Highway Administration, Washington, D.C. 20590.

Constituents of Highway Runoff, Volumes I–VI, Report Nos. FHWA/RD–81/042–047

by FHWA Environmental Division

The growing awareness of the threat to the environment from highway runoff has resulted in research to identify and quantify the effects and develop protection measures. FHWA is identifying the sources of these pollutants and migration paths from the highway to the receiving waters, analyzing the effects of these pollutants in receiving waters, and developing the necessary abatement/treatment methodology for objectionable constituents.

Results are documented in the following volumes which are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161.
Volume I—State of the Art Report (Stock No. PB 81 241895)
Volume II—Procedural Manual for Monitoring of Highway Runoff Characteristics (Stock No. PB 81 241901)
Volume III—Predictive Procedure for Determining Pollutant Characteristics in Highway Runoff (Stock No. PB 81 241911)
Volume IV—Characteristics of Runoff From Operating Highways, Research Report (Stock No. PB 81 241929)
Volume V—Highway Runoff Data Storage Program and Computer Users Manual (Stock No. PB 81 241937)
Volume VI—Executive Summary (Stock No. PB 81 241945)

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

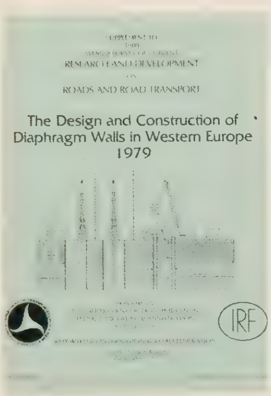
The following are brief descriptions of selected items that have been recently completed by State and Federal highway units in cooperation with the Implementation Division, Office of Development, Federal Highway Administration (FHWA). Some items by others are included when they have a special interest to highway agencies.

U.S. Department of Transportation
Federal Highway Administration
Office of Development
Implementation Division (HDV–20)
Washington, D.C. 20590

Items available from the Implementation Division can be obtained by including a self-addressed mailing label with the request.

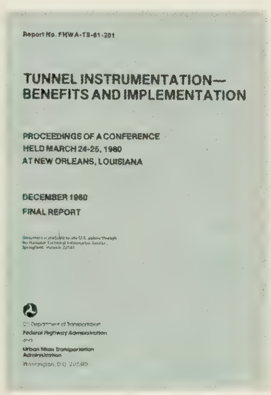
The Design and Construction of Diaphragm Walls in Western Europe, 1979

by FHWA Implementation Division and the International Road Federation



Diaphragm or slurry trench walls have been used in the United States since the early 1960's but have been used much more extensively and for a wider range of applications in Western Europe. Current practice dealing with the design, contracting, and construction of slurry trench walls in Western Europe was reviewed. The review involved discussions with designers, contractors, government personnel, and university professors; inspection of the sites of diaphragm walls that have been or are being built in several Western European countries; and evaluations of 26 projects in 5 countries. This report summarizes the findings of the review. Each of the 26 projects is described and illustrations are included.

Limited copies of the report are available from the Implementation Division.



Tunnel Instrumentation—Benefits and Implementation, Report No. FHWA–TS–81–201

by FHWA Implementation Division

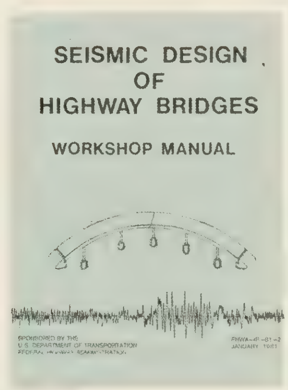
This report includes papers and edited versions of presentations and discussion sessions from the March 1980 seminar on the benefits of using instrumentation in the planning, design, construction, and performance of tunnels. FHWA and the Urban Mass Transportation Administration (UMTA) sponsored the seminar held in New Orleans, La. The seminar was directed toward transportation agency decisionmakers who are responsible for the engineering and construction of tunnels.

The reasons for instrumentation, methods and equipment for an effective instrumentation program, and selected case histories are discussed.

Limited copies of the report are available from the Implementation Division.

Seismic Design of Highway Bridges, Implementation Package 81–2

by FHWA Implementation Division



Highway bridges must continue to function following an earthquake when protection of lives and property depends on the efficient movement of emergency traffic. This requires that bridges subjected to earthquakes maintain both structural integrity and accessibility. Critical areas in the seismic design process should be identified and improvements implemented into the design as quickly as possible. This report presents an introduction and overview of the basic principles and tools needed to achieve this goal.

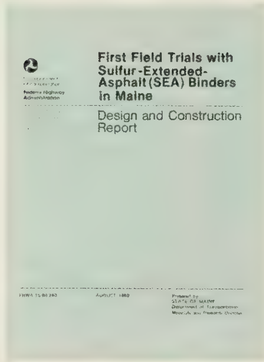
Limited copies of the report are available from the Implementation Division.

