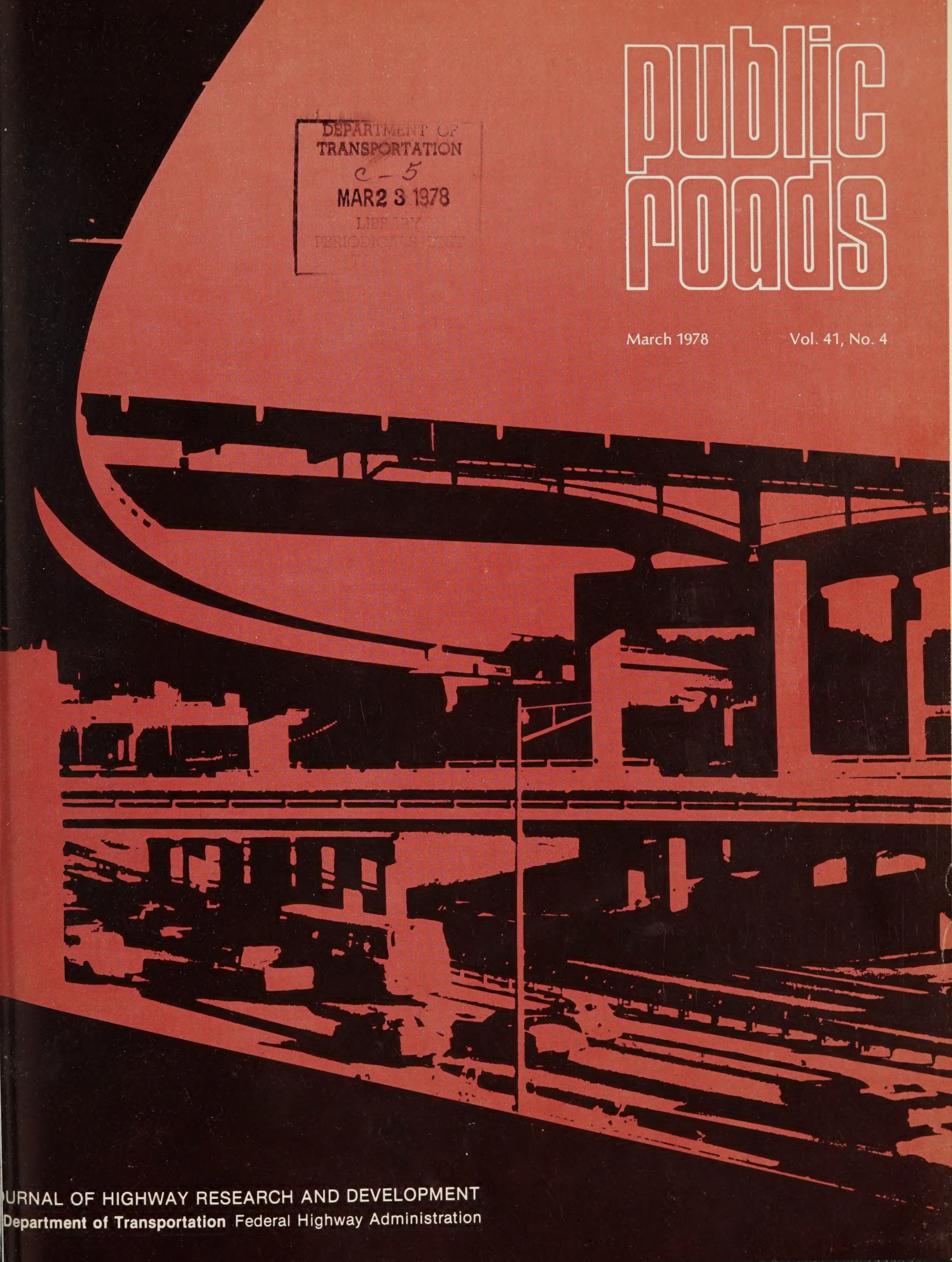

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COVER:
Sweeping curves in a simple, uncluttered design lend beauty to this four-level interchange between I–45 and I–20 at the edge of downtown Dallas, Tex.

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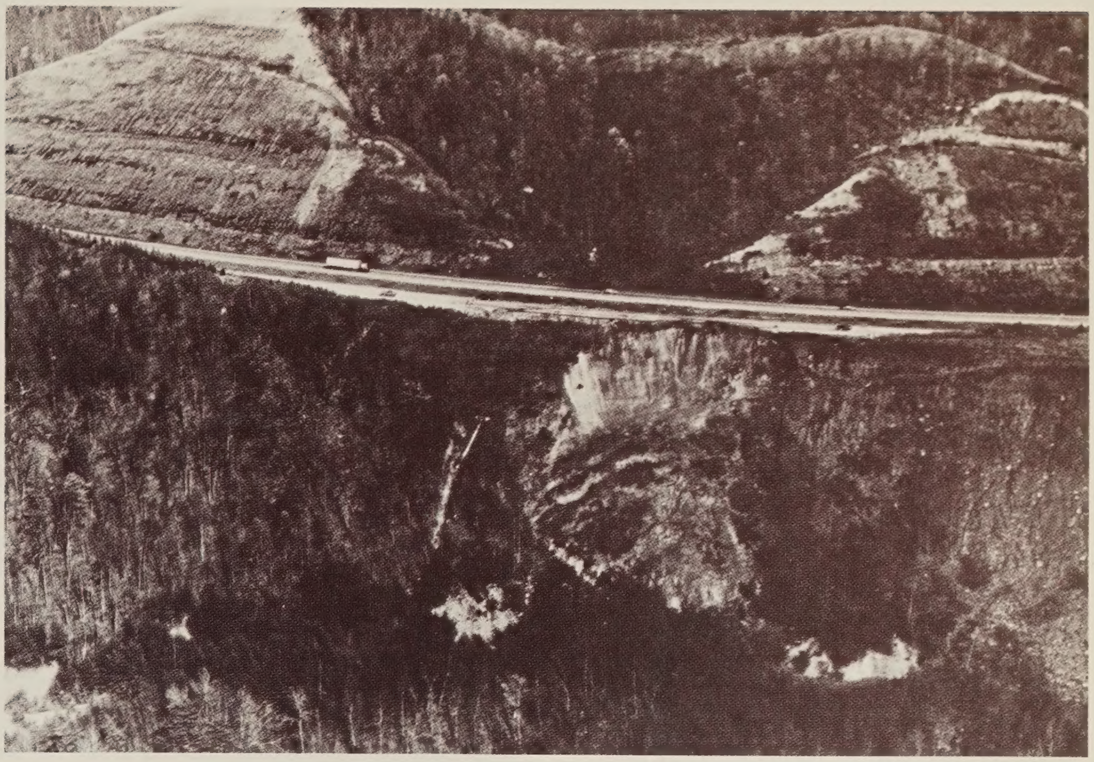
Status of Shale Embankment Research

by Albert F. DiMillio



Shale embankment failure on Interstate 75 in Tennessee.
—courtesy of the Tennessee Department of Transportation

Introduction

Shale, the transition material between soil and rock which exhibits a wide spectrum of engineering behavior, is one of the most common types of rock in the United States and also one of the most troublesome construction materials that highway builders have to deal with. For example, some shales are very hard and durable which allows them to be placed in embankments according to rock fill specifications; other shales are weak and must be treated like soil, that is, mechanically broken down and placed in thin compacted lifts. Shales which fall between the two extremes are difficult to classify in performance categories and present the most problems in design.

In recent years there have been numerous problems caused by slope failures and excessive, nonuniform settlements of highway embankments containing large amounts of nondurable shale materials that were not properly treated prior to placement in the embankment. The repair schemes most often used are very expensive and frequently require lane closures for long periods of time. For example, reconstruction costs in excess of $2 million and numerous lane closures resulted from slope failures on a short section of Interstate 74 in Indiana. Numerous other embankments along this 5-mile (8 km) section of highway have exhibited serious signs of pavement distress (bumps, waves, and cracks) and minor slope failures which were thought to be an indication of future embankment failures. Investigative and remedial work continues along this major highway route.

Figures 1 and 2 show the results of a failure of an embankment on Interstate 74 that was designed and constructed without the benefit of adequate guidelines for the treatment of nondurable shale. This failure occurred because the shale material, which appeared hard and durable when taken from the cut area, slowly deteriorated when it was placed as a rock fill. The deterioration caused a reduction in the void spaces resulting in settlement within the fill. The settlement caused cracks and breakup to occur in the concrete pavement and subsurface drainage structures; water infiltrated the fill material and accelerated the deterioration process of the shale. The reduction in shear strength caused by the deterioration process eventually resulted in a very large slope failure.

The obvious remedy for problems associated with shale materials is to mechanically process the materials during construction so that additional deterioration occurring during the service life of the embankment will not cause any significant embankment distress. The ease or difficulty of breaking down the shale material will determine the cost of the required processing. For example, a shale material which is mechanically hard when it comes out of the source area and nondurable (considerable slaking with time) is the most expensive shale material to use in an embankment because it is difficult to process to the point where subsequent deterioration will not cause excessive, nonuniform settlement and/or slope failure.

After numerous failures occurred, some highway agencies resorted to a more conservative approach where all shale materials became suspect. As a result, many hard, durable shales were needlessly treated like soil materials because of uncertainty regarding their performance characteristics. It is difficult to estimate the magnitude of overdesign that has occurred; however, it is thought to be very high.

Numerous cases of over- and under-design occur because of the wide spectrum of engineering characteristics of the various shale materials found in the United States. If the highway engineer is able to assess the susceptibility of a shale to deterioration in the projected service environment, he can design a safer, more functional, reliable, and economical embankment. At this time there is no comprehensive set of design and construction guidelines available to perform this task. The absence of proven criteria for classifying shale durability and predicting the long term performance has perpetuated the problem.

In order to provide the geotechnical engineer with the tools necessary to design safe and economical embankments, the Federal Highway Administration has sponsored a comprehensive research study by the United States Army Corps of Engineers at the Waterways Experiment Station (WES) in Vicksburg, Miss.

Figure 1.—Western portion of embankment failure on Interstate 74.
—courtesy of the Indiana State Highway Commission

Shale Research in Progress

The WES study was designed to gather information about the factors responsible for the deterioration of shale placed in a fill and to develop a technical manual containing design and construction control methodology for shale embankments. In addition, the study will develop techniques for evaluating existing embankments and will provide guidance on remedial treatment methods for existing, distressed shale embankments.

The study is divided into three major phases with numerous tasks assigned to each phase. In order to take advantage of the large amount of experience and documented evidence of various State highway departments and governmental agencies, the WES researchers were asked to survey the state of the art during the first year of the study and report the results in usable form for practicing highway engineers. The remaining 3 years will be spent developing the laboratory tests and other tools necessary to fill the gaps in the state of the art determined during the initial year of the study.

One of the most comprehensive research programs on shale embankments at a local level is being conducted by Purdue University in cooperation with the Indiana State Highway Commission (ISHC). The ISHC has provided most of the funding for this research, a coprincipal investigator for some of the work, and has performed numerous laboratory and field tests in support of the shale research. The Kentucky Department of Transportation is also conducting a comprehensive in-house research project on shale embankments.

Figure 2.—Eastern portion of embankment failure on Interstate 74.
—courtesy of the Indiana State Highway Commission

In addition to the valuable correlation data obtained, it is interesting to note the extensive in situ field testing performed to augment the results obtained from laboratory testing. Twenty-three pressuremeter tests were conducted in ten boreholes using the Menard pressuremeter. Since reduction of the pressuremeter field data by hand is tedious and often subjective, the consultant developed a computer program which reduces the data, plots the test results, and performs a graphical analysis of the pressuremeter curves to derive the stress path.

A special shear testing apparatus was also developed for the ISHC (fig. 3) and was used in several test pits in the median strip of Interstate 74. The equipment is designed to perform direct shear tests on in situ block samples (16 in by 16 in by 8 in [406 mm by 406 mm by 203 mm]) of soil or relatively soft rock.

Several other States, such as Georgia, Virginia, and West Virginia, are currently performing shale research for embankment design. Significant contributions have also been made by California, Colorado, Illinois, Missouri, Montana, Ohio, Oklahoma, Pennsylvania, Tennessee, and Utah.

It is the consensus of researchers that the highway designer finds degradation and slaking to be the most important shale properties. Degradation is the reduction in particle size that results from construction processing, and slaking is the decomposition of the shale materials due to weathering within the new environment (embankment). In both cases, the amount of deterioration of fresh material (parent shale from the borrow or cut source) must be predictable to allow a proper design of the embankment. The degree of slaking that will occur after placement in the embankment will affect the geometry of the embankment and/or the amount of processing the shale will require during construction. As previously mentioned, the ease or difficulty of breaking down the shale material determines the cost of the required processing.

In addition to the research program in Indiana, the ISHC also commissioned a geotechnical engineering firm to make a detailed study of embankment performance within the problem area along Interstate 74 near Cincinnati, Ohio. (*1*)[1] Significant portions of this study are extremely valuable to researchers studying the shale embankment problem.

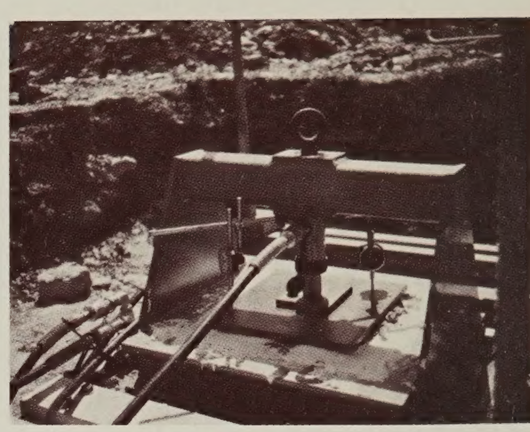
[1]Italic numbers in parentheses identify references on page 161.

Phase I of WES Study

The results of Phase I were reported in 1975 and include the results of a comprehensive literature search and personal interview program to define the problem areas and determine the current practices of various State and Federal agencies involved in shale embankment construction. (*2*) Available information was sought on classification and material properties, physical and chemical tests, other design guidelines, construction control procedures, and sampling and testing procedures for in situ shales and compacted shale mixtures.

Saw in operation.

Lower frame of testing device being positioned over sawed specimen.



Testing device in operation.

Figure 3.—Direct shear tests on in situ block samples of Interstate 74 shale. *—courtesy of the Indiana State Highway Commission*

Shale occurrence

An important accomplishment of this phase of the study was the mapping of the general occurrence of shales in the States being studied in relation to the distribution of problem shales. Three sampling units were established for identifying the occurrence of shale—shale predominates, shale subordinates, and nonshale areas.

In general, the States east of the Mississippi River have had more problems with shales in highway embankments than those west of the Mississippi River. The WES researchers have identified rock-stratigraphic units associated with problem shales and have presented a generalized description of their important geological features. (*2*) A geological time chart was also developed and the ages of the formations were grouped geographically. The formational characteristics were analyzed in a similar manner. Distinctions between the younger shales of the Western United States and the older formations of the East are discussed in this report.

Types of problems being experienced in the various States were identified during discussions held with representatives of the highway departments. (Summaries of these experiences are presented in reference 2.)

Another significant accomplishment of this study involves the summarization of current highway department construction procedures for shale embankments. (*2*) These procedures were divided into three broad categories—preconstruction, construction, and remedial measures.

It was also found that acceptance or rejection criteria for placement of shale in an embankment differ considerably between States, varying from fairly rigid measurements to a subjective judgment. The most important decision to make is whether to place shale as a soil or rock. Shamburger et al., have listed processing requirements for each method, have cited some noteworthy practices of certain States, and have listed and discussed a number of statements that State highway representatives identified as contributing to shale embankment distress or failure. (*2*) Most of the cited causes can be linked to one basic problem, that is, the lack of tests and criteria to predict shale performance with time. If this type of information were available prior to construction, the appropriate controls could be established and adhered to during construction.

An indepth review of the Corps of Engineers' experiences and practices is also provided, as well as a brief discussion of the U.S. Bureau of Reclamation practices. (*2*) Both agencies have had success with compacted shale fills using heavy compaction equipment and procedural specifications developed for each material to produce a dense fill with adequate shear strength. Although highway embankments do not require the same stringent specifications as earthen dams, the more conservative procedures used by dam builders can serve as a guide until the highway industry can develop more realistic specifications.

The above-noted practices for application to the construction of highway shale embankments are evaluated and presented as interim recommended measures until the overall study is complete. These recommendations center around processing and compaction requirements, drainage considerations, and materials testing.

Classification and composition

Several investigators have attempted to classify shale materials for engineering purposes; however, there still appears to be insufficient test data (as well as disagreement on test procedures) to develop an engineering classification system for shales. One of the objectives of this study is to develop such a system.

Purdue University developed an engineering classification system for the Indiana State Highway Commission which is used routinely by their engineers to determine shale durability. (*3*) In the next several years, a large data bank on Indiana shales will be available for study and refinement of the system.

Most of the researchers have concluded that more work is necessary to correlate laboratory results with field performance. This objective will be completed under Phase III of this study.

A discussion of the relationship between engineering properties and the composition of shales is also included in reference 2. The major engineering properties considered were plasticity, swell potential, durability, and strength. These properties were related to texture, mineralogy, geochemistry, and rock fabric. The WES researchers concluded that no single test (physical, chemical, or mineralogical) entirely indicates shale suitability, and therefore recommended that each shale material encountered in the soil profile be subjected to the following battery of tests:

- X-ray diffraction.
- Slake durability.
- Jar slaking.
- Scleroscope hardness.
- pH. (*2*)

Factors contributing to material degradation

In addition to the nature of its constituents, a shale material's suitability for use in embankment construction is a function of its geologic history and present-day environment. The geologic age, tectonic history, metamorphism, and the geological processes of weathering result in shales that possess varying degrees of soundness, strength, and durability.

The postconstruction changes that result from weathering of the shale embankment material are time-dependent and difficult to predict. Since water is the driving force of the weathering process, researchers recommend close control of the drainage aspects of the embankment design. It is also helpful to study the material in outcrops and recent exposures to determine an approximate weathering rate from the degree of altered rock and/or soil developed over the fresh material.

Reference 2 presents examples of the following types of intrinsic causes of shale deterioration and some geological processes which may contribute to embankment distress: expansive clay minerals, dispersive clay minerals, clay mineral weathering, cement weathering, unloading, and crystallization pressures.

Laboratory examination and testing techniques

The testing of shales for highway embankment purposes should provide the answer to one very basic question: Should the shale material be treated as a rock or a soil? Other questions to be addressed include the following: (1) What likely forms of deterioration will the shale experience, and (2) what other properties of the shale will influence the design? These three questions relate to the shale's resistance to three basic modes of deterioration which may be categorized as follows: (1) chemical weathering (breakdown of primary mineral components), (2) physiochemical (clay mineral hydration, swelling, and dispersion), and (3) physical (including relation to rock strength and measure of rippability).

The WES researchers have subdivided the laboratory testing of shales into three categories: (1) mineralogical and petrological tests (amount and nature of rock constituency), (2) soil mechanics tests (classification, plasticity, strength, grain size, and moisture-density), and (3) durability tests (slaking, soundness, and hardness).

Although all are important, the major factor is the degree of durability exhibited by the shale material and how this durability may be expected to change with time. The WES researchers were unable to confirm a single test which adequately covers the three modes of deterioration. They therefore recommended the battery of tests previously listed.

All of the tests investigated by WES suffered from at least one of the following drawbacks: they are not quantitative, experience with them is limited, they are too severe, or they are generally impractical.

The tests selected (X-ray diffraction, jar slaking, slake durability, scleroscope hardness, and pH) were based on the following criteria: previous success, general acceptability, time requirements, costs, simplicity of procedure, and indication of shale variability.

Problem shales and their variability

The WES researchers performed a limited assessment of shale deterioration as a cause of embankment distress by comparing index test results to embankment performance. Generally, it was found that problem embankments had intermediate to low slake durability indexes, low jar slake values, and high water contents. The WES researchers discovered exceptions to these general characteristics, which pointed to the need for conducting the complete battery of tests. For example, one case history embankment suffered considerable distress despite high slaking indexes and low moisture content. However, low pH values indicated pore water of high acidity conducive to chemical weathering which leads to the softening and deterioration of the shale materials. Accordingly, the WES researchers recommend the pH test be conducted to provide inexpensive information on the chemistry of the pore water and perhaps an indication of the ion exchange taking place in the clay minerals.

The jar slaking test was devised by the WES researchers to supplement the slower but more quantitative slake durability test described by Franklin and Chandra. (*4*) The latter test, which is accepted as a standard by the International Society of Rock Mechanics, measures resistance to disintegration from drying and agitation in a water bath (two cycles). Based on the WES investigation, which identified the parallelism between the two methods, materials testing engineers can take advantage of numerous jar slake tests (inexpensive, quick results) to provide continuous characterization of shale materials which could not be done economically by the expensive, time consuming slake durability test.

Phase II of the WES Study

Phase II of the study used the same data collection procedures as Phase I to identify and define the best available current practices of State and Federal agencies involved in shale embankment construction. Bragg and Zeigler (*5*) have divided embankment problems into two major types: settlement and slope failure. The causes of these problems are discussed in more detail than in Phase I and guidelines are presented for the evaluation and treatment of shale embankment distress. (*5*)

Embankment problems

The primary reason for shale embankment problems appears to be the susceptibility of shale materials to degrade or deteriorate after placement in the fill. The adverse effects of deterioration can be lessened significantly if the shale is processed as a soil, that is, thinner lifts, moisture-density controls, and effective subsurface drainage.

Since many highway engineers have not had the opportunity to develop a working knowledge of clay mineralogy and petrology, the identification and classification of problem shales should be presented in simpler terms if possible. Some investigators have separated shales into two categories—problem shales and no problem shales. Others prefer "soillike shales" and "rocklike shales." Perhaps the most descriptive shale classification scheme was developed by Purdue University and consists of the following categories:

1. Mechanically hard, durable.
2. Mechanically hard, nondurable.
3. Mechanically soft. (*3*)

The first and third category shales present very few problems to the highway engineer except when mechanically soft shales are found interbedded with harder shales, limestone, or sandstone. Category 1 shales can be treated as rock and placed in thick lifts without special processing methods. Category 3 shales are placed in soil lifts and controlled in the same manner as fine-grained soil materials.

Category 2 shales are responsible for the majority of shale embankment problems facing the highway engineer, especially when these materials are mistaken as Category 1 shales. The difficulty and expense of processing Category 2 shales as a soil in order to prevent distress caused by time-dependent deterioration within the fill can be significant if the shale is very hard and of low durability. The question of how much processing effort is required for this type of shale is still being studied. A liberal approach may result in subsequent maintenance and reconstruction problems, whereas a conservative design will be expensive to construct.

The report on Phase II of the study presents specific case histories which describe several large shale embankment failures that have occurred in various States. Some of the elements that usually combine with shale deterioration to cause failure are also noted. (*5*)

Evaluation techniques

When embankment and/or pavement distress reaches the point where routine maintenance procedures are ineffective for stopping or slowing the rate of distress increase, the extent of the shale embankment problem should be evaluated. Excessive settlement and surface slides could be indications of marginal stability. These signs of distress could be indications of shale deterioration occurring within the fill which could eventually lead to large-scale failures of the pavement and embankment slopes.

The evaluation process with which the highway geotechnical engineer must be knowledgeable is outlined as follows:

- Historical review.
  - —Design details
  - —Construction methods used
  - —Area geology
  - —Foundation conditions
  - —Materials data
- Instrumentation plan.
- Drilling, sampling, and site reconnaissance.
- Laboratory and in situ testing.
- Settlement analysis.
- Slope stability analysis.

After gathering the necessary data and performing the appropriate analyses, the highway geotechnical engineer must assess the validity of the information and use it to predict the future performance of the embankment. If only settlement is a problem, a forecast of the amount, rate, and location of future settlements must be made to assist in the planning of remedial treatments. If slope stability is marginal, the factor of safety must be determined and the potential for economical improvement investigated.

