VOLUME

41

HIGHWAY RESEARCH BOARD

1962

HIGHWAY RESEARCH BOARD

Proceedings

41ST ANNUAL MEETING

1962

VOLUME 41

PROCEEDINGS

41st Annual Meeting

National Academy of Sciences — National Research Council

DIVISION OF ENGINEERING AND INDUSTRIAL RESEARCH

HIGHWAY RESEARCH BOARD

VOLUME 41

Proceedings

OF THE

41st ANNUAL MEETING

Washington, D. C., January 8-12, 1962

FRED BURGGRAF
Director

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Editor

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1962

WASHINGTON, D. C.

NAS-NRC PUBLICATION 1024

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1962	K. B. WOODS, Head, School of Civil Engineering, and Director, Joint Highway Research Project, Purdue University.

[1] Died Aug. 27, 1961.

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[1] From Nov. 11, 1920.
[2] Temporary Chairman.
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DEPARTMENT OF ECONOMICS, FINANCE AND ADMINISTRATION

Management Improvement Programs in State Highway Departments

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One of the principal objectives of the National Highway Management Conferences, jointly sponsored by the American Association of State Highway Officials and the National Highway Users Conference is to stimulate activity in the States, to help define management problems, and to indicate ways in which State highway departments may undertake management improvement projects.

More than 20 State highway departments have undertaken management conferences for executive and supervisory personnel. These follow the pattern of the National Conferences and utilize as study material a manual developed as a product of the AASHO-NHUC sponsored program. Although most States have held the management conference sessions just for the upper level executives, several States have had numerous sessions including personnel down to the project supervisors. In some cases these sessions have taken the form of management problem-solving workshops.

Several management improvement projects of a special and promising character are now under way.

• THIS PAPER is an appraisal of highway management improvement programs that have been initiated in the States, stimulated by the National Highway Management Conferences and related activity under sponsorship of the American Association of State Highway Officials and the National Highway Users Conference.

The National Highway Management Conferences—one week in length—were initiated in 1956. Since then, there have been two each year. To meet the increasing demand for attendance there will be three National Conferences in 1962.

The purpose of the National Management Conferences is to provide for highway officials a program for discussion of management theory and practice somewhat on the pattern of the many executive development conferences carried on by industries and universities throughout the country.

The highway management conferences include academic discussion of the principles of management, consideration of what large, private businesses are doing in the areas of management improvement, and exchange of ideas between the highway administrator participants.

Although there are more than 200 still active highway administrators who have attended one or more National Conferences, over 30 of the chief administrators of the highway departments have attended, and only two States have never had a representative at a Conference, it has always been recognized that the National Conferences in themselves can do but a small part of the management improvement job. In the first place, there are thousands of individuals carrying on important administrative jobs in highway departments. The National Conferences can never accommodate more than a small percentage of this total. Furthermore, it is recognized that the Conferences are the beginning, not the end, of a management improvement program. Therefore, the States have been encouraged to carry on their own conference programs—generally called seminars—to provide the groundwork on which to build management improvement programs in the States.

STATE HIGHWAY MANAGEMENT SEMINARS

More than 20 States have had one or more management seminars. The State seminars have been patterned to a degree on the National Conferences.

Most of the States have used the "Manual for a Highway Management Seminar" as a principal text. This was published in 1957 by the American Association of State Highway Officials and the National Highway Users Conference.

The majority of the States have limited seminar programs to one or two, with attendance only from the upper level of management.

In some cases the programs have been for a continuous full week very much like the National Management Conferences. In other cases programs have been for shorter periods—three days, for example. Also, there are instances when the seminars were scheduled for short sessions, either a day or half-day, at weekly intervals over a period of several weeks.

In contrast with the holding of only one or two seminars for top management, several States have "gone all out" with the intent, over a few years, of reaching all levels of supervision.

At least one State has a carefully scheduled program for a series of annual seminars—each three days in length—to be attended by the same groups of top and middle management personnel. One year they concentrate on one group of related topics, the next year on another.

Because of the wide variety of seminar programs, it should be possible to evaluate results and develop some conclusions as to what are good or bad programs. It would be ideal if influence of the seminars could be measured directly on the individual managers who participate. Then equations could be set up representing various elements that go into the programs and be solved for the several variables. The best that can be done, however, is to reflect one's own judgment and that of many highway administrators with respect to the influence of some of the variables in the seminar programs.