Using a Dryer-Drum in the Construction of Sulfur-Extended-Asphalt (SEA) Pavements, Design and Construction Report, Report No. FHWA–TS–80–243

by FHWA Implementation Division

This report describes the procedures and testing used during the design and construction of an SEA trial section on Interstate 10 in Arizona. A dryer-drum plant was used to mix the SEA binders and aggregates to construct a wearing course. The SEA binder was formulated by using a static in-line blender to mix the sulfur and asphalt before introducing the material to the dryer-drum. Standard aggregate gradations were used, and the sulfur/asphalt weight ratio was 30/70.

Limited copies of the report are available from the Implementation Division.



First Field Trials With Sulfur-Extended-Asphalt (SEA) Binders in Maine, Design and Construction Report, Report No. FHWA–TS–80–240

by FHWA Implementation Division

This report describes the procedures and testing used during the design and construction of SEA trial sections at two sites in Maine. A full depth trial is being conducted at the Interstate 95 site. A "thin" overlay trial is being conducted at the Route 2 site. The sulfur/asphalt weight ratio at both sites was 30/70, and the SEA binder was formulated by separately adding sulfur and asphalt to a batch plant weigh bucket and then mixing in the pugmill with aggregate (direct mixing). Softer asphalt was used in the SEA trial sections than in the asphalt control sections.

Limited copies of the report are available from the Implementation Division.

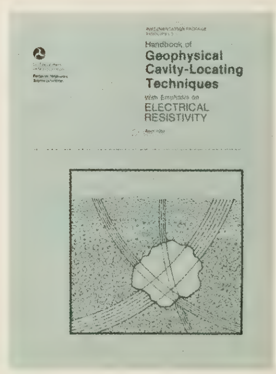
Handbook of Geophysical Cavity-Locating Techniques With Emphasis on Electrical Resistivity, Implementation Package 81–3

by FHWA Implementation Division

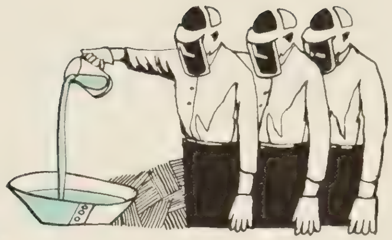
The accurate location of subsurface cavities is a problem that has faced engineers and planners for years. Highway subsidence, foundation collapse, and dam leakage are only a few of the often catastrophic problems associated with subsurface cavities. Billions of dollars have been wasted in attempting to salvage projects damaged by structural instability from underlying or adjacent cavities. For this reason, it is imperative that a rapid and efficient method of locating and designing for cavities and fractures be made available to highway engineers.

Although no method to locate all cavities under all conditions currently exists, several methods have been highly successful. This handbook presents various geophysical techniques for subsurface exploration and a means to tailor an exploration program from these techniques to meet specific project needs. Several promising geophysical techniques —micro-gravity, high-resolution seismic reflection profiling, and ground-probing radar—are examined briefly, as are aerial techniques. Three chapters of the handbook are devoted to resistivity techniques. The first chapter explains general principles of electrical resistivity methods, the second chapter examines alternative methods for conducting field surveys to obtain data, and the third chapter illustrates several data reduction and interpretation methods. Computer program listings for batch mode and hand-calculator operation also are provided.

Limited copies of the handbook are available from the Implementation Division.



New Research in Progress



The following items identify new research studies that have been reported by FHWA's Offices of Research and Development. Space limitation precludes publishing a complete list. These studies are sponsored in whole or in part with Federal highway funds. For further details, please contact the following: Staff and Contract Research—Editor; Highway Planning and Research (HP&R)—Performing State Highway or Transportation Department; National Cooperative Highway Research Program (NCHRP)—Program Director, National Cooperative Highway Research Program, Transportation Research Board, 2101 Constitution Avenue, NW., Washington, D.C. 20418.