Remedial treatment

The material on remedial treatment techniques presented in the Phase II report is organized as follows: pavement overlay; drainage systems; slope flattening, berms, and buttresses; retaining walls; chemical stabilization and reconstruction. Surface and subsurface drainage measures are emphasized as an integral part of most remedial treatment methods. The advantages and drawbacks of each of these methods are discussed in detail. None of the cited approaches is particularly peculiar to the treatment of shale embankment distress; however, the discussion includes pictures, illustrations, and descriptions of actual case histories of repaired shale

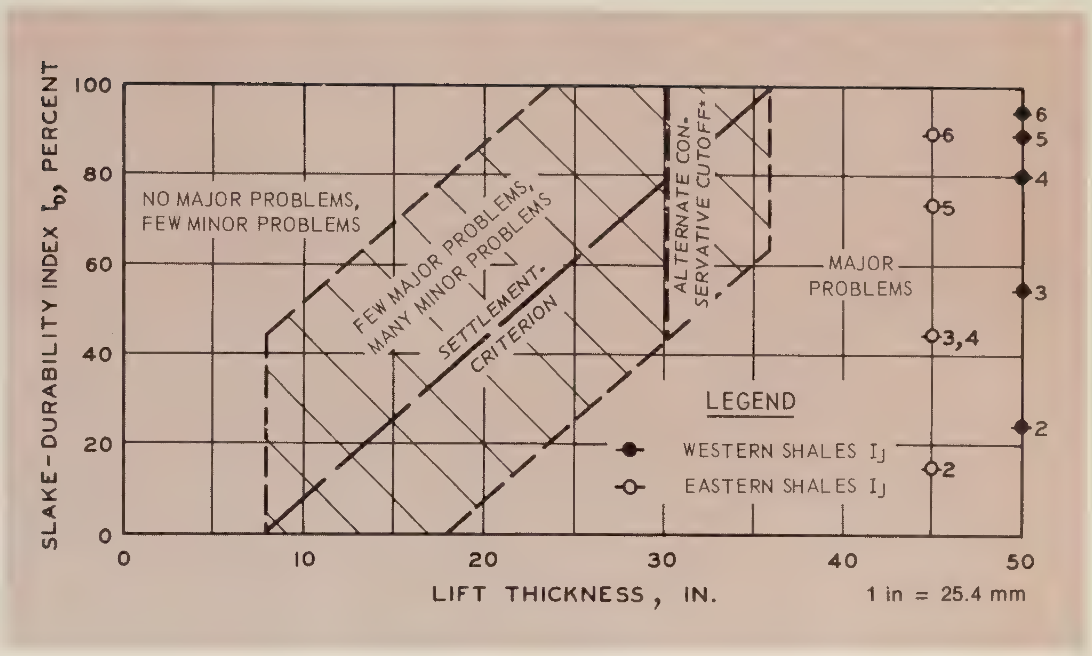


Figure 4.—Criterion for evaluating embankment construction on basis of behavior of materials.

embankments. A methodology for use in the evaluation of compacted shale embankments which shows signs of distress has been developed. The Phase II report describes this methodology in detail and includes a very helpful flow chart. (*5*)

Phase III of WES Study

Phases I and II were conducted simultaneously during the first year of the 4-year study. Until the overall study is complete and a technical manual is developed, the Phase I and II reports provide the best available treatise on compacted shale embankments. (*2, 5*) Phase III will be designed to fill in the gaps identified in the assessment of the state of the art.

One of the major tasks of Phase III involves the development of new index tests and/or the improvement of existing tests and the evaluation of them as techniques for obtaining compaction specifications for shale mixtures.

The WES researchers have developed a procedure for simply indexing any shale for valid predictions of the behavior of that shale in highway embankments. (*6*) Although field experiences played a big part in the development of this procedure, a complete evaluation of the recommended procedure through extensive on-the-job trial is still necessary. In fact, the data set used to develop the procedure was made up of contributions from various regions of the country; this indicates the need for each State to conduct a similar study to modify or supplement the WES procedure for its local shales and construction methods. A suggested work plan for this type of study was distributed to the appropriate States. Several States are presently conducting Highway Planning and Research studies of this type which will provide further validation data for the general WES procedure as well as the basis for local design guides.

Specific factors used to develop the WES procedure are as follows: (1) a battery of pertinent index tests, (2) test data on parent shale materials, (3) construction records of placement and processing, and (4) embankment

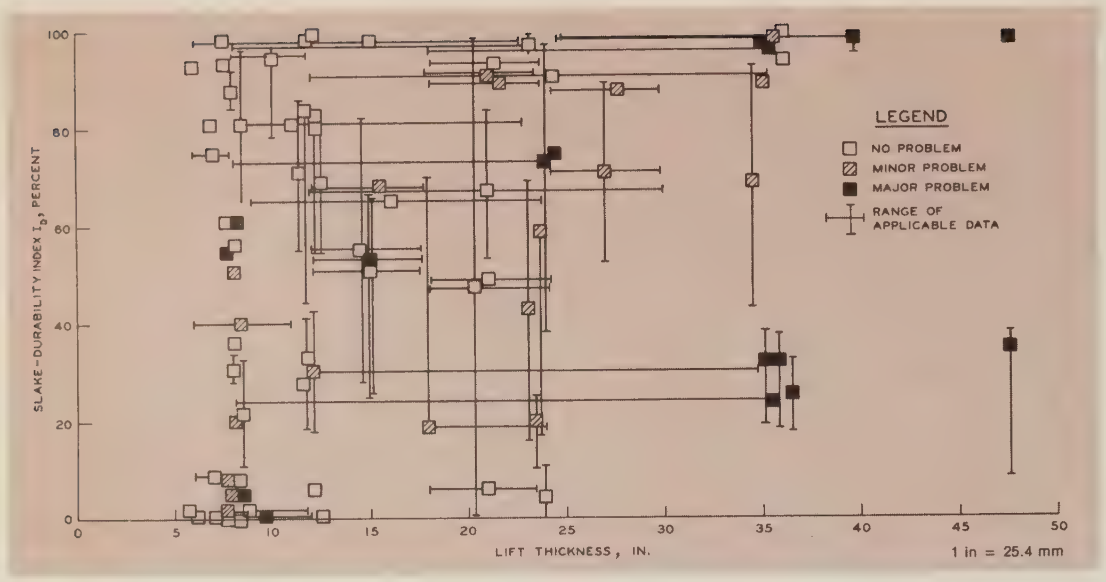


Figure 5.—Relationship between slake durability index (I_D), lift thickness, and performance of embankment.

performance data. (6) The collection of construction and service information is mostly new data gathered from the various State highway departments specifically for this study and provides a valuable data base for this and future research efforts by Federal and State agencies concerned with shale problems. Some of the data was obtained from the records of the State highway departments and some of it was generated by a comprehensive drilling, sampling, and testing program by the WES project team.

Several key embankments in five Eastern States (Indiana, Kentucky, Ohio, Tennessee, and West Virginia) were investigated by the WES researchers during the spring of 1976. Undisturbed samples were obtained from the inservice embankments and from the cut slope or borrow site where the embankment material originated. In situ strength tests (Menard pressuremeter) and other field data were recorded to supplement the lab data on the samples of "fresh" and "aged" shale materials obtained during the field study. Slaking and other index tests were also performed on groups of fresh samples representative of each formation and compared to embankment construction and performance data. The reference 6 report includes basic information to describe the various stratigraphical settings for the shale samples and the results of an examination of the variability of intrinsic properties (such as fissility, mineral composition, and cementing agents).

Durability index service criterion

In addition to the development of index tests for predicting the behavior of shale in an embankment, the WES researchers have developed a shale indexing criterion (fig. 4) which is based on the interrelationships of three factors:

- Slake durability index.
- Construction lift thickness.
- Embankment performance.

An analysis of these interrelationships is presented in graphical form in reference 6. Slake Durability Index (I_D) values are plotted against lift thickness, and the performance of each embankment is designated by the pattern of each plotted point to delineate a hypothetical performance criterion line or band separating problem fills from no problem fills (figs. 5, 6).

The durability index service criterion is useful both in design and maintenance operations. For example, the appropriate lift thickness to be specified for new construction can be determined from the graph by the I_D value obtained from slaking tests on "fresh" shale material and in accordance with the specified significance level. When a conservative approach is dictated by the nature of the embankment service (such as bridge approach fill, high type pavement, or buried utilities), a design value is picked from the low range of the criterion band. In cases where construction practice is anticipated to be exceptionally good or the consequences of postconstruction settlement are minor, the designer may select a lift thickness value on the high side of the band. The amount of post-construction settlement and the risk of slope failure are approximately related to the plotted distance from the criterion line which bisects the criterion band.

The evaluation of existing embankments can also be made provided the construction records contain the appropriate information on lift thickness and source of material. Knowing the location of the parent material, slake durability tests are run on samples of "fresh" material to obtain an I_D value. A plot of I_D versus lift thickness used during construction will give an indication of problems that may occur during the lifetime of the embankment.

The technical manual

The end result of the FHWA/WES research contract will be a comprehensive engineering and design manual to provide guidance on design and construction methodology for highway geotechnical and construction engineers at the operating level. A listing of subject areas which will be discussed in detail in the manual is shown in figure 7.

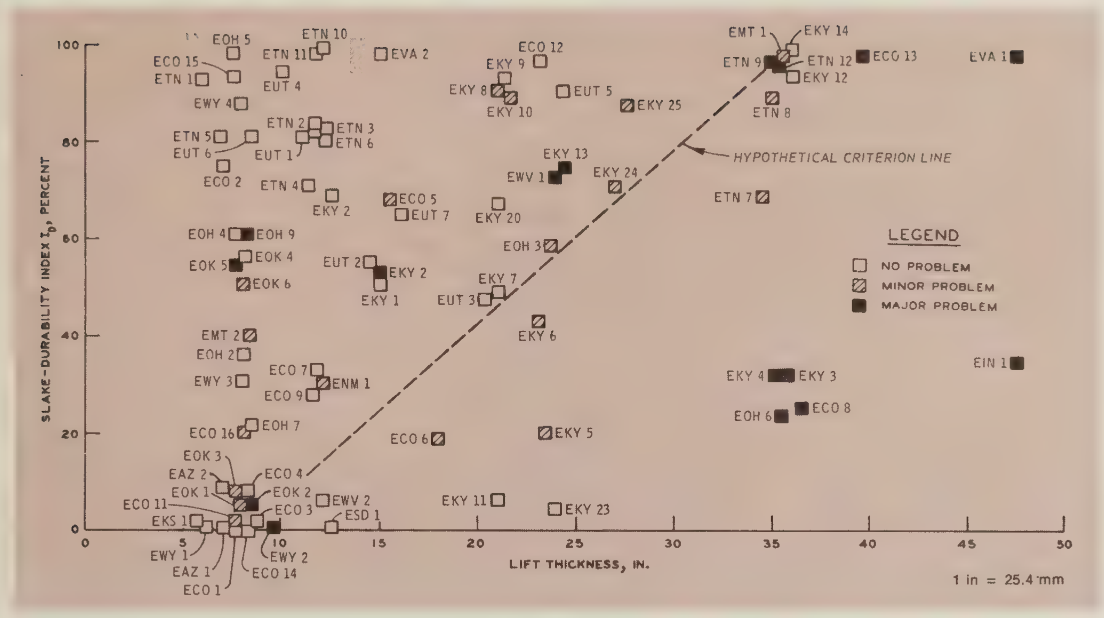


Figure 6.—Relationship between slake durability index (I_D), lift thickness, and performance of embankments with hypothetical criterion line of figure 4 superimposed.

The manual will be available in mid-1978, and distribution should occur during the fall of 1978.

A. Introduction
1. Background
2. Purpose
3. Scope
4. Applicability

B. General Considerations
1. Field exploration and sampling
2. Laboratory testing and classification of shales
3. Embankment design
4. Construction procedures

C. Evaluation Techniques

D. Remedial Treatments

E. Appendixes
1. Sample preparation and testing requirements
2. Test strip construction and testing
3. Case studies

Figure 7.—Shale technical manual outline.

Conclusions and Recommendations

Since shale materials exhibit a wide spectrum of engineering behavior, the highway designer must not treat all shales alike. Accordingly, it is recommended that designers identify and classify the shale materials to determine the amount of processing required. Other design and construction guidelines are also available in recently published reports. (*2, 5, 6, 7*)

Degradation and slaking are the shale properties of most importance to the highway designer. It is recommended that each shale material encountered in the soil profile be subjected to a battery of tests to assess the suitability of each shale for use in a compacted embankment. The battery of tests should include, but not be limited to, X-ray diffraction, slake durability, jar slaking, scleroscope hardness, and pH.

A relationship between the degree of construction processing, slake durability, and embankment performance has been identified and developed into a practical design tool by the WES research team. The procedure is presently being evaluated by several State highway agencies for adoption at the local level. Results from these local investigations will also be used to refine the generalized WES procedure. Accordingly, it is recommended that each State experiencing shale embankment problems undertake a similar study to relate performance with construction procedures and slake durability.

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What Network Simulation (NETSIM) Can Do for the Traffic Engineer

by Leslie Kubel, Gerald Bloodgood, Floyd Workmon, and David Gibson

This article is the first of two describing a practical traffic engineering analysis tool developed by the FHWA Offices of R&D. The first article describes the model, what is required to use it, what it tells the user in return, where to use it, its limitations, and costs. The second article will describe an application of the model to an analysis of a signalized intersection by the Traffic Engineering Division of the Utah Department of Highways. An important application of the model described in the second article is the simulation of a phasing arrangement which could not be studied in the field because it was not permitted by a State law.

NETSIM: An Introduction

The practicing traffic engineer has long needed a problem solving aid for evaluating the costs and benefits of alternative methods of traffic control. He or she has often found that any field experimentation was next to impossible due to limitations of time, cost, and the effects on motorists.

Simulation modeling has evolved as a tool with the advent of the computer. Modeling gives the engineer the ability to inexpensively choose the best of alternatives before actually committing financial resources to the implementation of the improvement in the field. In addition, it assists the traffic engineer in obtaining the most value for each dollar.

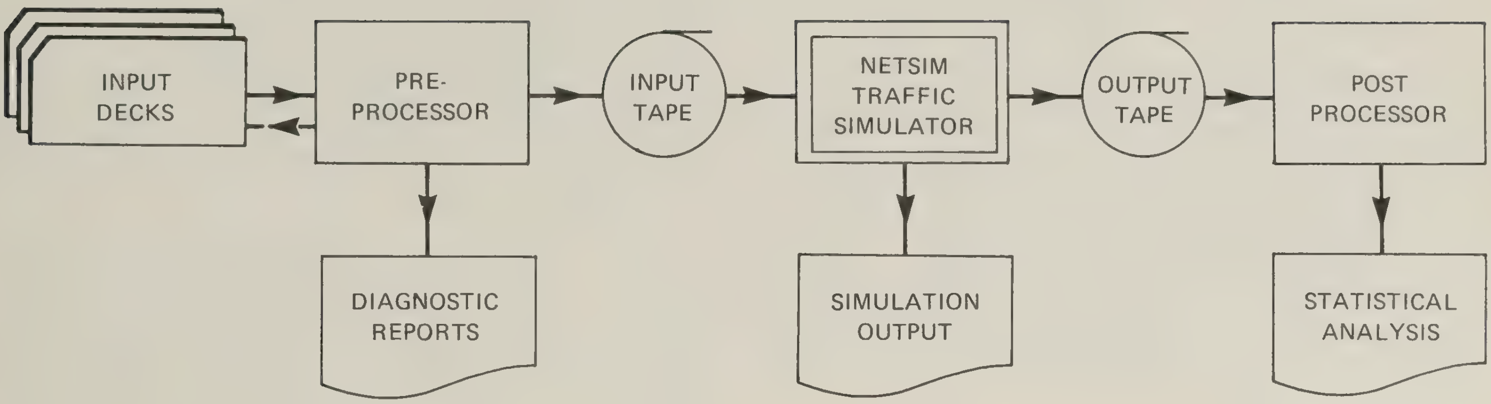


Figure 1.—NETSIM Model system.

The Network Simulation Model (NETSIM) is one such tool developed by the Federal Highway Administration to fill the traffic engineer's need. The basic elements of the NETSIM Model are shown in figure 1. The earlier, less powerful version of the model was called UTCS–1. The key to any modeling is in its ability to approximate "real world" conditions and resultant effects giving back meaningful results. The NETSIM Model has been validated against field data collected in Washington, D.C., Utah, California, and New Jersey. The model has been used successfully in numerous applications throughout the country in the last couple of years.

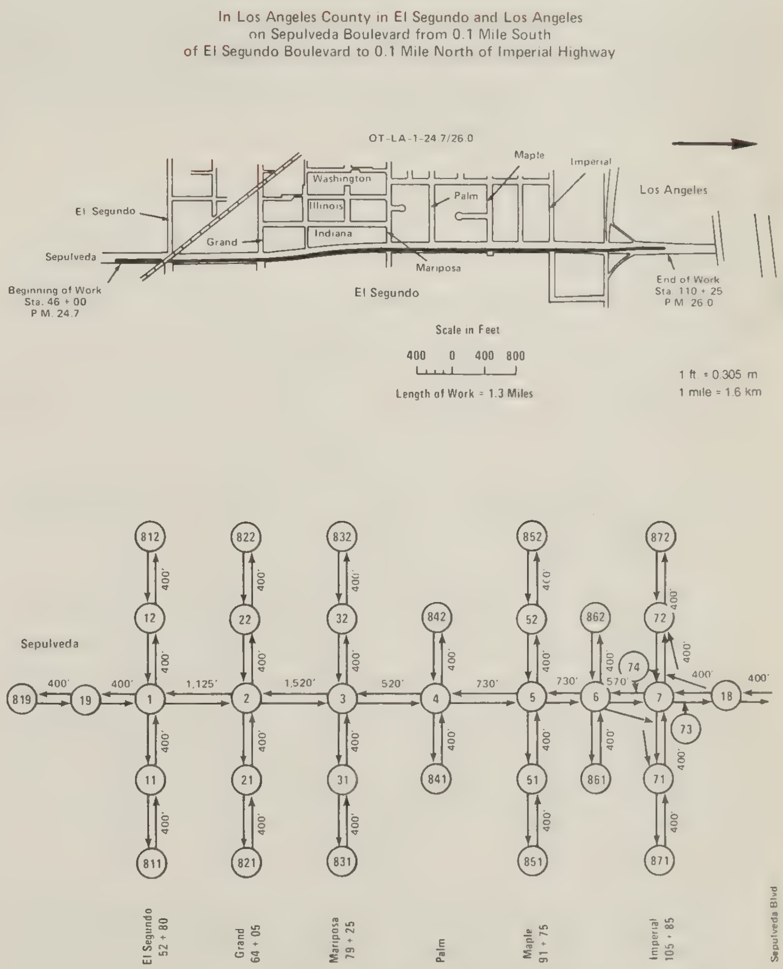


Figure 2.—Map of arterial and its associated link-node diagram.

What Is Required to Use It

A traffic engineer may say, "It's nice to have this new tool, but what's required from me in time, manpower, and machinery to use it?"

One should not be misled as to the minimum work requirement and the degree of expertise needed to use this simulation program. Like most large programs, the model requires a large computer. The present generation program uses approximately 500K Bytes of memory (290K Bytes overlaid). It required 5 man-days of experienced programer time to bring the program on board the State of California IBM 370–168 computer. Because of this, implementation of the simulation model package would most likely be achieved first by larger cities and States which have the expertise and computer facilities to use the package effectively. However, smaller municipalities could obtain aid from either larger agencies or knowledgeable consultants to evaluate a proposed project. This type of arrangement has been tried in California and Utah and is quite effective.

The most important requirements for use of the model by a traffic engineer are proper data input and the accuracy of the *link-node diagram*.

The first step taken in use of the model is the construction of a link-node diagram which represents the actual street network in the computer. Figure 2 shows a map of a small arterial and its associated link-node diagram. Links are the stretches of roadway interlinking nodes. They are unidirectional and may be either entry/exit type or internal to the system under study. Nodes are points at which vehicles enter, exit, or are controlled, such as a signalized intersection. The diagram should show the length of the link and direction of movement.

Once the proper link-node diagram has been drawn, the engineer must gather the necessary input data. This includes entering count data, turning movement counts (percentage or raw), road and intersection geometrics, channelization, type of control, operational timing desired, and detector placement if used. With this information tabulated, the engineer proceeds to code the simplified 80-column card forms (fig. 3) by use of the user's manual supplied with the model.

Upon completion of the form coding, the coded data is keypunched onto the 80-column FORTRAN cards. These cards should be verified and joined with the JCL (Job Control Language) Deck Cards. The engineer would now be ready to simulate using his or her computer system.

What the Printouts Tell You

The model printout first gives a listing of the input card deck (fig. 4). It then prints out link and network statistics (fig. 5).

If emission and fuel consumption results are desired, the coding of the type 99 cards will give tabulation and printout results in both a link-by-link and networkwide basis (fig. 6).

From these statistics, the engineer can formulate what effect a particular control strategy, control device, or channelization change has on a single link or on the total network.

The effects of a strategy are shown in changes in the number of stops per vehicle, stopped delay, total delay, travel time, signalized cycle failure, fuel consumption in gallons (litres) and mileage in miles per gallon (kilometres per litre), and the emissions generated (HC, CO, and NOX) in grams per mile

By controlling the variables, the traffic engineer can properly model the effects of each change to a system and the resulting benefits derived.

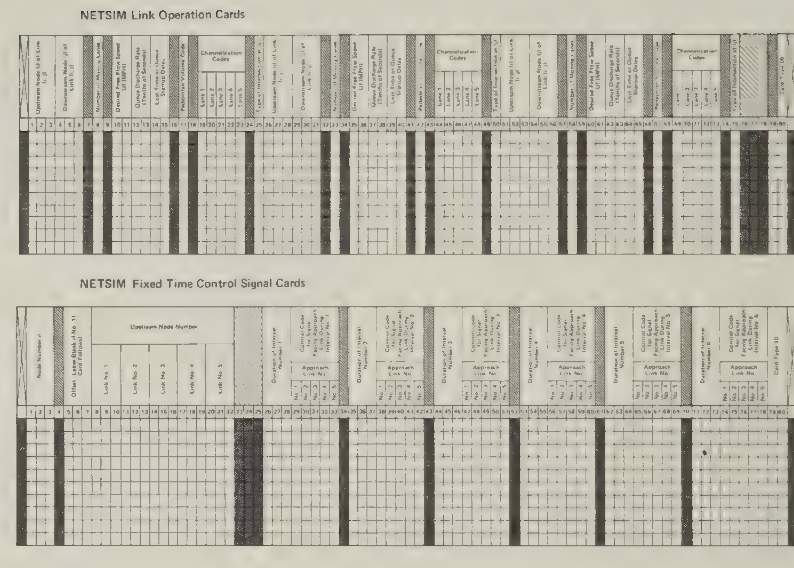


Figure 3.—NETSIM coding format.

CARD FILE LIST

```
TRAFFIC ACTUATED   VOLUME DENSITY   03-SUT-20                                    0
GRAY AVE              YUPA CITY                 CA                8 17 76       1
-400  1108                                                                       3
802   2 400        1       803    3 400        1                                 4
804   4 400        1       805    5 400        1                                 4
  2   1 4000810  4  3  5     3    1 4000910  5  2  4                             4
  1   2 4000      802        1    3 4000      803                                4
  4   1 4000 12  3  5  2     5    1 4000  6  2  4  3                             4
  1   4 4000      804        1    5 4000      805                                4
807   2 2        3 00      803  3 2          3 00                                5
804   4 2        3 00      805  5 2          3 00                                5
  2   1 2 35     3 00        3  1 2 35       0 00                                5
  4   1 2 30     3 00        5  1 2 30       3 00                                5
  1   2 2 35     3 00        1  3 2 35       0 00                                5
  1   4 2 30     3 00        1  5 2 30       0 00                                5
  2   1 10 85  5        3   1 20 75  5        4  1 35 50 15      5  1 25 50 25   7
  1   2    100          1   3    100          1  4    100        1  5    100     7
802   2    100        803   3    100        804  4    100      805  5    100     7
  2     802  1              12011                                               10
  3     803  1              12011                                               10
  4     804  1              12011                                               10
  5     805  1              12011                                               10
  1  A1100     1  2 20          2 20      3 0              2  3  4  5           15
  1  A0100     1  2 20          2 20      3 0              2  3  4  5           15
  1  B0101     3  5 40          2 5030 8  3 1              2  3  4  5           15
  1  C1100     1  2 20          2 20      3 0              2  3  4  5           15
  1  C0100     1  2 20          2 20      3 0              2  3  4  5           15
  1  D0100     1  2 30          1 40      3 1              2  3  4  5           15
  1  A11  34    5                4455          4455 5455                        16
  1  A02  34    5                5155          5955 5955                        16
  1  B01234    12     12         9955          9955 5555                        16
  1  C14  12    5                5544          5544 5545                        16
  1  C03  12    5                5515          5595 5595                        16
  1  D03412    12     12         5599          5599 5555                        16
802   2 684 5 80 3  3 728 5 804  4 460 2 805  5 39210                           20
  2   101 23002  23015   90                                                     25
  3   101 23002  23015   90                                                     25
  4   111 14512  14515 100                                                      25
  5   111 14512  14515 100                                                      25
 300               1200   A                                                     60
```

Figure 4.—Input data restatement.

