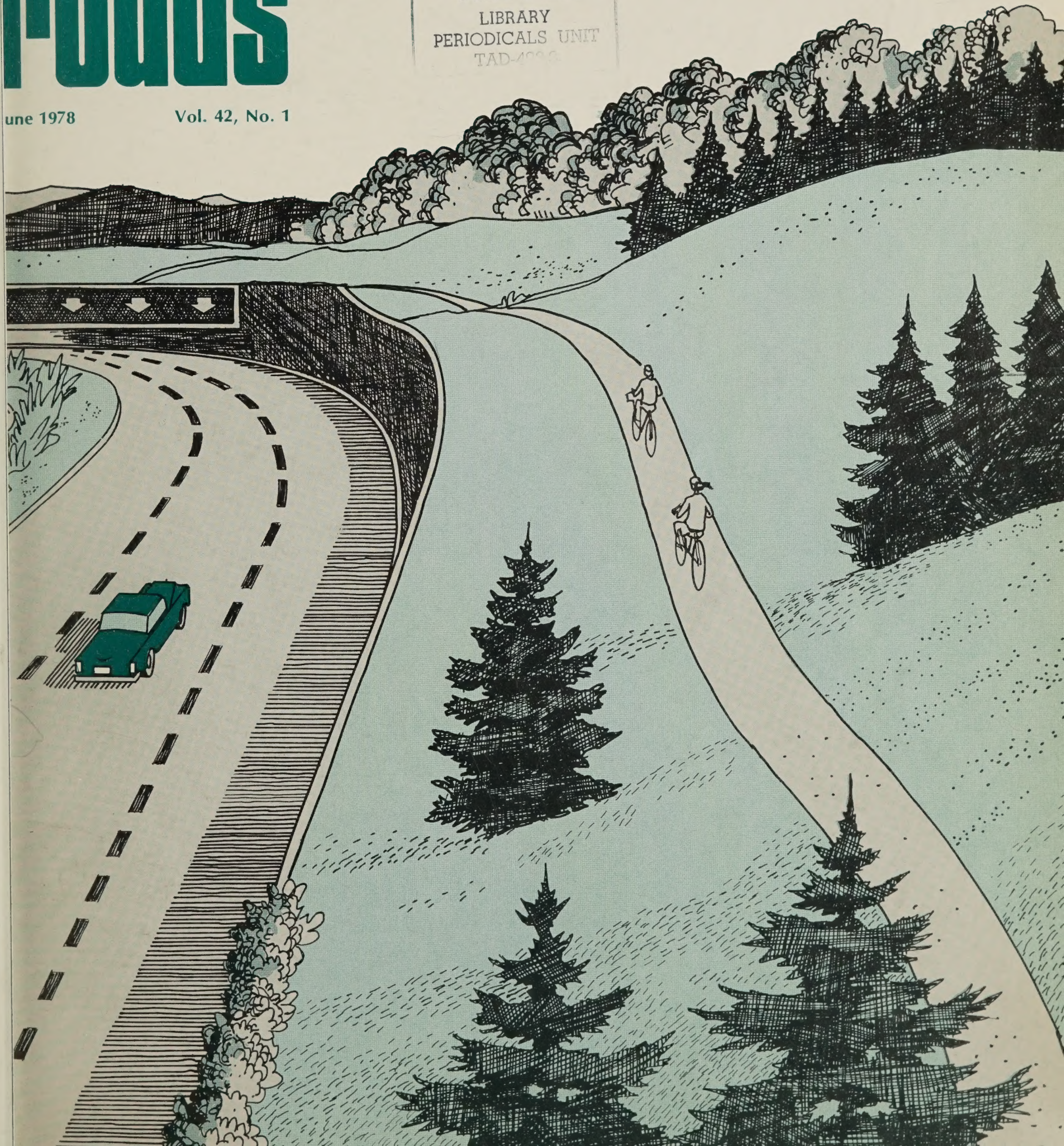
public roads

A JOURNAL OF HIGHWAY RESEARCH AND DEVELOPMENT
U.S. Department of Transportation Federal Highway Administration

une 1978 Vol. 42, No. 1



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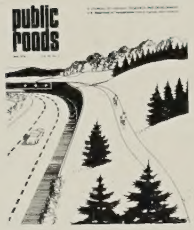

COVER:
Artist's concept of bicycle path paralleling the highway.

IN THIS ISSUE

Articles

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Public Roads Magazine, HDV–14
Federal Highway Administration
Washington, D.C. 20590

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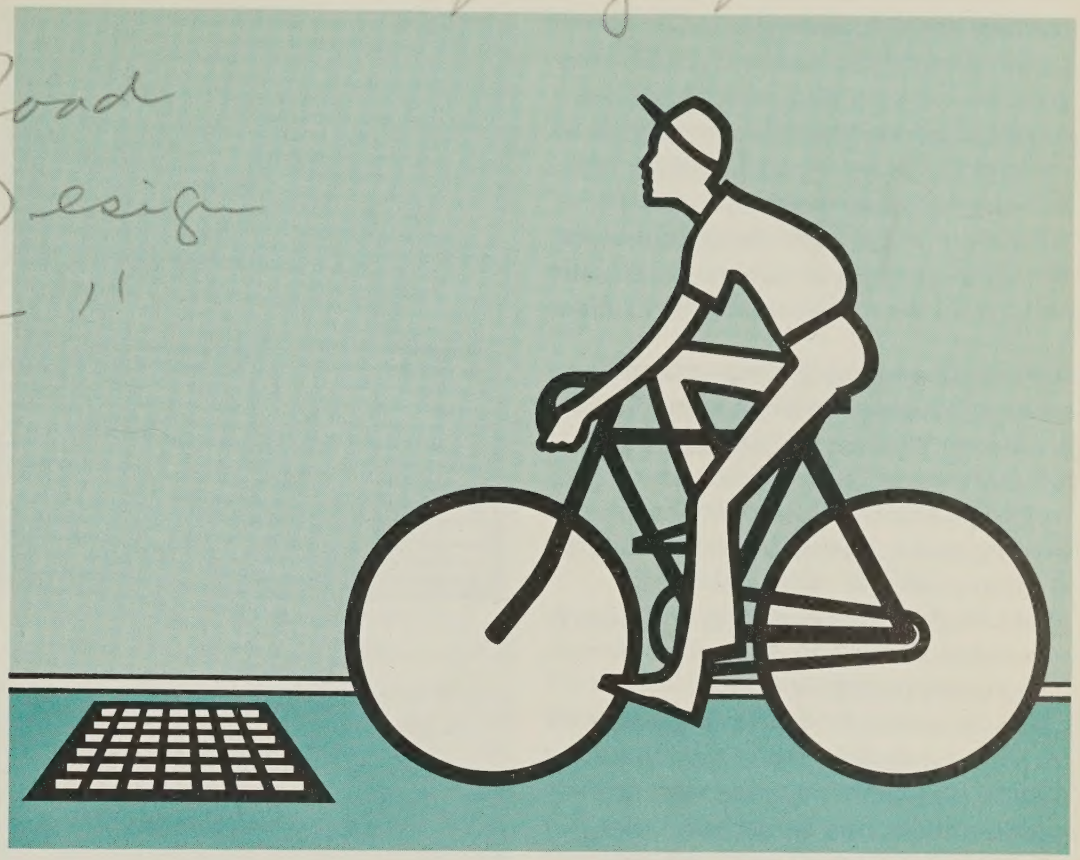
The Secretary of Transportation has determined that the publication of this periodical is necessary in the transaction of the public business required by law of this Department. Use of funds for printing this periodical has been approved by the Director of the Office of Management and Budget through March 31, 1981.



Bicycle-Safe Grate Inlets

by Dah-Cheng Woo

The common grate inlet takes many forms and is found wherever there is a need to intercept surface water for underground collection and disposal. Now that the bicycle is a serious competitor with the automobile for space on our streets, there is a need in many areas for a bicycle-safe, hydraulically efficient grate to replace the commonly used standard parallel bar grate. The U.S. Bureau of Reclamation conducted research for FHWA on 11 grate designs for bicycle and pedestrian safety, structural integrity, and hydraulic efficiency. Compatibility with current manufacturing techniques and cost of manufacturing were also considered. Three grate designs were found to be superior in all aspects of safety, hydraulic efficiency, and fabrication cost.

Introduction

The common grate inlet takes many forms, shapes, and sizes. It is usually located in the street gutter along the curb, but is also found in parking lots, playgrounds, golf courses, and other places where it is necessary to intercept surface water for underground collection and disposal.

Grates come in hundreds of different shapes and sizes, and tens of thousands are manufactured yearly for new structures or to replace broken or stolen grates. Approximately 15 to 20 million grates are in use throughout the United States. The primary purpose of the grate is to cover the sewer opening and allow water to enter, but there are other considerations in the design of most grates. They must be strong and heavy enough to support heavy vehicles and stay in place when run over by a fast-moving truck, heavy enough to discourage theft, shaped so they cannot be easily lifted from place and dropped into the hole, shaped to accept water but not collect debris, and shaped so a pedestrian's shoe heel or a child's foot will not be caught in the grate. Therefore, even something as traditional as the grate inlet must change in response to our changing society.

Early manhole covers in the United States were cast with a rough surface to provide a nonslip footing for horses, but the primary concern of the grate designer for the last 50 years has been the automobile. Grates have been designed to support heavy, fast-moving vehicles as well as effectively drain water from the streets. Flooded streets result in damaged property and stalled cars; with increased automobile speeds, hydroplaning is an additional problem. Because grates should be self cleaning, the parallel bar grate with its good drainage and self cleaning characteristics is widely used.

Although bicycles have been on U.S. streets for nearly 200 years, only recently in our energy conscious society has the bicycle become a serious competitor with the automobile for space. The conflict between the narrow-wheeled bicycle and the parallel bar grate is obvious to the cyclist. The solution, however, is not as simple as it may appear to be. Because very few roadway gutter grates are square, it is not possible to turn them 90 degrees so the bars are perpendicular to the flow of traffic. Even if the standard parallel bar grate could be turned, other problems would arise: an immediate reduction in hydraulic efficiency, an altering of the self cleaning characteristics, and a substantial increase in the potential for flooding. The consequences are equally serious for the motorist and the cyclist.

In many of our major cities, grates on selected routes have been altered to accommodate bicycle traffic. Parallel bar grates have been modified by welding transverse rods onto the existing grates; clamp on transverse bars are also available. Although these measures are good concepts and somewhat successful, they often defeat the primary purpose of the grate—draining the street. They also increase clogging, further reducing drainage capability and increasing maintenance costs. In addition, they often create a bumpy and dangerous ride for the cyclist and cause tire damage to vehicles. Where the inlet box can be enlarged to compensate for the change, grates may be rotated 90 degrees, but the cost of constructing a new box to provide extra surface area to improve the hydraulic efficiency makes such an alternative unattractive to most transportation authorities.

Criteria for Grate Design

A reasonable response to this dilemma is to design a grate which will provide adequate safety for both cyclist and pedestrian, be strong enough to support heavy vehicles, and have a high degree of hydraulic efficiency. In view of the large number of grates in the country, it is unlikely that it will ever be economically feasible, or necessary, to replace them all. But consideration must be given to a design which can be manufactured using existing technology and is within the current market price, so that bicycle-safe grates will be available for selective use.

Following preliminary feasibility studies, it was decided to pursue the

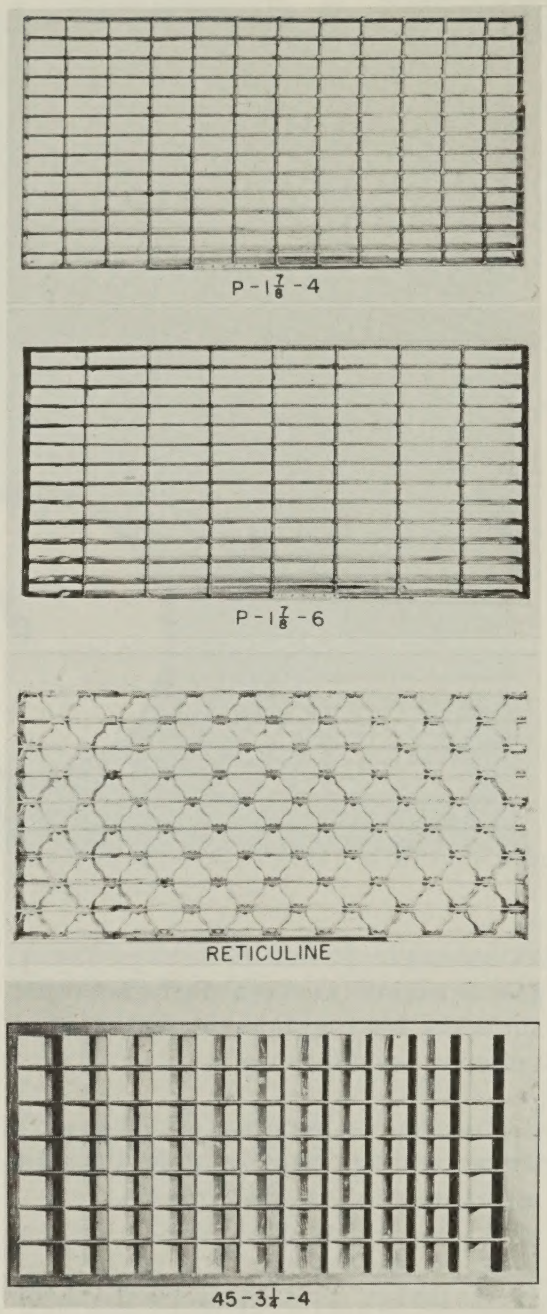


Figure 1.—Four of the grate designs tested in the bicycle-safe research.

design of a new grate, or series of grates, which would meet the following criteria:

- Safe for bicycles and pedestrians.
- Structurally sound.
- Hydraulically efficient.
- Free from clogging (self cleaning).
- Economical to manufacture and install.

The Engineering and Research Center of the Bureau of Reclamation at Denver, Colo., was selected to do this work because of their expertise in hydraulic testing and their excellent laboratory research facilities. This article summarizes three phases of the research: (1) An examination of manufacturing methods and structural analysis, (2) bicycle-pedestrian safety tests, and (3) hydraulic testing.

One very important aspect of the research was to select which grate designs should be tested. It would be impossible to test all of the bicycle-safe grates which have evolved in recent years; therefore, the selection of grates possibly able to meet the five criteria cited above was based on data collected in Federal Highway Administration laboratory studies (*1, 2*)[1] and from tests conducted in private industry, university, and government laboratories. (*3, 4, 5, 6, 7*) Fifteen grate designs were considered in various phases of the study. A grate design in this context refers to the grate bar configuration and the orientation and spacing of the bars, and not to the overall dimension of the grates. As the research progressed through the three phases, some grate designs were screened out and others were incorporated for comparisons. Eleven grate designs were used in the safety tests, eight were used in the basic hydraulics tests, and only three were used in the detailed hydraulics tests which included varying the overall dimensions to get generalized hydraulic design information. Figure 1 shows some of the grates which were selected for testing.

Manufacturing Methods and Structural Analysis

One of the first steps in the research program was to insure that the results would be implementable, that is, the designs proposed for testing could be reproduced by methods within the current state of the art of the

[1]Italic numbers in parentheses identify references on page 5.

manufacturing community. To avoid restricting future manufacturing techniques, both casting and fabrication processes were taken into consideration in developing the basic research plan. Representative manufacturers from the grate casting and fabrication industries met with the researchers to discuss potential problems associated with manufacturing the grates selected for testing.

The meetings established manufacturing constraints. Based on these constraints and the requirements as stated in the American Association of State Highway and Transportation Officials (AASHTO) "Standard Specifications for Highway Bridges," the U.S. Bureau of Reclamation's general purpose computer program, STR5, was used to perform structural analysis of the grates. The analyses varied somewhat between fabricated steel grates and cast iron grates, but the grates selected for safety and hydraulic testing met the required AASHTO specifications for structural integrity. (*1*)

Bicycle and Pedestrian Safety Tests

Because concern for cyclists' safety prompted the research study, an extensive test program was developed to insure that each grate design approved for hydraulic testing would be safe for the average cyclist and pedestrian. The tests were conducted at the Denver Federal Center under the supervision of a consulting bicycle safety specialist. Male and female cyclists, both adult and children, rode typical narrow-wheeled bicycles with different types of tires over an inlet for a total of 539 runs with 11 different grate designs in place. All grates were 24 in wide by 48 in long (610 mm by 1,220 mm). Grates were kept wet for all test runs to simulate the worst environmental conditions normally encountered by cyclists (fig. 2).

The test runs took place on a 22 ft (6.7 m) wide paved asphalt road with an average grade of 2 percent. An 8 in (203 mm) high concrete curb was provided along the approach to the grate for the uphill and downhill straight runs. The curb was removed for turning tests.

Three types of data were collected: measurements by a team of observers, bicyclists' perceptions, and several types of video records. The observers were to note skidding on grates, the need for cyclists to swerve to maintain control due to grate irregularities, and sudden reduction in speed resulting from wheels catching in grates (this did not occur). The cyclists' perceptions covered the comfort of the ride over the grate, the ability to recognize a safe grate before reaching it, the effect of the grate on steering control, grate-induced skidding, the impression of tire deformation resulting in a contact of the wheel rim with the grate, and the riders' feelings for control and skid on turning runs. Video data consisted of video tapes, high speed motion photography, and selected still photography.

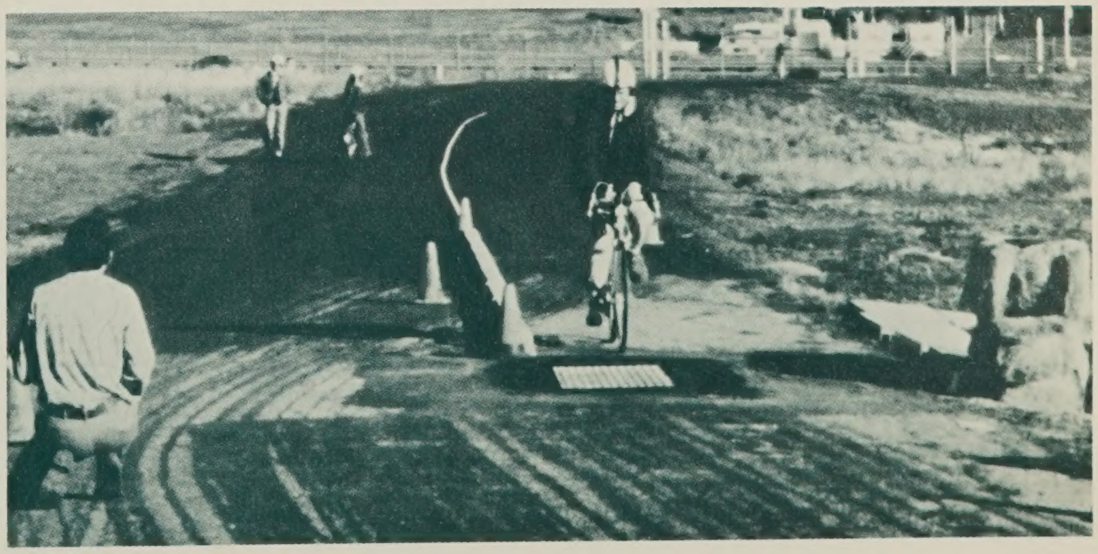
All of the grates tested proved to be markedly safer than many grates in use today; however, there were clear safety performance differences among the grates tested. Of the 11 grate designs tested, 4 were superior. Transverse bar spacing is a critical factor in bicycle safety performance of a grate. Transverse bars spaced beyond 4 in (102 mm) cause deterioration of grate safety. Skidding occurred on virtually all grates; wet grates may remain hazardous to cyclists crossing them while turning a corner.

In general, grate designs which proved least satisfactory for bicycle safety were also those which appeared least satisfactory from a pedestrian standpoint.

Hydraulic and Debris Tests

To test the hydraulic characteristics of grate inlets under conditions representative of a true field environment, a full-scale test facility was used. The tests were carried out in a specially designed flume 8 ft (2.4 m) wide and 60 ft (18.3 m) long. The gutter section was 2 ft (0.6 m) wide, leaving a 6 ft (1.8 m) traffic lane (one half of a normal 12 ft [3.7 m] lane and generally considered the allowable width of water flow spread). The inlet test section was located 40 ft (12.2 m) from the head box. By applying No. 16 sand (1 mm to 2.4 mm diameter) to wet epoxy paint on the bottom of the flume, it was possible to simulate surface roughness of

Figure 2—Typical test run for bicycle safety of a grate inlet.



average urban highways and streets. The longitudinal slope (S_O) could be increased up to 13 percent and its cross slope varied from 1/96 to 1/16. Maximum water flow was 5.6 ft^3/sec (0.16 m^3/sec). Figure 3 shows the hydraulic test facility.

Eight grate designs were tested. They included the four which were recommended from the bicycle-safe tests, two variations of these, a bicycle-safe grate provided by Los Angeles, and a standard parallel bar grate against which the hydraulic efficiency of the new grate designs could be measured.

Each grate design was tested on continuous grades for two sizes: 24 in by 24 in (610 mm by 610 mm) and 24 in by 48 in (610 mm by 1,219 mm). From these results, three grate designs with the best overall performance were further tested on continuous grades with four more sizes of two different widths, 15 in (381 mm) and 36 in (914 mm). All six sizes were then tested for sump conditions, that is, where the grate is located at the low point of a sag vertical curve. These six sizes were selected so that the test results can be used to provide information on hydraulic performance for any size grate.

A special test procedure for evaluating the debris passing characteristics was adopted for this study to simulate the field condition of debris accumulation in the gutter and its flow to the grate during a storm. One hundred fifty pieces of saturated 3 in by 4 in (76 mm by 102 mm) brown kraft paper were used to simulate wet leaves. The debris handling efficiency was calculated as the ratio of debris that passed through the grate, plus the debris that washed off the grate, to the total debris. The results were used for qualitative comparison only because the actual debris phenomena in the field are extremely complex to model.