FCP Category 1—Improved Highway Design and Operation for Safety

FCP Project 1N: Safety of Bicyclists, Moped Operators, and Pedestrians

Title: Pedestrian Risk Exposure Measures. (FCP No. 31N1012)
Objective: Identify pedestrian characteristics and behaviors. Develop exposure measures and apply to accident data to determine the relative hazardousness of particular pedestrian activities.
Performing Organization: Biotechnology, Inc., Falls Church, Va. 22042
Expected Completion Date: December 1982
Estimated Cost: $348,000 (FHWA Administrative Contract)

FCP Project 1O: Railroad-Highway Grade Crossings

Title: Alternative Ways to Improve the Visibility of Railroad-Highway Crossing Signals. (FCP No. 31O1184)
Objective: Analyze ways to improve crossing light dispersion and intensity. Determine hardware changes needed to improve signal visibility. Build and test a prototype of the proposed changes.
Performing Organization: Allard, Inc., Ellsworth, Kans. 67439
Expected Completion Date: September 1982
Estimated Cost: $240,000 (FHWA Administrative Contract)

FCP Project 1T: Advanced Vehicle Protection Systems

Title: Test Film Analysis and Video Tape Retrieval Library. (FCP No. 31T5094)
Objective: Catalog available test films and records, incorporating them into a database. Construct search capability compatible with database. Consolidate existing film library and convert to video tape. Develop video tape reviewing and editing laboratory.
Performing Organization: Native American Consultants, Washington, D.C. 20417
Expected Completion Date: July 1982
Estimated Cost: $277,000 (FHWA Administrative Contract)

FCP Category 2—Reduce Congestion and Improve Energy Efficiency

FCP Project 2J: Practicality of Automated Highway Systems

Title: Studies in Passive Guideway Automatic Vehicle Control. (FCP No. 32J1082)
Objective: Develop, test, and evaluate technologies required for automatic control of vehicles using passive guideways. Focus on the use of radar for lateral control.
Performing Organization: Ohio State University, Columbus, Ohio 43210
Expected Completion Date: May 1983
Estimated Cost: $542,000 (FHWA Administrative Contract)

FCP Category 3—Environmental Considerations in Highway Design, Location, Construction, and Operation

FCP Project 3F: Pollution Reduction and Environmental Enhancement

Title: Drip Irrigation Study. (FCP No. 43F1112)
Objective: Determine the water quality parameters that are important in drip irrigation to prevent clogging, bacteria growth, and mineral deposits and to improve maintenance.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95807
Expected Completion Date: June 1983
Estimated Cost: $169,000 (HP&R)

FCP Category 4—Improved Materials Utilization and Durability

FCP Project 4J: Coating Systems for Controlling Corrosion of Highway Structural Steel

Title: Improved Field Reliability of High-Performance Coating Systems. (FCP No. 34J2103)
Objective: Establish techniques, procedures, and guidelines to improve the reliability under field conditions of high-performance coating systems for highway bridges. Review field and laboratory tests and analyze application procedures and contracting practices. Investigate causes of failure and establish guidelines.
Performing Organization: Georgia Tech Research Institute, Atlanta, Ga. 30332
Expected Completion Date: May 1982
Estimated Cost: $121,000 (FHWA Administrative Contract)

FCP Project 4K: Cost Effective Rigid Concrete Construction and Rehabilitation in Adverse Environments

Title: Cathodic Protection of Bridge Substructures. (FCP No. 44K3602)
Objective: Develop cathodic protection systems that will stop or greatly reduce steel corrosion induced damage on piling, piers, backwalls, beams, and other concrete members. Test in the laboratory and field.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95814
Expected Completion Date: September 1984
Estimated Cost: $400,000 (HP&R)

FCP Category 5—Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

FCP Project 5B: Tunneling Technology for Future Highways

Title: Algorithm Research for Site Exploration Evaluation. (FCP No. 35B2192)
Objective: Develop and assimilate mathematical models (algorithms) for processing, analyzing, and displaying geotechnical data obtained by newly developing seismic, acoustic, electromagnetic, and resistivity probes for ground exploration.
Performing Organization: Southwest Research Institute, San Antonio, Tex. 78284
Expected Completion Date: January 1984
Estimated Cost: $481,000 (FHWA Administrative Contract)

FCP Project 5C: New Methodology for Flexible Pavement Design

Title: Evaluation of Illinois Flexible Pavement Design Procedures. (FCP No. 45C2112)
Objective: Use ILLI-PAVE and appropriate material testing procedures to design flexible pavements and develop recommendations for modifying the present flexible pavement design procedures.
Performing Organization: University of Illinois, Urbana, Ill. 61801
Funding Agency: Illinois Department of Transportation
Expected Completion Date: December 1983
Estimated Cost: $170,000 (HP&R)

FCP Project 5D: Structural Rehabilitation of Pavement Systems

Title: Pavement Rehabilitation By Means Other Than Overlay. (FCP No. 35D2704)
Objective: Investigate various means of restoring load transfer in jointed pavement and equipment and construction procedures necessary to install load transfer devices.
Performing Organization: Georgia Department of Transportation, Forest Park, Ga. 30050
Expected Completion Date: January 1983
Estimated Cost: $200,000 (FHWA Administrative Contract)