CUMULATIVE STATISTICS SINCE BEGINNING OF SIMULATION

PRESENT TIME IS 11 13 0, ELAPSED SIMULATION TIME IS 5 MINUTES, 0 SECONDS

LINK STATISTICS

LINK	VEH-MILES	VEH TRP	MOV. TIME V-MIN	DELAY TIME V-MIN	M/T	TOTAL TIME V-MIN	T-TIME /VEH. SEC	T-TIME/ VEH-MILE SEC/MILE	D-TIME /VEH SEC	D-TIME/ VEH-MILE SEC/MILE	PCT STOP DELAY	AVG. SPEED MPH	AVG. OCC.	STOPS /VEH	AVG SAT PCT	CYCL FAIL
(2, 1)	4.2	56	7.6	9.6	0.44	17.2	18.4	243.2	10.2	135.3	38	14.8	3.4	0.43	6	0
(3, 1)	4.7	62	8.0	12.9	0.39	20.9	20.2	267.1	12.4	164.3	46	13.5	4.1	0.34	7	0
(1, 2)	4.5	60	7.9	2.9	0.73	10.8	10.8	143.9	2.9	39.1	0	25.0	2.2	0.0	6	0
(1, 3)	5.2	69	8.8	4.2	0.68	13.0	11.3	150.1	3.6	48.3	10	24.0	2.6	0.07	8	0
(4, 1)	2.7	36	6.0	8.2	0.42	14.2	23.7	313.1	13.6	180.0	54	11.5	2.8	0.81	6	0
(5, 1)	2.5	33	5.0	6.3	0.44	11.3	20.5	271.0	11.4	150.8	44	13.3	2.3	0.79	5	0
(1, 4)	1.9	25	4.0	1.6	0.71	5.6	13.4	178.9	3.9	51.9	0	20.1	1.1	0.0	3	0
(1, 5)	2.4	32	4.9	2.1	0.70	7.1	13.2	175.9	4.0	52.7	6	20.5	1.4	0.16	4	0

NETWORK STATISTICS

VEHICLE-MILES = 28.15 VEHICLE-MINUTES = 100.1 VEHICLE-TRIPS (EST.) = 187 STOPS/VEHICLE = 0.59

MOVING/TOTAL TRIP TIME = 0.523 AVG. SPEED (MPH) = 16.88 MEAN OCCUPANCY = 19.8 VEH. AVG DELAY/VEHICLE = 15.32 SEC

TOTAL DELAY = 47.7 MIN. DELAY/VEH-MILE = 1.70 MIN/V-MILE TRAVEL TIME/VEH-MILE = 3.56 MIN/V-MILE

STOPPED DELAY AS A PERCENTAGE OF TOTAL DELAY = 36.2

1 mile = 1.6 km

Figure 5.—Link and network statistics.

Figure 6.—Emission and fuel consumption results.

CUMULATIVE VALUES OF FUEL CONSUMPTION AND OF EMISSIONS

LINK	FUEL CONSUMPTION						VEHICLE EMISSIONS (GRAMS/MILE)								
	GALLONS			M.P.G.			HC			CO			NO X		
	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	4.62	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 gal. = 3.8 l
1 mile = 1.6 km

BUS STATISTICS

LINK	NUMBER PROCESSED	MOVING TIME MIN	DELAY TIME MIN	M/T	NUMBER OF STOPS
(3, 4)	5	2.0	0.8	0.71	0
(4, 6)	5	2.1	0.5	0.82	2
(6, 5)	5	1.1	0.7	0.62	0
(5, 4)	5	1.2	0.0	0.98	0
(4, 3)	5	2.2	2.0	0.53	5
(70, 6)	5	1.4	0.7	0.68	1
(6,62)	5	0.4	0.6	0.38	0
(62, 6)	5	0.5	1.9	0.20	4
(61, 6)	5	0.5	2.0	0.21	4
(70, 7)	5	3.2	0.9	0.78	2
(7, 8)	5	1.2	0.9	0.55	1
(6,70)	5	1.5	0.5	0.76	0
(31, 3)	5	0.7	1.2	0.35	3
(3,31)	5	0.7	0.7	0.48	0
(6,61)	5	0.5	0.5	0.49	0
(7,70)	5	2.5	1.0	0.71	0
(71, 7)	5	0.6	1.6	0.27	5
(8,81)	5	0.6	0.7	0.49	0

BUS ROUTE DATA

ROUTE	BUSES PROCESSED	TOTAL TRAVEL TIME MIN	TOTAL DWELL TIME MIN	AVG. SPEED MPH
1	5	8.7	0.0	21.8
2	5	3.3	0.0	10.3
3	5	11.2	0.0	16.8
4	5	14.7	0.0	26.0

SEED FOR RANDOM NUMBER GENERATOR IS 76539661

1 mph = 1.6 km/h

Figure 7.—Bus statistics output.

It should be noted that the NETSIM Model is about the only reasonable technique in use to evaluate quantitative effects of traffic improvements in gas consumption and air pollution. This should prove extremely useful in evaluating environmental and energy considerations.

Where the Traffic Engineer Can Use It

There are many applications for this engineering tool. As previously mentioned, the engineer might desire to know the effects of a particular control strategy, control device, or channelization on a stretch of roadway. Also, in viewing this approach the engineer may change either the traffic volumes, turning movement percentages, or type of vehicles using the system. The advantage of a controlled environment represented by the NETSIM Model package offers the traffic engineer the ability to make an objective study of the benefits and cost trade offs of each variation.

Some typical applications of this model are as follows:

- Single isolated intersections simulating control methods and operational timing such as either full traffic actuated volume density signal control, two- and four-way stop signs, or fixed time signal control.
- Small to large arterial networks—effects of signal timing and coordination.
- Central business district grid networks and the effects of adding bus lanes.
- Effects of parking lot or shopping center development on the surrounding area.
- Effects of change in signed regulations, such as no-left-turn restrictions and no-parking regulations, on the network.
- Effects of geometric changes such as additions of left turn bays or right turn pockets.
- Use of the model to track a vehicle through the system and show the direct effects of change in travel time and so forth by altering the control or timing operations. This is a direct result benefit in dealing with a complaint from one or more motorists.
- Installation of future signal systems and the effects on surrounding systems.
- Analysis of the effects of the signal system on bus operation (fig. 7).

Limitations of the Model

The model has certain limitations in its present state. Considerable effort is being devoted to correct these deficiencies and upgrade the model's capabilities. None of the problems seriously demean the model. Valid useful results are presently being obtained from its application.

Some of these limitations are as follows:

- The inability to model all currently available real time control algorithms.

- The current generation program has a maximum of 160 links, 99 nodes, 60 entry links, and 1,600 vehicles tracked within the system.

- Extensive checking of input data validity and careful review of output results for reasonableness are required. Erroneous conclusions can be reached due to errors in input coding or careless construction of the link-node diagram.

- Non-standard traffic conditions require reliance on ingenuity of the user to transform the conditions into proper input.

- Any simulation model, no matter how complex, is a simplification of the real world.

What Does It Cost Per Return

The costs to model naturally vary upon the size of the system under study and the number of variations one wishes to simulate.

The following two examples of actual modeled systems show expected costs one might incur in typical applications. The examples are limited to two modeled variations, but they do project actual costs to run an initial system simulation and to vary the control of the system. It should be noted that man-hour costs assume that the person performing the tasks has a working knowledge of the NETSIM Model.

Costs not considered in the examples are a one-time cost of the 40 man-hours needed to bring the NETSIM program online and the data gathering expense. Data gathering costs may be minimal if the data is already in-house such as is usually the case for traffic counts, geometric information, and signal timing parameters. Other data may require onsite study which obviously will cost more than the data available in-house.

The dollar cost for CPU time is roughly estimated at $25 per minute on the State of California's IBM 370–168, based on overnight service and peripherals used for this program.

Example A

Single Intersection Signal System
Yuba City, California
03–Sut–20 PM 15.7
Colusa Avenue at Gray Avenue

(a) Initial run

1. Drawing link-node diagram and coding the initial system forms	16 man-hours[1]	$320.00
2. Keypunch and validate 40 data cards	$ .05/card	2.00
3. CPU time—full traffic actuated control, simulation study period of 15 minutes	24.8 seconds	10.50
4. Analysis of printout	1 man-hour	20.00
	Cost per initial run	$352.50

[1]Man-hour = $20/hour (including overhead)

(b) Variation run (semi-actuated control)

1. Modifying five data cards including coding and keypunch	4 man-hours	$ 80.00
2. CPU time—semi-actuated control simulation study period of 15 min.	28.8 seconds	12.00
3. Analysis of printout	1/2 man-hour	10.00
Cost per variation run		$102.00

Example B

Arterial System—One-mile (1.6 km) length with five signalized intersections in Los Angeles, California
07–LA–1 PM 24.7/25.9
Sepulveda Boulevard—Imperial to El Segundo

(a) Initial run

1. Drawing link-node diagram and coding the initial system forms	40 man-hours	$800.00
2. Keypunch and validate 142 data cards	$ .05/card	7.10
3. CPU time—coordinated semi-actuated control with study period of 15 min.	181 sec.	75.90
4. Analysis of printout	2 man-hours	40.00
Cost per initial run		$923.00

(b) Variation run (isolated full traffic actuated)

1. Modifying 40 data cards incl. coding and keypunch	8 man-hours	$160.00
2. CPU time—study period of 15 min.	205.0 sec.	85.00
3. Analysis of printout	1/2 man-hour	10.00
Cost per variation run		$255.00

These examples assume errorless coding and proper traffic logic. Fortunately, the computer program will detect many coding errors, but valid traffic logic is imperative. Care must be taken so that time and machine expenses can be kept to a minimum.

Summary

NETSIM is a tool which can reliably predict the effects of a traffic control strategy on an urban network and quantify these effects as stops, vehicle delay, gasoline consumption, and grams/mile (g/km) of pollutants. However, NETSIM does require a substantial investment of man-hours to prepare the data in a form acceptable to the computer and to prepare variation runs when a set of control strategies are being compared. Thus, the NETSIM Model is a valuable tool for analysis of alternatives but cannot be considered an optimization tool. An application of NETSIM to analyzing an individual intersection will be described in detail in the next issue of *Public Roads*.

The Fifth Annual Review of the FCP

by Debbie DeBoer

Research and development is an important function in many highway agencies. Because of this, there has always been a need for coordination of the efforts of various agencies to eliminate duplication of efforts. The role of the Federally Coordinated Program of Highway Research and Development (FCP) is to accomplish this task. Federal Highway Administration (FHWA) research and development personnel handle the overall coordination. They review all State highway agency research and development studies which are in part Federally funded. In addition, this review process provides a project focal point so that the objective of each study relates and contributes to all other activities related to the project.

Communication among participants in the FCP is necessary if the program is to work. Beginning in 1973, weeklong annual conferences have been held to aid in this communication. The objective of the FCP conferences is threefold: (1) to

R. C. Leathers, Director, FHWA Office of Development, welcomes Ohio conference participants.

Speakers at the welcoming session of the Atlanta conference (left to right): V. Crawford and D. O. Covault, both of Georgia Institute of Technology; D. Moreland, Commissioner and State Highway Engineer, Georgia Department of Transportation; H. Bryant, Georgia Division Administrator; J. D. Lacy, Region 4, Federal Highway Administrator; and C. F. Scheffey, Director, FHWA Office of Research.

D. L. Garwood, Assistant Director, Ohio Department of Transportation, at the opening session of the Ohio conference.

Ohio Department of Transportation's electric vehicle.


Atlanta conference participants involved in a project review.


Conferees at the opening session of the Atlanta conference.

review selected projects from the FCP, (2) to bring together members of each project team in an atmosphere of open discussion, and (3) to test the research approach of each project by discussion with individuals from State highway departments, operating offices, and FHWA field and headquarters offices.

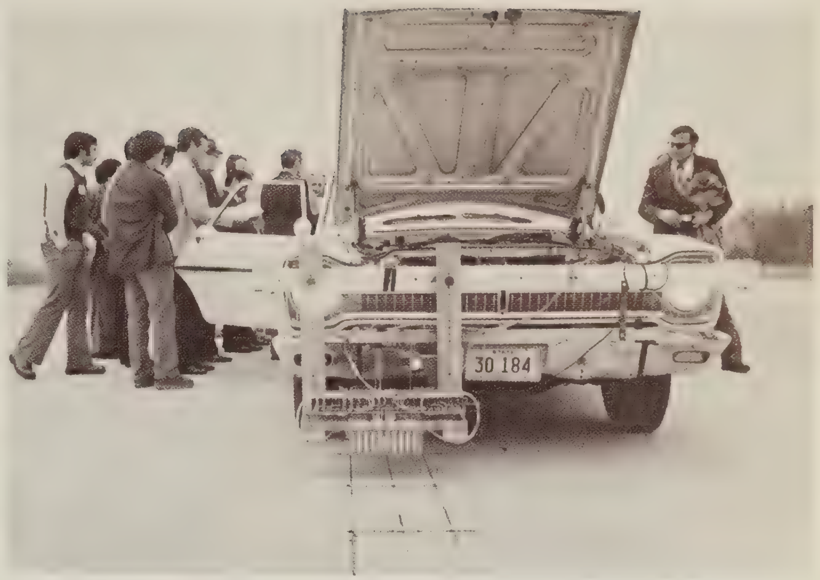
Because of the growth of the FCP, two weeklong sessions have been held for the past 2 years. Over 300 persons attended the first 1977 session which was held at the Atlanta Internationale Hotel in Atlanta, Ga., the week of October 3. The second session, which had 350 participants, was held the week of November 7 at the Fawcett Center for Tomorrow, Ohio State University, Columbus, Ohio.

The Atlanta session reviewed projects in Categories 4 and 5 of the FCP: Improved Materials Utilization and Durability, and Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety. After introductory remarks at the Monday morning opening session, overviews of the two categories were presented. Following this opening session, review of specific projects began and continued through the week.

Each project was introduced by the FHWA project manager, who was also the moderator for the entire review. Following the introduction, presentations were made by principal investigators on the project. Open discussion was an important part of each review.

This year the Third Annual U.S. Department of Transportation (DOT) Tunneling Conference was held in conjunction with the first FCP conference. It began on Thursday, October 6, and attendees of the FCP conference were invited to attend the sessions. After a keynote address on the need for research in tunneling technology, the general sessions began. There were four main sessions; the first three opened with an introduction and overview, followed by presentations of the keystone projects in that subject area. Session I was concerned with "Innovations in Planning, Analysis, and Environmental Concerns" and dealt with research sponsored by the Office of the Secretary. The second session, dealing with Urban Mass Transportation sponsored research, was entitled "Innovations in Ground Movement Prediction and Control." Session III, "Innovative Techniques in Site Exploration," presented FHWA sponsored research. The final session was a panel discussion critique of the DOT tunneling program.

Review of Project 4H, "Improved Foundations for Highway Structures."



Automated vehicle being tested at the Transportation Research Center of Ohio.

In addition to the DOT Tunneling Conference, other outside activities at the Atlanta conference included a tour of the Metropolitan Atlanta Rapid Transit Authority's Peachtree Station Pilot Tunnel and Test Section, and tours of Georgia Institute of Technology and Georgia DOT's research facilities.

The Columbus, Ohio, conference reviewed projects in FCP Categories 1, 2, and 3: Improved Highway Design and Operation for Safety; Reduction of Traffic Congestion, and Improved Operational Efficiency; and Environmental Considerations in Highway Design, Location, Construction, and Operation. A session on Transportation Systems Management was also held.

This second conference was very similar in format to the Atlanta conference. Monday morning there was an opening session with introductory remarks and an overview of the FCP. Following this the project reviews began; they were conducted in the same manner as at the Atlanta conference.

The Ohio conference included a tour of the Transportation Research Center of Ohio (TRC), one of the largest research facilities in the world. It has test facilities such as a crash simulator, a 7 1/2 mile (12 km) test track, rough road courses, and technical and support equipment. Ohio State University in conjunction with the FHWA uses TRC's skid pad for automated vehicle research and also tests skid trailers for State highway departments.

This conference also included an exhibit of an electric vehicle developed by the Ohio DOT. An engineer was available for discussion of the vehicle's performance and function. A *Public Roads* magazine exhibit, including copies of the magazine and a questionnaire, was also on display at Fawcett Center.

Ohio conferees attending a project review.

In addition to the formal sessions, there were occasions for the conference attendees to meet informally with their colleagues and discuss topics of common interest. These occasions included the coffee breaks, daily luncheons, and, at the Ohio conference, the Wednesday night banquet.

The two-part fifth annual FCP conference gave participants a chance to evaluate and discuss the progress that has been made in the FCP since its origin in 1969. As in previous years, the conference was an opportunity for communication between research and operations personnel. This type of forum for discussion and exchange of information is necessary if the FCP is to be a vital part of the national highway program.

Design of Segmental Bridges

by John E. Breen and
Craig A. Ballinger

Corpus Christi Bridge under construction.

This article is the second part of a paper presented at the 1976 Federally Coordinated Program of Research and Development Conference at Pennsylvania State University, during the review of Project 5F, "Structural Integrity and Life Expectancy of Bridges." It reviews the current state of the art for the design of segmental prestressed concrete box girder bridges. The first part of the article, which appeared in the December 1977 issue of *Public Roads*, **covered general design considerations, design sequence, conceptual and detailed design requirements, and analysis procedures. This part will cover substructure design, specifications, construction techniques, and materials considerations; followed by a brief description of the current status of segmental bridge construction in the United States.**

Pier Design

In most of the existing literature on precast segmental box girders, insufficient attention is given to pier design. Since the cantilever erection procedure imposes substantial moment requirements on the pier, it can greatly increase the cost of the piers. Several cases have been reported where the *increase* in pier cost to permit balanced cantilever construction amounted to 25 percent of the total superstructure cost. Careful attention should be given to the possibility of providing for the unbalanced moment with temporary struts, ties, or shoring, as shown in figure 1, so that the permanent pier does not have to have the built-in capability of resisting the full moment that occurs only during construction. In addition, considerable savings can be realized by using hollow piers that can develop the required strength and stiffness, but that do not need as much material as the solid piers nor require as many additional vertical supports. In difficult water crossings, the pier costs may be of the same magnitude as the superstructure costs and it is extremely important that careful attention be paid to the pier design. Several recent examples have indicated that erection on falsework is practical even in long spans if pier costs are high. Attention should be given to development of design procedures for slender, tapered piers of cellular cross section.

Applicable Specifications and Regulations

In design of relatively modest (up to 400-ft [122 m] span) segmental box girder bridges, existing design regulations are reasonably adequate. The examples in Report 121–3 utilize the 1973 American Association of State

Highway Officials (AASHO) regulations, the American Concrete Institute Building Code (318–71) provisions for shear and prestressed concrete as allowed by AASHO for prestressed concrete shear, and the 1969 Ultimate Design Criteria of the Bureau of Public Roads (BPR). (*1–4*)[1] This latter was used rather than the 1973 AASHO regulations because the designers were concerned about the combined load and ϕ factors permitted for this type of construction in the ASSHO regulations.

In the 1969 BPR ultimate design criteria, the basic load factors are 1.35 DL + 2.5 LL. In addition, the values of ϕ for flexure are 0.9 and for shear are 0.85. For this bridge type in the critical stage when cantilevering is almost complete, the structure is almost 100 percent dead load. The "safety factor" in flexure under the BPR criteria would then be $1.35 \div 0.9 = 1.5$. Using the 1973 AASHO regulations, the load factor would be 1.3 dead load and a ϕ factor of 1 could be used, since this could be interpreted as "factory produced precast prestressed concrete members." (*2*) This would give a total safety factor of 1.3 at this critical stage. The designers considered this insufficient with the relatively "tricky" construction and erection procedures required. The entire ultimate design concept for a segmental bridge differs greatly from that of a conventional structure, as shown in Report 121–5. (*5*) Regulations should reflect this difference.

[1]Italic numbers in parentheses identify references on page 180.

Other Types of Bridges Constructed in Cantilever

Although Report 121–3 (*1*) treated design of certain types of bridges constructed in cantilever, variations in the design procedure should be similarly documented for other types, such as those discussed below.

Multicell box girder

An alternative cross section is a multicell box girder, cast in full-width sections. The design procedure for this case is almost identical with the simpler box sections. However, temperature and shrinkage effects may be of more importance in this type.

Segments lifted from bridge superstructure

Recommendations are needed for construction load factors. In this case the impact load on the cantilever should be much higher (possibly 100 percent) and the live load and impact moments may constitute a substantial fraction of the total moment during construction. If this is so, it is probably best to design the top cables to balance a uniform load together with a concentrated load at the ends of the cantilevers (that is, at the center and ends of the completed bridge).

Superstructure rigidly connected to pier

Instead of the final simple support system used at Corpus Christi, it is possible to have the segments above the main piers permanently rigidly fixed by vertical prestressing cables. This will considerably modify the construction procedure and hence the design. The cantilever erection process will not be altered, but because of the fixity at the main piers it will no longer be possible to adjust the moments in the completed structure simply by jacking at the ends. Before closure at midspan, flat jacks will have to be inserted between the final segments at deck level and pressure applied to induce a positive moment. Tolerances will be more critical. Closure and placing of the main span bottom cables may be done before placing the side span bottom cables if desired. Specific design criteria and tolerance requirements for this type construction are needed.

Side span greater than half main span

If the side span is greater than half the main span, the final segments in the side span cannot be readily erected by the cantilever method. The simplest procedure is to erect the superstructure by cantilever on either side of the main piers to a distance of half the main span (minus the gap for the closing segment). The remaining segments in the side spans can be erected on falsework. Closure and insertion of the main span bottom cables can be done either before or after completion of the side spans, depending on the details of the structural system. Alternate systems should be evaluated.

Continuous viaducts

The construction and design of viaducts, comprising a large number of equal continuous spans, presents no special difficulties. However, provision must be made for expansion and careful attention paid to joint location and pier-girder connections. Muller treats this problem in some detail. (6) Specific design and construction recommendations should be developed for this case.

Other considerations

Cold weather construction. Substantial advantage may accrue from extending segmental construction seasons by development of cold weather epoxy

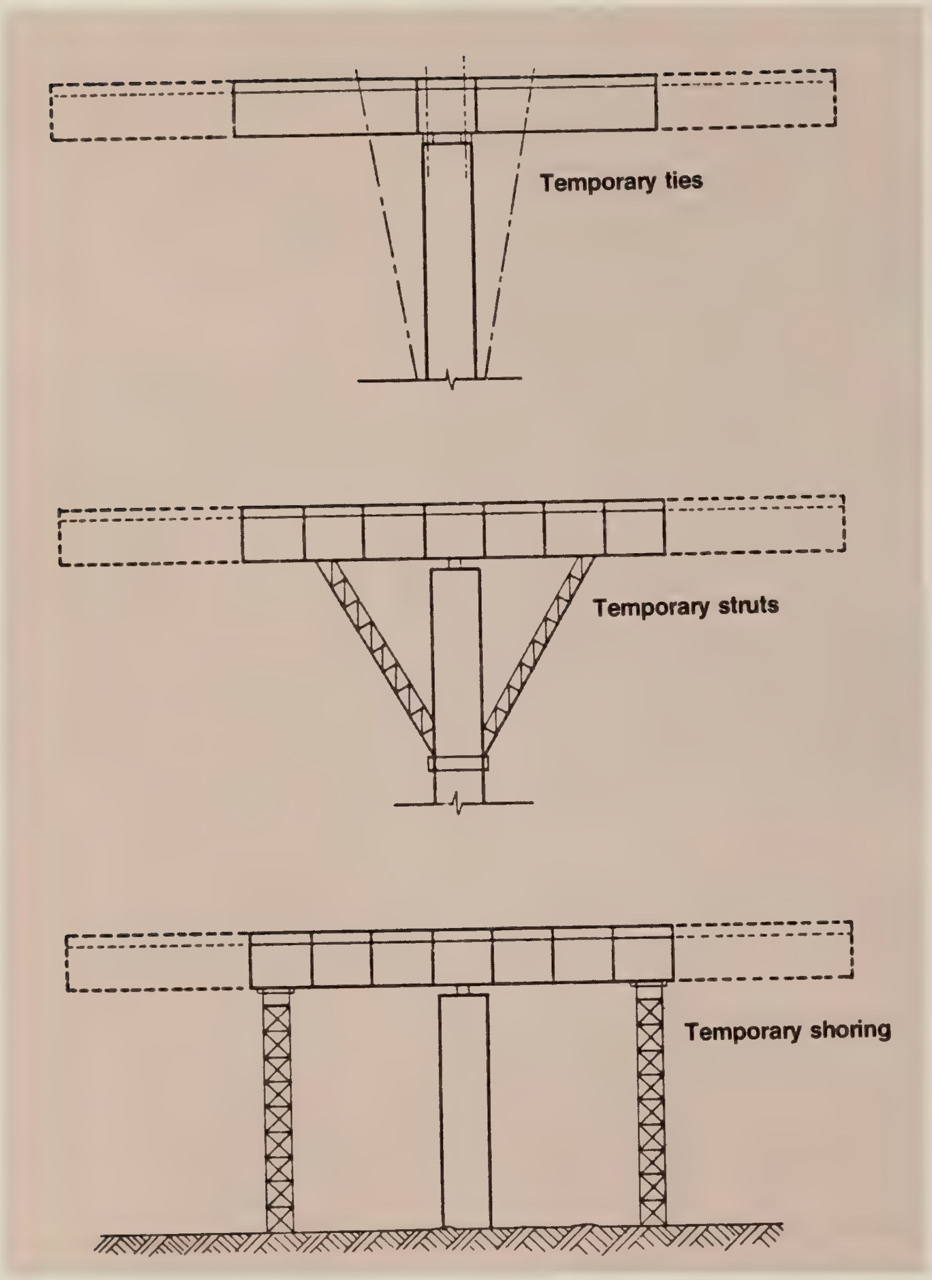


Figure 1.—Provisions for unbalanced moment.

formulations or detailed recommendations for accelerating epoxy curing by localized heating. Experimental programs should be undertaken to evaluate possible solutions to the cold weather problem.