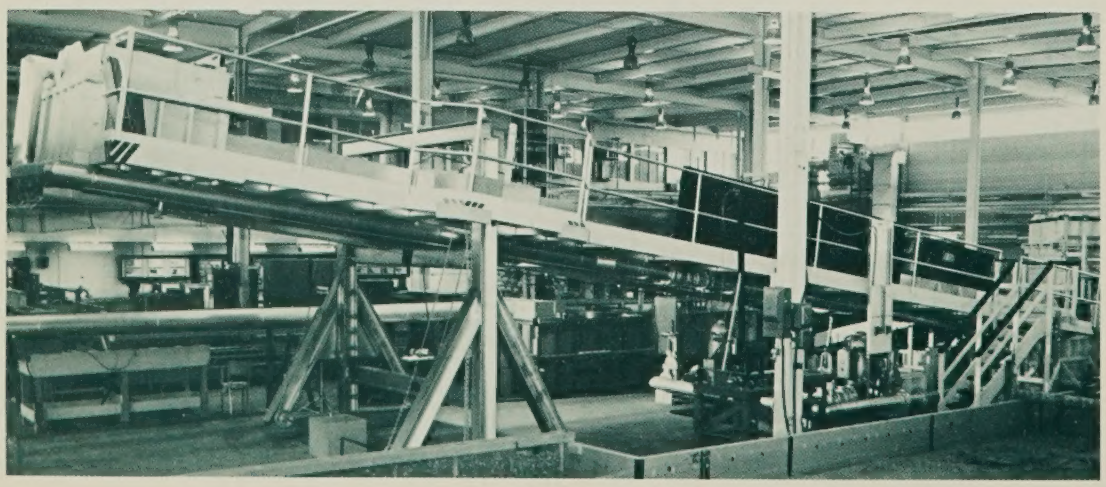
Summary of Test Results

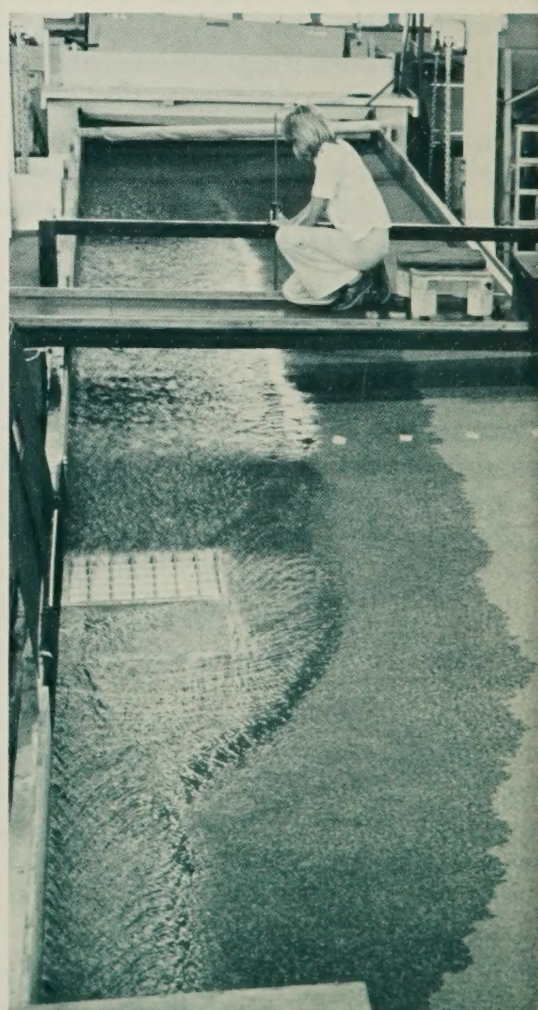
For flat longitudinal street slopes, the hydraulic efficiency of grates of a given width and length varied less than 6 percent; however, when longitudinal street slope was steep, the variation in some instances was as high as 34 percent. Possibly, roughening the top of bars of a grate would reduce the hazard of bicycle tire slippage while turning a corner. In general the grate designs which are rated high in bicycle safety are rated low in debris handling characteristics.

Three grate designs that are superior in safety characteristics show better overall hydraulic performances: the curved vane grate, the parallel bar grate with 1-1/8 in (29 mm) spacings and transverse spacers, and the parallel bar grate with 1-7/8 in (48 mm) spacings and transverse rods spaced at 4 in (102 mm). They were also found to be practical to fabricate or cast. Figure 4 shows some of the inlet hydraulic efficiency curves for the grates tested.

For mild longitudinal street slopes, other grate designs also perform well hydraulically. As the more efficient grate designs do not necessarily have better debris passing characteristics, the designer can choose the grate design which best suits his or her special conditions.

Figure 3.—The Bureau of Reclamation's test facility showing tilting flume and full-scale grate inlet test.





Future Research

The slotted drain which is used along the outside edges of a highway or in the gutter has gained increased recognition for its advantages under certain field conditions. Little information, however, is available on its basic hydraulic and structural characteristics. Research will be conducted on the various types of slotted drains so that hydraulic, structural, debris, and bicycle safety characteristics can be established, thus providing another option to the traditional grate inlet.

The bicycle-safe grate inlet research results will be published in four volumes. Volume 1, "Hydraulic and Safety Characteristics of Selected Grate Inlets on Continuous Grades," has been published and covers the structural analysis, bicycle and pedestrian safety tests, and hydraulic and debris tests for the 24 in by 24 in (610 mm by 610 mm) and the 24 in by 48 in (610 mm by 1,219 mm) selected grate designs. (*8*)

Volume 2 will cover the structural analyses and more detailed hydraulic tests for the three grate designs that were superior in safety characteristics and also had best overall hydraulic performance. The more detailed hydraulic tests for these grate designs included varying the widths to 15 in and 36 in (381 mm and 914 mm).

Volume 3 will cover the debris and hydraulic tests for the same three grate designs at all three widths (15, 24, and 36 in [381, 610, and 914 mm]) under sump conditions.

Volume 4 will cover the research results of the slotted drains. It will consist of analytical, experimental, and safety conditions.

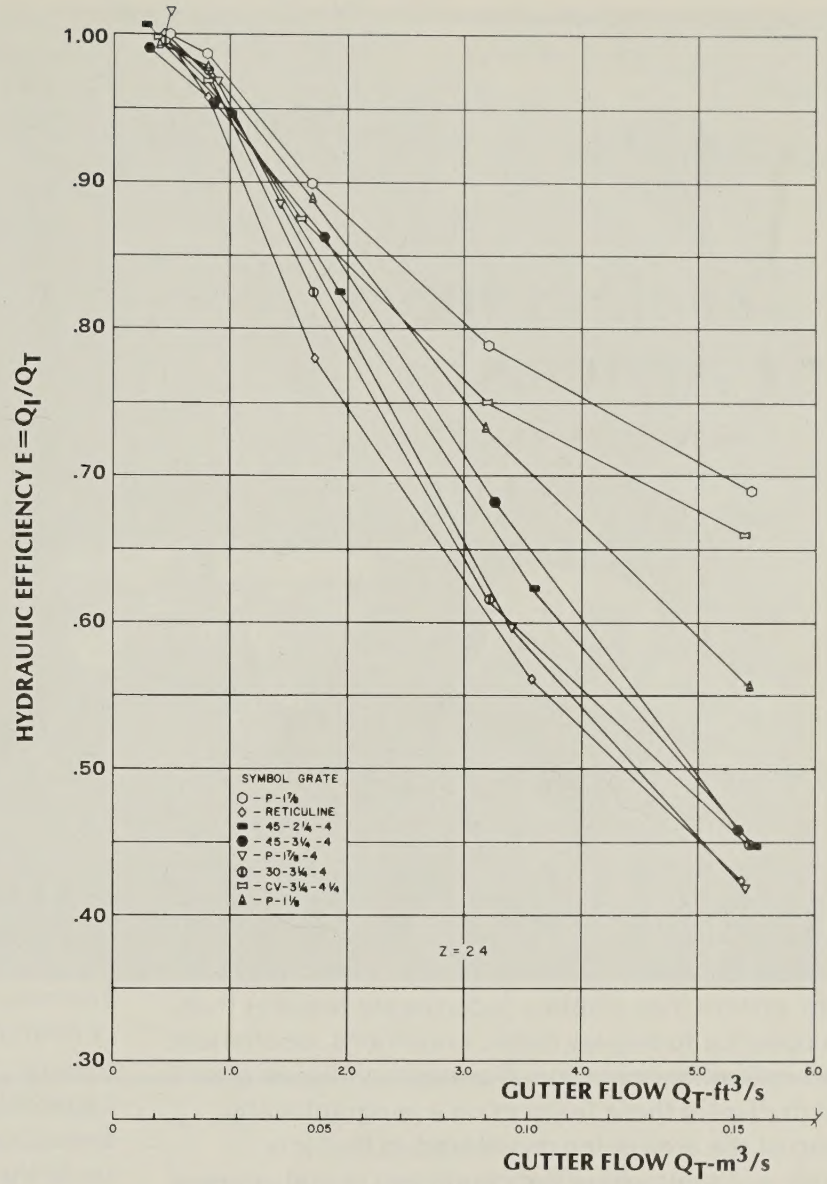


Figure 4.—Hydraulic efficiency versus gutter flow for the 24 in by 24 in (610 mm by 610 mm) grates. S_O *= 9 percent.*

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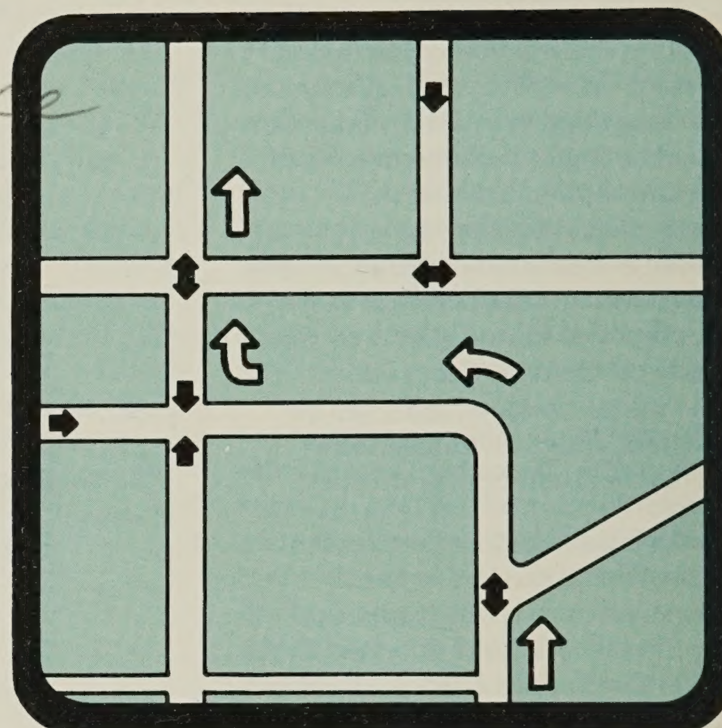
[2]Reports with PB numbers are available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161.

Map Displays for Traffic Systems

by Donald Sundberg



Introduction

The dynamic display has achieved status as a viable tool in the assessment and real-time operation of the traffic system, even though it was originally intended as a public relations device to demonstrate aspects of the traffic system to personnel not familiar with the traffic engineer's function. Most traffic system map displays incorporate features that permit an operator to display traffic conditions, equipment status, and controller operation. Various techniques are employed to display these features on a geographically related map of the area being monitored so that it is immediately apparent where the conditions or status being displayed exists. This article is intended to familiarize individuals involved with automatic surveillance and control systems with the design of a traffic control system map display.

Purpose

The central display serves two primary purposes. First, it provides for instantaneous recognition of sudden failures and abnormal degradation, which enables a small operating force, usually one system operator, to manage large and complex systems. Second, it provides traffic engineers with a real-time overview of existing traffic conditions, which is useful for system evaluation and analysis.

An additional advantage of the central display is the visibility provided for system installation and integration. As a troubleshooting tool, the display permits rapid visual assessment of the status of system implementation which facilitates the coordination of the various installation activities. The display has also been used in simulation situations where control strategies are imposed on the offline system and the response evaluated on a macroscopic basis.

The map display is invariably coupled to the control system central processor by means of an input/output device, thereby providing a highly flexible information display capability. New concepts and purposes are constantly being formulated; as the display's uses become more sophisticated, so do the presentation techniques.

Features

A geographic presentation of the system network with illuminated indicators representing the system surveillance and control elements is the fundamental ingredient of the traffic system map display (fig. 1). The shape and illuminated color of the indicators are usually configured to represent the physical aspects of an element. For example, indicators representing traffic signal controllers could be arrow-shaped to identify the main street green direction with illuminations of green or green/red corresponding to the illuminated state of the signal head.

In systems with requirements for multiple information displays, a single indicator could present several unique data. To reduce the quantity of indicators and improve the human factors of the map display, the information displays are requested on a mutually exclusive basis with legends conveying the current information mode incorporated in the display.

Traffic Engineering
Urban traffic control system
X UTCS Bus priority system
X BPS

Figure 1.—Metro Dade County—Traffic Control Center.

Design

Designing a display for the traffic control system requires an understanding of a number of diverse disciplines: a thorough knowledge of the system's functional requirements, a working knowledge of human factors engineering, and an understanding of the techniques available for display fabrication. Figure 2 outlines the general procedure for designing a display for the traffic control system.

Graphic Presentation

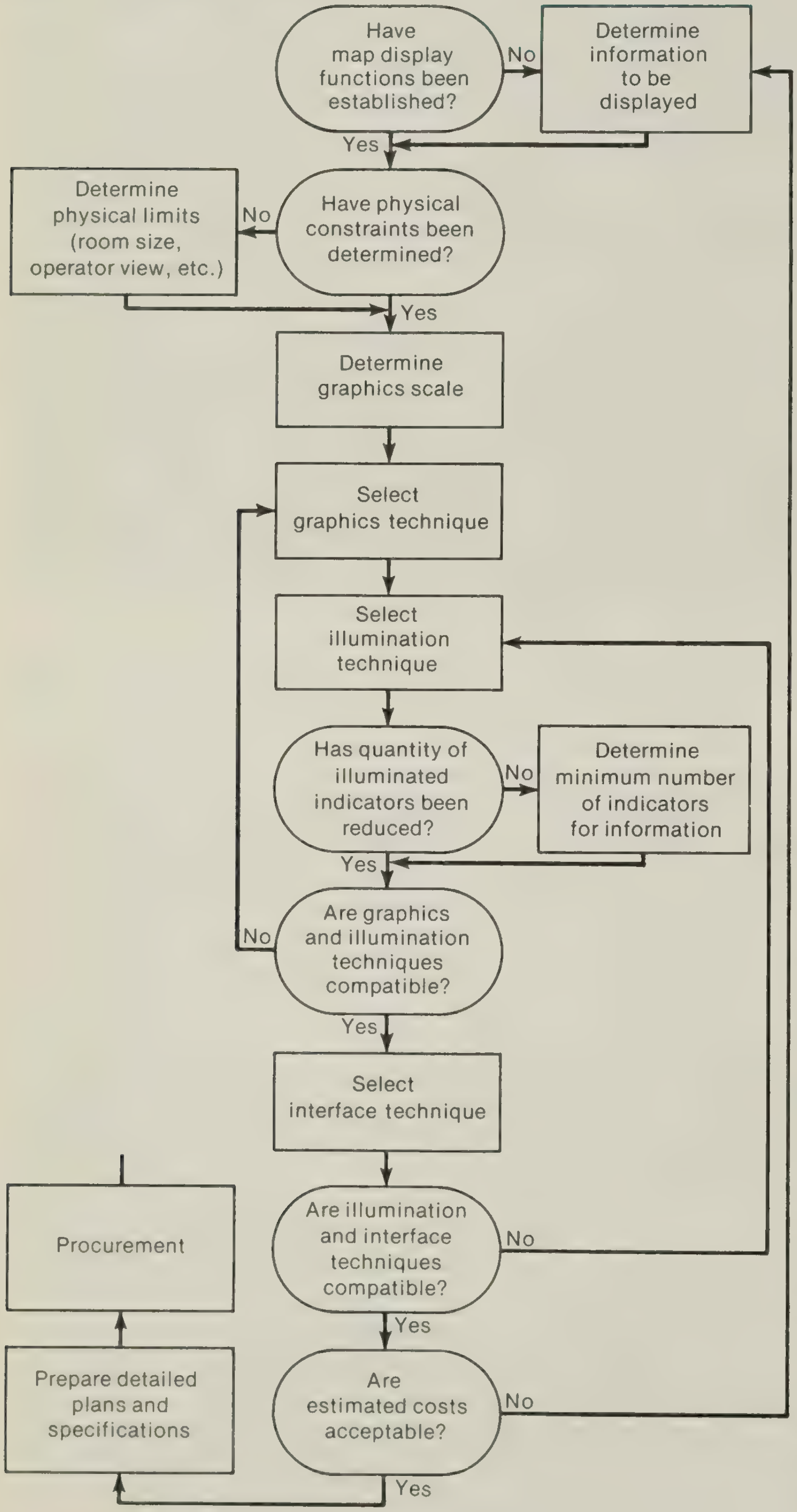
A geographic presentation of the network under surveillance or control is the primary and common ingredient of all traffic system displays. The graphics techniques employed in achieving this vary as a function of the number of display indicators, network configuration, size, expansion flexibility, and manufacturing ingenuity. The final selection of a particular graphic presentation technique is based on a study of the numerous graphical techniques presently available or under development. Two available techniques are as follows:

• *Enlargements of existing maps and aerial photographs.* Generally, the simplest and least expensive fabrication of a traffic map display includes enlargements of existing maps or aerial photographs. These are bonded to a hard backing suitable for the installation of lamp modules. Although the lower cost is enticing, the photographic approach is not usually recommended because trees, shadows, or other extraneous objects included in the enlargements reduce the display legibility. The relatively low cost of the enlargements ($6–$8/ft^2 [$0.56–$0.74/m^2]) enables an in-house staff to assemble a low cost display for study or temporary purposes. In this instance, illuminated indicators would be off-the-shelf items mounted in holes drilled directly through the display mounting board.

• *Custom artwork.* A more complex and costly approach is the use of custom artwork. In this technique, the display graphics are customized to eliminate superfluous information found with the enlargement of existing documentation, and essential geographic features are emphasized in order to provide an unambiguous, legible graphic display. In addition, the design of the format is "hand tailored" so slight distortions can be incorporated to accommodate the illuminated indicators.

Most backlighted displays, including the mylar maps which are used quite extensively for traffic system application, are designed with the custom artwork approach. With mylar maps, a full-scale layout of the system network is manually configured, incorporating provisions for the symbol illumination and lamp placement. The final version of the artwork is then photographically reproduced and mounted on a structure containing the lamp modules.

Figure 2.—Map display design flow chart.



Display manufacturers have made innovations in the photographic process and assembly of the graphics unit to provide for expansion and fabrication cost reduction. In one approach, the final transparency is processed with the entire artwork toned or minutely screened to create a translucent medium for light transmission. The artwork is then mounted on a translucent backing (usually fiberglass) with the light modules incorporating a shaped indicator mounted on the rear. This approach provides excellent flexibility for indicator placement and expansion; however, the transmitted light is weakened by the two translucent mediums and some brightness is sacrificed. The technique employed in developing the artwork for the Urban Traffic Control System (UTCS) display (figs. 3 and 4) had the street system clear and the background opaque black, with halftone symbols in the black background. The backing plate was aluminum painted white with holes drilled at all indicator locations for indicator mounting and light transmission. The traffic signal indicators, located behind the transparent roadways, suffered little loss of brightness. The toned sensor indicators in the opaque background were slightly discernible when not illuminated amber or yellow, so that the high contrast ratio of light source to background compensated for the light attenuation.

Holes were predrilled at all intersections within the artwork to account for expansion; however, the expansion capability is limited in comparison to the totally translucent display approach.

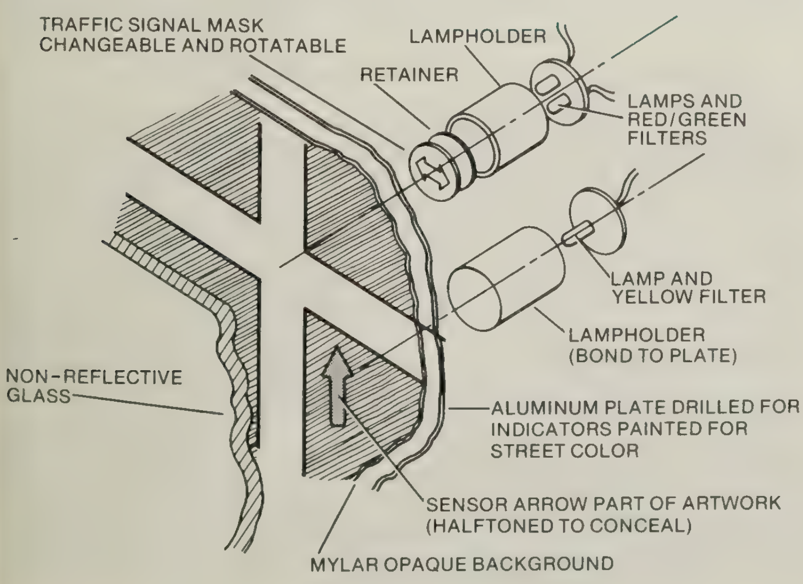
All backlighted displays employing photographically-produced artwork have glass or plexiglass front surfaces to protect the easily scratched film. Glass is the pre-dominant choice because of its rigidity, heat immunity, and nonyellowing characteristics. To avoid reflections, the glass is either sandblasted or chemically coated to provide a glare-free transparent medium.