Penetration

How far down the management ladder does the seminar program go? Should seminars be limited to top management? Should they include top and middle management? Or, should they be organized to include all managers down to first-line supervisors?

It was indicated earlier that some States have held one or two seminars just for top management personnel. In others, however, the seminars are being scheduled to reach and ultimately include first-level supervisors. In one State, there has been a seminar for top management and another for a select group of young lower-level managers. Therefore, there are at least three different "penetrations" with the seminar programs.

A seminar program should not be judged good or bad based on whether it is limited to top management or whether it is organized to include ultimately all levels of management. It may be that in different highway departments there is not the same concept of the seminar as a part of a management improvement program.

It is noteworthy, however, that the States that have extended the seminar program to lower management levels are some of the most enthusiastic and determined in their efforts to develop managers. Judging from their experience, it appears that the management seminar has great value at all management levels.

Location and Facilities for Seminars

Seminars have been held at a variety of places: headquarters conference rooms, university campuses, fairly isolated conference centers. Some were in crowded, poorly furnished rooms; others had excellent facilities.

Unfortunately too little attention is given to this problem of site and facilities. The seminar should be held at some site away from headquarters. Participants should be housed together. No one should be allowed to commute from home to seminar sessions. The seminar room should be of ample size to accommodate all participants around a U-shaped table. Acoustics should be such that everyone can hear everyone else. Ventilation is important, as is freedom from disturbing noises. Consideration should be given in advance to blackboards, slide and movie projectors and screens and any other facilities needed for discussion sessions. Name cards of large and legible design should be set in front of each participant. Stick-on white letters on black cards are excellent. Comfortable chairs and table arrangements are most important. The sessions are long and the participants are entitled to every consideration for their comfort.

Some seminars have been disappointing and the results not too good, and to a considerable extent this has happened simply because of a poor location and inadequate facilities.

Time Schedule

Some States have used the same schedule as the National Conferences. These run a week, starting Sunday afternoon and ending Friday night. A number of States have run three-day schedules. Several have had programs of single or partial days, repeated at regular weekly intervals.

There have been some fine programs held on each of these schedules. However, it is much more difficult to conduct a successful program on the week-to-week scheduling of a day at a time than to have the group together continuously for a longer period. There have been some programs scheduled from week to week that have not been satisfactory in the view of the State personnel themselves. On the other hand, there are no reports indicating dissatisfaction with programs scheduled for a full week.

There are some real values in getting participants away from their jobs for as long a period as possible. To get the maximum value from the seminar, they need to adjust to a different atmosphere and a new routine. This takes time.

When the National Conferences started in 1956, experienced executive development conference leaders said that this kind of job should not be tried for less than two weeks. They emphasized that it was not just the scope of the subject matter but, equally important, the need for people to adjust to the conference atmosphere. At that time, it did not seem possible to get highway officials away from their jobs for two weeks, so the conference was held for one week. This proved highly successful. If it could have operated two weeks it might have been even more successful. Three-day State conferences have also apparently been quite successful. It is probable that they would have been even more successful on a full-week schedule.

Size of Seminar Group

There have been seminars with as few as 20 participants. There also have been groups as large as 40. There can be no categorical position on this. If the seminar is conducted primarily as a discussion group, and not as a lecture course, with great emphasis on active participation, there are obvious advantages in a small number of participants. However, it may be found unfeasible or uneconomic to carry on programs with very small groups. This would most likely be the case if the program is to be extended to all levels of management where the total number of individuals ultimately to be accommodated is large. Furthermore, by using committees as work groups it is possible to generate broad participation even though the total number in the seminar may be as high as 40.

It must also be recognized that the length of the seminar, the management experience level of participants, relationships already existing between individuals, prior experiences, etc., are all factors influencing the group and individual response.

It is concluded from the experience with the National Conferences and work with the States that for a one-week seminar, with top management people and using committees as work groups, 30 represents a desirable seminar enrollment.

Selection of Seminar Participants

It has been suggested to States, as they undertake programs, that they start with top management in the initial seminar. It is apparent that any management improvement program, starting with seminars and continuing with other activities, must always have the understanding and full support of top management.

Several questions immediately arise, however. Should the very top boss or bosses be included—the highway director, the chief engineer, the chairman of the commission? Should any individual attend with his direct superior also in attendance? Can all the top managers be away from their jobs at the same time?