Geometric changes. Important design cases not considered in detail include skew bridges and bridges with variable depth. These should be investigated more fully.

Highway crossovers will often be skewed. The design and analysis of such skew bridges will require modifications to the procedures developed for normal bridges. A few computer programs are now available for analyzing box girder skew bridges and these could be used in the way MUPDI was used. Skew will also create some difficulties with segmental precasting. One solution is to cast a few segments near the ends of the span on the skew and the remaining segments normal to the roadway axis. Much will depend on the pier location details in this case.

Bridges having spans greater than 250 to 300 ft (76 to 91 m) generally vary in depth from a maximum at the piers to a minimum at midspan. In this way greater economy can be achieved. To a large extent the design procedure developed is applicable to bridges with variable depth. The finite element analysis program FINPLA2, developed by Scordelis, can be used for these bridges in place of the MUPDI analysis. (*1*) Unfortunately, no program corresponding to SIMPLA2 exists for this case. Such a program must be developed.

Optimization. The optimization techniques should also be extended to cases where the bridge span is a variable, as well as the cross sectional dimensions. Examples of such cases include three-span viaducts and river crossings, in which the total length is specified but not the span ratio. A wider study of effects of variables such as span and roadway width as well as pier cost should be undertaken using optimization techniques to provide further guidance for preliminary designs.

Strength

The general adequacy of ultimate design procedures for segmental precast box bridges with epoxy joints was conclusively demonstrated in the comprehensive one-sixth scale model test of the Corpus Christi bridge. (*5*) The test showed the following:

- The segmental bridge model safely carried the ultimate design loads for all critical moment and shear loading configurations on which its design had been based.
- The deflection under design live load in four lanes (only three lanes required by live load reduction factors) was approximately L/3200 in the main span. This is much smaller than L/300 which is generally considered as acceptable.

• Positive tendons in the main span were designed as if for an ideal three-span continuous beam. Since the completed bridge was supported on neoprene pads which have no vertical restraint against uplift, the outer ends were able to rise off their supports so that the structure did not act continuously at ultimate conditions under main span positive moment loading. Even so, there was sufficient reserve strength in the main span to carry design ultimate load.

• Under very high combined moment and shear loading flexural cracks appeared near the epoxy joints in the top slab near the main pier. However, they joined the diagonal tension cracks and did not extend along the joints. There was no sign of any direct shear failure at the joints. In tests of the full bridge model, approximately 75 percent of the theoretical ultimate shear load was applied in the maximum shear loading test prior to failure of the bridge by flexure during that test. No sign of shear distress was evident. Subsequent tests, of a three-segment model under severe shear loading as a cantilever section, indicated that full shear strength of the unit was developed. Hence, the epoxy joint technique used did not reduce the design shear strength.

• During erection of the first few segments, tensile stress occurred in the bottom slab as predicted in the design. Temporary prestress devices successfully controlled the effects of these stresses.

• Theoretical calculation of the load factor for live and impact loads required to form the first plastic hinge agreed very well with the experimental results.

• Near failure, major cracks concentrated near the epoxy joints which had no continuous conventional reinforcement. However, throughout the loading sequence cracks were generally well distributed because of the effective grouting and the strength of the epoxy joints.

• While behavior of the epoxy joints was quite satisfactory, it should be considered that the model segments were joined in a dry condition. The companion tests indicated that while most of the epoxy resins performed adequately for joining dry specimens, the strengths developed by most of the epoxy joints were very weak when joined with concrete segments in a saturated condition.

• Transverse moment capacity of the bridge cross section was very adequate, as shown by the punching shear load test results.

• There was no adverse effect of the epoxy joints on the slab punching shear strengths.

• Bolts used for the temporary connection of the pier segments to the main piers yielded locally under the most critical unbalanced loading, although the calculated direct compressive stress was less than the actual yield strength. The bolts used in the model were also below the yield strength specified for the bolts in the prototype. Yielding was apparently caused by the large gap between the pier segments and the pier, with consequent local bending, and was accentuated by the stress concentrations in the threads.

• Most of the theoretical calculations were in good agreement with the experimental results, although there were some appreciable deviations between the experimental and theoretical values of strain in the top slab in some stages of cantilever construction.

An elastic analysis finite segment program, BMCOL50, was very useful in predicting the behavior of the bridge during construction and for uniform loading tests. The BMCOL50 results agreed very well with the experimental results for longitudinal strains and deflections. The relatively simple data input for BMCOL50 is another advantage when compared to the folded plate theory programs.

The incremental constructing folded plate analysis program, SIMPLA2, reasonably predicted the variation of the longitudinal strain under very high stress levels across the top slabs of the newly erected segments.

The folded plate analysis program, MUPDI, which can be used only for a constant cross section, agreed very well with the experimental results at the service load level. The variation of cross section along this bridge was very small. MUPDI can be used to determine the transverse moments and shears under unsymmetrical loading and can be used effectively in designing the transverse reinforcement.

Volume Changes

Although the effects of temperature, creep, and shrinkage were minimal in the Corpus Christi study, Muller points out for this class of structure: "The effect of steel and concrete creep must be considered with regard to moment distribution, together with the possible effect of moment reversal. Final adjustment and compensation for shrinkage and concrete creep may help the structure to reach the optimum equilibrium." (*8*)

Problems have been reported on bridges currently under construction which indicate that differential volume changes in webs and flange tips are causing substantial tolerance errors. This should be carefully investigated.

Significant savings may be realized by use of lightweight aggregate concretes in segmental construction. However, these aggregates are less desirable in cast-in-place construction for two reasons:

- Early age stressing is required.
- Possible anchorage zone bursting stress weakness in thin web sections.

Both problems need study to develop acceptable procedures for application of lightweight aggregate concretes to this interesting construction type.

Epoxy Resins for Jointing

Based on the exploratory program outlined in Report 121–2 (9), experience in construction of the one-sixth scale laboratory models, and experience in construction of the prototype bridge at Corpus Christi, a revised set of specifications for epoxy jointing of precast concrete segmental box girder bridges was recommended for general usage in moderate and warm climates such as that of Texas. For cold weather applications, modification of the specifications was suggested.

Experience with the nine materials submitted for exploratory testing in connection with usage in the laboratory model and the Corpus Christi bridge indicates that no fully satisfactory material which met all of the original specifications was found in the programs. However, a number of promising materials were identified, which could be used with waiver of some parts of the specifications. Continued development by manufacturers indicates that the revised specification can be met by one or more American formulators. New formulations should be evaluated in a generally similar program.

The critical test condition in evaluation of the jointing capacity of the epoxy resin was the moist surface condition. In preliminary evaluations, testing can be limited to determinations of pot life, contact time, and moist surface condition flexural concrete prism specimens—to quickly determine potential adequacy. If the material passes these tests, a full series of evaluation tests could then be undertaken.

Because of the limited scope of the Corpus Christi epoxy study, the results are not totally conclusive. Further detailed examination of important variables—such as the effect of temperature on pot life, contact time, and rate of curing—should be undertaken in subsequent investigations. In addition, complete development of the specification for this application should include some methods of testing for long term resistance to weathering, temperature stability, and creep.

It should be noted that major problems have occurred during the construction of several recent segmental bridges because of unsuitable epoxy materials and jointing techniques. Thus, considerations for jointing techniques must be given substantially more attention.

Constructability

Methods for obtaining efficient and economical construction of long span bridges built with the use of segmentally precast box girders have been presented. (*1, 5, 9–12*) Thorough documentation of the initial U.S. project placed emphasis on reducing potential difficulties by describing problems and solutions which were experienced in construction of the model and prototype bridge at Corpus Christi, Tex. Many bridges have been similarly erected in other countries; a considerable potential may be realized with the use of these bridges in the United States. There are two general types of segmental bridges: those erected on falsework and those erected by the cantilever method. Many applications exist for both types, with economics determining the best one for each application. In general, only those bridges which can be erected on relatively inexpensive support systems will be designed for erection on falsework; erection by the cantilever method will be most common for inaccessible areas, water crossings, and locations where traffic patterns must not be interrupted.

The industrialization of this type of construction requires complete coordination; procedures must be established to insure that all persons concerned with any particular bridge are aware of their effect on the entire project. Coordination must begin with the initial site selection and continue through the design, production, and erection stages. Preliminary planning is very important in the attainment of maximum efficiency. Seemingly insignificant items may be of primary concern at some particular phase of the construction. The importance of standardization should be reiterated. Mass production techniques require a maximum of standardization. Depending on local conditions, every effort should be made toward the use of common details in all segments. This means economy through reduced materials, labor, and construction time. The use of standardized cross sections seems to offer the most benefits. Other items include the tendons, tendon layouts, anchorages, reinforcement, and shear keys. Typically, bridges with span lengths more than 300 ft (91 m) will be haunched, thereby precluding constant depth cross sections. However, proper planning can eliminate the need for expensive forming techniques.

The casting procedures can generally be the same whether the bridges are to be erected on falsework or in cantilever. Anchorage details are usually different between the two types. Casting segments end-to-end is the only practical means of achieving the accuracy which is required in the segments. Errors in the casting can cause serious alinement problems during the erection stage. However, any errors made in casting of one segment can usually be corrected with the succeeding unit. Forming procedures must be considered early in the planning stage of construction. Forms should be adaptable to suit various configurations in the tendon patterns, shear keys, reinforcement, and other details which may be present. Care must be exercised to insure that there are positive means of providing alinement in all of the ducts as required; misalinement can cause serious and costly delays. Internal stiffening of ducts is particularly recommended. In all bridges there will be certain segments which require special attention. These should be kept to a practical minimum. Other segments may be most economically produced in assembly-line fashion.

Erection of the segments on falsework can be an economical method of assembly, especially if the supports can be rather rapidly positioned. The use of falsework will depend largely on the available equipment, as well as the local terrain. Erection by the cantilever method is more complicated, but it can offer significant advantages. With the use of epoxied joints, erection rates are independent of the joint curing time. Pier segments are fastened to the piers in some manner to prevent rotation during the placement of other segments. Segments may be positioned by any of several methods. They are usually held in position by other equipment until stressing of the tendons is completed. The closure joints have to be cast in place because the tolerances would otherwise be too critical. A means must be provided to insure that the joint remains unstressed while the concrete cures. The completion of the structure involves procedures common to many other bridges in which the bearings are adjusted to a prescribed reaction.

Numerous minor problems which can occur in segmental cantilever construction—such as duct blockage, excessive friction, crossover of grout, and section spalling—are described in Report 121–6F. (*12*) Suggested solutions and precautionary procedures are given. Improvements in precasting procedures are suggested.

The most serious problem that occurred in the construction of the Corpus Christi bridge was the formation of web cracks in the vicinity of the anchorages, along the tendon ducts. The exploratory test program indicated that these cracks were caused by combined tensile stresses due to the conical type of anchor used, the normal bursting stress in front of a concentrated load, and the radial stresses due to tendon curvature in the vicinity of the anchor. Tests indicated that the supplementary reinforcement provided was insufficient to prevent formation of surface cracks but that the web reinforcement was adequate to carry structural forces and control crack size. Provision of active reinforcement in the form of spirals around the anchors and ducts (as used in the model construction) or by use of vertical post-tensioning would have prevented formation of the surface cracking.

Implementation and Summary

The Corpus Christi bridge is an excellent example for demonstrating the industrialization which is possible with segmentally precast bridges, within the traditional construction relationships in the United States. Close liaison was maintained throughout the duration of the project. Design changes were made to provide for easier construction and erection of the bridge. Training seminars were held to make all those concerned more familiar with the construction techniques. Many of the casting and erection problems were demonstrated and solved with the model. Information developed from this project should be widely disseminated to assist designers and constructors of similar structures.

Table 1.—Long span concrete bridges (segmental post-tensioned hollow box girders) in the United States

Name	Location	Length: Total	Length: Individual spans	Width	Precast or cast in place	Date of construction
		Feet	*Feet*	*Feet*		
Pine Valley Creek	I–8 San Diego, Calif.	1,746	450 380 345 286 285	84	Cast in place	1974
Muscatuck River	U.S. 50 Vernon, Ind.	380	95 (2)[1] 190	22	Precast	1975
Sugar Creek	FAS Rte. 1620 Parke Co., Ind.	361.5	180.5 90.5 (2)	30.5	Precast	1976
Napa River	Hwy. 29 Napa, Calif.	2,030	250 270 (2) 195 (2) 150 (7)	68	Cast in place	1976
Vail Pass, Gore Creek Bridge	I–70 Denver, Colo.	4 bridges 668 690 880 727	 134, 200, 200, 134 145, 200, 200, 145 151, 155, 210, 210, 154 153, 210, 210, 154	44	Precast	1976
Kishwaukee River	Winnebago Co. near Rockford, Ill.	1,090	250 (3) 170 (2)	82	Precast	Awarded 1976
Columbia River	Pasco to Kennewick, Wash.	1,794	main span—981 2 side spans—406.5	80	Segments precast in long spans and cast in place over piers and in approach spans	Stayed girder bridge under construction 1976
Pennsylvania DOT Test Track Bridge	Penn State University State College, Pa.	124	124 single span 10° curve	41	38 precast segments each 18.75 ft wide by 6.5 ft long	Under construction summer of 1976
Denny Creek	I–90 Snoqualmie Pass, Wash.	3,600	20 spans—140 to 188	52	Cast in place single cell box girder	Under contract
Turkey Run State Park	Parke Co., Ind.	360	180 (2)	30.5	Precast	Awarded Sept. 1976
Wabash River	U.S. 136 Covington, Ind.	935	187 (4) 93.5 (2)	46	Precast, assembled on shore and pushed out over pier	Awarded Oct. 1976

[1]Numbers in parentheses identify number of spans when more than 1.

1 ft=0.305 m

When several segmental bridge projects have been completed, a study committee should be established, perhaps on a national basis, to review specific procedures used in bridges built both in the United States and in other countries. Evaluations of the most successful methods should be provided as guidelines for designers and contractors, with specific recommendations for the various details which might be common to all types of segmental bridges. Cost studies should also be included. They could be analyzed to show the effect of changes in design or in construction procedures. Such a move toward standardization should not be started prematurely, as constructors in North America have had little opportunity to innovate in this type construction. Relatively more attention should be paid to pier design and to partial and full falsework methods of construction.

Status of Segmental Bridge Construction in the United States

Although acceptance and utilization of segmental prestressed concrete box girder bridges in the United States has been somewhat slow, this trend is beginning to change. Since the time of the design and construction of the bridge at Corpus Christi, Tex. (1973), several other bridges of this general type have been constructed in the

United States. A list of such bridges which have been constructed or are under construction is provided in table 1. Also, many segmental bridges are under design and segmental prestressed concrete options are likely for other proposed bridge sites.

Three of the major reasons for the slow acceptance of this type of bridge in the United States are as follows: (1) The lack of expertise in design and construction; (2) the high cost of the necessary formwork and lifting equipment, for example, gantry cranes; and (3) the lack of cost incentives for designing/redesigning for segmental prestressed concrete. The status of these problems has been gradually changing as state-of-the-art knowledge on design has become known and new techniques for constructing segmental prestressed concrete bridges have evolved. Also, several States are now using, or are considering the use of, a cost reduction incentive program which encourages contractor innovation.

Within the past few years, the techniques for constructing segmental prestressed concrete bridges have been changing. Initially such bridges were constructed by the balanced cantilever method. Such bridges could be constructed with either precast or cast-in-place concrete segments. Sometimes lightweight aggregate concrete has been used, which has reduced the deadload weight and permitted longer bridge spans. However, in the past few years several new methods and concepts have been developed for constructing segmental prestressed concrete bridges.

The most dramatic of the new concepts involves incrementally launching the bridge superstructure from one of the approaches, as cast-in-place concrete sections are cast, cured, and stressed together. This concept is shown in figure 2. The concept eliminates the

Figure 2.—Incremental launching of bridge superstructure.

need for expensive gantry or ground supported cranes, but normally requires establishment of a "short-line" casting bed with large steel forms. Additionally, hydraulic jacking and other specialized equipment is needed for pushing and guiding the box girder superstructure from one abutment to the other. This concept may be used for bridges with either straight or constant curved horizontal or vertical alinements. At least six segmental bridges have been constructed by this method in Western Europe. Also, the Wabash River bridge, near Covington, Ind., is being constructed in this manner (fig. 3).

Another concept has been developed whereby segmental bridges are constructed one span at a time. This "span-by-span" method normally involves the use of a ground supported form carrier. The form carrier supports the formwork for one entire span. However, the "span" is not the normal span between adjacent piers, but rather the span is offset by approximately 1/5 of the span length. The movable form carrier is supported on the edge of the previously completed span and by the next forward pier. Each span is prestressed prior to removal of the formwork. This concept offers many of the advantages of both precast and cast-in-place segmental construction, while eliminating the need for an overhead gantry crane. This concept is particularly suited to construction of long viaduct-type structures.

In summary, the use of segmental prestressed concrete box girder bridges in the United States is increasing, as developments in the state of the art have increased. It is interesting to note that almost every one of the segmental bridges in the United States has been different—in many ways. Each of the three construction/erection concepts or variations of them have been used. Both normal and lightweight aggregate concretes have been used. The traditional as well as newly developed prestressing systems have also been used. Some bridges have been designed for segmental construction, others permitted a segmental alternate, and others are segmental as the result of cost reduction incentive programs.

Figure 3.—Wabash River bridge under construction.

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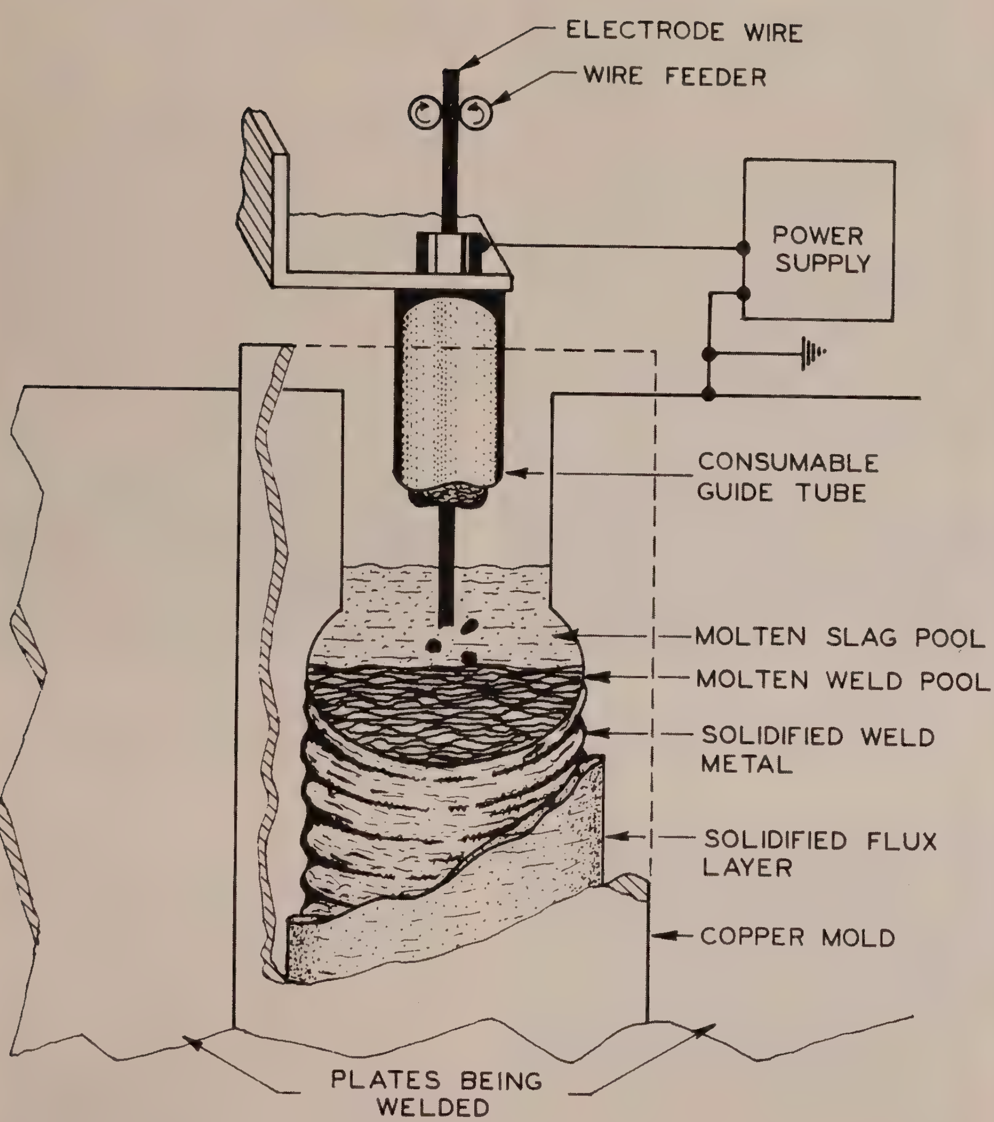

Schematic of consumable guide electroslag welding.

Electroslag Weldments: Performance and Needed Research

by James D. Culp

This article is the first part of a paper presented at the 1977 Federally Coordinated Program of 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. This article includes a process description, Michigan's fabrication experience, problem areas, and some research results. The second part of the article which will appear in the June 1978 issue of *Public Roads* will include additional research results and research needs.

Process Description

Welded butt joints in flange plates and cover plates, ranging from 1/2 to 4 in (13 to 102 mm) in thickness (sometimes thicker), are commonly encountered in the fabrication of steel plate girders and rolled beams used in highway bridges. Around 1970, the electroslag welding process began to gain acceptance as an

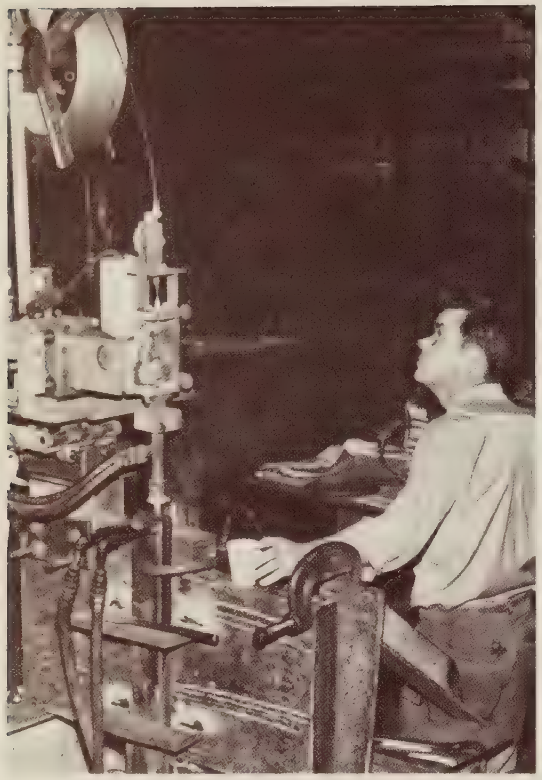

Figure 1a.—Electroslag welding procedure used by Fabricator A with water cooled retaining shoes.

Figure 1b.—Electroslag joint preparation showing strong-backs, starting sump, run-off tabs, consumable guide tube, and spacer ring.

Figure 1c.—Completed electroslag weld with retaining shoes removed; strong-backs, sump, and tabs are still in place.

alternative to submerged arc welding for fabricating these thick plate butt weldments. Electroslag welding was developed in Russia around 1950 as a single pass procedure for the butt welding of thick plates and castings.

Electroslag welding is defined as ". . . a welding process wherein coalescence is produced by molten slag which melts the filler metal and the surfaces of the work to be welded. The weld pool is shielded by this slag which moves along the full cross section of the joint as the welding progresses. The conductive slag is maintained molten by its resistance to electric current passing between the electrode and the work. Consumable guide electroslag welding is a method of electroslag welding wherein filler metal is supplied by an electrode and its guiding member." (*1*)[1] This process of welding is done in the vertical position and joints are usually completed in a single pass for any plate thickness.