Mosaic Displays

Custom artwork is also required for the fabrication of the mosaic or tile maps. In this case, the artwork is silk screened or engraved on a surface fabricated from plastic or aluminum tiles. The tiles are individually removable from the rear support structure that also serves as the lamp module holder. Wherever an illuminated indicator is required, the desired symbol shape is cut or stamped out of the tile, a clear or colored filter is inserted within the cutout, and the tile is replaced. A large percentage of the displays fabricated to depict flow processes (that is, refinery process control) use this approach. However, when a presentation depicts a large

Figure 3.—UTCS display, Washington, D.C.

Figure 4.—UTCS display construction.



quantity of oblique and randomly interconnected points, the cost of manufacturing unique light module tiles hampers the fabrication of an economically competitive display. For highway and freeway application, such as the system display in figure 5, the tile approach is generally economically competitive because of the large quantity of repetitive tile stampings which are possible (fig. 6). The advantage of the tile map is that indicator and artwork changes can be made from the front of the display without disturbing the entire artwork.

Furthermore, since the tile display can be packaged in smaller units, the difficulty of shipment and onsite installation is greatly reduced as compared to other displays fabricated as large rigid panels. Because the display is fabricated from small tiles and no protective front surface is required, a concave or convex display can be constructed.

Human Factors

Thus far only the techniques available for generating the graphic presentation have been considered, without regard to the human factors aspects of map display design. In addition to the artwork technique, display size, layout, and colors, the relationship of the various display elements is also of prime concern. Overemphasis on an esthetically dramatic presentation can result in a display that is unsatisfactory for information transfer.

Display size is essentially a function of the area coverage and the quantity and configuration of the illuminated indicators. Generally, for urban system displays the scales range from 1 in = 150 ft (1:1,800) for heavily instrumented systems to 1 in = 300 ft (1:3,600) for the average system. Scaling less than 1 in = 300 ft (1:3,600) usually requires some compromise in the information display with certain geographic features; noninstrumented intersections are the first to be deleted. Often two or three different scales can be used without seriously distorting the presentation. For example, the New Orleans System Display uses a scale of 1 in = 470 ft (1:5,640) for the total presentation and another map with a scale of 1 in = 200 ft (1:2,400) for the presentation of the central business district (fig. 7). Restricting the display size is essential, not only from the economic or system operator's viewpoint but also for installation convenience. Most displays are mounted in locations where the only delivery access, short of window removal, is through standard elevators, doorways, and corridors.

Bulb replacement is another consideration in determining display size. Paneled walls constructed to incorporate the display are the ideal situation since they permit rear access for bulb replacement and maintenance, while serving as a partition to conceal equipment and lessen noise. In installations where rear access is not permitted because of physical limitations, additional hardware is required to facilitate the displacement of the display unit. Even with the provisions for displacement, the larger displays are usually cumbersome to move and degraded display operation from lack of bulb replacement could result if the physical aspects of the display make periodic maintenance difficult.

Restraint should also be exercised in the selection of the display colors. Basically, three colors—one for the background, one for the grid system, and one to highlight terrain or other features—are sufficient for an unambiguous display. Occasionally, additional distinction is required to clarify complex geometric relationships. In these instances, the differences are defined by variations in shade rather than in color. Graphic color selection is also governed by the illuminated indicator color, with a high contrast ratio between the illuminated color and the background most

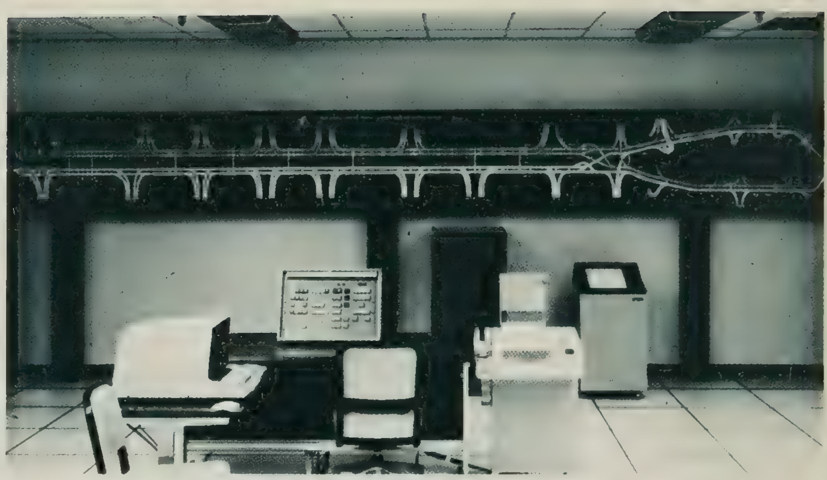


Figure 5.—New Jersey Turnpike System—Traffic Control Center.

Figure 6.—Mosaic map—construction.

Figure 7.—New Orleans System—Traffic Control Center.

desirable. Amber or yellow indicators on a dark background are more discernible than the same indicators on a bright yellow or white background. In addition, an operator using a predominantly bright-colored display over prolonged periods will experience visual fatigue.

Illumination Techniques

When considering the myriad of possible candidates, selecting the right illumination approach for traffic system display application can be perplexing. However, after the various technologies are compared, including light emitting diodes (LED) and liquid crystal displays (LCD), the incandescent indicator is invariably selected as the most practical for the intended application. The advantages of the incandescent indicator are as follows:

- Excellent brightness.
- Color selection.
- Installation flexibility.
- Easily obtained off the shelf.
- Low cost.

The first two advantages are easily understood because the map display must be readable in a fairly bright environment. Brightness, an expression common to all the illumination elements, is difficult to assess because there is no industry standard of measurement or specification. Correlating units of foot candles to candle power or to lumens per ft^2 (lumens per m^2) is confusing; when the calculations are complete, the expression still remains elusive. The expression "excellent brightness" in the context of the traffic system display application is derived from the direct viewing of the candidate elements in the intended environment. This "real" comparison is strongly recommended whenever a trade off study is conducted of illumination elements.

Because of the broad frequency spectrum emitted by the incandescent element, virtually any color can be generated by transmitting the light through a colored filter on the lamp or within the display surface.

For obvious reasons, green and red are the colors used for traffic signal presentation. Because of the high contrast ratio between the illuminated indicator and a subdued background, amber and yellow are commonly used to present the sensor or surveillance data. Additional colors may be employed to indicate levels of operation, sign messages, and direction; however, the use of numerous illuminated colors is discouraged. Fortunately, the display manufacturer limits the color selection by supplying six or seven standard filter colors; nonstandard colors are treated as specialty items.

Interface versatility has also contributed to the popularity of the incandescent lamp for traffic signal display application. With a wide selection of operation voltage and current coupled with several mounting configurations, almost unlimited combinations are available to relate the display to driver interface. For lower cost displays, off-the-shelf single-hole mount holders with integral filters are mounted on the front surface of the display artwork. The disadvantages of this approach are that the indicators are nondirectional, although this can be overcome by adding markings to the artwork or the indicator, and the scale of the display must be further enlarged to accommodate the lampholder hardware. On the other hand, backlighted displays employ lampholders that are custom tailored by the display manufacturer.

The lampholders and supporting structure on the back of the display are concealed from the front view; only the illuminated points are presented to the observer. Another advantage of the incandescent lamp backlighted approach is that two or more illuminated colors can be projected through a single symbol in the display graphics. This technique is incorporated in the New Jersey Turnpike Display, where one of three colors (red, amber, or green) is projected through a single aperture to display the current status of changeable message signs located at each turnpike interchange. The projected colors in this instance correspond to other than the normal sign messages, with nonilluminated sign symbols indicating the normal sign position.

Light Sources

Other factors, such as lamp life and the display environment, are important when evaluating the illumination elements. The rated life specified for incandescent lamps is obtained in a controlled laboratory environment with the lamps tested on 60 Hz alternating current at their design voltage. Environmental conditions, such as shock, vibration, and voltage fluctuations can contribute to a shorter lamp life and should be accounted for. Shock or vibration problems other than those associated with shipping or display mounting are not normally encountered with the traffic display maps and can be easily resolved. Fluctuation or transients, such as those produced by the initial turn-on current passing through a cold filament, can seriously shorten lamp life especially in the traffic display application where lamps could be cycled 2 or 3 times per minute. Inrush current, which can be as much as 10 to 12 times greater than the normal operating current, is generally reduced by placing "Keep Alive" resistors in series with the lamp filaments to produce a low value of filament preheat current.

Since incandescent lamp life varies inversely as the 12th power of the voltage (fig. 8), lamp life can be increased by driving the lamp below rated current. The disadvantages of lowering the voltage are that the light output varies directly with the applied voltage raised to the 36th power and as the voltage is lowered, the light output is displaced toward the red end of the spectrum, disproportionally affecting lamp visibility through green and blue filters. The power supply provided for lamps should feature a variable output to enable the voltage to be lowered if permitted by the surrounding light conditions.

Heat is another factor that can reduce lamp life. Lamp life can be increased by adequate ventilation within the display. This can usually be accomplished by louvered doors, fans, or, in a situation where central air conditioning is available, by the extension of plenums into the rear of the display.

Fiber Optics

The fiber optics approach has been used in displays for large city and freeway traffic systems where the illuminated indicators are so closely spaced that lamps cannot be mounted directly behind the indicator. In this approach, glass or plastic light pipes are routed from lampholders mounted a short distance from the rear of the display to the display artwork. The light is then projected either through a backlighted artwork or terminated with a lens cap on the outside of the display front surface.

There are some problems with fiber optics that should be considered. First, even with a lens cap the surface area of illumination is very small and difficult to discern. Second, the transmitted light will be weakened before reaching the display surface due to the end losses inherent with fiber optics. To compensate for these problems, the light immediately in front of the display is usually reduced, and the color of the display artwork is coordinated to provide a high contrast ratio.

New Illumination Concepts

New illumination concepts are rapidly becoming available for use in traffic system displays. Of particular interest are the recent developments in the LED and woven fiber optic technologies. The trend toward larger LED devices with at least four available colors is very promising, especially with the advantage that they are solid state devices and have a practically unlimited lifetime. The older concept of woven optics has had limited success in certain large sign applications but, perhaps due to cost, has not received much attention in the area of process control displays. The advantage of woven optics is that a small shaped indicator can be incorporated in a densely illuminated display with a light source remotely located.

When considering different illumination elements, it is important to make comparisons with actual devices in the anticipated environment. Examination of the real thing is the only positive approach that will resolve the outstanding uncertainties.

Interface Techniques

Techniques available for driving traffic system map displays are varied and depend primarily on the information presentation and system equipment capability. For example, systems using time division multiplexing (TDM) hardware do not have communication equipment driven displays since discrete outputs for each sampled point are not readily accessible. On the other hand, systems employing DC or frequency division multiplexing (FDM) communications equipment usually have discrete points available for each sampled sensor and controller.

In instances where communication equipment discretes are available, inexpensive minimum information displays can be considered. With this approach, the controller "A Phase Green" and "Hold" signals and the sensor data return signals are direct-wired to the map display indicators from the computer to communication equipment interface. Continuous display of these signals is one advantage of this approach. However, no additional system information can be presented. Also, the continuous flashing of the sensor indicators could be distracting to the operator and should be evaluated accordingly.

For informational displays beyond the limitations of the communication equipment drive, a display to computer link must be established. Once instituted, the flexibility of this approach is extremely significant; with sufficient programing, virtually any type of geographically related system information can be presented to the system operator. In addition to presentations related to modes of controller operation, processed sensor or surveillance data can be continuously displayed without the system operator visually interpreting the significance of the flashing indicators.

One disadvantage of the direct channel from the computer to the display is the loss of display information, including the basic traffic signal controller timing when the computer is not servicing the traffic control system. To prevent this problem, several systems have incorporated electronic switching circuits within the interface link to enable the display of "A Phase Green" information without an active computer. In this configuration the "A Phase Green" display is obtained from the communication equipment, and unique controller displays are handled by the computer. In the absence of computer commands the electronics automatically switches to a display of the controller "A Phase Green" simultaneously with the clearance of all other display information.

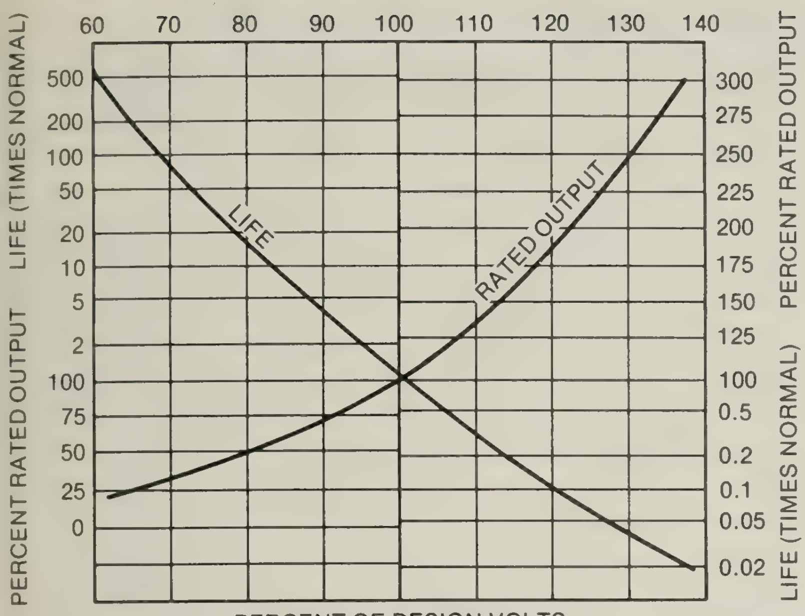


Figure 8.—Incandescent lamp life and brightness vs. voltage.

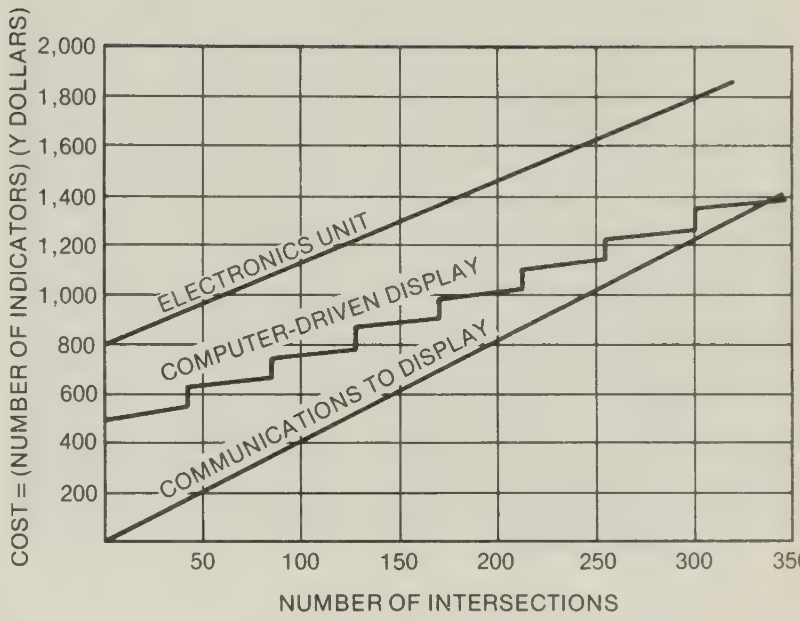


Figure 9.—Interface cost comparisons.

The cost of establishing the display interface depends on the technique. However, the selection of a particular technique is not clearly defined, and closer examination is warranted before deciding on the approach. For example, assume that the cost of a single indicator with a transistorized driver is "Y" dollars. In systems where the communications equipment permits access to the discrete "A Phase Green" return, controller "Hold on Line" (HOL), and sensor return signals, the cost of implementing the interface would be (2C + S)"Y" where C equals the total number of controllers and S equals the total number of system sensors. Furthermore, if we assume that two sensors are associated with each instrumented intersection, the interface cost of a communication equipment display would be 4CS"Y". The cost of the display will be a linear function of the number of instrumented intersections (fig. 9).

In comparison, a computer-driven display would permit mutually exclusive operation of the controller indicator and for this example, the number of indicators per instrumented intersection would be reduced to 3C. Another factor, the cost of the computer driver, has to be introduced to establish the cost of this interface. Computer hardware is usually configured to handle multiple discrete input/output signals and subsequently lamp drivers are not available on an individual basis. For this example, the computer packaging provides the lamp drivers in groups of 128 outputs. One-fourth of the cost of implementing the total interface would be a series of steps at equal intervals of intersections with a reduced slope connecting the intermediate points (fig. 9). In order to provide for as many factors as possible in this example, a cost of 400Y is introduced into figure 9 to account for the additional system software required to exercise the display.

To further explore the interface approach, a plot of the electronics unit interface is also provided in figure 9. The slope plotted on the basis of one indicator driver is approximately equal to 1.1 times that of the first or communications approach. The advantages of mutually exclusive operation have also been introduced along with the factors related to additional hardware and engineering costs (400Y). Unlike the computer interface where the lamp drivers are packaged in large groups, the electronics are usually tailored to provide indicators in groups of 8 or 16.

A comparison of the plots for the computer and communications interface shows that the cost is not significantly different for larger quantities of instrumented intersections (fig. 9). Additionally, the reduction in the number of indicators required and the increased flexibility in presenting the information leaves little doubt that the computer driver interface is superior for all but the simplest applications.

Although the plot for the electronics unit interface shows no cost advantage, the approach cannot be easily dismissed. A unit designed and manufactured by an extremely competitive specialty house employing the latest in electronic technology could compare favorably in cost with the standard interfaces provided by the computer manufacturers. Although ascertaining a cost relationship for the advantages of a continuous display in the absence of computer is difficult, the advantage does justify consideration.

Figure 10.—UTCS display modes.

Display Modes

Controller traffic signal displays include the following:

- A Phase Green—Basic signal timing.
- Controllers Failed—Nonoperational controllers.
- Controller Status—Controllers under computer control.
- Critical Intersection Control Status—Controllers in active critical intersection operation.
- BPS Status—Controllers in active bus priority operation.

Surveillance sensor displays include the following:

- Occupancy—Percent.
- Speed—mph (km/h).
- Volume—Vehicles Per Hour.
- Queue—Vehicles.
- Stops—Stops/Cycle.
- Delay—Seconds.
- Surveillance Equipment Failure.
- Bus Gain—Passenger/Minute/15 Minutes.
- Bus Helped—Bus/15 Minutes.
- Bus Surveillance Equipment Failure.

Planning the Display

The planning for the Urban Traffic Control System/Bus Priority System (UTCS/BPS) map display serves as an excellent example of the steps necessary in developing a traffic system map display (fig. 3).

The UTCS/BPS system is a heavily instrumented traffic and bus priority control system installed in Washington, D.C., with approximately 201 intersection controllers and 600 sensors managed by a central computer. The area implemented encompasses a typical roadway grid found in large urban areas. The network includes a core area approximately 20 blocks square with two arterials projecting from opposite ends of the grid.

The necessity for a computer-driven display was seen early in the system planning stage. Once established, the computer interface permitted access to a significant quantity of traffic system data. However, the traffic system operator would only be interested in map display information that could be logically presented on a macroscopic basis, and the display of the detail data was assigned to the system cathode ray tubes or printers for analysis. The information displays (fig. 10) were chosen as the most desirable for the UTCS/BPS operation.

Mutually exclusive display modes, requested from an operator's console, further reduced the quantity of indicators required to present the various displays. Only 1 of the 5 available controller display modes could be operated simultaneously with any 1 of 10 available surveillance display modes. Furthermore, the control console incorporated a series of switches that permits the operator to interrogate the selected surveillance parameter at specific thresholds (that is, locations where speed is less than 45 mph [72 km/h]). Legends indicating the particular display mode and a four-digit LED numerical readout were also incorporated within the display to inform observers of the display status and threshold.

Based on the analytic aspects of the system, indicators tracking the turning or staggered traffic signal timing on complex intersections were desired in addition to the basic "A Phase Green" display. Likewise, the concentration of sensors at many of the intersections required the clustering of an above average number of indicators around a display intersection. To further reduce the quantity of sensor indicators, surveillance information was defined on a per approach (link) basis with one indicator in many instances representing information from three individual sensors.

Once the information and process of display were determined, the remaining areas of indicators, graphic technique and scale, display size, and expansion capability could be defined.

Indicators representing the traffic signals were required to be arrow shaped and to present both the green and red states of the traffic signal. The decision to present both states of the traffic signal added flexibility in the usage of the display and eliminated the confusion resulting from lamp failures. The arrow shape details the exact multiphase operation and also coincides with one- or two-way streets. Changing the direction or shape of the traffic signal indicator was required to permit updating of the display.

Arrow-shaped sensor indicators were required to depict the exact location and direction (through or turning) of the surveillance information displayed. Unlike the traffic signal indicator, sensor indicators are only illuminated to correspond to a specific parameter and threshold selection, otherwise the indicators remain extinguished.

The quantity and type of indicators desired eliminated all graphic techniques with the exception of those used with a backlighted display. In order to accommodate the number of indicators clustered at an intersection, a display scale of 1 in = 150 ft (1:1,800) was considered reasonable. By comparison, the backlighted display developed for the New Orleans traffic system permitted a display scale of 1 in = 200 ft (1:2,400) for the central business district, a scale reduction of 25 percent from the UTCS/BPS display. The scale reduction for the New Orleans display was warranted since there were no requirements for the intersection to display multiphase controller operation and a concentration of sensors.

With the display scale determined, the overall size of the graphics unit was calculated. Initial calculations indicated that a 10 ft high by 24 ft wide (3.05 m by 7.32 m) display would result if the entire control area, including the projecting arterials, were depicted at the desired scale (1 in = 150 ft [1:1,800]). A display of this size was considered unwieldy for the intended application; to reduce the overall size, the projecting arterials were depicted in two sections at a reduced scale of 1 in = 200 ft (1:2,400). This approach permitted the display size to be decreased to 6 ft high by 18 ft wide (1.8 m by 5.5 m).