The first two questions, to a degree, are the same: should a man attend a seminar at which his direct superior is in attendance? The answer is definitely no. In most instances at a number of seminars where bosses and immediate subordinates are attending the same seminar, the subordinate will not participate in the same way he would if he were attending on his own, which is a perfectly natural and understandable situation. The effective superior-subordinate relationship contemplates that when they attend a meeting together, the superior speaks for the organization. In their own sphere of work activity they may have differing views and discuss them freely, but when they move into sessions with people from other segments of the organization, or with outsiders, they have only one position with regard to their work and that is generally presented by the boss.

The management seminars deal with principles and with hypothetical problems and need not get down to problems involving the work area of the individual. This is generally true, although it is very effective to have participants bring up problems from their own areas of work. However, even in discussion of principles and non-highway problems the subordinate—by training—defers in the group meeting to his superior.

It is a rare situation to get the same participation from subordinates with and without their bosses present. Many bosses sincerely do not believe this because they are so anxious that it not be true; they feel it is a reflection on them personally, on their tolerance as bosses and that it implies a lack of mutual trust and respect between them and their subordinates.

It is therefore concluded that the director, the chief engineer, and the chairman of the commission should stay away from the seminar being held for their subordinates. Most States accept this and the top officials, as well as some others in the department, go to the National Management Conferences and receive their exposure to management theory, principles, and practices.

For positions below the top, there may be difficulty in completely separating superiors and subordinates. When only one seminar is held for top management, it is almost certainly necessary to have a number of men and their immediate bosses in attendance. Though not ideal, this is better than not having some key people attend at all. Immediate superiors and subordinates would, of course, be separated on committee work assignments. If two seminars can be held for top management personnel, it is possible to set the two groups with minimum, if any, superior-subordinate pairs in the same seminar.

The question was raised as to whether all the top managers could be away from their jobs attending a seminar at one time. In one State, literally all top brass were away for three days. The department operated in their absence without a crisis. No emergency telephone calls required the absence of any participant from the conference sessions. This might indicate that nothing was done in the highway department for those three days. However, it probably indicates the top staff people are good administrators and have things organized so their subordinates are prepared to carry on in their absence. In another State, all but a few of the top managers were away for a full week. The few excepted from attendance had been to National Conferences and represented a skeleton force of top management during the one-week seminar.

The best arrangement for top management seminars is to split the group in two. One-half attends one week; the other attends another week. As indicated earlier, this minimizes attendance of superiors and subordinates at the same seminar. It also reduces the problem created by many key people being away from their offices at the same time.

There is another problem in the selection of seminar participants: what do you do to management personnel who are not included? Unless well-defined and impersonal distinction can be made of the management levels that are included, there will be individuals who feel left out. John Smith is selected for attendance. George White, who thinks he has a comparable job, is left out. Why? Can this be explained to George?

If seminar participation includes all division or assistant division heads and all district engineers, this provides a clear and impersonal basis of selection. If the seminar program is being extended to all supervisory personnel on an established schedule basis, this is likewise clear and impersonal. If George White was not scheduled this year, he will expect to go next year. However, if there is no firm policy for future seminars and George White feels that he has been left out, he and his counterparts throughout the organization will suffer varying degrees of demoralization.

Seminar Methods

Seminars can be—and some have been—organized with the idea of encouraging discussion by participants as much as possible. On the other hand, a considerable portion of the program can be lecture sessions and some seminars have been handled in this manner. Committee work groups can be utilized for evening problem assignments and for periods during the day as well. There is considerable variation in the degree to which this has been done. Reading assignments are sometimes distributed for advance study by participants some weeks before the seminar. Likewise, in many programs, related readings have been assigned as the seminar is in progress. Formal textbooks are used in addition to the AASHO-NHUC "Manual for a Highway Management Seminar." Special reprints and mimeographed material are used by some discussion leaders for advance assignments and also for handouts at seminar sessions.

In attempting to appraise what has been done, it must be recognized that the individuals who are responsible for the program (the discussion leaders, primarily) have developed special techniques and established certain procedures for handling seminars. It would be unrealistic to expect them to change. Furthermore, techniques which one individual uses effectively may not be so effective for another. Finally, it must be recognized that as a seminar program moves ahead in a State, as succeeding seminars are held, and as broader concepts are developed by participants, the character of the program and the methods for handling sessions can be modified.