Using one or more electrodes, electroslag welding can be performed with either a consumable or nonconsumable guide tube through which the welding wire passes. We shall consider the consumable guide tube method because it is the method predominantly used due to its simplicity in equipment and setup. The physical setup used in consumable guide electroslag butt welding is shown in the display photograph. As the molten slag pool and weld pool move up the joint, they are contained by two copper molds or shoes that are clamped to the plate surfaces. These shoes are slightly recessed in the middle to allow weld reinforcement to be built up on the plate surfaces. This reinforcement is later ground off flush with the plates. The shoes may be solid copper or hollow for circulating cooling water, called the "dry shoe" and the "cooled shoe" methods, respectively. A sump or starting tab is required at the bottom of the joint to assure that both the slag depth and the width of fusion are adequate when the plates to be welded are reached. Likewise runoff tabs are provided at the top edge of the plates to avoid lack of fusion and other flaws that occur at the stopping point of the weld. These starting and runoff tabs are later removed flush with the plate edges by flame cutting and grinding.

Michigan's Fabrication Experience

Two fabricators were the predominant users of the electroslag process in Michigan bridge fabrication. Fabricator A used the water cooled shoe method where heat is continually extracted from the joint by water circulated through the shoes (fig. 1). An additional feature of this setup was that the guide tube was oscillated across the joint during welding on plates thicker than 2 in (51 mm). This distributes the heat input more uniformly throughout the

[1] Italic numbers in parentheses identify references on page 192.

joint and allows a single wire and tube to be used on thick plates. Fabricator B used the dry shoe method and his setup is shown in figure 2. In this method, the only cooling supplied to the shoes is the heat loss to the surrounding air and base metal. Fabricator B used stationary guide tubes only (that is, no oscillation); when the plate thickness exceeded 2 in (51 mm), a double guide tube setup was used (fig. 2b). Electroslag welding has been used on American Society for Testing and Materials (ASTM) steel types A 36 and A 588 in Michigan bridge fabrication. Table 1 presents a listing of typical welding parameters used for this welding. (Additional detail on the welding setups and procedures is presented in reference 2.)

One of the main advantages of the electroslag butt welding process is the speed of welding thick plates with a single pass and an accompanying savings on labor costs. It has been Michigan's experience that fabricators using electroslag welding on butt joints realize about 50 percent savings on labor costs over using the submerged arc process on the same joints. Thus, the cost savings on normal girder fabrication actually appear quite small with respect to the total fabrication cost. Conceivably, greater cost savings could be realized on joints larger or more complex than normal.

Problem Areas

The main disadvantage of the electroslag welding process is the metallurgical structure that is produced by the very high heat input and prolonged thermal cycle with accompanying slow solidification and cooling rates. This thermal cycle results in a "coarse casting" type of weldment structure with anisotropic (properties vary with direction) and non-homogeneous (properties vary with position in the weld) large-grained weld metal and extremely large heat-affected zones. This type of weld metal structure has many physical and metallurgical deficiencies and is normally not considered acceptable for use in the "as welded" condition when Charpy V-notch impact requirements are specified, that is, where fracture toughness is an important design parameter. (3) The "as welded" electroslag structure is likewise highly suspect concerning its fatigue sensitivity and its corrosion behavior when placed in unpainted exposures of ASTM A 588 steel girders.

From early 1970 to June 1974, the consumable guide electroslag process was approved for use by the Michigan Department of State Highways and Transportation and gained a dominant role in the fabrication of steel plate girders. At least four fabricators were qualified in its use and two of these used the process almost exclusively. The electroslag process was qualified in

Figure 2a.—Electroslag welding procedure used by Fabricator B with dry retaining shoes.

Figure 2b.—Double guide tube used in 3-in (76 mm) thick joint (note flux coating on guide tubes).

accordance with the specifications set forth in the American Welding Society's (AWS) Specifications for Welded Highway and Railway Bridges, D 2.0–69. It was used on approximately 53 bridges of ASTM A 36 and 72 bridges of ASTM A 588 self-weathering (unpainted) bridges for a total of 125 electroslag welded bridges. The use of the electroslag welding process on Michigan bridges was suspended in June 1974. This was a result of the findings of Michigan's research on the properties of the weldments. Based on the preliminary results of the Interstate 79 bridge failure near Pittsburgh, Penn., the Federal Highway Administration prohibited the use of electroslag weldments on main structural tension members on any Federal-aid project and called for a rigorous inspection, using both radiography and ultrasonic testing, of any nonredundant main load carrying tension members in existing structures that have been welded by the electroslag process. This testing program is now in progress and several defective welds have been located and reinforced in the Pittsburgh area and in Atlanta, Ga. Michigan has identified their critical structures but has not yet inspected them.

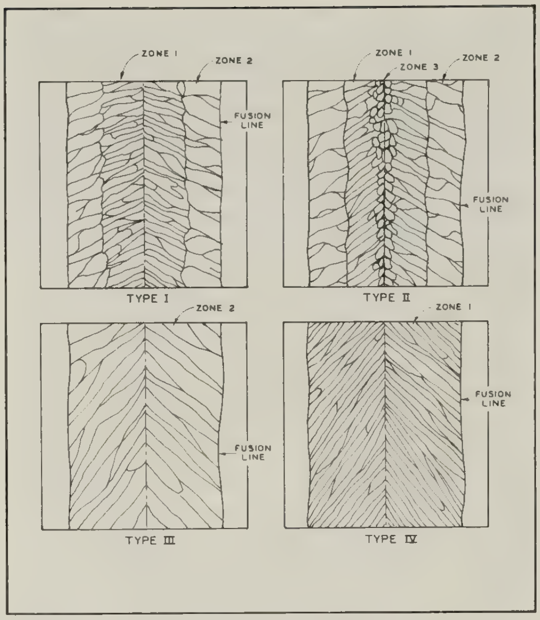

Figure 3.—Sketch of electroslag joint types, longitudinal section through the weld.

Since 1970, several research projects have been initiated on electroslag weldments and the results confirm that physical problems are present in weld metal. (*2, 4*)[2] The central zone of the electroslag weld has a lower yield point and Charpy V-notch energy than the surrounding weld metal and is sometimes lower than the base metal. (*2, 4–6*) Other studies also confirm this result and further report a loss in ductility in the weld metal as compared to the base metal. (*3, 7, 8*) Post-weld heat treatment, such as normalizing and tempering, is often required on electroslag weldments to elevate these properties to the specified values. (*6–10*) Sometimes the heat-affected zones surrounding an electroslag weld will have impact properties inferior to those of the base metal. (*9*) This is almost always true in high yield strength, and quenched and tempered steels; thus, the process is normally prohibited on such steels. (*11*) There is still only a limited amount of fatigue strength data available on electroslag weld metal. Noel, Harrison, and Almquist rate electroslag weld metal as having a fatigue strength approximately equivalent to that of the parent metal in smooth specimen testing. (*4, 12*)[3] (For a complete listing of foreign and domestic literature on the subject of electroslag welding see reference 13.)

[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.

Table 1.—Welding variables—electroslag process

Weld identification[1]	Base metal ASTM designation	Plate thickness	Electrode type	Flux type	Amps	Volts
		Inches				
ES 588–A1	A 588	1 3/4	Hobart 25P	(*)[2]	600–650	36–38
ES 588–A2	A 588	3	Hobart 25P	(*)	750–800	42–44
ES 36–A1	A 36	1 3/4	Hobart 25P	(*)	600–650	36-38
ES 36–A2	A 36	3	Hobart 25P	(*)	750–800	42–44
ES 588–B1	A 588	1 3/4	Linde 36	Linde 124	525–550	36
ES 588–B2	A 588	3	Linde MC–70	Linde 124	1100	36
ES 588–B2a	A 588	3	Linde WS	Linde 124	1100	36–38
ES 36–B1	A 36	1 3/4	Linde 36	Linde 124	525–550	36
ES 36–B2	A 36	3	Linde MC–70	Linde 124	1100	36

[1]Identification symbolism: 1 in=25.4 mm

ES—electroslag weldment.

588 or 36—ASTM A 588 or A 36 steel, respectively.

A or B—Fabricator A or B, respectively.

1 or 2—1 3/4 or 3 in plate thickness, respectively.

[2](*)=Hobart—PF 203—starting, PF—201 running.

Research Results

The following results were obtained by an extensive testing program on weldments made by Fabricators A and B cited above and were essentially identical to electroslag weldments being put into Michigan bridges. The procedures used were considered to be properly qualified.

Metallurgical structure and alloy composition

In his book on electroslag welding, Paton describes the four different types of grain structure arrangements that can be produced in electroslag weldments

[3]"The Fatigue Strength of Electroslag Welded Joints," by G. A. Almquist. Lecture given to the Swedish Welding Association, Värmland Section, September 1964.

(fig. 3.) (5) Type I weld structure consists of an outer zone of coarse columnar crystals (Zone 2) and an interior zone of thin, elongated columnar crystals (Zone 1). Type II welds have a zone of coarse, equiaxed crystals in the very center of the weld in addition to the coarse and fine columnar crystals of Type I. Type III welds consist of only coarse columnar crystals throughout the cross section, and Type IV welds consist of only fine columnar crystals. All weldments studied in Michigan's research were either of Type I or Type III. Type IV welds have been encountered in some of Michigan's fabrication work with the cooled shoe electroslag welding, but these joints have not been reproduced in the research studies.

Figures 4 and 5 illustrate the various electroslag weldment structures produced by Fabricator A using the cooled shoe technique. The figures show transverse cross sections taken normal to the longitudinal axis of the weld. The macrosections reveal the primary structure of the welds as belonging to Type I as defined above. In the center of the weld, the Zone 1 thin columnar crystals appear and the coarse columnar Zone 2 crystals appear at the periphery of the weld. This structure is viewed on another plane in figure 6 which is a longitudinal section taken at the midthickness of the plate. This section shows the columnar nature of the crystals with the Zone 1 crystals being alined nearly parallel with the weld axis and the Zone 2 crystals growing inward and upward, pointing in the direction of the welding. Microstructures on the transverse plane are repeated in figure 6 for comparison with those on the longitudinal plane. Between the fusion line and the outlying base metal, two heat-affected zones (HAZ) appear. The first one, HAZ 1, appears dark in the macrosection and

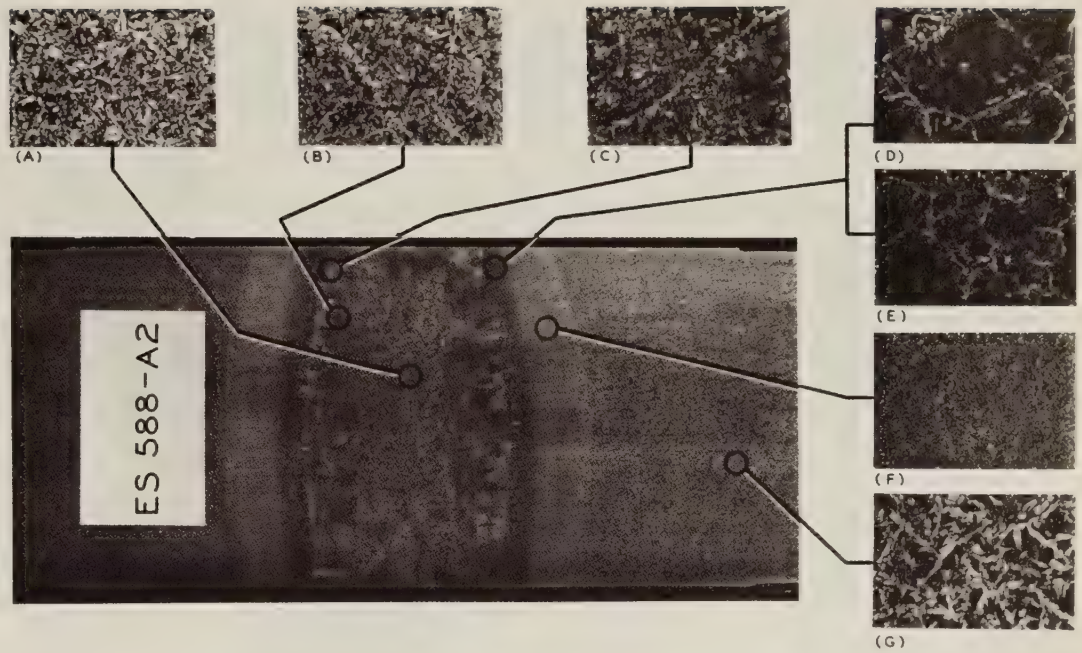

Figure 4.—Cooled shoe electroslag weld by Fabricator A on 3-in (76 mm) A 588 steel. Microstructures shown are: (A) Zone 1 weld metal, (B) Zone 2 weld metal, (C) fusion line (HAZ 1 on right), (D) HAZ 1 just beyond fusion line, (E) HAZ 1 further out, (F) HAZ 2, (G) unaffected base metal. (Microstructures at 100x, etchant 3 percent nital.)

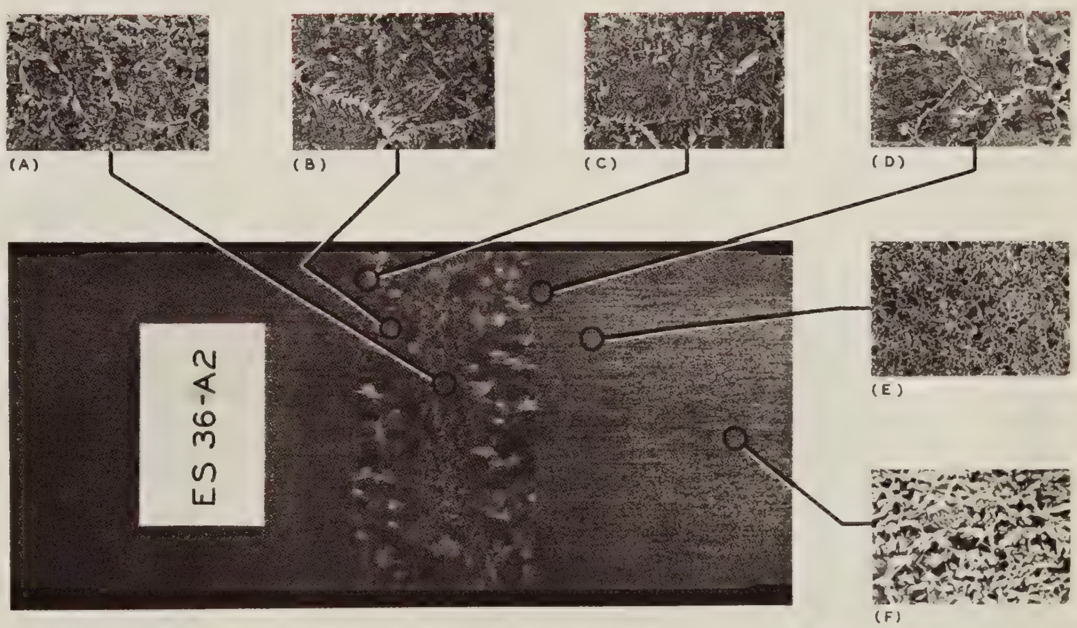

Figure 5.—Cooled shoe electroslag weld by Fabricator A on 3-in (76 mm) A 36 steel. Microstructures shown are: (A) Zone 1 weld metal, (B) Zone 2 weld metal, (C) fusion line (HAZ 1 on bottom), (D) HAZ 1 beyond fusion line, (E) HAZ 2, (F) unaffected base metal. (Microstructures at 100x, etchant 3 percent nital.)

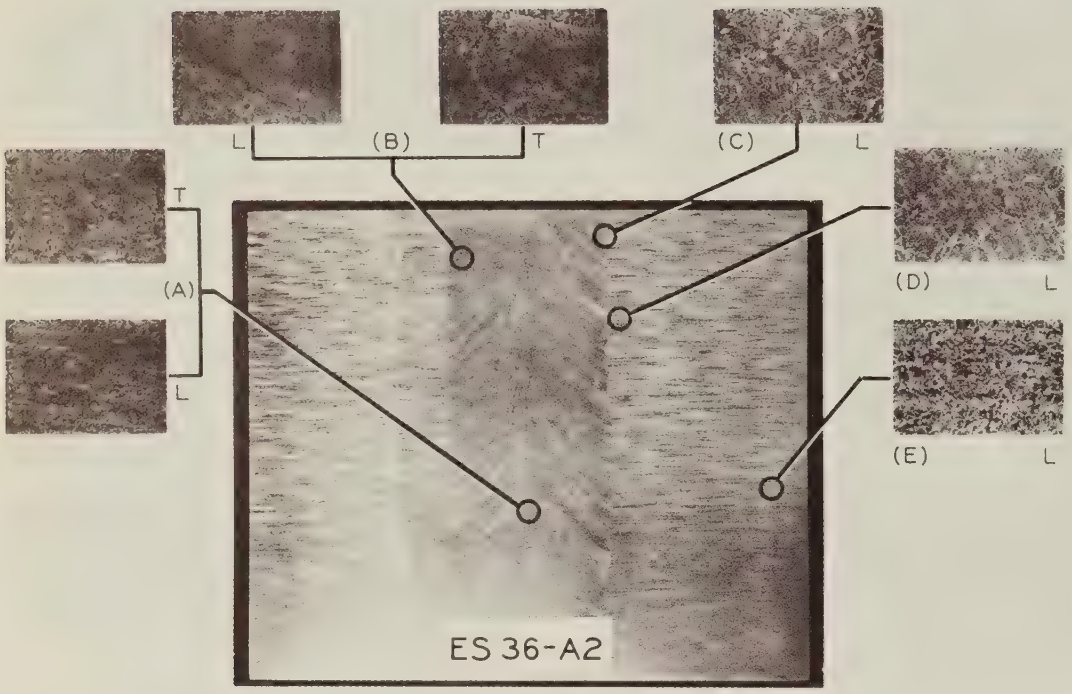

Figure 6.—Typical midthickness longitudinal section through a cooled shoe electroslag weld by Fabricator A on 3-in (76 mm) A 36 steel. Microstructures shown are: (A) Zone 1 weld metal, (B) Zone 2 weld metal, (C) fusion line (HAZ 1 on right), (D) HAZ 1 beyond fusion line, (E) unaffected base metal. Microstructures marked "L" are taken in the longitudinal plane shown and "T" are in the transverse direction. (Microstructures at 50x, etchant 3 percent nital.)

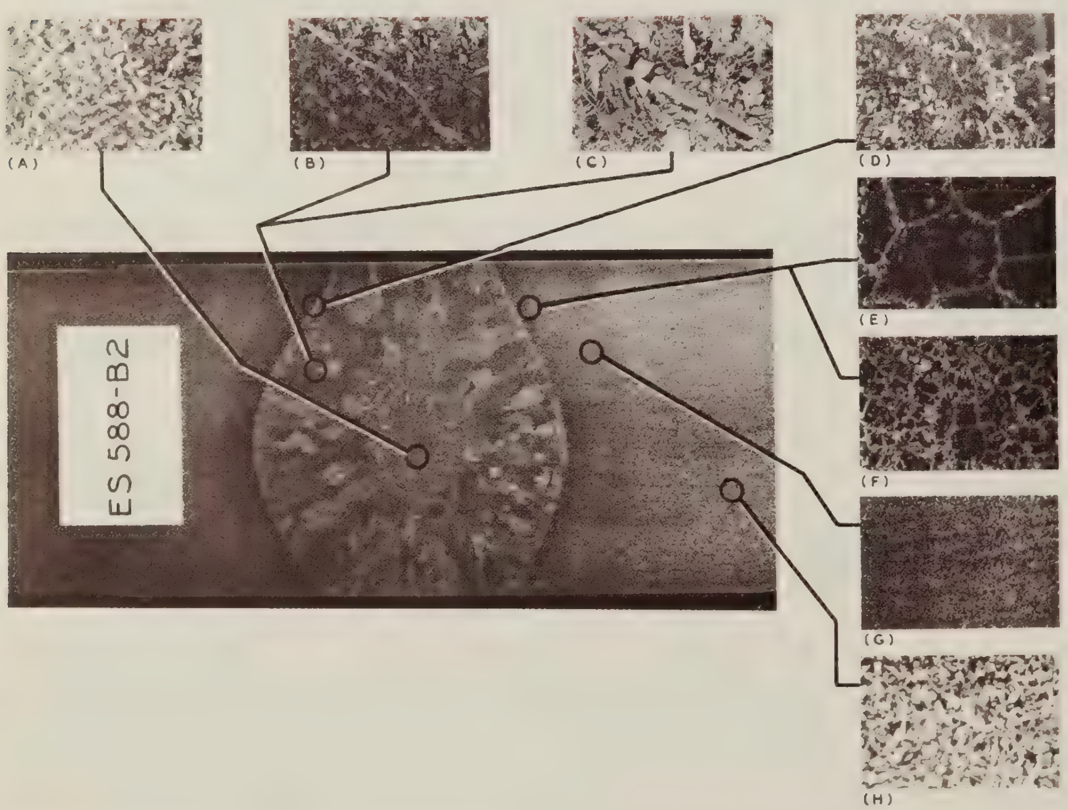

Figure 7.—Dry shoe electroslag weld by Fabricator B on 3-in (76 mm) A 588 steel. Microstructures shown are: (A) Zone 1 weld metal, (B) Zone 2 weld metal, (C) ferrite band in Zone 2 weld metal (200x), (D) fusion line (HAZ 1 on right), (E) HAZ 1 just beyond fusion line, (F) HAZ 1 further out, (G) HAZ 2, (H) unaffected base metal. (Microstructures at 100x, etchant 3 percent nital.)

is a zone of grain coarsening as shown in the microstructures. The grains in HAZ 1 decrease in size as the second heat-affected zone (HAZ 2) is approached. HAZ 2 appears light in the macrosection and is seen to be a zone of grain refinement. Similar structures are shown in figures 7 and 8 for dry shoe electroslag weldments made by Fabricator B.

An understanding of the metallurgical structures present in an electroslag weldment is essential to the assessment of the nonhomogeneous and anisotropic nature of the properties that are present.

One of the most important features of the electroslag weld metal structure is the ferrite bands that outline the primary crystal structures of the zones. These bands form a continuous path or network of least resistance to a propagating crack. The resistance to crack propagation is highly dependent on the crack direction with respect to the direction of the ferrite bands. Thus, it is expected that such a weld structure would be significantly nonhomogeneous and anisotropic in its properties.

In the electroslag welding process, the high heat input spreads the fusion area to the extent that the weld metal is diluted by 40 to 60 percent with base metal throughout the entire cross section of the weld. Due to this high dilution, the resulting weld chemistry is highly dependent on the base metal chemistry. Tables 2 and 3 list the weld metal chemistry for Zones 1 and 2 of the weldment types listed in table 1. Note that Zones 1 and 2 weld metals have virtually identical compositions. This implies that the difference in their crystal structures is primarily a function of the different cooling cycles experienced and not due to a difference in alloy composition. Analyses run on HAZ 1 and HAZ 2 show them to have compositions identical to the base metal. The types of cast structures present in electroslag weldments do lead to alloy segregation on the microscopic level, but no work was done to quantify this. Such segregations, especially along the ferrite network, can lead to preferential paths for cracking and corrosion attack.

The main points to note on the chemical compositions of the weld metal are as follows: (1) The high carbon content, undoubtedly due to the high dilution rate with base metal; (2) the deficiencies in the A 588 steel weldments of nickel, chromium, and copper due to metal dilution and

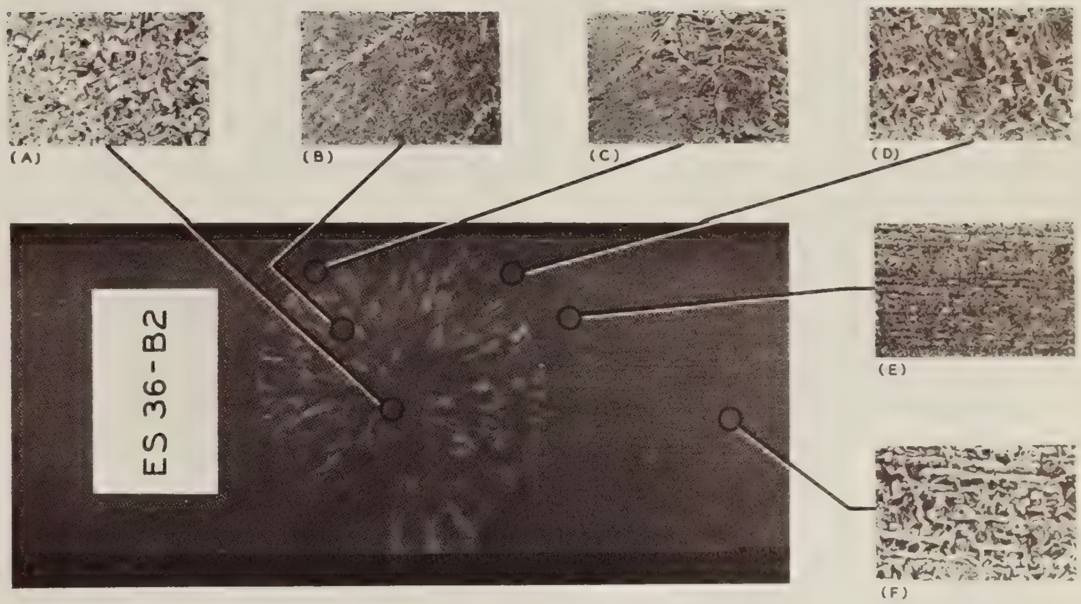

Figure 8.—Dry shoe electroslag weld made by Fabricator B on 3-in (76 mm) A 36 steel. Microstructures shown are: (A) Zone 1 weld metal, (B) Zone 2 weld metal, (C) fusion line (HAZ 1 on right), (D) HAZ 1 beyond fusion line, (E) HAZ 2, (F) unaffected base metal. (Microstructures at 100x, etchant 3 percent nital.)