The physical location of the display with respect to the observer was considered next. Ideally, a perpendicular view angle of the entire display unit from the operator's position is desired. After several layouts were generated, a three-section angled configuration, consisting of a 8-ft (2.4 m) center section and 5-ft (1.5 m) sections angled toward the operator from each side of the center, was selected as a satisfactory approach (fig. 11).

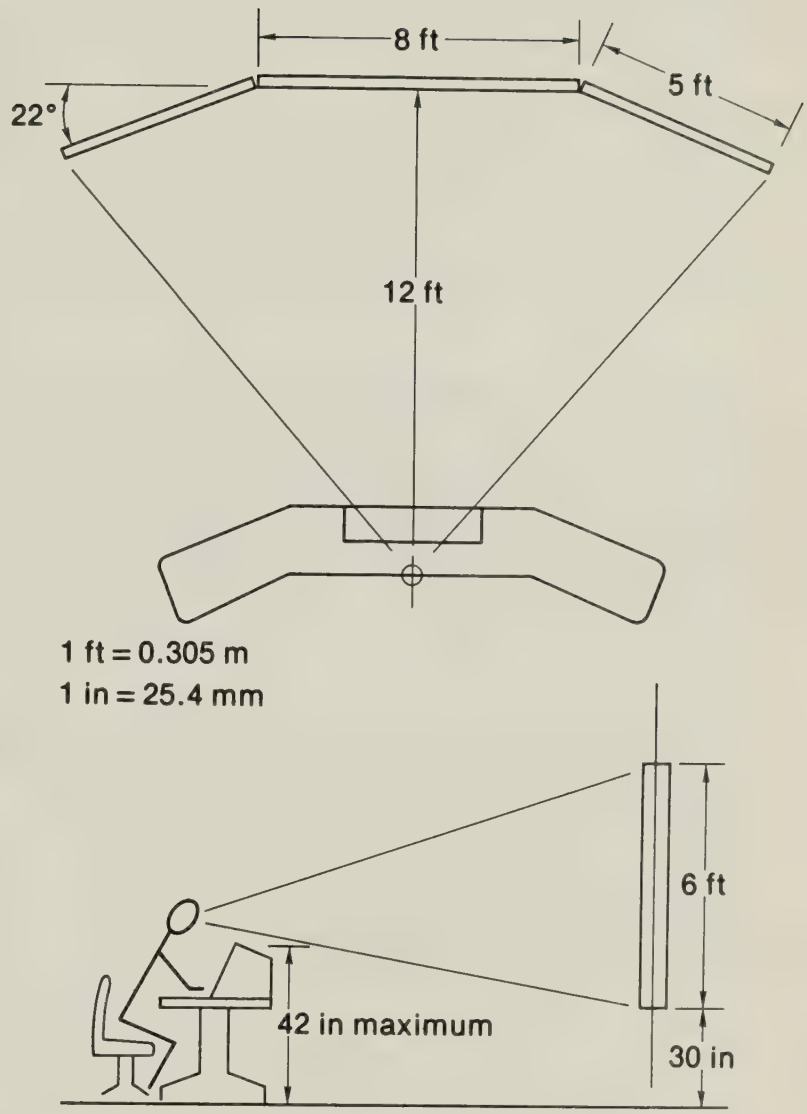


Figure 11.—Layout of UTCS display.

The need for expansion was recognized and specified accordingly. The method of expansion varies depending upon the particular techniques available to the display contractor. For the UTCS/BPS display, the contractor chose to mount the artwork on an aluminum backplate, and holes were predrilled at all intersections to provide for the addition of traffic signal and sensor indicators. To further accommodate future expansion, the mylar artwork was installed in six individual sheets that can be removed to permit backplate drilling or artwork replacement. Finally, to account for major modifications, four individual aluminum backplates were installed. Each is screw-fastened to the display unit frame to allow for backplate replacement without dismantling the entire display unit.

Trade offs for defining the computer interface were conducted after the total number of indicators, including those for expansion, were estimated. It was estimated that 650 lamp drivers were required to accommodate the actual system installation and near term expansion, with the capability of adding another 350 lamp drivers for future growth.

The application of an electronics unit was concluded to be the most practical in establishing the computer to display interface. The cost of the electronics unit was comparable with the direct drive approach and with the particular equipment configuration anticipated for the system; additional display features were possible. One important feature is the ability to display "A Phase Green" information directly from the communications equipment during periods when the computer is offline.

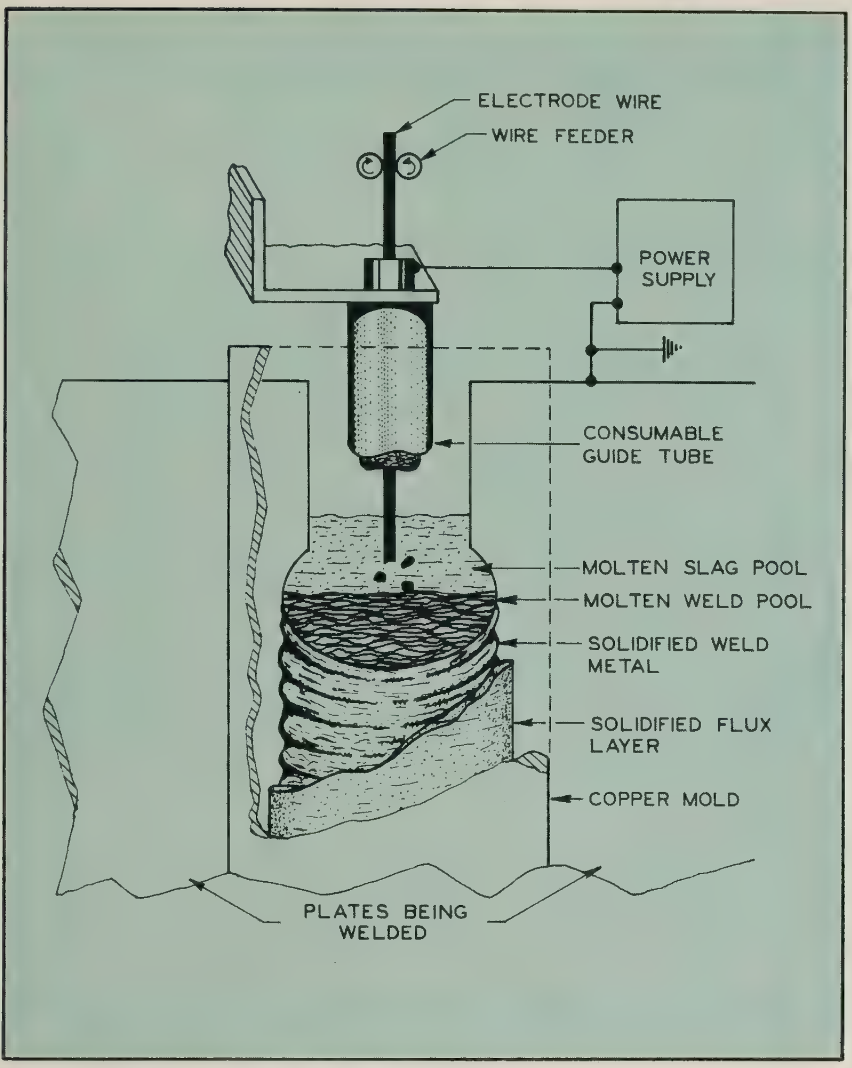
As with the display unit, the ability to expand was recognized and incorporated within the equipment specifications. The electronics unit furnished for the UTCS/BPS system had sufficient electronics and cabling to permit near term expansion by inserting additional printed circuit boards. Further expansion is accomplished by adding a plug-in prewired chassis and an appropriate quantity of circuit boards.

The power supplies furnished with the unit are capable of supplying power for a completely expanded configuration of approximately 1,000 incandescent indicators.

Procurement Considerations

With the exception of less expensive cut-and-paste standard component assemblies, traffic system displays are usually built to specifications by manufacturers that specialize in display systems. To avoid conflicts that occur from interpreting loosely defined terms, the procurement specification should clearly state the desired design and constraints including size, readability, view, color, and ambient light conditions. Prior to specifying the preferred display approach for the particular traffic system application display, vendors should be interviewed to establish a working knowledge of the available display techniques. Also, the manufacturers may suggest alternate approaches to satisfy the design requirements with working models presented for evaluation. These discussions should provide the cost and scheduling information required for trade offs.

After a particular manufacturer is selected, close cooperation is mandatory. All facets of the display design fabrication and installation should be carefully reviewed and discussed with the manufacturer to insure that the goals of the display are achieved.



Schematic of consumable guide electroslag welding.

Electroslag Weldments: Performance and Needed Research

by James D. Culp

This article is the second part of a paper presented at the 1977 Federally Coordinated Program of Highway Research and Development Conference at Atlanta, Ga., during the review of Project 5L, "Safe Life Design for Bridges." The objectives of this new project are to refine and improve the existing fabrication and inspection techniques through the development of new instrumentation, develop guidelines for effective quality control, and improve the safety and performance of structures. The first part of this paper, which appeared in the March 1978 issue of *Public Roads*, included a process description, Michigan's fabrication experience, problem areas, and some research results. This article includes additional research results and research needs.

Research Results

Charpy V-notch impact evaluation

Series II—Energy transition-temperature testing. The Charpy impact energy of a steel or weld metal is dependent on the test temperature of the specimen. The energy transition-temperature characteristics of the electroslag weldments were studied over the temperature range of -40° F to +40° F (-40° C to 4° C), that is, the lower range of expected service temperatures for the Northern United States. Acceptance criteria often specify a minimum Charpy impact energy, such as 20 ft-lb (27 J), at some test temperature, such as 0° F (-18° C). This is equivalent to requiring the weld metal to possess a 20 ft-lb (27 J) transition-temperature of 0° F (-18° C) or below. This implies that any testing on the material above 0° F (-18° C) will yield energies equal to or greater than 20 ft-lb (27 J), and any testing below 0° F (-18° C) will yield energies equal to or less than 20 ft-lb (27 J). However, if the service temperature of the material goes significantly lower than 0° F (-18° C), say -30° F (-34° C), then the amount of decrease in energy below the 0° F (-18° C) level may be very important to the performance of the structure.

Electroslag weldments similar to those tested in the previous acceptance testing (*1*)[1] were tested over the temperature range of -40° F to +40° F

[1] Italic numbers in parentheses identify references on page 23.

(-40° C to 4° C). These weldments were made in 3-in thick by 16-in wide (76 by 406 mm) plates. The weldments were sectioned into five blocks and Charpy specimens (in sets of five) were removed from all the various weld metal, heat-affected, and base metal zones in each block. Each weldment zone was then tested at the five temperatures -40°, -20°, 0°, +20°, and +40° F (-40°, -29°, -18°, -7°, and +4° C). The results of these tests are plotted as energy transition-temperature curves in figures 1 through 4. Any trends in the transition-temperature curves that show an increase in the impact energy with a decrease in temperature are due to one of two reasons. First, all of the zones as defined are still somewhat nonhomogeneous, which leads to such variations. Second, due to the narrow width of some of the heat-affected zones, part of an adjoining zone sometimes was inadvertently included in the specimen and slightly biased the breaking energy.

A comparison of the differences in results using the cooled shoe and using the dry shoe methods in electroslag weldments in A 588 steel is discussed first. Results of the cooled shoe method (fig. 1) show that the Zone 1 weld metal (thin columnar crystals oriented in the direction of welding) exhibits 20 ft-lb (27 J) at +40° F (4° C) and drops to 5 ft-lb (6.8 J) at -40° F (-40° C). Note that this Zone 1 falls short of the required 15 ft-lb (20 J) at 0° F (-18° C) as was the case in the previous Series I on acceptance testing. (*1*) The dry shoe weldment in figure 2 had no Zone 1 weld metal.

The Zone 2 weld metal (coarse columnar crystals oriented at an acute angle with the direction of welding) exhibits very high toughness in both the cooled and dry shoe weldments. Even at -40° F (-40° C), the absorbed energies are 15 and 30 ft-lb (20 and 41 J), respectively. The difference in impact strength exhibited by these two zones of weld metal is mainly due to the orientation of their long grain boundaries with respect to the direction of crack propagation in the Charpy test; this will be demonstrated in the anisotropic test series.

Figure 1 shows that the first heat-affected zone, HAZ 1 (a zone of grain coarsening due to overheating by the adjacent weld metal), exhibits higher toughness than the unaffected base metal at all temperatures. This can be explained by the fact that even though the primary structure of HAZ 1 is coarser than that of base metal, the secondary structure is much finer, thus actually improving the impact toughness of the moderately tough base metal. On the other hand, HAZ 1 had the opposite effect on A 588 base metal with extremely high toughness (fig. 2). This illustrates that in a high toughness base metal, the grain coarsening present in the HAZ 1 primary structure can degrade the toughness and it points out the need to critically evaluate the effects of the heat-affected zones on the properties of the base metal when using electroslag welding.

The effect of these heat-affected zones is always going to be relative to the initial properties of the base plate being welded. As would be expected by the grain refinement present in HAZ 2, its impact strength is greatly elevated over the parent metal. It is interesting to

Figure 1.—Energy transition-temperature curves for a cooled shoe electroslag weldment made in 3-in (76 mm) A 588 steel.

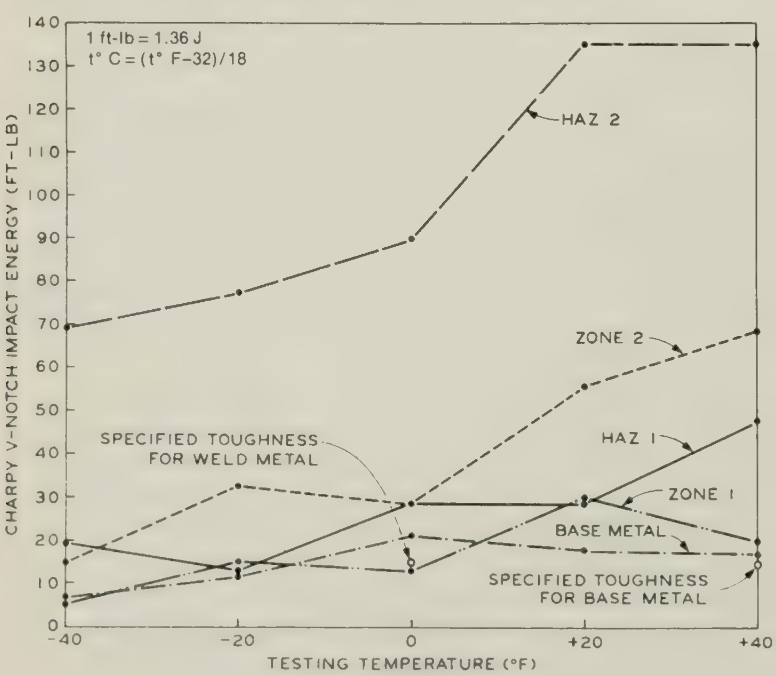
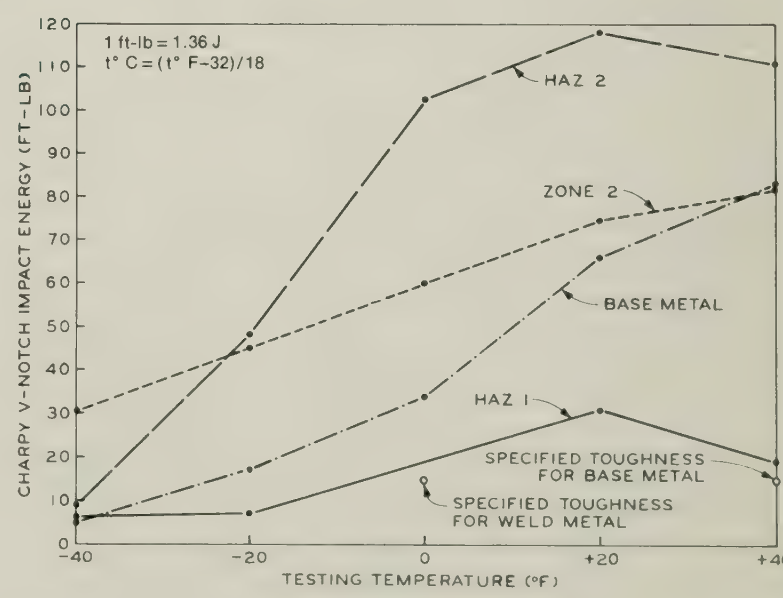


Figure 2.—Energy transition-temperature curves for a dry shoe electroslag weldment made in 3-in (76 mm) A 588 steel.



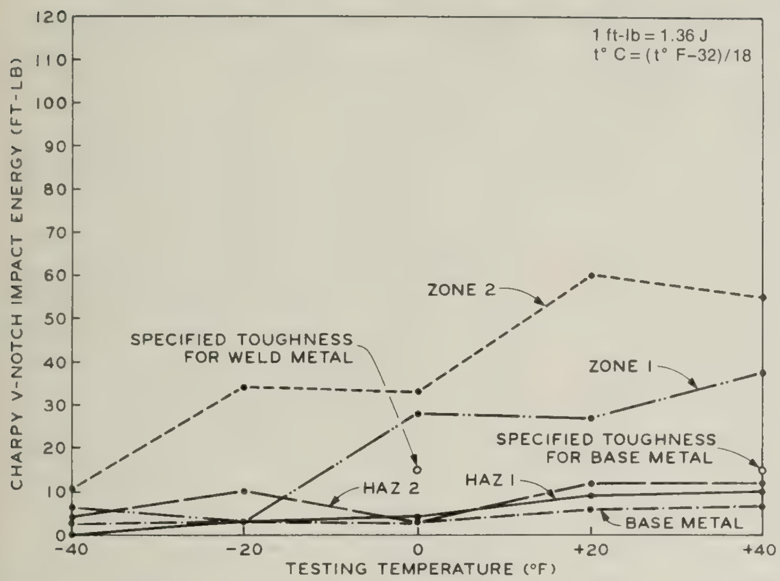


Figure 3.—Energy transition-temperature curves for a cooled shoe electroslag weldment made in 3-in (76 mm) A 36 steel.

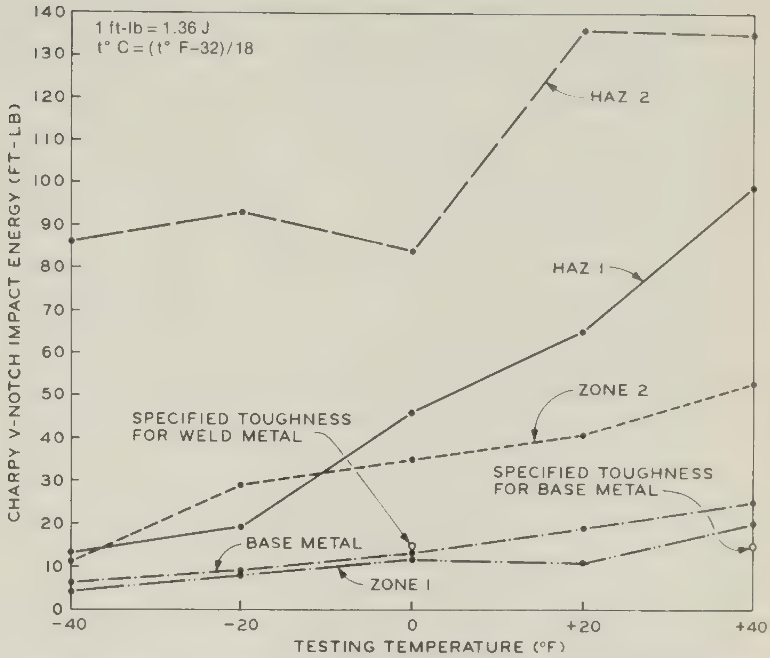


Figure 4.—Energy transition-temperature curves for a dry shoe electroslag weldment made in 3-in (76 mm) A 36 steel.

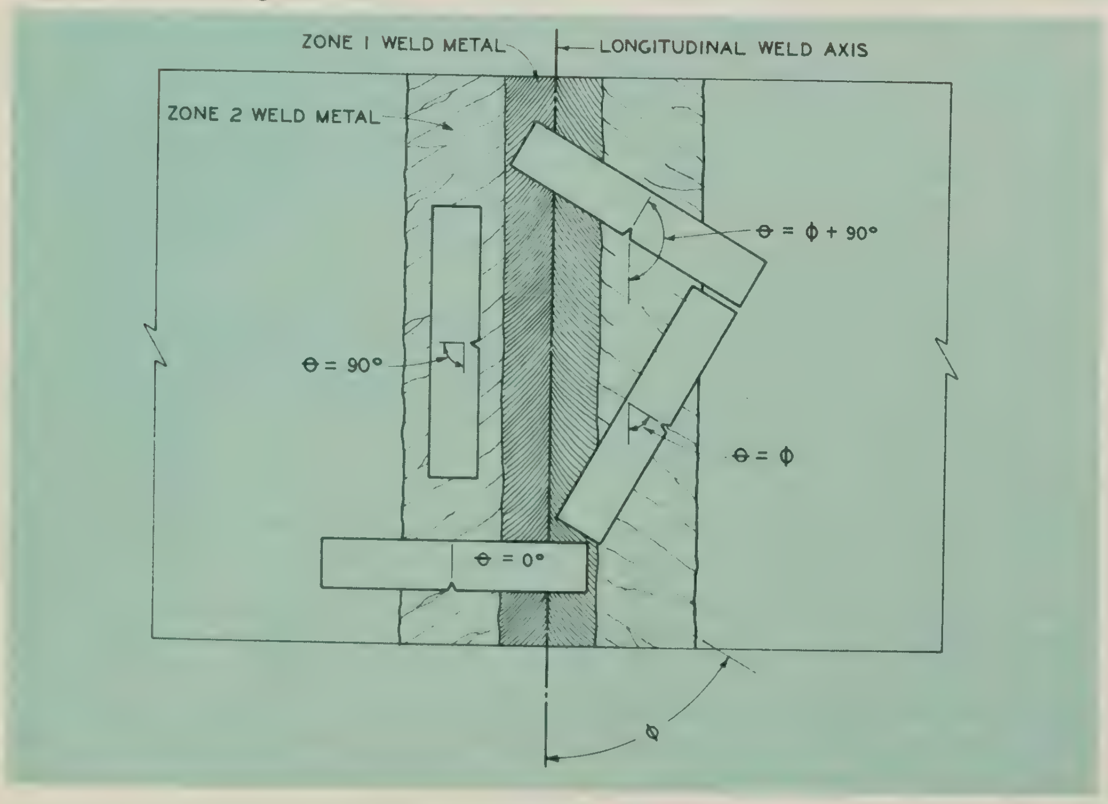
note that the HAZ 2 in figure 2 (dry shoe weldment) drops down to the base plate toughness at -40° F (-40° C) while the HAZ 2 in figure 1 (cooled shoe weldment) remains at 70 ft-lb (95 J) at -40° F (-40° C). Comparing the transition curves for the base plate, we see that even though the plate in figure 2 is extremely tough at temperatures above 0° F (-18° C), the plates are almost equivalent at -20° F (-29° C) and below. This illustrates the fact that setting a certain toughness requirement on steel at some arbitrary temperature, such as 40° F (4° C), does not necessarily account for its behavior at lower temperatures since the energy transition can vary drastically with decreasing temperature.