In this evaluation only a few generalizations are made:

1. Some reading assignments should be sent out well in advance of the seminar. These assignments should include academic treatment of basic management principles and some readings that present practical problems and practices in big business. Much greater progress can be made during the seminar if the participants have done some homework in advance.
2. Reading assignments for evening study are desirable supplements to the seminar sessions.
3. Committee work groups are excellent means for getting extra value from the time outside of the regularly assembled seminar sessions.
4. Active participation in discussions is preferable to lecture sessions.

Discussion Leaders

It is difficult to appraise discussion leaders except to reiterate what has just been mentioned. Leaders who do a minimum of lecturing and a maximum of stimulating discussion are preferable. In any case, discussion leaders should be given advance information about the highway department organization, the highway work program, external relationships, current problems, etc. A good discussion leader will appreciate getting this background information and will make effective use of it in preparing for and conducting seminar sessions.

There is another consideration with regard to discussion leaders. Should only one or two men be used exclusively, or should a greater number be scheduled? Should they be from the academic community? Should industry representatives be used in spots? Is there a place for highway department employees as discussion leaders? Every conceivable type and combination of discussion leaders probably has been used in the State seminars.

Some of the advantages in using a minimum number of discussion leaders for a week-long seminar are the following:

1. Leaders and participants get acquainted. They may be expected to establish a rapport that is conducive to spontaneous discussions.
2. The leaders will make changes in the handling of sessions as the seminar progresses and they learn individual interests and problems.

Some of the disadvantages are the following:

1. There is likely to be less change of pace, less variety in technique.
2. More leaders would bring more viewpoints and greater breadth of concept to the seminar.

The following are some of the advantages of having industry representatives to supplement the academic staff:

1. It provides a practical application of management principles.
2. It contributes a change of pace.

A possible disadvantage of using an industry discussion leader may be the difficulty participants have in relating their problems to the industry presentation. Only rarely should this occur, however, if the industry representative previously has been given adequate background information about the seminar program and the participants.

Using highway department employees as discussion leaders has been successful in some cases but not in others. If participation as a discussion leader is considered a training technique, which it is, and if the department is willing to take a gamble on an occasional dull session, it is quite appropriate to use department employees as discussion leaders. It is suggested, however, that outside leaders carry the major responsibility, at least until some local talent has demonstrated unusual competence.

In any case, one man should be designated conference leader and follow through as coordinator for the entire program. Preferably, this should be a professional in the business of handling conferences.

Seminar Discussion Topics

As would be expected, the general area of discussion topics has been fairly much the same for all seminars. An effort is made to cover the principles of organization, planning and controlling, and communications. The human relations aspects of management are emphasized and appear to be new and stimulating to many of the highway administrators.

The real differences between seminars seem to be in the way the topics are handled. One extreme is to present the principles on a topic-by-topic basis, following the classical process in formal education. The other extreme is to use cases and problems as discussion topics, each providing illustrations of management situations in which certain of the management principles may be involved. The cases, like most management problems, are never susceptible to solution with a simple, single correct answer. The problem technique has been called the "Harvard Case Method" because of its development and intensive use in the graduate school program at Harvard University.

No State seminar appears to have followed the first extreme precisely. Most States have had programs that start from this base but use cases and problems as a supplementary teaching technique. This, in effect, ends up in a combination of the classical technique with varying usage of the case method.

At least one State has conducted a seminar designed almost wholly on the case method technique. This particular seminar program was not considered really satisfactory, but there were some very unfavorable factors, which had nothing to do with the case method, that rendered this neither a reasonable nor conclusive test.

It has been observed, nonetheless, that for short-period seminars such as are involved here, the all-out use of the case method is not desirable. It is a time-consuming process, and it is a frustrating process to the uninitiated. For programs of longer duration or for individuals who have some background in the method, it is, no doubt, an excellent educational and training process.

As indicated earlier, most State seminars have been organized to use cases, problems, or incidents to supplement more formal consideration of topics. For this purpose there is a wide variety of material. There are case problems from industry, some highway department problems, and there are films that present management problems in an effective way. The trick with any of these is to select problems and evoke discussion of them to bring out the management principles. The tendency of participants new to the case technique is to try for the "one answer" rather than to explore and analyze all aspects.