Title: Subsurface Drainage Materials. (FCP No. 45D3382)
Objective: Analyze materials used in drainage collector systems as they affect outflow. Conduct a detailed hydrothermal analysis of the collector system.
Performing Organization: University of Illinois, Urbana, Ill. 61801
Funding Agency: Illinois Department of Transportation
Expected Completion Date: January 1984
Estimated Cost: $138,000 (HP&R)

FCP Project 5K: New Bridge Design Concepts

Title: Assessment of Deficiencies and Preservation of Bridge Substructures Below the Waterline. (FCP No. 55K2032)
Objective: Develop guidelines for assessing structural deficiencies and rating their severity. Develop methods for arresting structural deterioration below the waterline.
Performing Organization: Byrd, Tallamy, and MacDonald, Falls Church, Va. 22042
Expected Completion Date: July 1982
Estimated Cost: $150,000 (NCHRP)

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FCP Category 6—Improved Technology for Highway Construction

FCP Project 6C: Use of Waste as Material for Highways

Title: Kiln Dust-Fly Ash Systems for Pavement Bases and Subbases. (FCP No. 36C2242)

Objective: Evaluate the effectiveness of substituting kiln dust collected during the production of lime and cement for hydrated lime in lime-fly ash stabilization systems.
Performing Organization: Valley Forge Laboratories, Inc., Devon, Pa. 19333
Expected Completion Date: August 1982
Estimated Cost: $159,000 (FHWA Administrative Contract)

Title: Latex Improvement of Recycled Asphalt Pavement. (FCP No. 46C4314)

Objective: Determine the feasibility of using rubber latex and waste rubber latex as the supplementary binder in recycled cold-milled asphalt mixtures.
Performing Organization: University of Akron, Akron, Ohio 44325
Funding Agency: Ohio Department of Transportation
Expected Completion Date: April 1982
Estimated Cost: $51,000 (HP&R)

FCP Project 6D: Pavement Rehabilitation

Title: Correlating Dynamic Deflections With Pavement Performance. (FCP No. 46D1412)

Objective: Select a deflection testing device for use during Statewide pavement inventories. Demonstrate the level of correlation between pavement performance and deflection measurements made with Benkelman Beam, Road Rater, and Falling Weight Deflectometer. Compare the elastic layer properties derived from the measurements.
Performing Organization: Alaska Department of Highways, Juneau, Alaska 99801
Expected Completion Date: March 1983
Estimated Cost: $86,000 (HP&R)

Title: Analysis of Low Modulus Interlayers in Existing Pavements. (FCP No. 46D2784)

Objective: Investigate the mechanistic behavior of stress-absorbing membrane interlayer (SAMI) in the overall pavement overlay system to optimize the use of this stress-absorbing concept in different applications to establish criteria for overlay design. Perform laboratory testing, field verification, and an analytical study of SAMI.
Performing Organization: Arizona Department of Transportation, Phoenix, Ariz. 85007
Expected Completion Date: June 1983
Estimated Cost: $57,000 (HP&R)

FCP Category 0—Other New Studies

Title: Performance Evaluation of an Air Duct Ground Stabilization System. (FCP No. 40M1793)

Objective: Develop a design methodology based on the heat transfer coefficient determined from the heat loss and heat transfer characteristics of low velocity air flow through various culvert configurations.
Performing Organization: University of Alaska, Fairbanks, Alaska 99701
Funding Agency: Alaska Department of Highways
Expected Completion Date: September 1982
Estimated Cost: $58,000 (HP&R)

Title: Asphalt Composition Analysis. (FCP No. 40M3672)

Objective: Develop a detailed databank that can be used at a later date to correlate bituminous concrete field performance with raw asphalt chemical composition, providing such a correlation exists.
Performing Organization: Maryland State Highway Administration, Baltimore, Md. 21203
Expected Completion Date: February 1983
Estimated Cost: $97,000 (HP&R)

Title: Development of a Materials Tests Data Information System. (FCP No. 40M5593)

Objective: Develop a computerized data system to process and analyze laboratory data rapidly to provide management with up-to-date materials testing information.
Performing Organization: University of Puerto Rico, Mayaguez, P.R. 00708
Funding Agency: Puerto Rico Department of Transportation and Public Works
Expected Completion Date: June 1982
Estimated Cost: $75,000 (HP&R)

Title: Incidence Assessment of Transverse Cracking in Concrete Bridge Decks. (FCP No. 40M5611)

Objective: Identify the causes of specific incidences of transverse cracking in bridge decks. Investigate design, materials, and construction factors. Perform field observations and measurements during the construction of approximately 18 bridges.
Performing Organization: North Carolina State University, Raleigh, N.C. 27650
Funding Agency: North Carolina Department of Transportation
Expected Completion Date: March 1983
Estimated Cost: $76,000 (HP&R)

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