The next comparison to be made is of the electroslag weldments made in A 36 steel for the cooled shoe method (fig. 3) and for the dry shoe (fig. 4). As seen in figure 3, Zone 1 weld metal has high impact strength at 0° F (-18° C) and above but falls off rapidly to around 3 ft-lb (4 J) at -20° F (-29° C). Its resistance to brittle fracture at a service temperature of -20° F (-29° C) would be expected to be low, even though the weld meets the requirement of 15 ft-lb (20 J) at 0° F (-18° C). Note that this Zone 1 weld metal tested considerably higher at 0° F (-18° C) than a similar weldment, ES 36–A2. (*1*) The Zone 1 weld metal in the dry shoe weldment exhibits low toughness (fig. 4). It fails to meet the required 15 ft-lb (20 J) at 0° F (-18° C) and remains inferior to base metal throughout the entire temperature range. The Zone 2 weld metal for both welding methods has very high toughness, not dropping below 30 ft-lb (41 J) until tested below -20° F (-29° C). Again it is the orientation of the columnar crystals that causes the drastic difference in the impact strengths of the two weld metal zones. The effects of the heat-affected zones in A 36 steel are again dependent on the toughness of the base plate itself. Figure 3 shows that both HAZ 1 and HAZ 2 have only a slight improvement over an extremely low toughness base plate. However, figure 4, where the base plate welded had a moderately high toughness, shows that HAZ 1 and HAZ 2 improve the toughness dramatically as was the case in the A 588 steel. We would further expect, as was shown in A 588 steel, that in an A 36 plate of very high toughness the HAZ 1 would degrade the toughness due to its grain coarsening of the primary structure. The base metal itself represents low toughness A 36 steel that would possess little resistance to brittle fracture (fig. 3). In figure 4, the base plate exceeds the required 15 ft-lb (20 J) at 40° F (4° C) and would be much more reliable in service. (Until recently there were no toughness requirements on A 36 steel and many structures with thick plates, some with electroslag welds, are in service today with toughness curves similar to that shown in figure 3.)

The pertinent question, concerning how much Charpy V-notch impact energy at what temperature will preclude the possibility of brittle fracture in a welded structure, is undergoing scrutiny at the present. One opinion is that some minimum acceptable value, such as 20 ft-lb (27 J), should be provided by the material at its lowest service temperature. A less conservative point of view, usually employed in bridge construction, is that if some toughness level is specified, say 20 ft-lb (27 J), at a temperature above the lowest service temperature, such as 0° F (-18° C), then we have "screened out" the brittle material and will get adequate toughness from the rest. The shortcoming of this approach is that one has no idea of how the energy transitions below the testing temperature, which can vary considerably as shown in the previous discussion. The most recent and promising approach to the problem is that proposed by Barsom and Rolfe (*2, 3*) who have established the validity of a relationship between the Charpy V-notch test and the plane strain fracture toughness, K_{IC}, as defined by American Society for Testing and Materials (ASTM) E 399, for certain structural steels. This approach also accounts for the relationship between the strain rate experienced in service loading and that applied in testing. If these correlations prove valid for weld metal, toughness criteria can be based on the Charpy impact test and related to inherent weld metal toughness as measured by the K_{IC} test. This approach would allow the actual "design" against brittle fracture occurring in a structure. However, before such an approach can be assumed, much work needs to be done to establish the temperature-transition characteristics of weld metal toughness as measured by the Charpy V-notch specimen and the K_{IC} specimen and to establish how they relate.

Series III—Anisotropic properties of electroslag weld metal impact toughness. Grain structures such as those present in Zone 1 and Zone 2 electroslag weld metal are highly susceptible to anisotropies in mechanical properties. Such directional variation is expected in the property of impact toughness because the large columnar grain boundaries may provide a path of low resistance to crack propagation. These grain structures undoubtedly will affect the tensile properties of yield point, tensile strength, and ductility as measured by percent elongation or reduction in area, although no testing was done to quantify these effects.

Figure 5.—Specimen orientations used in testing the anisotropic nature of the Charpy impact toughness of electroslag weld metal.



Three types of electroslag weldments were tested for variations in Charpy impact toughness relative to variations in direction of crack propagation with respect to the long axis of the columnar crystals of Zone 1 and Zone 2 weld metal. Figure 5 defines the specimen orientations used in assessing these directional variations. The angle θ is taken to be the angle between the direction of crack propagation in the Charpy specimen and the longitudinal axis of the weld. The standard orientation for impact testing of the weld metal is $\theta=0°$. The angle ϕ is the angle between the long axis of the Zone 2 coarse columnar crystals and the longitudinal axis of the weld. Zone 1 weld metal was tested at the angles $\theta=0°$, which gives the crack a direction that is nearly parallel to the fine columnar crystals present in the welds tested, and $\theta=90°$ which causes fracture across the grains. Zone 2 weld metal was tested at the angles $\theta=0°$, 90°, $\phi+90°$, and ϕ (fig. 5). The results of these tests are shown in table 1.

In the electroslag weldment made by the cooled shoe process in A 36 steel (ES 36–A2), the Zone 1 weld metal tested in the standard testing direction, $\theta=0°$, had a toughness of 10 ft-lb (13.6 J) at 0° F (-18° C). However, testing at $\theta=90°$, the same Zone 1 weld metal had a toughness of 41 ft-lb (55.6 J) at 0° F (-18° C). Thus, in the A 36 alloy, the Zone 1 weld metal exhibited four times greater toughness in a direction transverse to its columnar grain structure than it had in a direction nearly parallel to the long axis of the grains. In the similar electroslag weldment made in A 588 steel (ES 588–A2), the Zone 1 weld metal had a toughness of 22 ft-lb (30 J) at the angle $\theta=0°$ and 19 ft-lb (26 J) at the angle $\theta=90°$. Thus the alloy composition of A 588 steel seems to compensate for the anisotropic effects of the Zone 1 grain structure on impact toughness. This is understandable because the amounts of nickel and chromium in A 588 steel could strengthen the prior austenite grain boundaries (that is, ferrite bands)

Table 1.—Anisotropic Charpy impact test results on electroslag weld metal

(*Test temperature 0° F [-18° C]*)

Weldment type	Impact toughness in the θ direction $\theta=0°$	$\theta=90°$	$\theta=\phi+90°$	$\theta=\phi$
	ft-lb	*ft-lb*	*ft-lb*	*ft-lb*
ES 36–A2				
Zone 1 weld metal	10(3)[1]	41(2)		
Zone 2 weld metal	51(3)	52(1)	54(3)	25(3)
ES 36–B2				
Zone 1 weld metal	N.T.[2]	N.T.		
Zone 2 weld metal	69(3)	N.T.	64(3)	33(2)
ES 588–A2				
Zone 1 weld metal	22(3)	19(2)		
Zone 2 weld metal	33(3)	32(5)	34(3)	23(3)

1 ft-lb = 1.36 J

[1]Numbers in parentheses denote the number of specimens tested to give the average value shown.

[2]N.T. denotes that the zone was not tested.

enough to overcome their unfavorable orientation. Thus, the Zone 1 weld metal is anisotropic in impact toughness in A 36 steel, but the alloy composition of the A 588 steel seems to compensate for the crystal anisotropy.

Zone 2 weld metal exhibits similar characteristics. In weldments ES 36–A2 and ES 36–B2 (electroslag welds in A 36 steel produced by the cooled shoe and dry shoe methods, respectively), the lowest toughness is obtained by propagating the Charpy test crack in the direction $\theta=\phi$, which is parallel to the long axis of the grains and forces the fracture along the grain boundaries. In weldment ES 36–A2, the directions $\theta=0°$, 90°, and $\phi+90°$ are all transgranular types of fractures and produce twice the impact toughness of that measured at $\theta=\phi$. In weldment ES 36–B2 the test directions $\theta=0°$ and $\phi+90°$ produce approximately twice the toughness of the direction $\theta=\phi$. In weldment ES 588–A2, a cooled shoe weld in A 588 steel, the directions $\theta=0°$, 90°, and $\phi+90°$ give approximately a 40 percent increase in the impact toughness as compared to that measured in the direction $\theta=\phi$. The alloy strengthening of the grain boundaries in A 588 steel is still present but not as effective as it was in the Zone 1 weld metal. Therefore, the large grain structure present in the Zone 2 weld metal produces a significant reduction in the impact toughness in the direction parallel to the long axis of the grains but is partially compensated for in A 588 steel by the alloying effects in the grain boundaries.

When an electroslag butt joint weldment is loaded in uniaxial tension, such as in a bridge beam, the anisotropies that have been shown to exist are not too serious. The Zone 1 weld metal has a severe weakening of impact properties along the main axis of the weld ($\theta=0°$) but fortunately this is the direction of crack propagation in the standard acceptance testing. The problem that does exist is that most specifications do not require testing of the Zone 1 weld metal. An additional problem is the probable anisotropic nature of the Zone 1 weld metal ductility that was mentioned in the discussion of tensile properties. (*1*) The plane of minimum toughness in the Zone 2 weld metal will usually lie between 50° and 65° (that is, $\theta=\phi$) from the longitudinal axis of the weld. Under the normal uniaxial loading condition, the toughness measured at $\theta=0°$ corresponds to the applied loading. However, if an electroslag weldment is put into a highly restrained joint or a state of biaxial loading, the low toughness measured along the axis of the columnar crystals could become critical. Careful analysis and testing should be carried out if such an application were to arise in an electroslag welded joint. As a precaution, fracture toughness testing should be carried out in all directions whenever "as welded" electroslag joints are subjected to a biaxial or triaxial stress condition.

Another point that needs to be made is that the anisotropic nature of electroslag weld metal properties is dependent on the alloy composition. No generalization can be made to predict the direction of minimum strength (impact or tensile) with respect to the grain structures present. Only by testing in the various directions can this be determined. An additional problem arising in the use of "as welded" electroslag butt joints is the variation that occurs along the length of the weld in the angle ϕ. A slight shift in ϕ could significantly alter the state of stress in the plane of the columnar grain boundaries. Such fluctuations in the state of stress along the ϕ direction would have to be accounted for in the original design to preclude the possibility of exceeding a failure condition along this plane (especially in biaxial or triaxial stress conditions). The primary lesson to be learned here is that the coarse grain structure resulting from this type of welding can significantly alter the properties of the metal, depending on the location and orientation of the applied stress or strain. This could have serious consequences, especially in highly restrained weldments subjected to biaxial or triaxial states of stress.

Fatigue-notch sensitivity evaluation

A program of constant amplitude, axial fatigue testing of notched tensile specimens was conducted to determine the qualitative effects of the various metallurgical zones present in electroslag and submerged arc weldments in ASTM A 588 and A 36 steels on fatigue crack initiation from a preexisting flaw or discontinuity. In fatigue failures of bridge beams, the fatigue cracks usually originate at some fabrication flaw or geometric discontinuity and the major part of the life of a beam is consumed in the initiation of a fatigue crack from the flaw or discontinuity. The notched specimen fatigue work was designed and conducted in accordance with ASTM E 466, "Constant Amplitude Axial Fatigue Tests of Metallic Materials." The objective of this evaluation was in accordance with the stated "significance" of ASTM E 466 that such tests ". . . may be used as a guide to the selection of metallic materials for service under conditions of repeated direct stress." (*4*) In other words, this type of test is recognized as a qualitative method of ranking various metallurgical zones in the weldments with respect to resistance to crack initiation in the presence of a controlled notch condition. These results cannot be directly applied to design conditions in a girder because the conditions of service loading are not exactly paralleled in the test. However, the results can reveal any significant differences in the fatigue susceptibility of the various weldment zones, and differences among welds produced by different welding processes.

The results of this testing are discussed in detail in reference 5. The main conclusions are as follows:

A 588 steel weldment:

(1) At a 42.5 ksi (293 MPa) stress range with a circular crack starter (theoretical elastic stress concentration factor, $K_t = 2.6$), the electroslag weld metal zones and HAZ 2 had lower cycle lives than the base metal. The cycle lives of the submerged arc weld metal (made in identical base metal as the electroslag weldments) and its HAZ exceeded the life of the base metal and were at least two times greater than the lives of the electroslag weld metal.

(2) At a 21 ksi (145 MPa) stress range with a double edge notch crack starter ($K_t = 28$), both of the electroslag weld metal zones had cycle lives that were greater than or equal to those of the base metal. In one case the HAZ 2 had a lower cycle life than the base metal. The submerged arc weld metal and its HAZ had cycle lives greater than or equal to the cycle life of the base metal. The submerged arc weld metal had a significantly higher life than the corresponding electroslag weld metal zones.

A 36 steel weldment:

(1) At a 38 ksi (262 MPa) stress range with a circular crack starter ($K_t = 2.6$), both the electroslag weld metal zones and HAZ's exceeded the cycle life of the base metal. Likewise the cycle lives of the submerged arc weld metal (made in identical base metal as the electroslag weldments) and HAZ exceeded that of the base metal. The submerged arc weld metal had a significantly higher cycle life than the corresponding electroslag weld metal.

(2) At a 28 ksi (193 MPa) stress range using a double edge notch crack starter ($K_t = 28$), both of the electroslag weld metal zones and HAZ 2 have cycle lives less than or equal to base metal. The submerged arc weld metal and its HAZ at this condition are still greater than or equal to base metal in cycle life. The submerged arc weld metal is seen to have a higher cycle life than the corresponding electroslag weld metal.

Based on the results of these tests, it is concluded that, qualitatively, the submerged arc weld metal is more resistant to fatigue crack initiation than electroslag weld metal in the presence of a flaw condition. The electroslag weld metal was in some instances less resistant to fatigue crack initiation than the base metal welded. This conclusion must be qualified within the limitations of the test conducted. Because the testing conditions do not parallel service loading conditions it cannot be stated that this difference would appear in service. However, these results indicate that the electroslag weld metal is susceptible to fatigue crack initiation in the presence of a flaw at a lower than expected number of loading cycles.

Research Needs

Based on Michigan's experience with electroslag welding of butt splices in steel plate girders and recent observations of field performance, the process does not appear to be suitable for use for tension members. Any efforts to overcome the deficiencies of the weldment, such as post weld heat treatment or the use of "super alloy" types of filler metal, do not seem warranted because of the small economic advantage of the process over the alternate submerged arc welding process. The following four research areas are essential to the understanding and evaluation of existing electroslag welded structures and are vital to any consideration of future use of the process. If proper knowledge of these areas were obtained, the electroslag process could be applied to compression butt weldments and hence realize whatever economics may be offered in areas where fatigue and brittle fracture considerations would be minimal.

• *Nondestructive testing (NDT) of electroslag weldments*. Considerable concern has been generated recently over the failure of standard NDT methods (that is, X-ray and ultrasonic testing) to adequately inspect the electroslag weld metal for rejectable flaws. Several recent investigations have determined that some flaws overlooked by X-ray are picked up by ultrasonics and vice versa.[2,3] This is

[2] "Acceptance Criteria for Electroslag Weldments in Bridges," by W. P. Benter, Jr., et al. Phase I final report of NCHRP Project 10-10, not yet published.

[3] "Bridge Girder Butt Welds—Resistance to Brittle Fracture, Fatigue, and Corrosion," Michigan Project 75 F-144 in cooperation with the Federal Highway Administration, Highway Planning and Research Program, initiated July 1976.

most likely attributable to the X-ray's failure to delineate the "intergranular cracking" that can occur in electroslag weld metal and the increased attenuation and backscattering of ultrasonic pulses through the coarse grain structures. These problems need to be carefully resolved before reliable inspection can be achieved.

• *Fracture toughness and related properties*. Before a fracture-safe design of electroslag weld metal can be achieved, a complete understanding of its fracture toughness characteristics must be determined. This information is particularly crucial to the evaluation of existing bridge structures fabricated by the electroslag process. Specifically, the following research information is needed:

(1) Fracture toughness testing of electroslag weld metal using either linear elastic techniques to measure K_{IC} or elastic plastic techniques to measure J_{IC} at temperatures and strain rates typical of bridge applications. These evaluations must assess the nonhomogeneous and anisotropic natures of the electroslag weld structure and how they influence design considerations. The end goal of such an evaluation would be to estimate a critical flaw size that the electroslag weld metal can tolerate before unstable crack propagation occurs and to specify toughness requirements to assure a reasonable factor of safety against brittle fracture. Michigan's current research project[4] includes this goal among its objectives for both electroslag and submerged arc weldments.

[4]"Bridge Girder Butt Welds—Resistance to Brittle Fracture, Fatigue, and Corrosion," Michigan Project 75 F-144 in cooperation with the Federal Highway Administration, Highway Planning and Research Program, initiated July 1976.

(2) The energy transition-temperature relationships of electroslag weld metal must be studied. In particular, the possible existence of a relationship between the Charpy V-notch specimen (with or without fatigue precracking) and the fracture mechanics K_{IC} type of specimen at comparable strain rates is of great interest as are the effects on the energy transition-temperature curves with the varying of strain rate. It cannot be assumed that the relationships currently accepted for base metals are equally valid for weld metal, especially in a complex structure like electroslag weld metal. These relationships also can be expected to vary with the alloy composition of the weld metal.

• *Corrosion behavior of electroslag weld metal in unpainted exposures*. Unpainted electroslag weldments in ASTM A 588 steel bridges are prevalent. The potential for increased corrosion attack of the complex crystal structures of electroslag weld metal is a serious problem, especially in light of some of the alloy deficiencies that have been pointed out. Before confidence can be placed in the corrosion resistance of such unpainted welds, this problem needs to be examined. This area is also undergoing scrutiny in Michigan's current research,[5] and close surveillance is planned for the many existing electroslag welded structures in the unpainted condition.

• *Fatigue properties*. There still is little qualitative information on the fatigue properties of electroslag weld metal. Of special concern is the resistance of the various weldment zones to the initiation of fatigue cracks from the inevitable "flaws" which escape inspection and repair in fabrication. Sometimes a flaw repair itself can become a serious source of fatigue crack initiation. Because the major portion of the fatigue life of a structural member is consumed in this initiation stage, resistance to such flaw growth is very important. Certainly existing work has shown that the electroslag weld metal is inferior to submerged arc weld metal in this area and, considering the metallurgical structures present, a serious problem could exist. A thorough inspection of existing electroslag welded structures should provide some interesting data.

[5]"Bridge Girder Butt Welds—Resistance to Brittle Fracture, Fatigue, and Corrosion," Michigan Project 75 F-144 in cooperation with the Federal Highway Administration, Highway Planning and Research Program, initiated July 1976.

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(3) J. M. Barsom, "The Development of AASHTO Fracture-Toughness Requirements for Bridge Steels," *American Iron and Steel Institute*, Washington, D.C., February 1975.

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(5) J. D. Culp, "Fracture Toughness and Fatigue Properties of Steel Plate Butt Joints Welded by Submerged Arc and Electroslag Welding Procedures," Research Report No. R-1011, *Michigan Department of State Highways and Transportation*, 1976.

Intersection of 13th East and 5th South Streets in Salt Lake City, Utah.

Analyzing Intersection Performance With NETSIM

by Willard D. Labrum, Ralph M. Farr, William J. Kennedy, Jr., and David Gibson

This article is the second of two describing a practical traffic engineering analysis tool developed by the FHWA Offices of Research and Development. This article describes an application of the NETSIM Model to an analysis of a signalized intersection by the Division of Safety of the Utah Department of Transportation, including an analysis of a phasing arrangement which could not be field tested as it was prohibited by State law. The first article, "What Network Simulation (NETSIM) Can Do For the Traffic Engineer," was published in the March 1978 issue of *Public Roads*. It described the model, what is required to use it, what it tells in return, where it may be used, and its limitations and costs.

Introduction

Attaining optimum performance of signalized intersections has long been a major goal of traffic engineers. An intersection is often signalized under difficult conditions with demands by individuals and public officials influencing the design. Political pressure for installation of special features such as left turn phases is difficult to resist when only general objections can be offered. This article will show that delay savings from 22 percent to 28 percent can be obtained through the use of simpler phasing.

In May 1973, the Division of Safety of the Utah Department of Transportation undertook a project to apply computer simulation to traffic flow problems. Preliminary investigations revealed that the Federal Highway Administration (FHWA) had developed a computer-traffic simulation program. (*1*)[1] Further effort led to acquisition of a copy of the program tape which was then used to simulate a Salt Lake City network.[2]

The computer simulation results of the Salt Lake City application were accurate enough to warrant further experimentation with the program. A high volume signalized freeway interchange in Salt Lake City was simulated to determine if delay could be reduced. The interchange was simulated with the existing controller settings. The simulation was repeated with modified signal control parameters. A 30 percent savings in delay was indicated. The improved traffic signal controller settings determined with the NETSIM simulation were then applied to the freeway interchange with a significant improvement in traffic operations.