Table 2.—Chemical composition of electroslag weldments in A 588 steel

Weldment type[1]	Typical analysis (weight, percent) Carbon	Manganese	Phosphorus	Sulfur	Silicon	Nickel	Chromium	Copper	Vanadium	Aluminum	Electrode/flux type
ES 588–A1											
Zone 1 weld metal	0.15	1.20	0.018	0.021	0.39	0.06	0.26	0.15	0.02	0.01	
Zone 2 weld metal	0.16	1.19	0.017	0.021	0.41	0.07	0.25	0.15	0.03	0.01	Hobart 25P/Hobart PF 201
Base metal	0.17	1.17	0.006	0.026	0.24	0.18	0.56	0.30	0.05	0.04	
ES 588–A2											
Zone 1 weld metal	0.15	1.18	0.019	0.018	0.42	0.05	0.18	0.12	0.02	0.01	
Zone 2 weld metal	0.15	1.20	0.019	0.017	0.45	0.06	0.20	0.13	0.02	0.01	Hobart 25P/Hobart PF 201
Base metal	0.19	1.14	0.015	0.022	0.22	0.15	0.54	0.32	0.05	0.04	
ES 588–B1											
Zone 2 weld metal	0.14	1.29	0.012	0.020	0.22	0.20	0.35	0.20	0.03	0.01	Linde 36/Linde 124
Base metal	0.13	1.00	0.012	0.020	0.25	0.39	0.61	0.32	0.04	0.01	
ES 588–B2											
Zone 1 weld metal	0.12	1.28	0.015	0.020	0.30	0.09	0.28	0.19	0.03	0.01	
Zone 2 weld metal	0.13	1.30	0.013	0.021	0.31	0.09	0.32	0.20	0.03	0.01	Linde MC–70/Linde 124
Base metal	0.18	1.15	0.014	0.022	0.23	0.13	0.53	0.31	0.05	0.03	
ES 588–B2a											
Zone 2 weld metal	0.16	0.92	0.006	0.026	0.25	0.30	0.52	0.37	0.04	0.01	Linde WS/Linde 124
Base metal	0.16	1.23	0.011	0.016	0.28	—	0.56	0.32	0.06	—	
Specified weld metal composition range[2]	0.12 max.	0.50/1.10[3]	0.03 max.	0.04 max.	0.35/0.80	0.40/0.80	0.45/0.70	0.30/0.75	0.05 max.[4]		

[1]See table 1.

[2]MDSHT Supplemental Specifications for Welding Structural Steel, for unpainted exposures. (14)

[3]AWS D1.1–72 requires 0.50/1.30 manganese.

[4]Applies only when post weld heat treatment is required.

Table 3.—Chemical compositions of electroslag weldments in A 36 steel

Weldment type[1]	Carbon	Manganese	Phosphorus	Sulfur	Silicon	Copper	Vanadium	Electrode/flux type
	Typical analysis (weight, percent)[2]							
ES 36–A1								
Zone 1 weld metal	0.14	1.10	0.019	0.021	0.41	0.05	0.01	
Zone 2 weld metal	0.15	1.11	0.017	0.020	0.38	0.05	0.01	Hobart 25P/Hobart PF 201
Base metal	0.21	1.00	0.012	0.020	0.22	0.05	0.01	
ES 36–A2								
Zone 1 weld metal	0.14	1.10	0.018	0.022	0.40	0.05	0.01	
Zone 2 weld metal	0.14	1.12	0.017	0.020	0.40	0.05	0.01	Hobart 25P/Hobart PF 201
Base metal	0.23	1.07	0.012	0.018	0.23	0.05	0.01	
ES 36–B1								
Zone 2 weld metal	0.17	1.34	0.016	0.016	0.22	0.06	0.01	Linde 36/Linde 124
Base metal	0.21	1.00	0.012	0.020	0.22	0.05	0.01	
ES 36–B2								
Zone 2 weld metal	0.16	1.28	0.016	0.016	0.28	0.01	0.01	Linde MC–70/Linde 124
Base metal	0.23	1.07	0.012	0.018	0.23	0.05	0.01	

[1]See table 1.

[2]No specification on weld metal composition, except 0.05 max. V when post weld heat treatment is required.

welding wire deficiencies; and (3) the low silicon content of the dry shoe electroslag weldments. Nickel, chromium, copper, and silicon are the major corrosion inhibiting elements and these deficiencies make the weldments unacceptable for bare, unpainted exposures. The high carbon and low nickel can also reduce the toughness of the weld metal. It is difficult to control the chemical composition of an electroslag weldment when 40 to 60 percent dilution with base metal occurs.

Tensile properties

The tensile properties and side bend test results for electroslag weldments are shown in table 4 for A 36 steel and in table 5 for A 588 steel. The only weldment type that failed to meet the minimum required yield strength was ES 588–A2, the cooled shoe electroslag weld in 3-in (76 mm) A 588 steel. Note that in all of the electroslag weldments, Zone 1 weld metal has both lower yield and tensile strengths than the Zone 2

Table 4.—Tensile properties and bend test results for electroslag weldments in A 36 steel

Weldment type[1]	Yield strength[2]	Tensile strength	Elongation (2 in gage)	Reduction of area	Guided side bend test
	psi	*psi*	*Percent*	*Percent*	
Values required by specification[3]	36,000	58,000–80,000	24	—	
ES 36–A1					
Zone 1 weld metal	43,000 (2)[4]	74,100	33	64	Pass
Zone 2 weld metal	49,300	75,400	—	59	Pass
Base metal	40,700	71,400	32	—	Pass
ES 36–A2					
Zone 1 weld metal	46,100 (2)	73,800	—	69	Pass
Zone 2 weld metal	48,000 (2)	75,500	28	64	Pass
Base metal	39,000	64,400	33	—	Pass
ES 36–B1					
Zone 2 weld metal	48,100 (3)	74,000	28	63	Fail
Base metal	40,700	71,400	32	—	Pass
ES 36–B2					
Zone 2 weld metal	42,700 (3)	70,700	34	65	Pass
Base metal	39,000	64,400	33	—	Pass

[1]See table 1.

1 psi=6.89 MPa
1 in=25.4 mm

[2]0.2 percent offset method used for weld metal, 0.5 extension under load method used for base metal.

[3]Require electroslag weld metal to match the properties of the base metal (*14*) single values shown are minimum requirements.

[4]Numbers in parentheses are number of specimens tested (when greater than one) to give average values shown.

Table 5.—Tensile properties and bend test results for electroslag weldments in A 588 steel

Weldment type[1]	Yield strength[2]	Tensile strength	Elongation (2 in gage)	Reduction of area	Guided side bend test
	psi	*psi*	*Percent*	*Percent*	
Values required by specification[3]	50,000	70,000	21	—	
ES 588-A1					
Zone 1 weld metal	53,100	80,900	27	65	Pass
Zone 2 weld metal	54,200 (2)[4]	82,400	29	51	Pass
Base metal	66,700	95,200	22	—	
ES 588-A2					
Zone 1 weld metal	47,400 (2)	76,400	32	65	Pass
Zone 2 weld metal	49,400 (2)	77,500	—	57	Pass
Base metal	66,600	96,000	22	—	Pass
ES 588-B1					
Zone 2 weld metal	57,600 (3)	82,500	26	51	Pass
Base metal	67,500	97,000	25	—	Pass
ES 588-B2					
Zone 1 weld metal	52,000 (2)	79,700	32	66	Pass
Zone 2 weld metal	56,200 (2)	82,800	27	47	Pass
Base metal	56,100	83,200	32	—	Pass
ES 588-B2a					
Zone 1 weld metal	54,000 (1)	83,000	—	67	Fail
Zone 2 weld metal	57,500 (2)	85,500	24	45	Fail
Base metal	57,400	86,400	26	—	Pass

1 psi=6.89 MPa
1 in=25.4 mm

[1]See table 1.

[2]0.2 percent offset method used for weld metal, 0.5 percent extension under load method used for base metal.

[3]Require electroslag weld metal to match the properties of the base metal (*14*), single values shown are minimum requirements.

[4]Numbers in parentheses are the number of specimens tested (when greater than one) to give average values shown.

weld metal. This agrees with results previously reported (*5, 6*) and is a function of the metallurgical structures present in these zones. A potentially serious anisotropy exists in the tensile properties of these weld metal zones, especially because the direction of testing is oriented 90° to the direction of service loading. (*2*)

Charpy V-notch impact evaluation

Three series of Charpy V-notch impact tests were performed on the electroslag weldments: (1) an evaluation of the impact strength at 0° F (−18° C) for comparison to the existing acceptance criteria; (2) impact tests over a temperature range of −40° to +40° F (−40° to 4° C) to determine the temperature transition characteristics; and (3) impact tests on specimens taken at different angles to the standard direction to determine the directional variation (anisotropy) of the toughness.

Series I—Acceptance testing. The impact requirements for electroslag butt welds are 15 ft-lb at 0° F (20.3 J at −18° C) according to AWS specifications. (*11*) One set of five specimens is required to be taken from the quarter thickness point on the weld centerline in a standard procedure test weld assembly. This requirement gives no consideration to the nonhomogeneous nature of an electroslag weld nugget and most of the time locates the test specimens in the toughest portion of the weld, that is, Zone 2 weld metal. Since 1974, the Michigan specification (*14*) has placed additional requirements on the impact testing of electroslag weldments to remedy the deficiencies in the existing code, but no one has since attempted to meet the new requirements.

A 3-in (76 mm) section was saw cut from both the starting end and the top (finishing) end of the electroslag weldments. In the machining and testing of the Charpy specimens all requirements of the specifications were adhered to, except for the positioning of the specimens in the cross section of the weldment. Sets of five specimens were located at various positions throughout the weldment to test all the weld zones and heat-affected zones present. The specimens were precisely located by etching of the weld prior to any cutting.

The results of these tests are presented in tables 6 and 7 for welds made in A 588 and A 36 steels, respectively. Each value entered in the table represents either an average taken from a set of specimens as prescribed by the specifications (*14*) or an average of several such sets taken from one particular zone of the weldment. Figure 9 presents cross-sectional drawings of four electroslag weldments showing numbers that correspond to the average impact strength measured at the points located by a cross.

A comparison of the impact values representing the various electroslag weldment zones in tables 6 and 7 shows that the Zone 1 weld metal has a lower toughness than the Zone 2 weld metal. The only exception is weldment ES 588–B2a which has very low toughness in both weld metal zones (table 6). This result agrees with previously reported tests on electroslag weldments (*5, 6*) and is to be expected when consideration is given to the orientation of the grain structures with respect to the direction of crack propagation in the Charpy test specimen. The standard orientation for removing a Charpy specimen from the weld directs the crack propagation parallel to the longitudinal axis of the crystals that comprise the Zone 1 weld metal; failure that occurs is intergranular, that is, the crystals separate along their grain boundaries. In Zone 2 weld metal, the coarse columnar crystals are oriented at an angle varying between 40° to 65° with the direction of crack propagation; the failure occurs in a transgranular mode, that is, the crack propagates across the crystals. This usually requires more energy than crack propagation along the grain boundaries depending on the alloy composition of the weld. Some Zone 2 specimens did exhibit a partial intergranular type of fracture, but the plane of the crack is forced to deviate from its original plane to follow the grain boundaries. This results in a higher energy absorption than that required to propagate the crack parallel to the crystal axis. These statements will be further substantiated by the impact data from the anisotropic test series which will be presented in part 2 of this article.

Table 6.—Impact toughness (CVN) of electroslag weldments in A 588 steel

(Test temperature 0° F)

Weldment type[1]	Impact toughness: Zone 1 weld metal	Zone 2 weld metal	HAZ 1	HAZ 2	Base metal
	ft-lb	*ft-lb*	*ft-lb*	*ft-lb*	*ft-lb*
ES 588–A1					
Top end	18*[2]	46	N.T.[3]	81	13
Starting end	13*	34	N.T.	69	13
ES 588–A2					
Top end	11*	35	18	77	12
Starting end	14*	29	18	123	14
ES 588–B1					
Top end	N.P.[4]	22	8	128	14
Starting end	20	34	N.T.	52	9
ES 588–B2					
Top end	6*	14*	18	30	16
Starting end	5*	7*	25	33	18
ES 588–B2a					
Mid-section[5]	5*	6*	N.T.	N.T.	51

[1]See table 1.

1° F=−17.2° C
1 ft-lb=1.4 J

[2](*) denotes that the set of specimens failed to meet the specified impact requirements.
[3]N.T. denotes that the zone was present but not tested.
[4]N.P. denotes that the zone was not present in the weldment.
[5]Specimens were located in accordance with AWS D2.0–69.

The first heat-affected zone past the fusion line, HAZ 1, which is a zone of grain coarsening due to the overheating experienced, exhibits impact energy equal to or greater than the unaffected base metal. This grain coarsening is known to be detrimental in other types of steel, especially high yield strength and quenched and tempered steels, but apparently causes no loss of toughness in the base metal for the ASTM A 588 and A 36 steels evaluated. This is the zone that requires careful evaluation if no previous data are available on the effect of such grain coarsening on a particular type of steel. The next heat-affected zone, HAZ 2, which was a zone of grain refinement, is greatly enhanced in impact strength. This type of fine grained structure, which is produced by the recrystallization effect of the heating and cooling cycle experienced during the welding, would be expected to yield high impact strengths. The only exceptions to this high toughness in HAZ 2 were in weldments ES 36–A2 and ES 36–B2 where the base metal had extremely low toughness (table 7). In these two cases, the grain refinement present in HAZ 2 did more than double the low level of toughness present in the base plate, but these elevated values were still quite low.

The impact energies measured from specimens taken from the starting end of the weld were usually equal to or somewhat less than those measured from the top end of the weldment. The only exceptions to this are seen in Zone 2 weld metal in weld ES 588–B1 and in weld ES 36–A2. The higher impact energy measured in weldment ES 588–B1, in Zone 2 from the start of the weldment, was actually due to a change in the orientation of the large columnar crystals with respect to the axis of crack propagation, which resulted in a lower impact energy measured at the top of the weld. If the specimen sets had been positioned at the same point in the cross section, possibly this difference would not have occurred. In weldment ES 36–A2, the values at the starting end in Zone 2 do not exceed those at the top end, probably also due to the crystal orientation. (The Zone 1 weld metal in this weldment does follow the trend of lower toughness on the starting end.)

Table 7.—Impact toughness (CVN) of electroslag weldments in A 36 steel

(Test temperature 0° F)

Weldment type[1]	Impact toughness: Zone 1 weld metal	Zone 2 weld metal	HAZ 1	HAZ 2	Base metal
	ft-lb	*ft-lb*	*ft-lb*	*ft-lb*	*ft-lb*
ES 36-A1					
Top end	42	47	N.T.[2]	N.T.	50
Starting end	21	50	N.T.	167	51
ES 36-A2					
Top end	20	37	6	5	4
Starting end	16*[3]	53	5	14	4
ES 36-B1					
Top end	N.P.[4]	42	57	140	47
Starting end	N.P.	22	N.T.	161	49
ES 36-B2					
Top end	N.P.	51	6	6	3
Starting end	N.P.	57	8	7	3

1° F=−17.2° C
1 ft-lb=1.4 J

[1]See table 1.

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

[3](*) denotes that the set of specimens failed to meet the specified impact requirements.

[4]N.P. denotes that the zone was not present in the weldment.

Thus, qualified by the two above exceptions and their explanation, it appears that the lower toughness levels in the electroslag weldments tested (16 in [406 mm] long) occur in the weld metal at the starting end of the weld. This would be an important fact to incorporate into specifications governing the acceptance testing of such welds. In fact, because the impact toughness of the Zone 2 weld metal is so sensitive to the orientation of the large columnar crystals which does vary throughout the weldment, it might be justified to conduct impact tests at both the start and finish of the weld and at several positions within the cross sections, or to selectively orient the specimens within the grain structure.

A comparison of the results obtained for these electroslag weldments with the recommended AWS acceptance criterion of 15 ft-lb at 0° F (20.3 J at −18° C) shows that for A 588 steel the Zone 1 weld metal fails to qualify in all but one of the weldments. The only exception is the dry shoe electroslag weldment made in a 1 3/4-in (44.5 mm) A 588 plate (table 6). The comparison also shows that the Zone 2 weld metal failed to qualify only in the two dry shoe electroslag welds made in 3-in (76 mm) A 588 plate. Failure of the weld metal to meet specified requirements may mean the following: (1) does not exceed an average of 15 ft-lb (20.3 J); (2) has more than one specimen of the three averaged below the minimum required average of 15 ft-lb (20.3 J); or (3) the value for one of the specimens was below the minimum value permitted of 10 ft-lb (13.6 J). (*11*)

For weldments made in A 36 steel, all zones passed with the exception of Zone 1 weld metal at the starting end of the water cooled shoe weld made in the 3-in (76 mm) plate (table 7). These observations point out two important facts. First is the need to carefully test the Zone 1 weld metal as well as the Zone 2 weld metal when evaluating the fracture toughness of the weldment. The procedure used in the AWS specification (*11*) for locating the specimens at the quarter thickness is not adequate for testing electroslag joints. This location will usually place the specimens in Zone 2 weld metal or in a combination of Zone 1 and Zone 2 weld metal, either of which will lead to a higher impact toughness than that measured in Zone 1. If only one set of specimens is tested, they should be located at the midthickness on the weld centerline at the starting end of the weld. This location will measure the lowest toughness value present in the weld metal. Even when Zone 1 is absent from the weld, the Zone 2 coarse columnar crystals will have their lowest impact toughness at the weld center because of the way the grain boundaries collimate at the center and the influence of the coarse secondary structure found in the center. The second fact that can be concluded from the data is that the electroslag weld metal structure is much more detrimental in high strength, low alloy steel (A 588) than it is in the construction grade of carbon steel (A 36). The complex alloy systems present in the various grades of A 588 steel result in a weld metal chemistry that is hard to control and unfavorable to the "as welded" crystal structures produced by electroslag welding. Without post-weld heat treatment, it appears to be impossible to reliably weld A 588 steel by the electroslag process, using the present commercially available electrodes, if the impact toughness requirements are to be strictly adhered to.

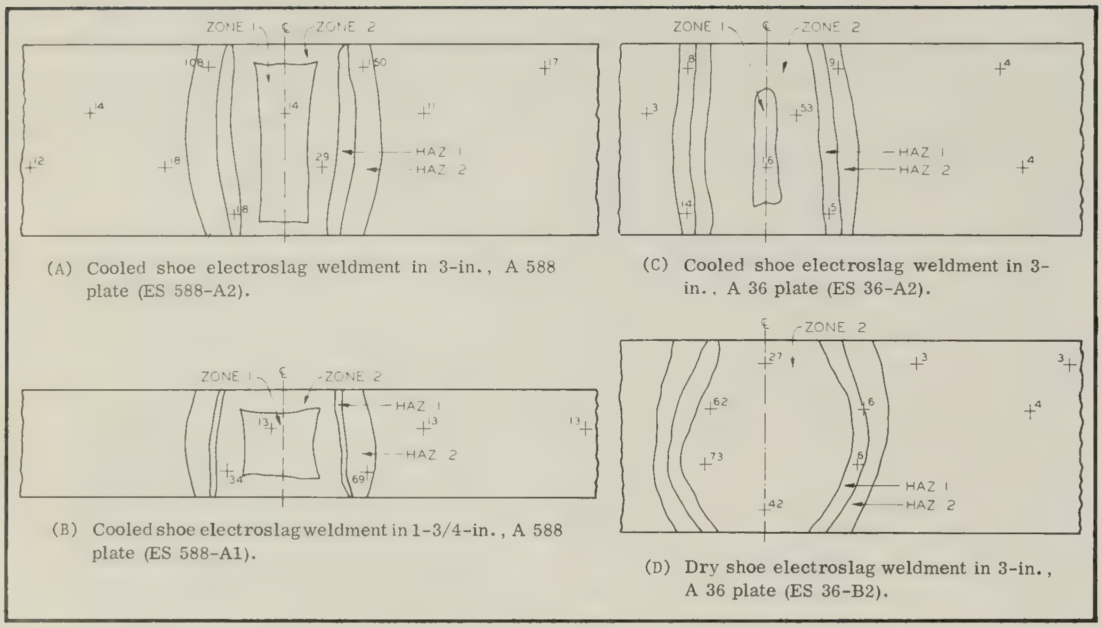

Figure 9.—Impact toughness (CVN) variation in electroslag weldments tested at 0° F (−18° C). Each value represents a set of five Charpy specimens averaged as prescribed by the specification. (14)

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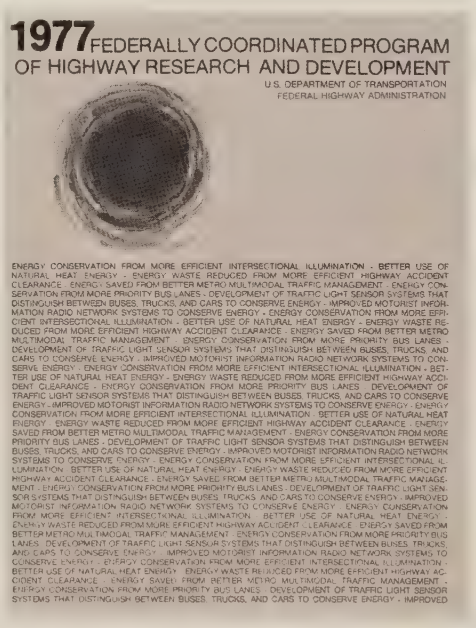
New Publications

Highway Statistics Summary to 1975 provides a general historical summary of information dealing with highway use—the ownership and operation of motor vehicles; highway finance—the receipts and expenditures for highways by public agencies; and the highway plant—the extent and changing characteristics of the mileage of public highways, roads, and streets in the Nation. The report presents a comprehensive statistical review of highway development in the United States through 1975.

Included in the summary are statistical and analytical tables of general interest on motor fuel, motor vehicles, driver licensing, highway mileage, and highway finance data for counties, townships, and other units of local government. A listing of the data is given in the table of contents, and a brief description is given in the text accompanying each section. The data presented have been obtained largely from Federal and State annual reports and through the cooperative efforts of Federal, State, and local officials, particularly those of the State highway departments.

The summary may be purchased for $4.75 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00129-2). It is also available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 for $4.75 (Report No. FHWA-HP-HS-S75).



The Offices of Research and Development, Federal Highway Administration (FHWA), have announced release of their fiscal year 1977 **Annual Report on the Federally Coordinated Program of Highway Research and Development** (FCP).

This is the fourth year for the issue of this detailed account of FHWA's progress in highway research and development (R&D). The 72-page document consists of two major parts. The first is an indepth treatment of the agency's R&D activities, organized by five major topics: for improving safety through highway design, traffic controls, and pedestrian improvements; for increasing highway efficiency; for responding to adverse environmental conditions through highway design and engineering; for preserving the environment through highway location and design; for construction cost effectiveness, low maintenance, and energy conservation. This part is preceded by a 1-page statement on the background of the FCP and a 2-page summary of the year's major achievements. The second major part deals with the agency's mission and goals, organization, facilities, contracting operations, and includes directories of project engineers and key agency personnel.

While the report's primary focus is on direct research and development efforts conducted by FHWA staff and research contractors, it also covers other highway research financed with Federal-aid highway funds, such as that conducted under the Highway Planning and Research Program and the National Cooperative Highway Research Program.

Copies of the report are available for $2.50 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00130-6).

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.