Because of its success, the simulation program was applied to other situations, including those listed below. (*2*)[3]

- A benefit-cost analysis was made of a freeway interchange.
- Simulation was used to compare the feasibility of constructing a detour versus accepting delay by routing detour traffic through a signalized intersection.

[1] Italic numbers in parentheses identify references on page 29.

[2] An earlier version of the NETSIM Traffic Network Simulation Model, UTCS-1, was used for this study.

[3] "Use of Computer Simulation to Determine Cost Effectiveness of Detour Construction on Interstate 80," by W. D. Labrum and R. M. Farr. Unpublished report, Utah State Department of Highways, July 1974.

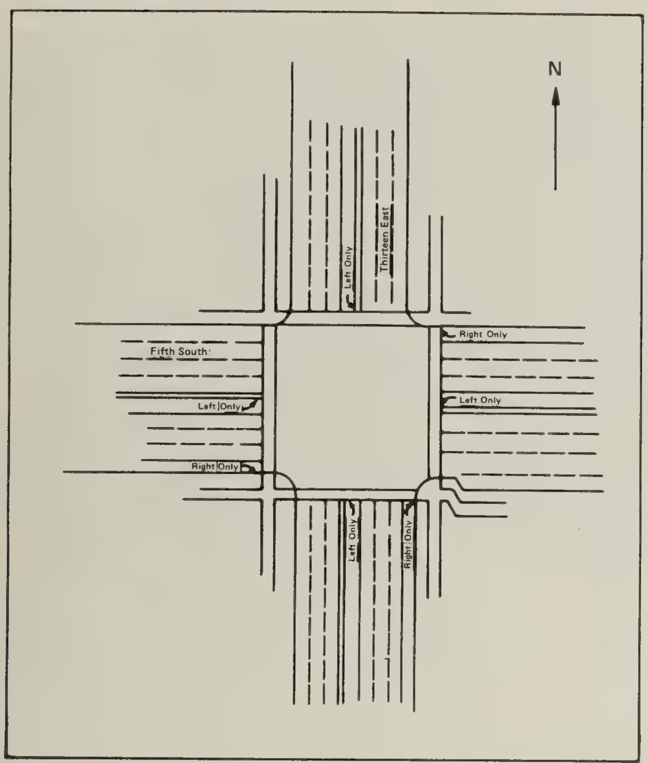

Figure 1.—Intersection plan for 13th East and 5th South Streets.

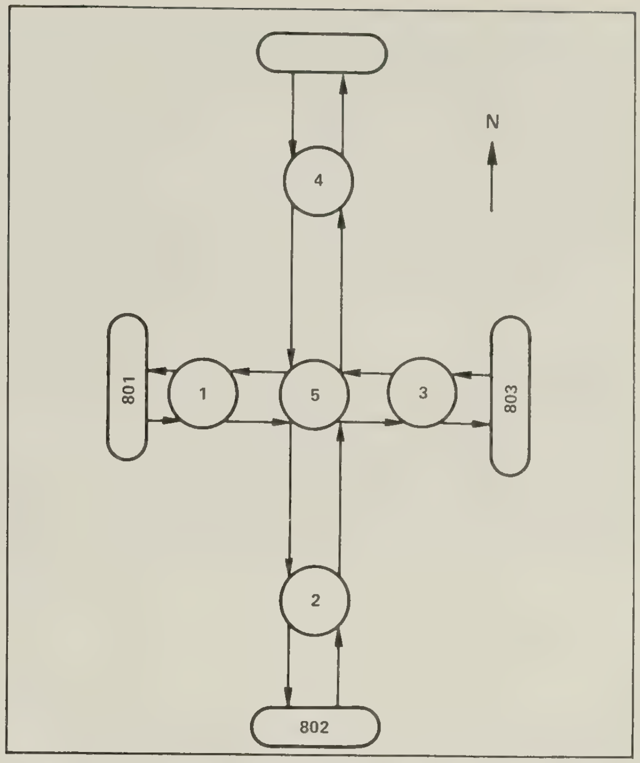

Figure 2.—Link-and-node diagram for 13th East and 5th South Streets.

- Analyzing signalized intersection performance.

This article describes the results of applying the simulation program to the third situation.

Description and Results of Study

It appeared that the NETSIM Model could be used to compare the effectiveness of different traffic control strategies proposed for an urban intersection.

Typical urban intersections in Utah consist of two wide arterial streets which intersect at right angles. Peak traffic volumes at these intersections often cause excessive delays, with vehicles queuing for long distances. The intersection of 5th South and 13th East Streets in Salt Lake City has these characteristics and was selected for this study (display photograph, figs. 1 and 2).

Fifth South (US–40) is a major east-west arterial into the Salt Lake City central business district. It carries an average daily traffic of 32,055 vehicles, varying from 2,740 vehicles per hour during the evening peak to 1,180 vehicles per hour in the offpeak periods. Thirteenth East is a heavily traveled north-south route providing access to the University of Utah. It also carries traffic from the southern suburban areas to the Salt Lake City central business district. The 1973 average daily traffic on this street was 22,530, varying from 2,540 vehicles per hour in the evening peak to 1,020 vehicles per hour in the offpeak daytime periods. Additional smaller peaks are associated with activities at the university.

Five traffic control strategies for this intersection were compared using the NETSIM Network Simulation Model. Two strategies assumed the use of existing equipment and current Utah State Department of Highways operating policies. The three additional strategies either had been used elsewhere or were of interest to the Division of Safety. The phasing diagrams of the control strategies are shown in figure 3.

The following five control strategies were modeled:

1. *Optimized two-phase, fixed-time control—2 dial system.* Two different control plans per day were considered. The northbound and southbound traffic moved during the first phase; the eastbound and westbound traffic moved during the second phase. No separate signal phases were provided for traffic turning left. These vehicles could turn left only when sufficiently long gaps occurred in the opposing traffic or as left turn green leaders or laggers. The controller setting were chosen for the peak and offpeak periods. Cycle length and phase durations were determined from traffic counts taken for the day studied. Phase splits were computed by allotting available green time in proportion to the *average* through lane flow rate on each of the critical approaches. Flow rates used were those observed during the peak 15

minutes of the time periods under consideration—a.m. peak, p.m. peak, noon peak, and offpeak. Right turning traffic was not considered in totals used to calculate means; this movement is not controlled by the traffic signal because of the "right turn on red" ordinance in effect throughout Utah.

2. *Optimized two-phase, fixed-time control—4 dial.* Four traffic patterns per day were used. This control strategy duplicated strategy 1 with four selected patterns rather than two.

3. *Four-phase actuated control—leading left.* The left turn movement leads the through movement. This was the phase sequence in use on the day traffic volumes were measured. There were four phases in this actuated control mode. In phase 1, traffic turned left from southbound to eastbound and from northbound to westbound. The length of this left turn phase depended on the number of vehicles crossing the vehicle detectors embedded in the left turn lanes. This traffic was then stopped, and phase 2 began: The northbound and southbound through and right turning traffic proceeded. When this traffic was stopped, the turning traffic from eastbound to northbound and from westbound to southbound proceeded (phase 3). When phase 3 traffic was stopped, the eastbound and westbound through and right turning traffic proceeded until stopped (phase 4). Then the four phases repeat.

4. *Four-phase actuated control with permissive left turns—leading left.* The exclusive left turn phase leads the through movement, but left turns are also allowed during the through traffic phase when gaps permit. Such a system is experimental and is not permitted under current Utah law. This strategy was of interest to determine if efficiency would increase over strategy 3.

5. *Four-phase actuated control and permissive left turns—lagging left.* The left turn phase lags or follows the through movement rather than leading it. Left turns are permitted during the through movement when gaps in the opposing traffic are adequate. This control strategy is currently used in some States and is of considerable interest to FHWA. It is illegal in Utah at the present time.

Once the control strategies and the traffic conditions to be modeled were chosen and the decision made as to the number of runs required for each set of conditions, the data input for the computer program was prepared and verified. A link-and-node diagram was drawn for the intersection (fig. 2). A node was placed at the main intersection with "dummy" nodes set at arbitrary distances from the intersecting streets. These dummy nodes enabled the program to provide meaningful statistics about the network. After the link-and-node diagram had been prepared, the computer input was coded as outlined in the user's manual for the NETSIM Model (*1*), using available geometric and traffic data.

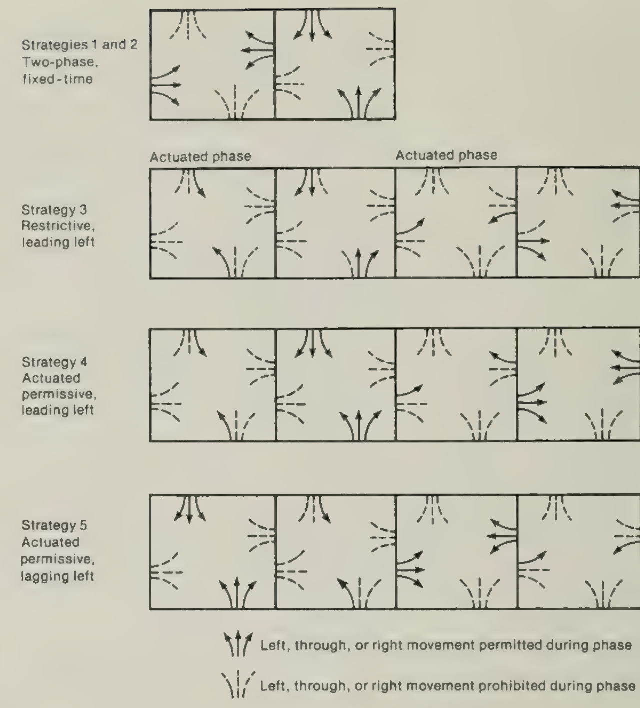

Figure 3.—Phasing diagrams for control strategies.

Before the final study runs were undertaken, the model input was verified by comparing the preliminary model output with field observations. The only field observations that could be conveniently collected were vehicle counts. Data comparison for the 7:30 a.m.–7:45 a.m. period on April 25, 1974, is presented in table 1.

The greatest difference between the simulation and the observed vehicle count was less than 2 percent. The average difference was -0.375 vehicles. These differences showed that the data had been correctly entered into the computer.

Table 1.—Model-generated vehicle counts versus observed counts

Link	Model-generated vehicle counts	Observed vehicle counts	Differenc
1,5	237	235	2
5,2	165	162	3
5,1	223	223	0
2,5	454	458	-4
5,3	310	306	4
3,5	256	260	-4
5,4	429	437	-8
4,5	179	175	4

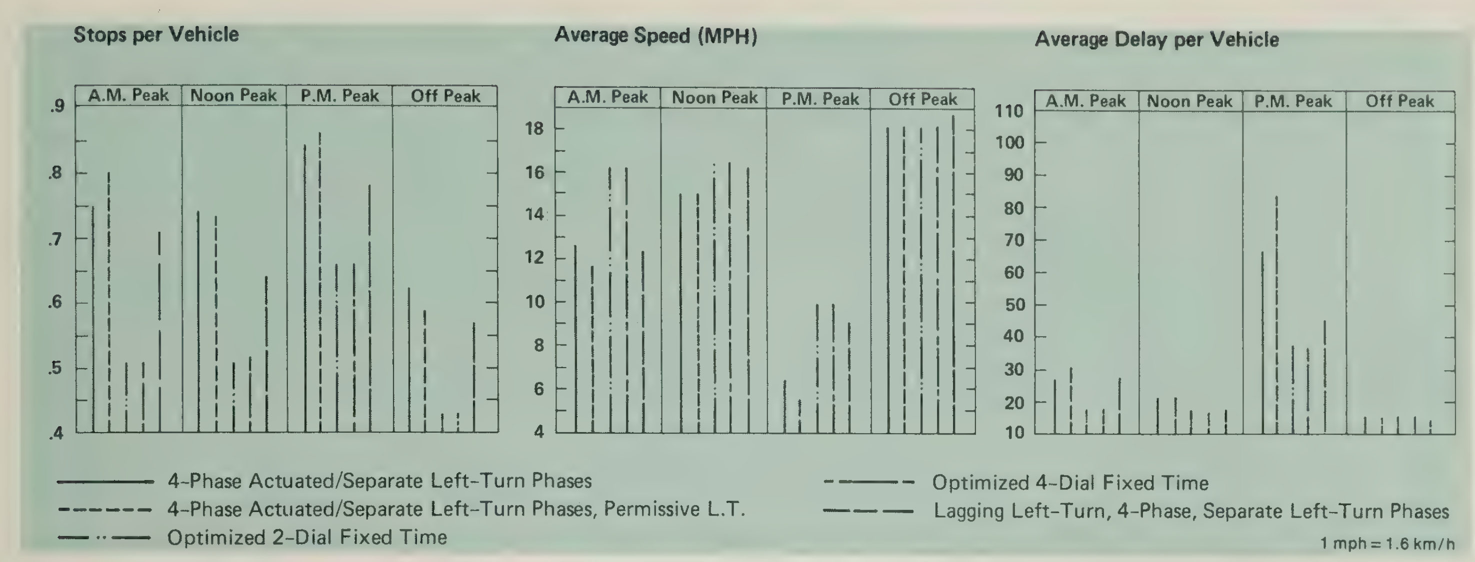

Figure 4.—Comparative efficiency measures for five signal control strategies and four traffic patterns.

After verifying that the data were being correctly read into the computer program, the chosen simulation runs were made. Three measures of traffic flow efficiency were selected from among those available in NETSIM—stops per vehicle, average delay per vehicle, and average vehicle speed through the network. These are tabulated in figure 4 and table 2. (*3*) The values shown in figure 4 are means; the variability in simulation output is shown in table 2. A statistical analysis of the data in table 2 (two-way analysis of variance together with Duncan's Multiple Range Test) showed a statistically significant difference ($\alpha = 0.01$) in traffic flow measures between the fixed time signal control strategies and the actuated strategies.

Measures of Effectiveness

Two optional measures of effectiveness are now available with NETSIM that were not in the program when this study was conducted. They are of particular relevance to the

Figure 5.—Fuel and emission statistics output.

CUMULATIVE VALUES OF FUEL CONSUMPTION AND OF EMISSIONS

LINK	FUEL CONSUMPTION						VEHICLE EMISSIONS (GRAMS/MILE)								
	GALLONS			M.P.G.			HC			CO			NO X		
VEHICLE TYPE =	1	2	3*	1	2	3	1	2	3	1	2	3	1	2	3
(2,1)	0.5	0.1	0.0	7.1	3.1	0.0	5.3	0.0	0.0	92.5	0.0	0.0	10.9	0.0	0.0
(3,1)	0.6	0.2	0.0	6.7	3.5	0.0	5.7	0.0	0.0	101.0	0.0	0.0	11.0	0.0	0.0
(1,2)	0.4	0.1	0.0	10.4	3.7	0.0	3.4	0.0	0.0	53.3	0.0	0.0	8.4	0.0	0.0
(1,3)	0.5	0.1	0.0	9.2	3.6	0.0	3.8	0.0	0.0	60.8	0.0	0.0	9.4	0.0	0.0
(4,1)	0.4	0.0	0.0	7.7	3.4	0.0	5.5	0.0	0.0	96.1	0.0	0.0	9.3	0.0	0.0
(5,1)	0.3	0.1	0.0	7.8	3.9	0.0	5.4	0.0	0.0	94.4	0.0	0.0	9.6	0.0	0.0
(1,4)	0.2	0.0	0.0	8.6	3.3	0.0	3.9	0.0	0.0	63.8	0.0	0.0	8.7	0.0	0.0
(1,5)	0.3	0.0	0.0	8.6	2.9	0.0	4.0	0.0	0.0	64.5	0.0	0.0	9.1	0.0	0.0
NETWORK-WIDE STATISTICS															
	3.10	0.61	0.0	8.15	3.50	0.0	0.62	0.0	0.0	77.93	0.0	0.0	9.65	0.0	0.0

*VEHICLE TYPE 1 = COMPOSITE AUTO, TYPE 2 = TRUCK, TYPE 3 = BUS

1 mile = 1.6 km
1 gal = 3.8 L

Table 2.—Statistical results for simulation runs

Signal phasing	Measure of efficiency	Time period[1] A.M. peak	Noon peak	Offpeak	P.M. peak	Averages
4-phase traffic actuated with separate leading left turn phase	Stops per vehicle	0.71	0.73	0.64	0.85	
		0.79	0.75	0.59	0.84	0.74
	Average speed (mph)	13.53	14.85	18.16	6.29	
		11.68	14.90	18.35	6.59	13.04
	Average delay per vehicle	24.35	21.63	15.36	68.42	
	(in seconds)	30.65	21.48	14.88	65.12	32.74
	Total delay (in minutes)	445.5	337.8	77.3	1,426.5	
		566.9	335.9	74.9	1,347.9	576.6
4-phase traffic actuated with left turn permitted during through and separate leading left turn phases	Stops per vehicle	0.80	0.73	0.59	0.88	
		0.79	0.73	0.58	0.84	0.74
	Average speed (mph)	11.95	15.02	18.29	4.44	
		11.62	14.91	18.51	6.52	12.66
	Average delay per vehicle	30.05	21.19	15.06	101.06	
	(in seconds)	30.91	21.46	14.80	65.09	37.45
	Total delay (in minutes)	560.4	331.0	76.0	2,125.5	
		573.8	335.8	75.0	1,335.3	676.6
4-phase traffic actuated with left turn permitted during through and separate lagging turn phases	Stops per vehicle	0.70	0.64	0.59	0.76	
		0.72	0.65	0.55	0.80	0.68
	Average speed (mph)	12.03	16.36	18.62	10.51	
		12.59	15.98	18.92	7.74	14.09
	Average delay per vehicle	29.40	18.42	14.61	35.91	
	(in seconds)	27.34	19.08	14.00	53.81	26.57
	Total delay (in minutes)	544.0	288.0	73.8	767.9	
		505.7	297.9	70.7	1,166.7	464.3
2-dial 2-phase fixed-time	Stops per vehicle	0.50	0.51	0.41	0.62	
		0.52	0.52	0.44	0.69	0.53
	Average speed (mph)	16.24	16.46	18.06	9.83	
		16.20	16.62	17.85	10.20	15.18
	Average delay per vehicle	18.50	18.08	15.22	38.88	
	(in seconds)	18.64	17.93	15.54	36.81	22.45
	Total delay (in minutes)	346.0	283.9	77.4	834.5	
		348.2	280.7	78.8	789.6	379.9
4-dial 2-phase fixed-time	Stops per vehicle	0.52	0.51	0.41	0.64	
		0.52	0.53	0.44	0.64	0.52
	Average speed (mph)	16.24	16.69	18.06	9.84	
		16.20	16.52	17.85	10.31	15.21
	Average delay per vehicle	18.50	17.67	15.22	38.84	
	(in seconds)	18.64	18.01	15.54	36.27	22.34
	Total delay (in minutes)	346.0	257.7	77.4	835.7	
		348.2	281.6	78.8	779.2	375.6
Averages	Stops per vehicle	0.65	0.63	0.52	0.76	0.64
	Average speed (mph)	13.83	15.83	18.27	8.53	14.1
	Average delay per vehicle (in seconds)	24.7	19.50	15.02	54.02	28.3
	Total delay (in minutes)	458.5	303.0	76.0	1,140.9	494.6

[1]Two runs were made during each time period. The values for both runs are given.

1 mph=1.6 km/h

traffic engineer in this era of energy and environmental crisis. They are fuel consumption in gallons (litres) of gasoline and air pollution in grams of pollutants emitted by the vehicles in the network. These measures of effectiveness are available on both a link-by-link basis and on a networkwide basis and are illustrated in figure 5. The fuel consumption study conducted for FHWA illustrates the broad range of impacts which ordinary traffic operations tools such as one-way streets, cycle lengths, and bus lanes may have on fuel consumption. (4)

Conclusions

The following conclusions apply only to the particular traffic flows and geometry of this intersection and not necessarily to other intersections:

- Two-phase, fixed-time signals performed more efficiently than four-phase, actuated signals for the conditions simulated. This was true even for offpeak traffic conditions.
- When the fixed-time signals did not perform better, the difference between their performance and that of the other signals did not appear to be significant. The settings for the fixed-time signals were determined after traffic counts were taken. These same settings would probably not be optimum for all traffic conditions.
- The four-dial, fixed-time settings did not perform significantly better than the two-dial, fixed-time settings.
- A signal system which permits the optimal splits to be updated on a real-time basis would perform more efficiently than a fully actuated system.
- Benefits and costs of such a real-time, two-phase system should be compared with those of actuated systems before an installation decision is made.
- Computer simulation was effectively used by traffic engineers to give practical answers to traffic flow problems. Traffic engineers determined the traffic control systems to be tested and prepared the computer data input.
- Simulation appears to be a viable tool for future traffic control investigations.

A movie demonstrating the capabilities of the NETSIM Model is available on loan from the FHWA Office of Development, Implementation Division, Washington, D.C. 20590. A sample frame is shown in figure 6.

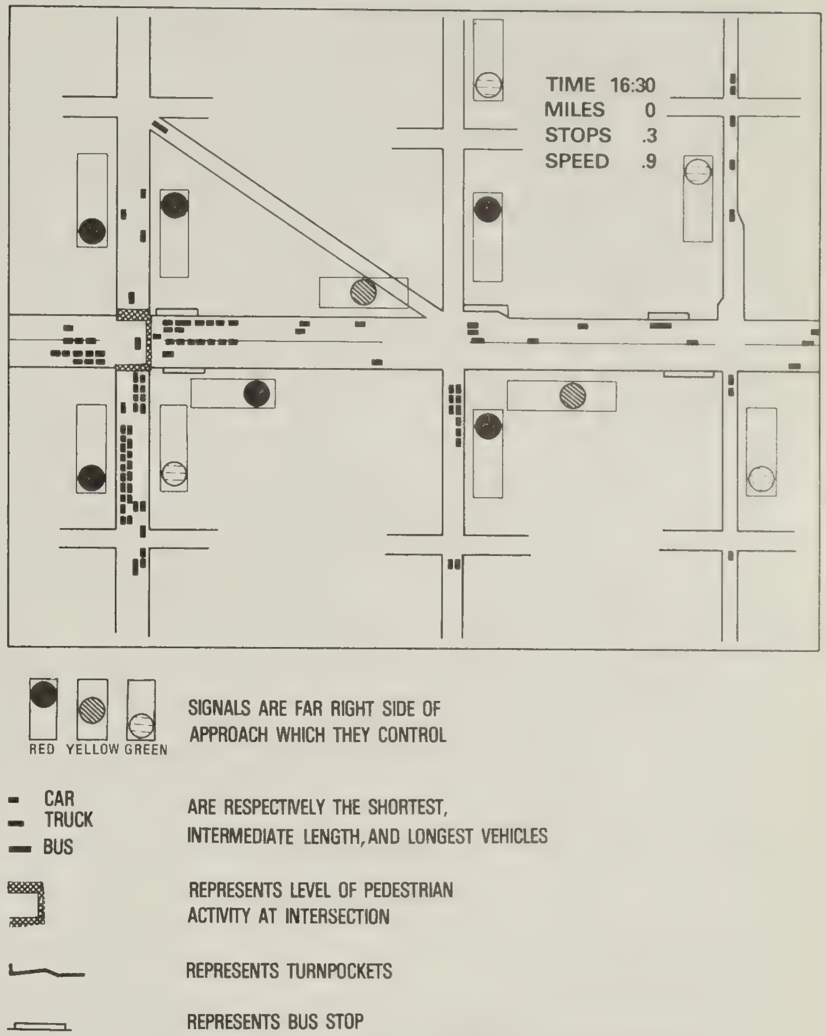

Figure 6.—Example frame from movie of NETSIM Traffic Network Simulation Model.

REFERENCES[4]

(1) R. D. Worrall and E. B. Lieberman, "Network Flow Simulation for Urban Traffic Control Systems—Phase II," Report Nos. FHWA-RD-73-83/73-87, *Federal Highway Administration*, Washington, D.C., March 1974. PB Nos. 230760-230764.

(2) W. D. Labrum, R. M. Farr, and W. J. Kennedy, Jr., "Application of Cost Effectiveness to Traffic Signalization Problems," *Proceedings of the Third Cost Effectiveness Symposium*, Washington Operations Research Council, March 1974.

(3) A. R. Pennell, "Practical Applications of Statistical Design of Experiments," *The Journal of Industrial Engineering*, vol. XIX, No. 10, October 1968.

(4) "Fuel Consumption Study," Report No. FHWA-RD-76-81, *Federal Highway Administration*, Washington, D.C., February 1976. PB No. 259003.

[4]Reports with PB numbers are available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161.

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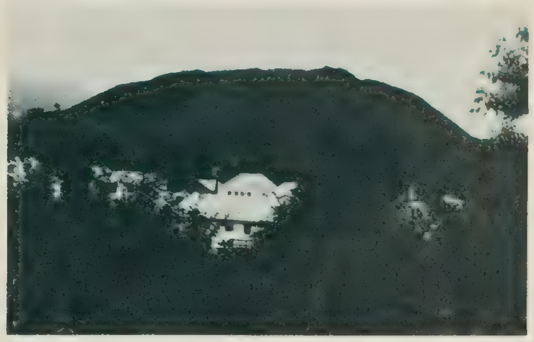
William J. Kennedy, Jr., is an associate professor of industrial engineering at the University of Utah. His primary interest is in the application of operations research techniques to the problems which arise in engineering practice. He is currently doing research in traffic simulation, production control, and energy conservation.

David Gibson is a highway engineer in the Implementation Division, Office of Development, Federal Highway Administration. Before joining FHWA in 1974, he worked for the District of Columbia Department of Transportation in traffic engineering and developed a strong interest in techniques for improving traffic flow. He is currently working on developing computer programs to control traffic signals and to compute traffic control strategies.

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 Design and Control Division. The reports are available from the address noted at the end of each description.



Preferential Treatments for High Occupancy Vehicles, Report Nos. FHWA-RD-77-99 (Moanalua Freeway, Hawaii), FHWA-RD-77-100 (Kalanianaole Highway, Hawaii), FHWA-RD-77-114 (Shirley Highway, Va.), FHWA-RD-77-148 (I-95, Miami, Fla.)

by FHWA Traffic Systems Division

A series of research and development reports has been published on the use of reserved lanes for carpools and buses. These reports provide detailed evaluations of the operations and effects of restricting the use of traffic lanes to high occupancy vehicles. This attempt to move more people in fewer vehicles will reduce highway congestion, conserve fuel, and limit air pollution from motor vehicles.

To provide an incentive for the formation of carpools or to get drivers to ride the bus, 10 minutes or more must be saved by the high occupancy vehicles on the special use lanes. Restricting these lanes to high occupancy vehicles is a problem unless the lanes are physically separated from the regular lanes, such as the Shirley Highway in Virginia. The separated Shirley Highway lanes have very few accidents related to their operation. Accident rates on the Moanalua Freeway, the Kalanianole Highway, and Miami's I-95, however, are comparable with other roadways which do not have restricted high occupancy vehicle lanes.

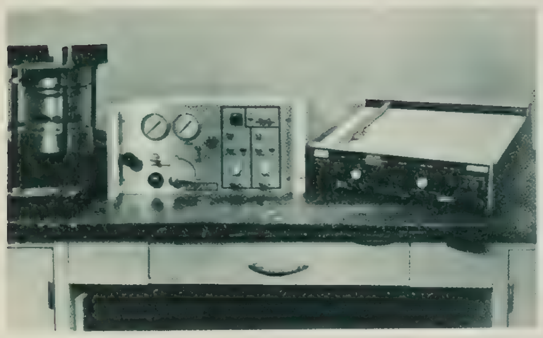
These reports are available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock Nos. PB 275205, PB 275232, PB 275233, PB 278854).

Availability of Mining Wastes and Their Potential for Use as Highway Material—Executive Summary, Report No. FHWA-RD-78-28

by FHWA Materials Division

Over 1.6 billion tons (1.5 Pg) of mining and metallurgical wastes are produced each year. A percentage of this material is being successfully used as highway construction material. A comprehensive literature survey was performed and knowledgeable personnel in the mining industry, governmental agencies, trade associations, and universities were contacted to obtain information on the availability of mining and metallurgical wastes in the United States and to assess their potential for use in various highway construction methods. This report discusses the potential use of mineral wastes which would require some degree of processing. Materials highly recommended for use in highway construction are gold gravels, steel slag, lead zinc chat, phosphate slag, taconite tailings, copper slag, and waste rock from the mining of copper, fluorspar, and iron ore.

The report is available from the Materials Division, HRS-20, Federal Highway Administration, Washington, D.C. 20590.



Determination of Consistency Characteristics of Soils, Report No. FHWA–RD–77–101

by FHWA Materials Division

This report presents a new, improved test method to measure the limits of soils. Many problems with the traditional consistency methods have been eliminated. The new test is performed on natural whole soil-aggregate samples without subjecting the sample to the harmful effects of predrying or pulverization, which often change the physical and chemical characteristics of the samples.

Researchers measured the strength or stiffness of soil specimens compacted for moisture-density determination. Subsequent testing of a range of soil and soil-aggregate materials established the coefficient of earth pressure at rest, K_O, as the best method. The K_O test device and procedure are fully described in the report. This test has the advantage of using a standard specimen prepared in most laboratories for other tests. The K_O test can be performed after the sample has been examined in freeze-thaw or permeability tests; it can be used to determine the effect of stabilization procedures that might be used in highway construction. The test provides data for the classification of soil materials for various highway uses and for the evaluation of the relative effects of moisture change on the strength of materials in the compacted condition.

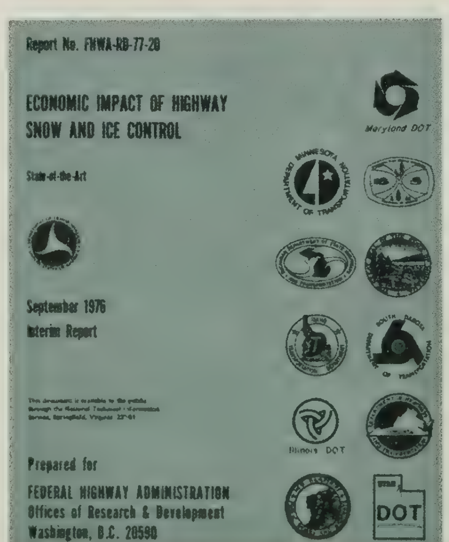
The report is available from the Materials Division, HRS–20, Federal Highway Administration, Washington, D.C. 20590.

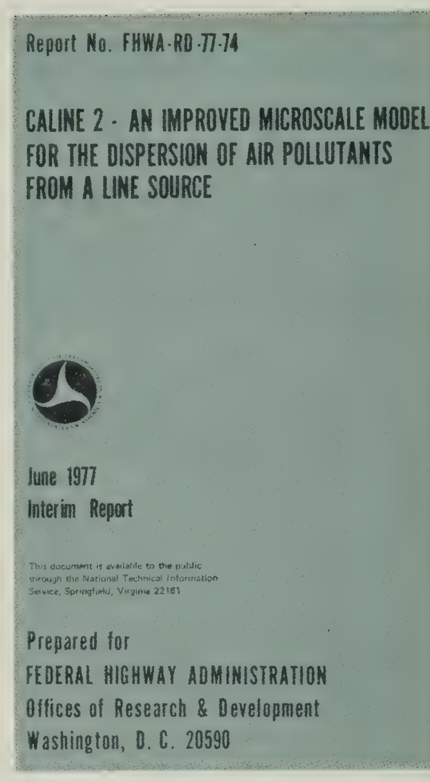
Economic Impact of Highway Snow and Ice Control, State-of-the-Art, Report No. FHWA–RD–77–20

by FHWA Environmental Design and Control Division

Many studies have been made on the economics of highway snow and ice control in an attempt to meet the public demand for clean highways in spite of funding limitations and increasing costs of maintenance. This report summarizes these studies and is intended as a reference volume. Areas discussed include environmental impact, levels of service, traffic characteristics, maintenance activities, and public opinion. The report should be of interest to all concerned with the economical, operational, and environmental aspects of winter maintenance.

The report is available from the Environmental Design and Control Division, HRS–42, Federal Highway Administration, Washington, D.C. 20590, and the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock No. PB 271250).





CALINE–2: An Improved Microscale Model for the Dispersion of Air Pollutants From a Line Source, Report No. FHWA–RD–77–74

A User's Manual for the CALINE-2 Computer Program, Report No. FHWA–RD–76–134

by FHWA Environmental Design and Control Division

In order for transportation planners and engineers to evaluate the air quality impact of a proposed project, mathematical means are required to describe the dispersion of air pollutants from roadways. The California Line Source Dispersion Model (CALINE–2) is one such mathematical approach and is presented and discussed in these two reports. CALINE–2 is based on the generalized Gaussian dispersion theory and simulates the dispersion of carbon monoxide. Preliminary results using carbon monoxide data from the Los Angeles region give an estimate of CALINE–2's predictive capabilities.

Other available reports related to this subject include the following:

• Air Pollution and Roadway Location, Design, and Operation—Project Overview, Report No. FHWA-RD-77-102.

• Air Pollution and Roadway Location, Design, and Operation—Preliminary Study of the Distribution of Carbon Monoxide On and Adjacent to Freeways, Report No. FHWA-RD-76-141.

• Variables Affecting Air Quality Instrument Operation, Report No. FHWA-RD-77-77.

• Mini-Computer Software Data Acquisition and Process Control System for Air Pollution Monitoring, Report No. FHWA-RD-76-145.

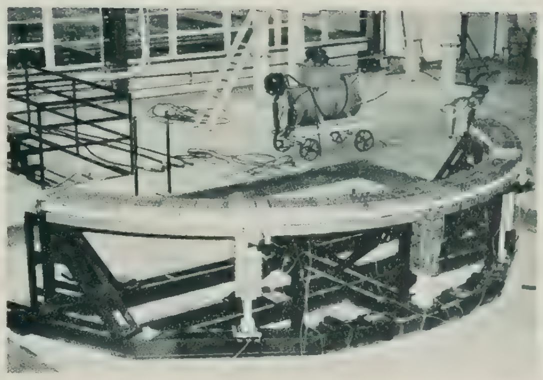
These reports are available from the Environmental Design and Control Division, HRS-42, Federal Highway Administration, Washington, D.C. 20590.

A computer magnetic tape containing the CALINE-2 computer program is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock No. PB 271105).

An Investigation of the Effectiveness of Existing Bridge Design Methodology in Providing Adequate Structural Resistance to Seismic Disturbances: Phase IV, Experimental Model Studies on the Seismic Response of High Curved Overcrossings, Report No. FHWA-RD-77-91.

by FHWA Structures and Applied Mechanics Division

An experimental structural model study was conducted to investigate the seismic resistance of high multispan curved highway interchange overcrossings of the type which suffered heavy damage during the 1971 San Fernando earthquake. The feasibility of developing a valid model that could be tested dynamically on the 20-ft (6.1 m)



square shaking table at the University of California at Berkeley is outlined.

The small amplitude dynamic characteristics of the microconcrete model, a 1:30 true-scale version of a hypothetical prototype, are examined. Experimental results compared satisfactorily with those predicted analytically. Model response is described for a series of progressively intense simulated seismic excitations applied horizontally in the bridge model asymmetric longitudinal direction and horizontally in the bridge model symmetric transverse direction. Both series were repeated with simultaneous vertical excitation.

The presence of expansion joints in the model bridge deck had much influence on the dynamic behavior of the structure. The joints were severely damaged by multiple impacting in both torsional and translational vibration modes, despite the inclusion of strong, ductile restraints against separation.

The report recommends that particular attention be given to the following when designing a bridge deck expansion joint to resist seismic forces:
(1) The stiffness and ductility of the restraint provided to prevent horizontal separation of the joint, (2) the proper length of the joint hinge seat to safely accommodate large horizontal movements, (3) the ability of the shear key to resist large transverse forces, (4) the combined effects of vertical shear and dynamic torsional impacting which produce severe vertical forces on the joint hinge seats, and (5) the advisability of incorporating vertical restrainers to prevent torsional lifting.

The report is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock No. PB 274459).

Stress Corrosion Susceptibility of Highway Bridge Construction Steels, Phase IIB, Report No. FHWA-RD-77-9

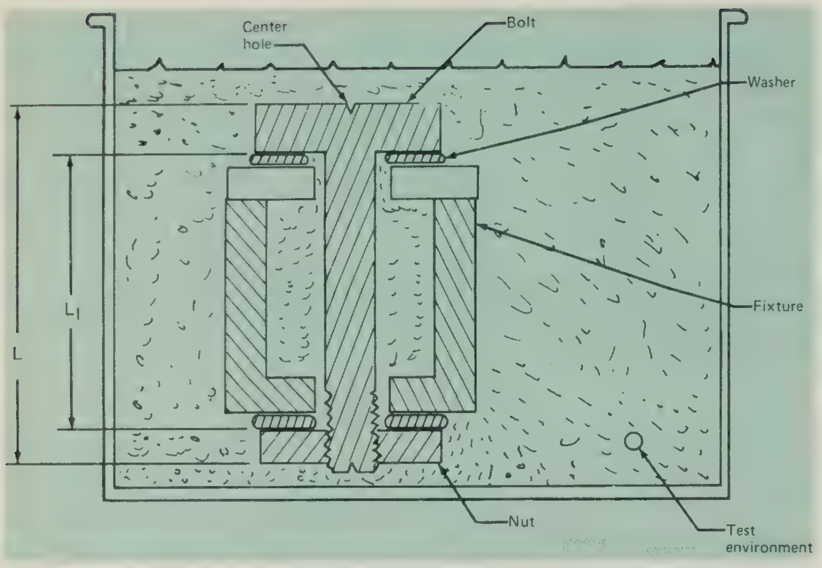
By FHWA Structures and Applied Mechanics Division

This report evaluates the potential danger of stress corrosion cracking in highway bridge construction steels. The fact that such cracking can occur in steel bridge members was brought out by the investigation of the failure of the Point Pleasant highway bridge in West Virginia.

Stress corrosion cracking of susceptible bridge steels may result from the combined action of a sustained static tensile stress above some minimum threshold level and one of several specific hostile environments. This report covers the stress corrosion testing of various structural steel plate materials used in highway bridge construction, structural bolts, cable wire used for suspension bridges, and prestressing strands.

The fracture mechanics approach to the evaluation of stress corrosion resistance of the plate steels involved testing of notched, precracked, double-cantilever beam specimens in which stress was induced by forcing a tapered wedge into the notch to displace the beam arms.

The study also included an investigation of the effects of welding and cold working on stress corrosion susceptibility and field exposure tests of selected steels in some potentially corrosive environments.



This report is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock No. PB 272493).

Integrated Motorist Information System (IMIS), Feasibility and Design Study: Phase I—Feasibility Study, Volume 1 (Report No. FHWA-RD-77-47), Volume 2 (Report No. FHWA-RD-77-48), and Volume 3 (Report No. FHWA-RD-77-49)

by FHWA Traffic Systems Division

Vehicular demands on many traffic facilities already exceed both the available and planned capacity. These heavy demands not only lead to the daily recurring congestion so familiar to commuters, but also result in erratic operation, accidents, and motorist aggravation. Traffic congestion problems also result from the presence of disabled motorists, other distracting roadside events, construction, and maintenance operations.

To combat these problems in the face of the increasing social and economic pressures against future construction of highways, highway traffic authorities must obtain better use of existing facilities. A study was performed to determine the feasibility of implementing an Integrated Motorist Information System (IMIS) in the northern Long Island Corridor of New York State. The objectives of IMIS are to demonstrate the benefits of an integrated approach to traffic management and to permit research in motorist information systems and advanced concepts in traffic flow optimization.

In the study, extensive corridor data were collected and analyzed to determine candidate alternate routes and connecting roadways to be included in the system network. Trade off studies were performed to identify the most suitable approaches and equipment types for implementing the IMIS features. Two traffic simulation programs were used to perform system configuration studies and develop fundamental relationships for various control policies. Among the concepts included in the various configurations were traffic surveillance, incident management, traffic diversion, ramp metering, and motorist aid. A set of nine alternative system designs was developed, varying in network configuration, equipment complement, and functional capability. For each design, benefits, costs, and benefit/cost ratios were calculated. Eight of the designs showed benefit/cost ratios greater than 2:1 and the ninth was only slightly less.

Volume 1, **Final Report,** and Volume 2, **Appendixes,** are very detailed. Volume 3, **Executive Summary,** consists of the first two sections of Volume 1 and is published separately for those who have no need for the detail of the first two volumes.

The reports are available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock Nos. PB 276090/276092).

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

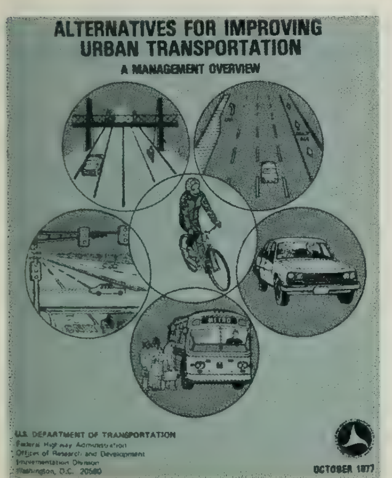
The following are brief descriptions of selected items which have been recently completed by State and Federal highway units in cooperation with the Implementation Division, Offices of Research and Development, Federal Highway Administration (FHWA). Some items by others are included when they have a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

Alternatives for Improving Urban Transportation: A Management Overview, Report No. FHWA-TS-77-215

by FHWA Implementation Division



There are various alternatives available for the improvement of urban transportation. This report, which outlines a 3-day training course, provides an overview of these alternatives, with emphasis on the application, the benefits, and the trade offs associated with each alternative. Concepts of managing the total transportation system and factors relating to the transportation process are discussed.

Alternatives presented include improving traffic operations, improving urban goods movements, ride sharing programs, demand management, transportation pricing, and improving public transit.

Information on the training course and the training kit is available from the Federal Highway Administration, National Highway Institute, HHI-2, Washington, D.C. 20590.

The manual is available from the Implementation Division.

Report of 1977 Symposium on Highway Construction Noise, Report No. FHWA-TS-77-211

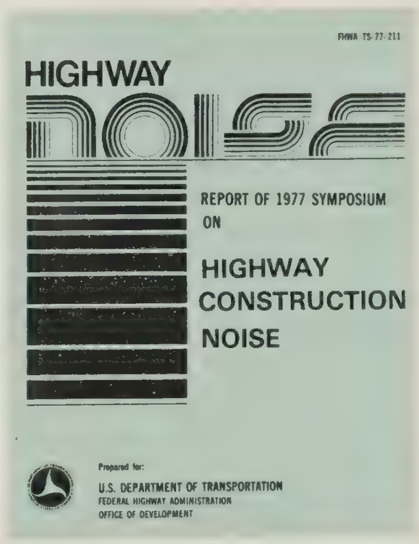
by FHWA Implementation Division

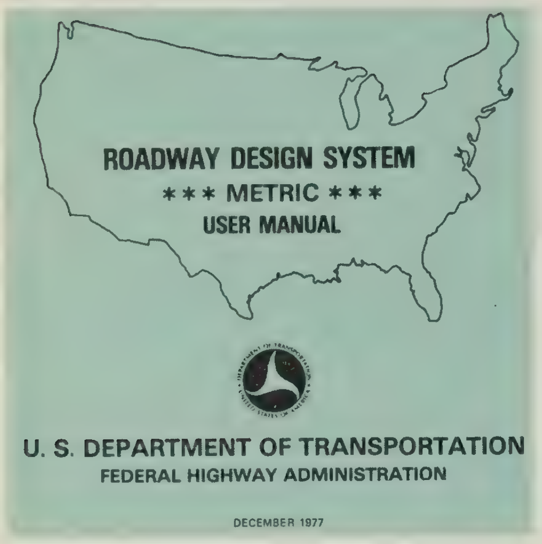
A limited amount of data is available on sound levels for existing construction equipment, use of equipment at the site, noise impact criteria, and practical highway construction noise abatement methods. In an attempt to discuss what has been done, what is being done, and what needs to be done to minimize highway construction noise impacts, individuals working in this relatively new field met in February 1977 in Lakewood, Calif. The proceedings of the symposium on highway construction noise are presented in this report. Attendees were from Federal and State highway agencies, the U.S. Environmental Protection Agency, the U.S. Army Corps of Engineers, universities, acoustical consulting companies, and equipment manufacturing and contracting companies.