Zero-Maintenance Pavements: Results of Field Studies on the Performance Requirements and Capabilities of Conventional Pavement Systems, Report No. FHWA-RD-76-105

by FHWA Structures and Applied Mechanics Division

Many highways in urban and suburban areas are being subjected to unanticipated heavy traffic loadings which cause rapid deterioration and premature failure of pavements. Considerable maintenance is therefore required, but scheduling of remedial and preventive maintenance is almost impossible without closing lanes and producing massive traffic jams, accidents, and delays to the traveling public. This report is concerned with the improved design and construction of conventional pavements to serve exceptionally heavy loadings without maintenance for a minimum of 20 years.

The report presents the findings and results from a nationwide survey of the performance of over 60 pavements with high traffic volumes. Five types of pavements—plain jointed concrete, reinforced jointed concrete, continuously reinforced concrete, flexible, and composite—were considered in the field studies. The results presented include the following: (1) An evaluation of the capability of commonly used pavement design procedures to provide zero-maintenance performance; (2) the identification of the causes of major types of distress observed in the pavements and the corrective maintenance required; (3) limiting criteria for zero-maintenance design; and (4) a determination of the differences in maintenance-free life associated with each type of pavement in different environments. It was concluded that each of the five pavement types studied can be designed to give maintenance-free life in excess of 20 years. In various geographic regions some types will be more economical and provide better performance than others.

This report is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock No. PB 270371).

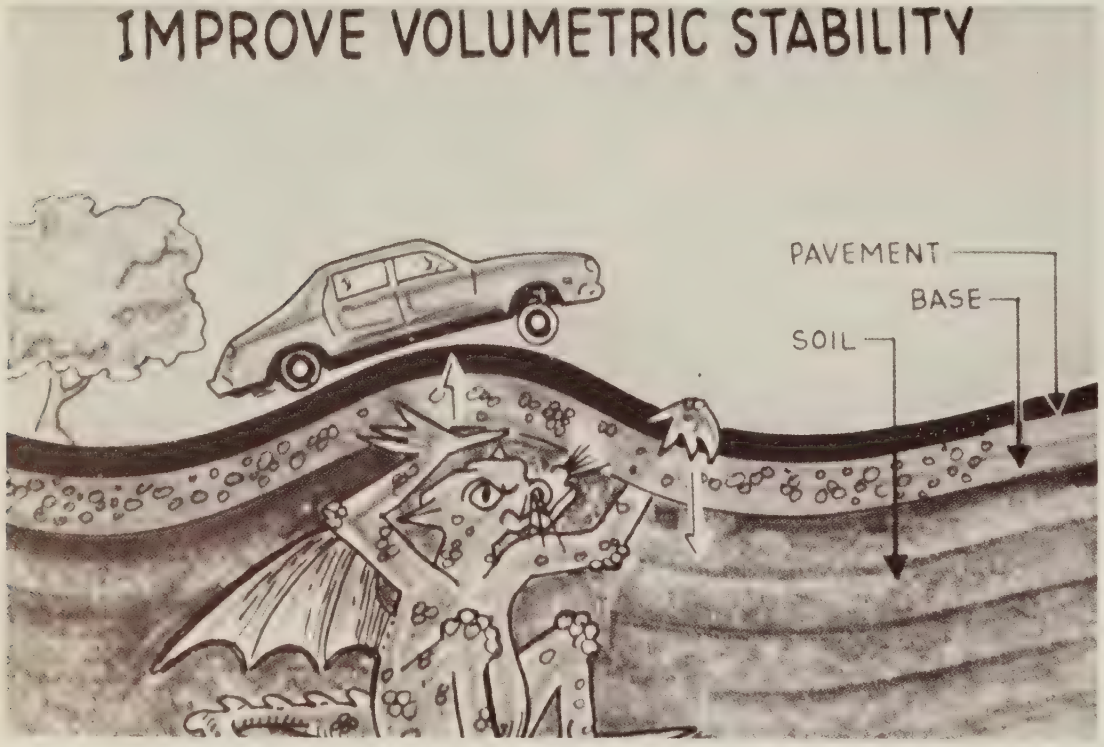
Sampling and Testing for Chloride Ion in Concrete, Report No. FHWA-RD-77-85

by FHWA Materials Division

One of the most severe problems facing the highway community is chloride ion-induced reinforcing steel corrosion and the subsequent deterioration of concrete bridge decks and marine structures. This report presents recommended methods for sampling and testing both total and water-soluble chloride ion in hardened concrete. The methods are updated and expanded versions of the earlier procedure developed by FHWA and contained in the report "Determination of Chloride in Hardened Portland Cement Concrete for Permanent Bridge Deck Repair," Report No. FHWA-RD-77-12.

Alternate procedures for sampling the hardened concrete with either a core drill or a rotary impact drill are included. In addition, two other methods of chemical analysis are presented, the original potentiometric titration method and a significantly more rapid method employing the Gran endpoint determination procedure. Both are compatible with either chloride or silver ion-selective electrodes.

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



An Evaluation of Expedient Methodology for Identification of Potentially Expansive Soils, Report No. FHWA–RD–77–94

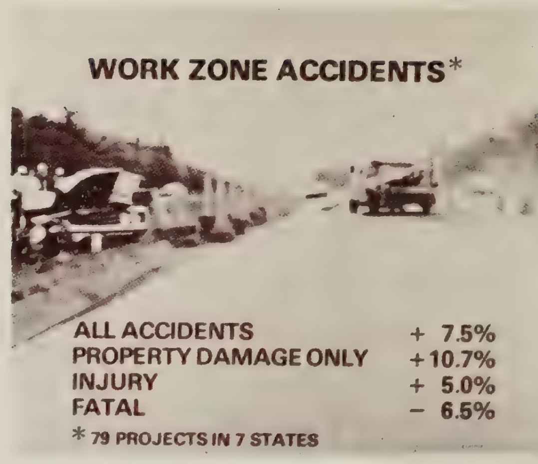
by FHWA Materials Division

This report concludes an evaluation of expedient methodology for identifying and qualitatively classifying potentially expansive soils. The need for qualitative characterization of potential swell is to warn the engineer during early planning stages of potential problems with expansive soil and to provide the necessary criteria for the application of a logical decision process to the problems associated with expansive soils in highway subgrades.

Seventeen published techniques used for identification and classification purposes were reviewed and evaluated using data collected from an extensive laboratory testing program. The results of the evaluation reveal that the best techniques, and thus the best indicators of potential swell, are the liquid limit and plasticity index.

A definition of potential swell that is more consistent with field simulation requirements and more representative of in situ volume change behavior is presented. A classification system more consistent with the definition of potential swell and using the liquid limit, plasticity index, and natural soil suction was developed and is presented. Guidelines for the recommended use of the classification system are given.

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



Accident and Speed Studies in Construction Zones, Report No. FHWA–RD–77–80

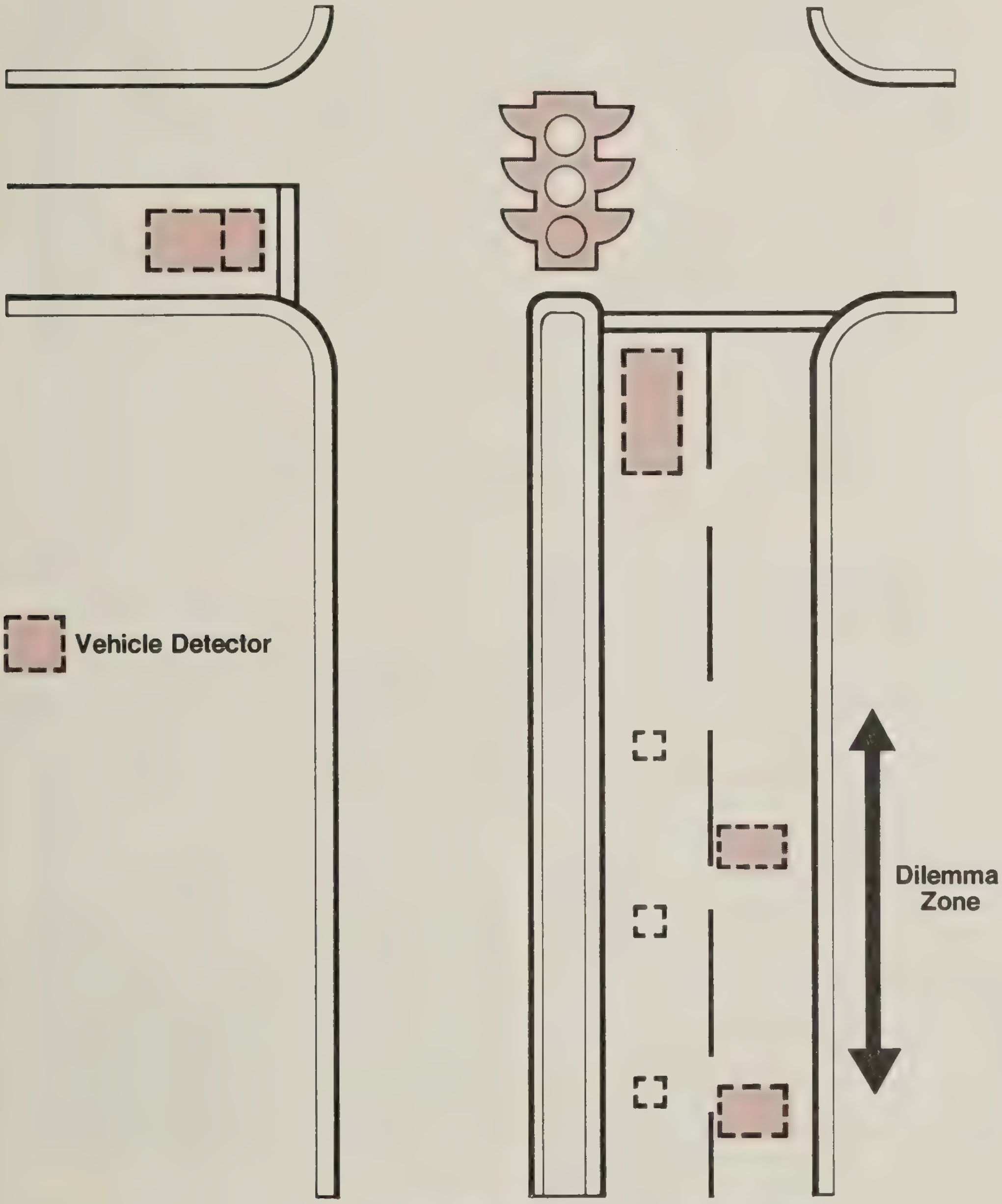
by FHWA Traffic Systems Division

To provide safe and expeditious movement of traffic through or around construction zones, the construction activity and the traffic controls must be coordinated. There is a need for rigorous management of traffic and construction operations. A research effort was initiated to determine the optimum speeds through work zones in construction areas and to develop speed design and control criteria to insure safe and efficient traffic operations.

The report resulting from this research effort presents the state of the art of managing traffic through construction zones. An attempt was made to relate accidents to construction zone characteristics based on the analysis of data from 79 construction projects located in seven States. Methods to reduce the speed of traffic through construction zones were tested in the field. Some guidelines for controlling traffic through construction zones are recommended.

The report examines the effect of the following construction zone parameters on vehicle speeds and safety: sequential flashing arrow boards, speed zoning, enforcement, transverse striping, taper length, lane width reduction, and active warning of speed zoning.

The report is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock No. PB 273290).



Vehicle Detector Placement

Vehicle Detector Placement for High-Speed Isolated, Traffic Actuated Intersection Control: Volume I (Report No. FHWA-RD-77-31), Volume II (Report No. FHWA-RD-77-32), and Volume III (Report No. FHWA-RD-77-33)

by FHWA Traffic Systems Division

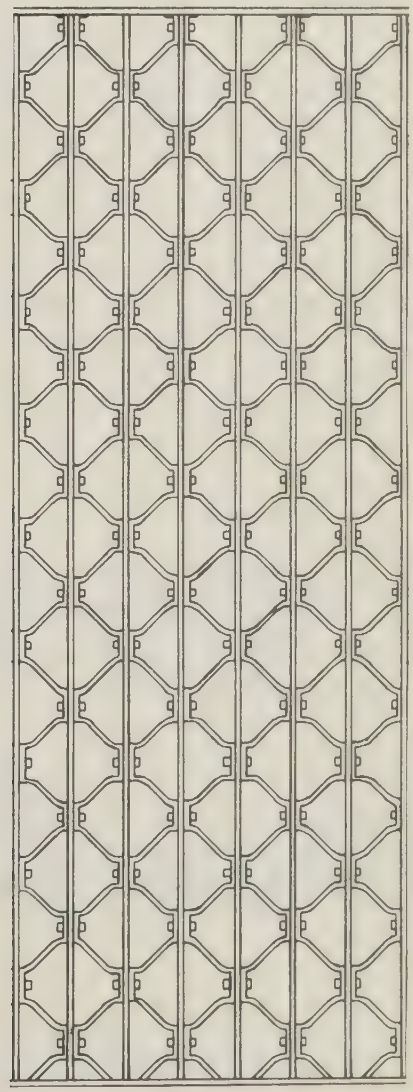
This three-volume report presents information on detector placement at high-speed isolated, traffic actuated intersections. This information has been critically reviewed and systematically integrated so that a more discriminating selection, design, and placement of detectors can be made in accordance with local requirements and resources.

Volume I, **Executive Summary,** presents an overview of the entire research effort, with key findings summarized and portrayed graphically.

Volume II, **A Manual of Theory and Practice,** describes and illustrates seven major alternative detection systems and provides clear and definitive guidelines to assist the traffic engineer in selecting among alternatives.

Volume III, **Case Study—Local Field Test and Evaluation,** documents the field testing done at two high-speed isolated, traffic actuated intersections in cooperation with local traffic engineers.

The reports are available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock Nos. PB 273380–PB 273382).



Bicycle-Safe Grate Inlets Study: Volume 1—Hydraulic and Safety Characteristics of Selected Grate Inlets on Continuous Grades, Report No. FHWA-RD-77-24

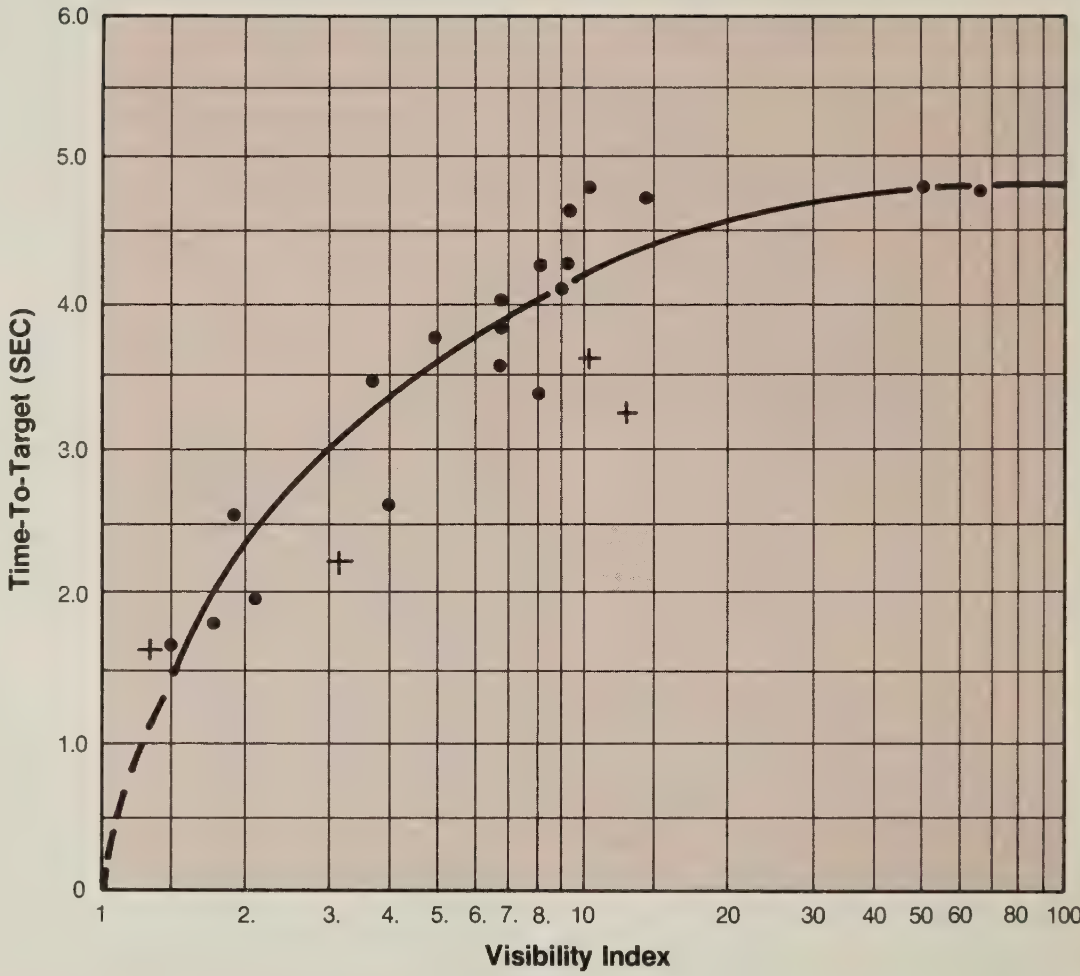
by FHWA Environmental Design and Control Division

In recent years Americans have shown increased interest in bicycling. This growing interest has resulted in increased bicycle accidents with vehicular traffic as well as with various highway-related structures. Proper surface drainage of streets and highways is one of many requirements for the safe movement of traffic and is normally accomplished with curb inlets, grate inlets, or a combination of both. This report identifies, develops, and analyzes selected grate inlets which maximize hydraulic efficiency, bicycle safety, pedestrian safety, structural sufficiency, economy, and freedom from clogging.

To accomplish the objectives of this investigation, 15 grate inlet designs were selected based on bicycle safety, hydraulic efficiency, and freedom from clogging. Eleven drain inlet grates were tested to evaluate their safety characteristics for bicycle as well as pedestrian traffic. Four of the grates that rated highest in the safety tests were selected for hydraulic testing. A parallel bar grate was included in the hydraulic test program as a standard with which to compare the performance of the other test grates.

Characteristics of the tested grates are presented in individual chapters, and a summary and discussion of results are included in the report.

The report is available from the Environmental Design and Control Division, HRS-42, Federal Highway Administration, Washington, D.C. 20590.



Effectiveness of Highway Arterial Lighting: Final Report (Report No. FHWA-RD-77-37) and Design Guide (Report No. FHWA-RD-77-38)

by FHWA Environmental Design and Control Divison

These reports describe a study to produce reliable methods for adequate and cost effective street lighting design. The results of an accident analysis have shown that total nighttime dry weather accidents are inversely related to visibility; higher visibility results in fewer accidents. Areas with high population densities have a much higher accident rate than low density areas, and central business districts have a much higher rate than other areas. For typical street configurations, more cost beneficial lighting systems can be designed using high pressure sodium sources rather than mercury luminaires.

Costs considered in the reports include initial installation, operation, and maintenance of various lighting treatments. Instructions for using a computer program to determine visibility on arterial streets are provided, and a framework for rational decisionmaking for conversion of existing lighting systems is described.

The report is available from the Environmental Design and Control Division, HRS-42, Federal Highway Administration, Washington, D.C. 20590.

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
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Office of Development
Implementation Division, HDV-20
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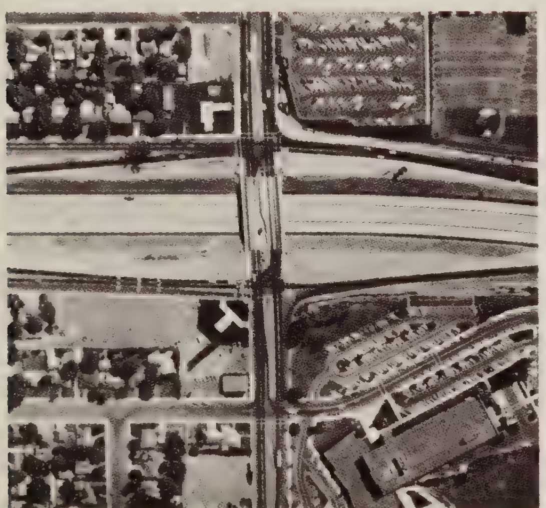


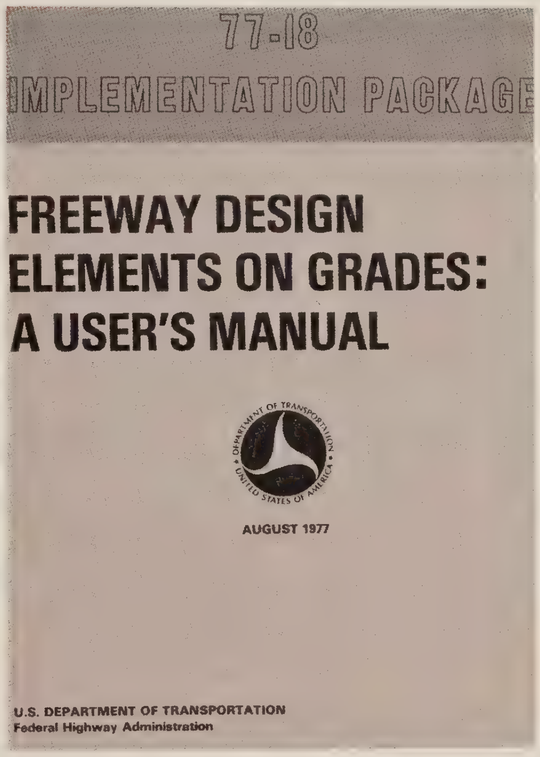
Improvements for Diamond Interchange Traffic Control

by FHWA Implementation Division

This motion picture film describes a research effort on diamond interchange traffic control that began in 1966. The film describes the development and testing of the real-time control system which evolved through simulation analysis using a series of candidate control algorithms, a constructed computerized system in Los Angeles, and field test results of an algorithm which minimized average vehicle delay. A 20 to 30 percent reduction in both delay and stops for motorists using the diamond interchange complex was indicated.

The hardware and software shown in the movie have become outdated and controllers designed around a microcomputer are now recommended for this application.

The movie is available from FHWA regional offices (see page 192.)


77-18
IMPLEMENTATION PACKAGE

FREEWAY DESIGN ELEMENTS ON GRADES: A USER'S MANUAL

AUGUST 1977

U.S. DEPARTMENT OF TRANSPORTATION
Federal Highway Administration

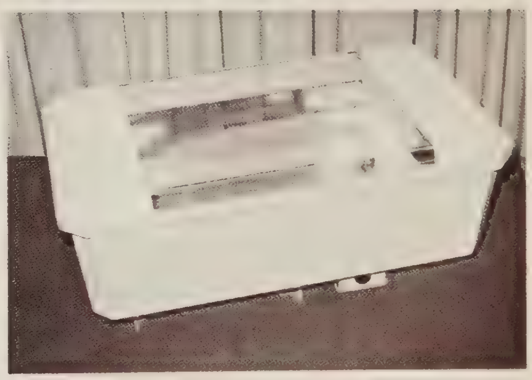
Freeway Design Elements on Grades: A User's Manual, Implementation Package 77-18

by FHWA Implementation Division

This user's manual is intended as a guide or design procedure to evaluate traffic operations on grades and to design climbing lanes. It provides charts and procedures for the calculation of operating speeds and service levels in mixed traffic flows of trucks and passenger cars on grades. Procedures which are applicable to traffic flows on two and three unidirectional lanes where access and roadside interference are negligible are described. Special attention is given to situations where the third lane is added to maintain service on an upgrade.

The procedures and guides outlined in the manual are based on results from a simulation model that has been extensively tested. The manual appeals to those who seek information on the source, character, and capabilities of the procedures and to those who wish to solve a specific problem.

The manual is available from the Implementation Division.



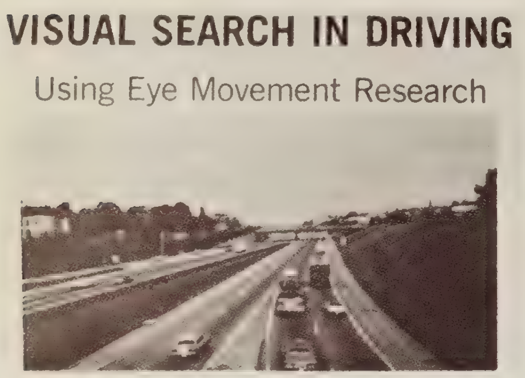
Special Crosswalk Illumination for Pedestrian Safety

by FHWA Implementation Division

This 5-minute movie describes a new technique for illuminating pedestrian crosswalks to reduce nighttime accidents. The film vividly portrays pedestrian crossing safety problems, describes lighting hardware used to illuminate the crosswalk, and includes photographs of pedestrians using the crosswalk during lighted and unlighted conditions.