It was concluded that there are methods to determine the highway construction noise and the preconstruction sound climate but no criteria to estimate the potential impact. A program to determine the public health and welfare effects of highway construction noise is recommended.

The proceedings of the symposium should be of interest to highway agencies in designing highway projects and preparing environmental impact statements.

The report is available from the Implementation Division.





Roadway Design System Metric User's Manual

by FHWA Implementation Division

The roadway design system is one large integrated computer program that assists engineers in the design of highways. It contains more than 375 integrated computer subroutines written in FORTRAN IV and four assembler language subroutines.

The original computerized roadway design system was developed in 1967 and has been modified several times since then. This manual develops a *metric* roadway design system program in recognition of this country's general move toward the use of metric units.

The manual is available from the Implementation Divison.

Guidelines for the Measurement of Interstate Motor Carrier Noise Emissions, Report No. FHWA-TS-77-222

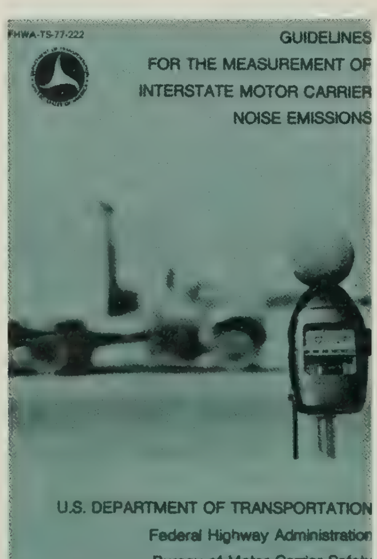
by FHWA Implementation Division

This manual presents and describes the elements of the measurement of sound, particularly from motor vehicles. Provisions and requirements of the Federal interstate motor carrier noise emission regulations are provided.

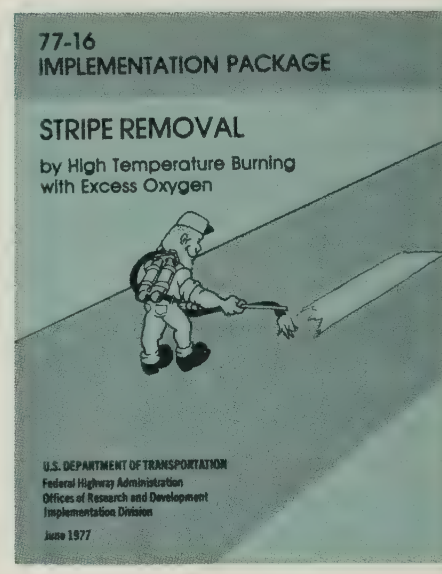
Although the manual is primarily intended to help field safety investigators determine compliance with the regulations, it may also serve as a guide to any agency or individual desiring to measure the noise emissions of highway vehicles.

After a preliminary discussion of the properties of sound and the characteristics and operation of sound measurement instruments, the succeeding chapters of the manual follow a typical testing sequence: site selection, preparation for testing, determination of site correction factors, visual inspection of the vehicle, sound measurement and vehicle operation, sound level limits, violation citations, and test data reporting.

A glossary of sound measurement terms and a listing of instrumentation availability are included as appendixes.



The manual is available from the Implementation Division.



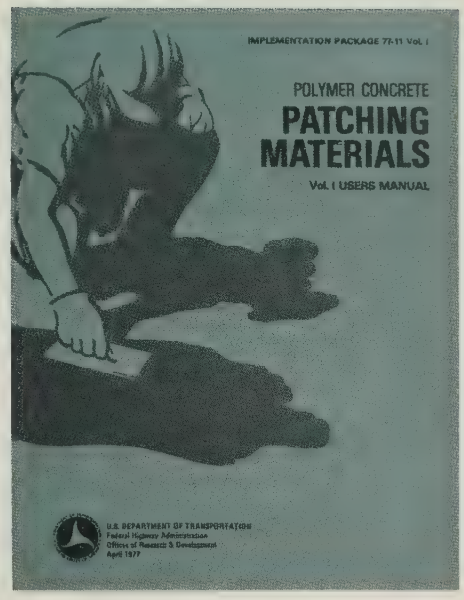
Stripe Removal by High Temperature Burning With Excess Oxygen, Implementation Package 77-16

by FHWA Implementation Division

Much attention has been directed toward the problem of traffic stripe removal, particularly in highway construction zones. While many methods are presently available to remove markings, all share the serious shortcomings that they discolor or scar the pavement surface or change its texture. The scars or traces left by these removal methods are often indistinguishable from the actual traffic marking. This type of situation is considered very hazardous and has been identified as a contributing cause in a number of serious construction zone accidents.

New concepts for removing pavement markings with as little damage or scarring of the pavement as possible were explored. This report describes the method of high temperature burning with excess oxygen. This method of stripe removal has the advantage of burning the unwanted marking material off rapidly with little damage to the pavement. Discoloration or contrast differences between the burned and unburned surfaces can vary from excellent to poor, depending upon the type, nature, and surface condition of the pavement. It is not expected that this method will replace all existing methods or be of use in all situations, although it does offer a number of advantages over most other methods. The equipment and its operation is simple and inexpensive.

The report is available from the Implementation Division.



Polymer Concrete Patching Materials— Volumes I and II, Implementation Package 77–11

by FHWA Implementation Division

Deterioration of concrete bridge decks is caused by chloride-induced corrosion of reinforcing steel which results in cracking and spalling of the concrete. This represents one of the most severe problems facing the highway industry today. In addition to being costly, bridge deck maintenance is difficult to perform under hazardous traffic conditions and presents delays and safety hazards to the traveling public.

This report discusses the use of polymer concrete patching in the repair of highway structures. Polymer concrete is a composite material consisting of aggregate and polymer, with the polymer serving as the binder material. Polymer concrete cures rapidly at ambient temperatures and has excellent bonding to existing concrete.

Volume I, **User's Manual,** outlines procedures for using polymer concrete as a patching material. The chemicals used are volatile, flammable, and toxic materials but can be safely used by maintenance personnel. The process technology, materials, equipment, and safety provisions used in the technique are discussed. The report is intended to inform potential users of the various steps necessary to insure successful field application of the material.

Volume II, **Final Report,** describes experimental work performed in developing Volume I. Data on the relationship between process variables and the effects of these variables on the properties of the polymer concrete are discussed.

The reports 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 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 1A: Traffic Engineering Improvements for Safety

Title: Identification of Traffic Management Problems in Work Zones. (FCP No. 31A1564)
Objective: Identify and define the problems associated with the movement of traffic through work zones.
Performing Organization: University of Tennessee, Knoxville, Tenn. 37916
Expected Completion Date: November 1978
Estimated Cost: $177,000 (FHWA Administrative Contract)

Title: Safety Aspects of Using Vehicle Hazard Warning Lights. (FCP No. 31A1774)
Objective: Investigate the present uses of vehicle hazard warning lights. Determine the safety benefits as well as the legal and operational problems for a variety of environmental conditions and traffic situations. Give special attention to the use of the lights on slow moving vehicles. Address the role of regulatory and warning signs.
Performing Organization: BioTechnology, Inc., Falls Church, Va. 22042
Expected Completion Date: June 1979
Estimated Cost: $149,000 (FHWA Administrative Contract)

FCP Project 1C: Analysis and Remedies of Freeway Traffic Disturbances

Title: Control Strategies in Response to Freeway Incidences. (FCP No. 31C3522)
Objective: Develop ramp control strategies for freeway incidents. Develop guidelines and procedures for specifying the location and spacing of freeway loop detectors and for functional specifications for an online control strategy evaluator.
Performing Organization: Orincon, La Jolla, Calif. 92037
Expected Completion Date: March 1979
Estimated Cost: $310,000 (FHWA Administrative Contract)

FCP Project 1H: Skid Accident Reduction

Title: Calibration and Correlation of Response-Type Road Roughness Measuring Systems. (FCP No. 51H3374)
Objective: Develop rapid and inexpensive methods for the calibration of response-type pavement roughness measurement systems. Determine the accuracy and time stability of measurement system, and identify factors affecting performance.
Performing Organization: University of Michigan, Ann Arbor, Mich. 48109
Expected Completion Date: June 1980
Estimated Cost: $250,000 (NCHRP)

FCP Project 1I: Traffic Lane Delineation Systems for Adequate Visibility and Durability

Title: Development and Evaluation of Mechanized Equipment for Installation of the RRM Delineation System. (FCP No. 31I3033)
Objective: Develop mechanized equipment to install the recessed reflective marker delineation system and evaluate the operational characteristics of the full-scale prototype equipment by field testing.
Performing Organization: HH Aerospace Design Company, Cambridge, Mass. 02138
Expected Completion Date: September 1980
Estimated Cost: $371,000 (FHWA Administrative Contract)

FCP Project 1O: Aids to Surveillance and Control

Title: Grade Crossing Active Advance Warning Signs. (FCP No. 31O1094)
Objective: Identify types of grade crossings with active devices where active advance warning signals would improve safety. Develop, test, and evaluate prototype active advance warning signals. Develop guidelines fo[r] use of active advance warning signals.
Performing Organization: JGM Associates, Stanford, Calif. 94305
Expected Completion Date: Septembe[r] 1979
Estimated Cost: $150,000 (FHWA Administrative Contract)

Title: Identification and Evaluation of Off-Track Train Detection Systems for Grade Crossing Applications. (FCP No. 31O1104)
Objective: Develop specific conclusions and recommendations on the technical feasibility and cost effectiveness of off-track train detection concepts for activating rail-highway grade crossing warning systems.
Performing Organization: Gard, Inc., Niles, Ill. 60648
Expected Completion Date: May 1980
Estimated Cost: $158,000 (FHWA Administrative Contract)

FCP Project 1T: Advanced Vehicle Protection Systems

Title: Analysis of the Performance of Railing Systems on Nonlevel Terrain. (FCP No. 31T5043)
Objective: Establish guidelines on treating deviations from ideal level ground configuration by determining and documenting information on variations in collision performance due to placing guardrail and median barriers on slopes, for both new and retrofit construction.
Performing Organization: Texas A&M Research Foundation, College Station, Tex. 77843
Expected Completion Date: September 1979
Estimated Cost: $172,000 (FHWA Administrative Contract)

FCP Project 1U: Safety Aspects of Increased Size and Weight of Heavy Vehicles

Title: Evaluation of Techniques to Counteract Truck Accidents on Steep Downgrades. (FCP No. 31U2042)
Objective: Develop a prototype grade severity rating system, evaluate and determine cost effectiveness of techniques to reduce the incidence and severity of truck downgrade accidents, and determine warrants for the development and deployment of these techniques.
Performing Organization: Systems Technology, Inc., Hawthorne, Calif. 90250
Expected Completion Date: January 1980
Estimated Cost: $313,000 (FHWA Administrative Contract)

Title: Motorists' Attitudes Toward Large Trucks. (FCP No. 31U3032)
Objective: Identify motorists' attitudes toward large trucks and their situational context and relate the presence of these attitudes to motorist behavior.
Performing Organization: Lawrence Johnson and Associates, Inc., Washington, D.C. 20009
Expected Completion Date: March 1979
Estimated Cost: $222,000 (FHWA Administrative Contract)

FCP Project 1Y: Traffic Management in Construction and Maintenance Zones

Title: Development of a Driver Based Method for the Evaluation of Traffic Control Systems for Freeway Construction and Maintenance Operations. (FCP No. 41Y1682)
Objective: Develop, validate, and utilize a driver based method for evaluating traffic control systems for rural freeway construction and maintenance operations.
Performing Organization: Ohio State University, Columbus, Ohio 43210
Funding Agency: Ohio Department of Transportation
Expected Completion Date: October 1980
Estimated Cost: $246,000 (HP&R)

FCP Category 2—Reduction of Traffic Congestion, and Improved Operational Efficiency

FCP Project 2K: Metropolitan Multimodal Traffic Management (MMTM)

Title: Urban Curbside Pickup and Delivery Operations. (FCP No. 32K5012)
Objective: Conduct a complete investigation of the truck loading and unloading process as it occurs on public rights-of-way and construct models thereof.
Performing Organization: Polytechnic Institute of Brooklyn, Brooklyn, N.Y. 11201
Expected Completion Date: October 1979
Estimated Cost: $269,000 (FHWA Administrative Contract)

FCP Category 4—Improved Materials Utilization and Durability

FCP Project 4C: Use of Waste as Material for Highways

Title: Investigation of the Use of Coal Refuse-Fly Ash Compositions as Highway Base Course Materials. (FCP No. 34C3062)
Objective: Determine the potential for combining fly ash with coal mine refuse to function as a strong, durable, and economical highway base course material. Conduct a comprehensive laboratory testing program to develop the optimum blends with and without the use of various stabilizers.
Performing Organization: GAI Consultants, Inc., Monroeville, Pa. 15146
Expected Completion Date: March 1980
Estimated Cost: $150,000 (FHWA Administrative Contract)

FCP Project 4F: Develop More Significant and Rapid Test Procedures for Quality Assurance

Title: Automation of Aggregate Sampling and Gradation Testing. (FCP No. 34F7043)
Objective: Identify and study those automatic techniques which can be applied to aggregate sampling and gradation testing. Design modifications and adaptations of these techniques to develop systems yielding quick, accurate, inexpensive results. Construct and test a prototype employing the most promising of the techniques investigated.
Performing Organization: Rexnord, Inc., Milwaukee, Wis. 53201
Expected Completion Date: January 1980
Estimated Cost: $151,000 (FHWA Administrative Contract)

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

FCP Project 5D: Structural Rehabilitation of Pavement Systems

Title: Test Methods and Use Criteria for Filter Fabrics. (FCP No. 35D3142)
Objective: Identify criteria for engineering use of filter fabrics, particularly in subdrainage, erosion control, and soil reinforcement applications. Also modify existing test methods and develop new test methods for evaluating engineering properties of filter fabrics. Determine properties of filter fabrics critical to each proposed application.
Performing Organization: Oregon State University, Corvallis, Oreg. 97331
Expected Completion Date: March 1980
Estimated Cost: $205,000 (FHWA Administrative Contract)

FCP Project 5E: Premium Pavements for "Zero Maintenance"

Title: Flexible and Composite Structures for Zero Maintenance Pavements. (FCP No. 35E2022)
Objective: Develop design and construction procedures for conventional heavy-duty flexible and composite type pavements to minimize deterioration and structural maintenance: (1) Identify causes of deterioration, (2) develop procedures to minimize deterioration, (3) evaluate design and construction procedures, (4) conduct analysis, and (5) prepare user manual.
Performing Organization: Austin Research Engineers, Austin, Tex. 78746
Expected Completion Date: September 1979
Estimated Cost: $141,000 (FHWA Administrative Contract)

Title: Material Property Requirements for Zero Maintenance Pavements. (FCP No. 35E4032)
Objective: Identify optimized sets of materials properties required to eliminate pavement distress for at least 20 years for various structural systems, traffic levels, and environmental conditions. Portland cement concrete, asphaltic concrete, and composite pavements are included.
Performing Organization: Austin Research Engineers, Austin, Tex. 78746
Expected Completion Date: September 1979
Estimated Cost: $174,000 (FHWA Administrative Contract)

FCP Project 5H: Protection of the Highway System from Hazards Attributed to Flooding

Title: Performance of Stream Channel Changes. (FCP No. 35H1012)
Objective: Document highway channelization performance; isolate instability factors.
Performing Organization: U.S. Geological Survey, Menlo Park, Calif. 94025
Expected Completion Date: September 1979
Estimated Cost: $125,000 (FHWA Administrative Contract)

FCP Project 5K: New Bridge Design Concepts

Title: Time-Dependent Behavior of Segmental Cantilever Concrete Bridges. (FCP No. 45K1032)
Objective: Determine the time-dependent behavior of a segmental post-tensioned cantilever bridge through a field study, a laboratory study, and an evaluation of measured versus calculated deformations.
Performing Organization: Portland Cement Association, Skokie, Ill. 60076
Funding Agency: Illinois Department of Transportation
Expected Completion Date: May 1982
Estimated Cost: $133,000 (HP&R)

FCP Project 5L: Safe Life Design for Bridges

Title: Fatigue Behavior of Full-Scale Welded Bridge Attachments. (FCP No. 55L3012)
Objective: Examine the fatigue strength of beams with web and flange lateral attachment plates, provide a comprehensive data base, and examine the influence of lateral bracing members on the out-of-plane distortion of the lateral plate.
Performing Organization: Lehigh University, Bethlehem, Pa. 18015
Expected Completion Date: March 1980
Estimated Cost: $125,000 (NCHRP)

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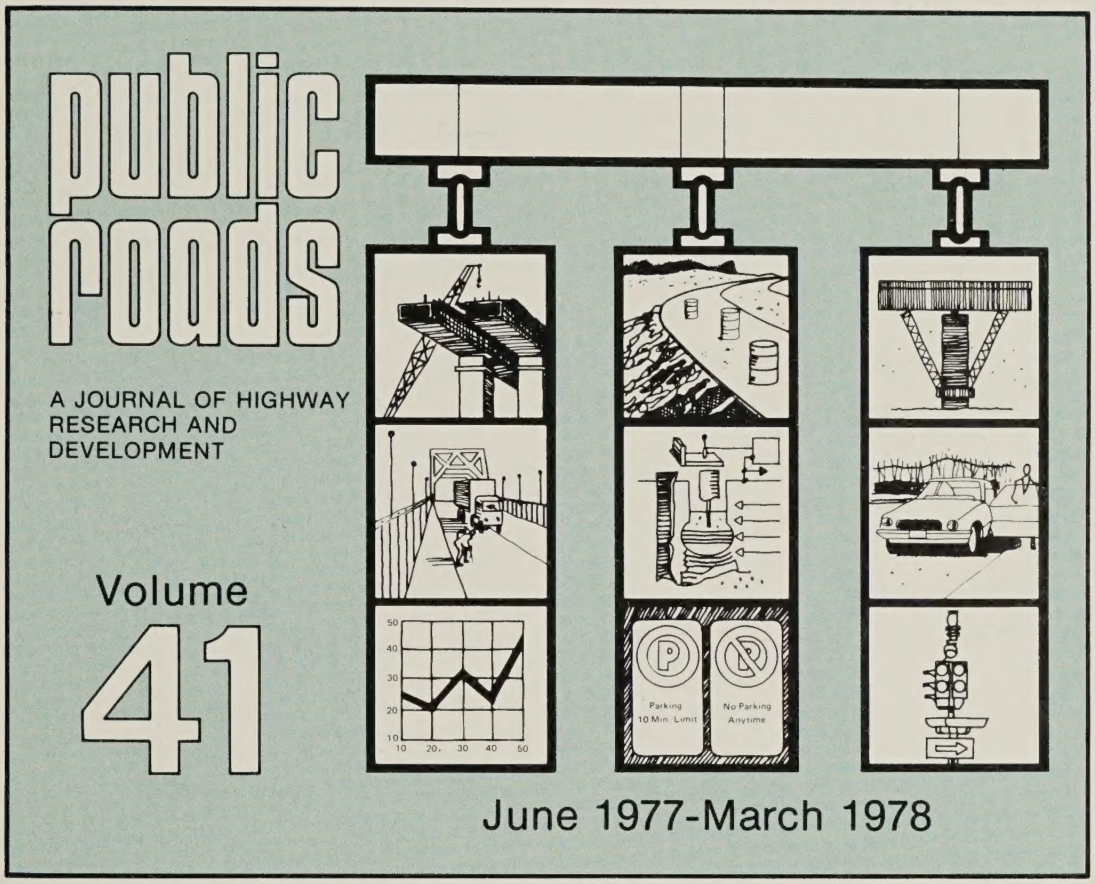
TITLE SHEET, VOLUME 41



The title sheet for volume 41, June 1977–March 1978, of Public Roads, A Journal of Highway Research and Development, is now available. This sheet contains a chronological list of article titles and an alphabetical list of authors' names. Copies of this title sheet can be obtained by sending a request to the editor of the magazine, U.S. Department of Transportation, Federal Highway Administration, HDV–14, Washington, D.C. 20590.

Price Decrease for PUBLIC ROADS

Effective with this issue, the annual domestic subscription rate for *Public Roads* is decreased to $6.25 ($1.60 additional for foreign mailing). The price decrease is attributed to a change in the mailing procedure. The Federal Highway Administration produces the magazine. The Superintendent of Documents, U.S. Government Printing Office, establishes subscription rates and conditions of sale.

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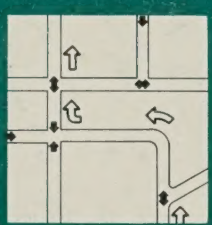

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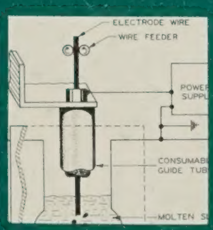

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