The film can be used alone or as an introduction to a more detailed 30-minute slide-tape presentation which is available. The presentation includes warrants for crosswalk lighting, design of the crosswalk, and crosswalk lighting costs.

The movie and slide-tape presentation are available from FHWA regional offices (see page 192).



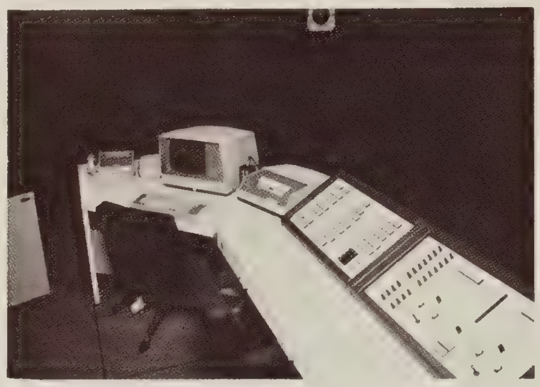
Visual Search in Driving Using Eye Movement Research

by Ohio Department of Transportation and FHWA Implementation Division

This 21-minute, color, sound film describes an eye movement recording system which is used to gather research information about the visual information system. The unique system allows researchers to observe and record the eye movements of drivers in actual road driving situations. While driving, a small light source reflected from the driver's eye is combined with the road-scene picture and other measurement data and is recorded on video tape. Once the data is recorded on the video tape, it is taken to the Driving Performance Laboratory for data reduction and statistical analysis.

This research technique has been applied to study sign design effects, curve negotiation, night driving, visual search of new drivers, and the effects of alcohol, fatigue, and carbon monoxide on drivers. This available data on driver information acquisition can lead to the design of safer highways and vehicles.

The movie is available from FHWA regional offices (see page 192).

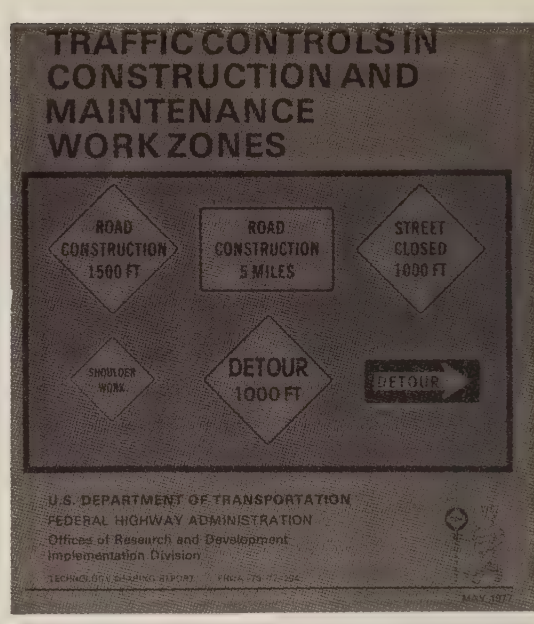


Traffic Engineering Technology Transfer

by FHWA Implementation Division

This slide-tape presentation describes immediately available technology transfer items dealing with the general subject of traffic engineering. The presentation includes descriptions of handbooks and manuals to aid with many aspects of design, installation, and maintenance of traffic systems; training courses on management of complex traffic systems and traffic control techniques; seminars on computer traffic control technology; implementation packages of traffic control system hardware; the latest traffic simulation models; and a variety of computer programs.

The slide-tape presentation is available from FHWA regional offices (see page 192).



Traffic Controls in Construction and Maintenance Work Zones, Volume I (Report No. FHWA-TS-77-204) and Volume II (Report No. FHWA-TS-77-203)

by FHWA Implementation Division

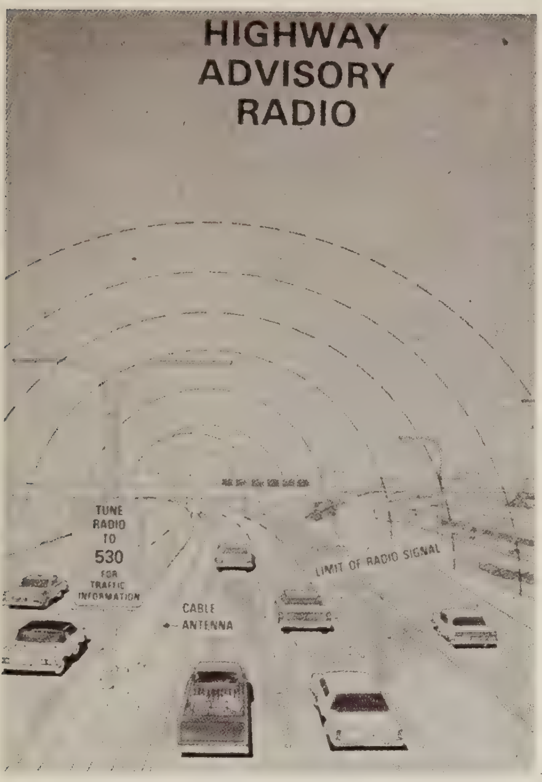
This two-volume manual provides information that can be used to supplement local, State, and national standards to promote uniformity in the application of provisions to protect the traveling public, construction and maintenance personnel, and pedestrians. The report illustrates many typical worksites and describes the most common conditions encountered.

Volume I, **Office Function,** emphasizes the value of a good public information program in meeting the needs of specific projects and in building community support for future improvements.

Volume II, **Field Function,** is intended primarily for job supervisors, addressing common work zone situations and problems and offering solutions.

Both volumes are in conformance with the national Manual on Uniform Traffic Control Devices.

The reports are available from the Implementation Division.



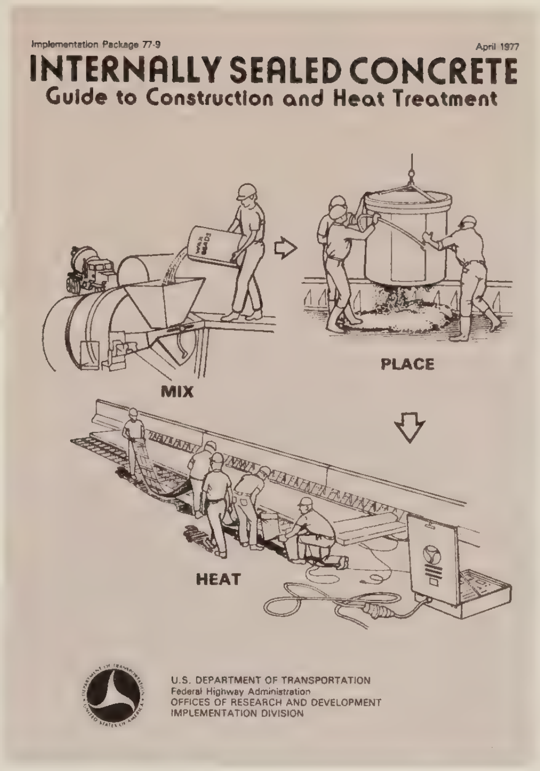
Highway Advisory Radio

by FHWA Traffic Systems Division

This movie presents a history of roadside radio from 1940 to current highway advisory radio (HAR) activities. It explains the HAR concept and describes HAR hardware.

HAR techniques used in State and national parks, drive-in theaters, and amusement parks are depicted. Major sites employing highway advisory radios, such as the Los Angeles Airport and the Walt Whitman Bridge in Philadelphia, are featured. Cable types, antennas, and special in-vehicle highway advisory radio equipment are shown.

The movie is available from FHWA regional offices (see page 192).



Internally Sealed Concrete: Guide to Construction and Heat Treatment, Implementation Package 77–9

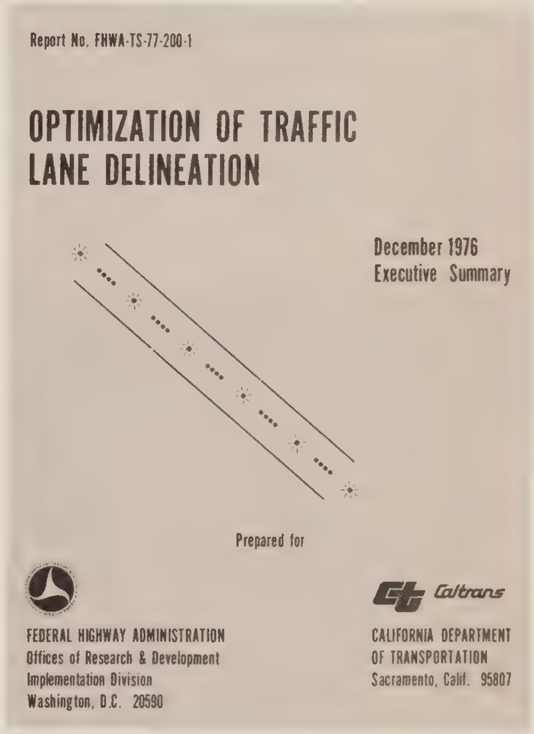
by FHWA Implementation Division

Steel rebar corrosion caused by deicing salts or ocean spray is considered the major cause of early deterioration of reinforced concrete bridge decks. Preventing chloride intrusion into the concrete has been widely studied as a means of eliminating the problem. One promising solution is set forth in this manual which outlines and discusses the use of wax beads to internally seal concrete and prevent the intrusion of the chloride ion.

Internally sealed concrete is a standard portland cement concrete modified by replacing a small portion of aggregate with a similar volume of wax beads. The concrete is heated, causing the wax to melt. The melted wax flows into capillaries and pores, providing an effective seal against penetration by moisture and chlorides.

During the past year, nine internally sealed concrete bridge decks were constructed and heat treated. All of the decks were surveyed and no freeze-thaw damage or delamination of the overlays was found. The lack of damage on the internally sealed decks is an excellent indicator that sealing was achieved.

The manual contains complete information on guideline procedures and specifications useful to the engineer and contractor. The report is available from the Implementation Division.



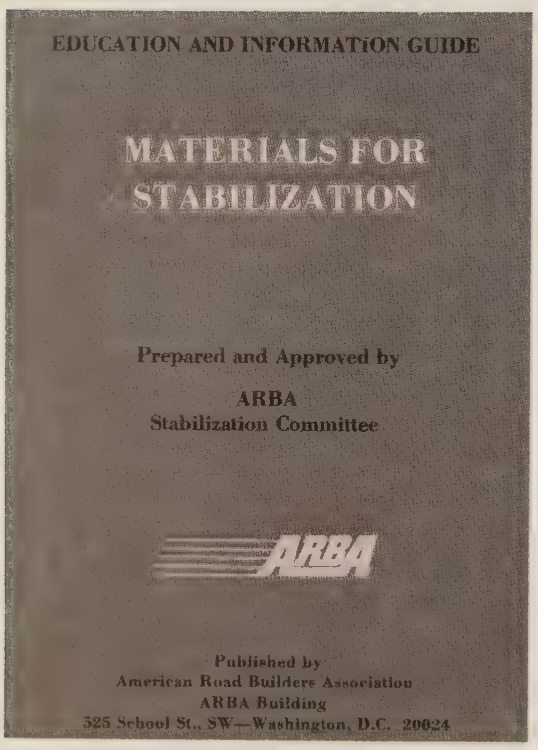
Optimization of Traffic Lane Delineation, Final Report (Report No. FHWA–TS–77–200) and Executive Summary (Report No. FHWA–TS–77–200–1)

by FHWA Implementation Division

The purpose of this study was to review and evaluate existing pavement delineation practices, investigate initial costs, maintenance costs, application methods, and determine an optimum system. The reports summarize the results of an investigation of three systems of pavement delineation—paint, raised pavement markers, and hot spray thermoplastic—on different types of highways.

This study reviewed existing practices in California and selected States throughout the Nation to investigate eight areas where possibilities of improvements to current practices were anticipated. Cost data were also developed for the existing and proposed systems. The major consideration in the reports' recommendations is to provide a system that will not decrease the guidance provided motorists or adversely affect safety.

The reports are available from the Implementation Division.



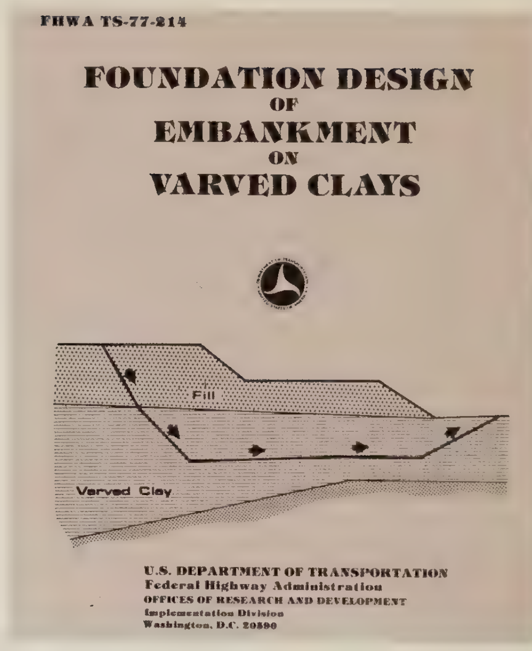
Materials for Stabilization: Education and Information Guide, Report No. HC–100A

by American Road Builders Association and FHWA Implementation Division

This report includes nine short articles on several soil stabilization techniques using asphalt, calcium chloride, cement, fly ash, lime, and sodium chloride. It is intended as an educational and informational guide for highway engineers interested in the application of soil stabilization techniques.

The cost effective use of soil stabilization for pavement bases and subbases is becoming increasingly important because of dwindling supplies and the rising costs of good aggregate. As a result of this, much research has been conducted in the laboratories and on actual construction projects in the last 20 years which establishes the feasibility, practicality, and limitations for the use of various stabilization techniques. Information from many of these studies is available in several separate reports; this report provides one convenient source for the information on the potential methods of soil stabilization and sources for obtaining further information on specific product application.

The report is available from the Implementation Division.



Foundation Design of Embankment on Varved Clays, Report No. FHWA–TS–77–214

This report presents procedures applicable to the foundation design of embankments constructed on soft foundations. The described techniques can be applied directly to foundations composed of horizontal deposits of varved clays found in the Northeastern United States.

Much of the information and data presented in the report is also applicable to soft foundations of heterogeneous deposits of clays, silts, and sands. The report is intended to provide guidance in selecting soil design parameters, predicting soil strength increases, and estimating rate of consolidation and associated deformations for embankment foundations, including applications of surcharging and sand drains. A comprehensive design problem is presented to facilitate understanding of the design procedures and to provide guidance for analyzing various embankment-foundation slope stability problems.

The report is 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: Guidelines for the Application of Arrow Boards in Work Zones. (FCP No. 31A2554)
Objective: Develop guidelines for the application of arrow board devices in work zones with respect to the conditions under which the devices should be used and where in the zone the devices should be placed to provide for safe and efficient movement of traffic.
Performing Organization: Midwest Research Institute, Kansas City, Mo. 64110
Expected Completion Date: November 1978
Estimated Cost: $154,000 (FHWA Administrative Contract)

FCP Project 1J: Improved Geometric Design

Title: Procedures and Guidelines for the Rehabilitation of Existing Freeway-Arterial Highway Interchanges. (FCP No. 31J4032)
Objective: Develop cost, safety, and operational effective geometric design procedures and guidelines for the upgrading of freeway-arterial highway interchanges by quantifying the effect of cost, safety, and operational trade offs. Interchanges to be investigated include those upgraded to accommodate an increase in traffic volume, maximize safety benefits, and minimize costs.
Performing Organization: Midwest Research Institute, Kansas City, Mo. 64110
Expected Completion Date: September 1980
Estimated Cost: $184,000 (FHWA Administrative Contract)

FCP Project 1T: Advanced Vehicle Protection Systems

Title: Impact Attenuators for Heavy Vehicles—A Feasibility Study. (FCP No. 31T3054)
Objective: In Phase I, determine the feasibility of developing an impact attenuator system capable of safely decelerating both compact and heavy vehicles. If feasible, design, construct, and full-scale test a prototype model in Phase II.
Performing Organization: Southwest Research Institute, San Antonio, Tex. 78284
Expected Completion Date: October 1979
Estimated Cost: $279,000 (FHWA Administrative Contract)

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

Title: Simulation of Effects of Increased Truck Size and Weight. (FCP No. 31U4012)
Objective: Modify and exercise the MVMA-HRSI truck simulation program for investigating the dynamics of heavy vehicle trains, their response to control imputs, and their stability in the presence of disturbance inputs.
Performing Organization: University of Michigan, Ann Arbor, Mich. 48105
Expected Completion Date: March 1979
Estimated Cost: $108,000 (FHWA Administrative Contract)

Title: Instrumentation of Test Trucks for Measuring Pavement Vehicle Interactions. (FCP No. 31U4034)
Objective: Review literature, select parameters, prepare system design for truck instrumentation, assemble and install instrumentation on a truck, validate by test runs, and document research with manuals and reports.
Performing Organization: Systems Technology, Inc., Hawthorne, Calif. 90250
Expected Completion Date: October 1978
Estimated Cost: $100,000 (FHWA Administrative Contract)

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

Title: Traffic Management During Urban Freeway Maintenance Operations. (FCP No. 41Y1672)
Objective: Develop practical traffic control and management systems for temporary maintenance work zones along urban freeways in Texas cities which improve the safety of both

workmen and motorists in the work area with minimum loss of facility usefulness for the duration of the maintenance operation.
Performing Organization: Texas Transportation Institute, College Station, Tex. 77843
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1979
Estimated Cost: $130,000 (HP&R)

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

FCP Project 2D: Research on Priority Techniques for High Occupancy Vehicles

Title: Priority Use of Transportation Facilities. (FCP No. 42D1574)
Objective: Prepare for and assist the Texas Department of Highways and Public Transportation in the implementation of priority treatment projects on transportation facilities which include both restricted travel lanes and park and ride techniques.
Performing Organization: Texas Transportation Institute, College Station, Tex. 77843
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1979
Estimated Cost: $372,000 (HP&R)

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

Title: Determination of Origin-Destination Matrices from Link Volumes and Turning Movements. (FCP No. 32K2022)
Objective: Develop a software package to estimate the origin-destination matrix of a given network given the volumes on all links of the network and the turning movements at all nodes of the network.
Performing Organization: John Hamburg and Associates, Chicago, Ill. 60603
Expected Completion Date: February 1979
Estimated Cost: $190,000 (FHWA Administrative Contract)

Title: Signing and Delineation for Special Usage Lanes. (FCP No. 32K4032)
Objective: Determine the information required by both users and nonusers of special usage lanes on urban streets and freeways and develop signing and delineation systems to assure compliance and appropriate use of these lanes.
Performing Organization: BioTechnology, Inc., Falls Church, Va. 20042
Expected Completion Date: June 1979
Estimated Cost: $383,000 (FHWA Administrative Contract)

Title: Requirements and Specifications for Off-Hours Delivery. (FCP No. 32K5022)
Objective: Identify financial and institutional restraints which impede off-hours truck pickup and delivery operations in central business districts. Produce guidelines which will provide reasonable assurance that a viable system of off-hours delivery can be established in a given metropolitan area.
Performing Organization: Organization for Environmental Growth, Washington, D.C. 20017
Expected Completion Date: January 1979
Estimated Cost: $97,000 (FHWA Administrative Contract)

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

FCP Project 3H: Procedures for Integration of Public Needs and Citizen Concerns in the Location and Design of Highways

Title: Effects of Freeway Noise on Hearing Level and Academic Achievement of Children. (FCP No. 43H1072)
Objective: Measure freeway noise in elementary school classrooms and correlate noise level with academic achievement. Develop empirically based interior noise limit for school.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95814
Expected Completion Date: June 1979
Estimated Cost: $350,000 (HP&R)

FCP Category 4—Improved Materials Utilization and Durability

FCP Project 4D: Remedial Treatment of Soil Materials for Earth Structures and Foundations

Title: Stability of Slopes in Shale and Colluvium. (FCP No. 44D5142)
Objective: Improve maintenance practice and the design of correction measures through an improved understanding of the behavior of slopes in shales and clay.
Performing Organization: Ohio State University, Columbus, Ohio 43210
Funding Agency: Ohio Department of Transportation
Expected Completion Date: June 1980
Estimated Cost: $213,000 (HP&R)

FCP Project 4G: Substitute and Improved Materials to Effect Materials and Energy Conservation in Highways

Title: Economic Asphalt Treated Bases. (FCP No. 44G1093)

*U.S. Government Printing Office: 1978—720-428/2

Objective: Develop criteria for mix design and material selection of bituminous stabilized bases using emulsified asphalt, sand asphalt mixtures, and foamed asphalt. Investigate use of gyratory compactor curing and strength properties by laboratory testing. Take surface deflections and roughness measurements. Use cost data to develop economic model that will balance material and structural properties.
Performing Organization: Texas Transportation Institute, College Station, Tex. 77843
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1980
Estimated Cost: $135,000 (HP&R)

Title: Assessment of Impermeability of Rigid Concrete Members. (FCP No. 34G3022)
Objective: Develop rapid destructive and nondestructive test methods and equipment to determine the degree and depth of impermeability of rigid concrete materials.
Performing Organization: Portland Cement Association, Skokie, Ill. 60076
Expected Completion Date: August 1979
Estimated Cost: $156,000 (FHWA Administrative Contract)

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

FCP Project 5B: Tunneling Technology for Future Highways

Title: Fabrication and Research Evaluation of a New Combination Sensing System to Determine Ground Conditions for Tunnel Design and Construction. (FCP No. 35B2202)
Objective: Develop, fabricate, and demonstrate a prototype multiple sensor borehole probe system capable of obtaining exploratory subsurface information relevant to transportation tunnel design and construction, using electromagnetic, acoustic, and earth resistivity measurements. Perform an indepth analysis and evaluation of the system.
Performing Organization: Southwest Research Institute, San Antonio, Tex. 78284
Expected Completion Date: May 1979
Estimated Cost: $500,000 (FHWA Administrative Contract)

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

Title: Modification of the System EAROMAR. (FCP No. 35E1022)
Objective: Update data, models, and basic relationship that EAROMAR system uses to determine costs of pavement maintenance; modify the system so pavement deterioration will be sensitive to a variety of variables; modify the system to provide greater flexibility in pavement management systems; and determine economic warrants for constructing premium pavement for varying traffic characteristics and pavement designs.
Performing Organization: CMT, Inc., Cambridge, Mass. 02139
Expected Completion Date: September 1979
Estimated Cost: $100,000 (FHWA Administrative Contract)

FCP Project 5F: Structural Integrity and Life Expectancy of Bridges

Title: Dynamic Loading of Skewed Bridges. (FCP No. 25F4082)
Objective: Experimentally investigate the dynamic behavior of skewed highway bridges through the use of a scale multistructure. Verify dynamic load factors.
Performing Organization: Federal Highway Administration, Washington, D.C. 20590
Expected Completion Date: October 1980
Estimated Cost: $81,000 (FHWA Staff Research)

FCP Project 5K: New Bridge Design Concepts

Title: Influence of Casting Position and of Shear on the Strength of Lapped Splices. (FCP No. 45K1014)
Objective: Determine the influence of bar position during casting on the capacity of lapped splices and anchored bars, and the influence of shear (or moment gradient) on the behavior of anchored bars.
Performing Organization: University of Texas, Austin, Tex. 78712
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: March 1980
Estimated Cost: $90,000 (HP&R)

Title: Structural Behavior of a Skew Reinforced Concrete Box Girder Bridge Model. (FCP No. 45K1024)
Objective: Design, construct, instrument, and test a 1:2.82 scale model of a typical skew, continuous, two-span, four-cell reinforced concrete box girder bridge.
Performing Organization: University of California, Berkeley, Calif. 94720
Funding Agency: California Department of Transportation
Expected Completion Date: June 1980
Estimated Cost: $410,000 (HP&R)

FCP Project 5L: Safe Life Design for Bridges

Title: Rehabilitation of Bridges on Local Roads. (FCP No. 45L3032)
Objective: Determine properties of wrought iron and steel produced early in this century. Study the problems of welding this material and develop welding procedures. Develop a rapid means of identification to distinguish between wrought iron and steel.
Performing Organization: University of Illinois, Urbana, Ill. 61801
Funding Agency: Illinois Department of Transportation
Expected Completion Date: July 1981
Estimated Cost: $106,000 (HP&R)

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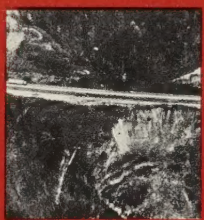
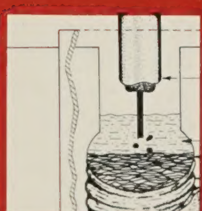


Status of Shale Embankment Research

What Network Simulation (NETSIM) Can Do for the Traffic Engineer

The Fifth Annual Review of the FCP

Design of Segmental Bridges



Electroslag Weldments: Performance and Needed Research

public roads

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