Appraisals by Participants

Some States have had participants make appraisals upon completion of a seminar. After examination of some of the State and National Conference appraisals, it appears that those by participants are a waste of time. The conference leader, who should be involved in the planning of the seminar and be present for all sessions, will be able to make an effective appraisal of the program. It is doubtful that he will find any real value in voluminous recordings of participant post-session attitudes.

If some sessions did not go over, an alert conference leader will know it when the session is going on. He will appraise the situation on the spot much more effectively than will a composite appraisal a week or so later. He will do a lot of informal visiting with participants during the off-hours and will listen in on discussions between participants that are much more revealing than a bundle of evaluation forms.

The seminar program should be appraised, but not formally, by the participants. Appraisal is the responsibility of the conference leader. Furthermore, appraisal should be directed to something deeper than an evaluation of the seminar, per se. It must be directed toward the underlying problems of the department. What has the seminar revealed as to areas of organizational conflict? What are the participants' attitudes about developing people, their (the participants') responsibilties, the department's responsibilities? These and other important management questions should be the subject of evaluation by the conference leader.

Appraisal by Conference Leader

As implied in the preceding paragraph, this is the area in which more emphasis needs to be placed if full value is to be obtained from management seminars. The seminars should be planned with two major purposes in mind:

1. They should be as they generally are, designed to give the participants an opportunity to learn something about the principles and practice of

management and to carry back to their jobs an interest in making some constructive management improvements.

2. They should be, as they generally have not been, organized with the definite concept of developing from them some guide lines for department policy and action in the accomplishment of organized department-wide management improvement activities.

If the seminars are planned with the latter objective in mind, the conference leader (or leaders) must make a continuing appraisal as the seminar is in progress. Subsequently, the conference leader should, on the basis of his appraisal, sit down with top management of the department and map an action program for management improvement. He should be admirably prepared on the basis of seminar discussions to know where improvements are most needed and the degree to which they are recognized by department personnel. This recognition is important, because a management improvement program can be effective only if it has the understanding and support of key personnel.

ACTION PROGRAMS FOR MANAGEMENT IMPROVEMENT

As indicated, one of the purposes of seminars should be to provide the guide lines for continuing management improvement programs. The point was made that this potential of the management seminars has not been adequately appreciated or exploited. However, there are continuing management improvement programs that have been initiated in a number of States. Some have been conscious products of the seminars. Others have been started fairly much as independent efforts.

A canvass of the States and the Bureau of Public Roads was made in fall 1961 for the AASHO-NHUC Highway Management Advisory Committee to see what the individual States were doing in carrying on specific management improvement activities. Over 30 States and the Bureau of Public Roads responded with a huge array of projects. Selected excerpts from the highway department reports were distributed with an AASHO-NHUC "Management News," November 13, 1961.

Even though there is tremendous interest in management improvement and a wide variety of activities being undertaken, there are some rather obvious shortcomings in the efforts of most highway departments.

Top Management Involvement

First, there is the need for top management to set the stage, to demonstrate its vital interest in management improvement activities and to set objectives and policies that define clearly the intent of top management. Many highway departments do not have well-defined objectives and policy manuals which establish clearly the intent of top management.

Top management might well ask itself whether there is a written statement of the objectives and general policies of the organization and whether it is reviewed and revised periodically. Presumably the concept is accepted that the basic objectives of the department are "to develop and operate an adequate highway system." However, policies and objectives need to be defined clearly. They need to be reviewed constantly and to be fully understood by all management personnel.

To illustrate this, about 15 years ago the author was active in developing an in-service training program for graduate engineers in the Connecticut Highway Department. A fair job of recruiting was done and a carefully scheduled two-year program worked out for each recruit, but there was a poor job of defining a department policy. The total problem of developing people was not examined. No cognizance was taken of the development needs of all supervisory and professional personnel. Attention was simply on the new recruits.

It is now generally recognized that a good management improvement program must be geared to providing opportunity for the fullest possible development of all supervisors and potential supervisors. Objectives and policies should be defined accordingly.

Participation

To refer again to the "ill-fated" in-service training program of 15 years ago, there was a further error in establishing the program. It was created and organized almost entirely by the author and the personnel division. However, the individuals to be trained were, of course, assigned to supervisors of divisions in headquarters and to district engineers, each of whom was told what he was supposed to do. In other words, the program was organized dependent on the line-operating personnel to carry it out.

Looking back, it is easy to see that the training program should have been set up in the planning stage with participation by all those who were to be involved. It should have been the supervisors' program, not a headquarters program.

This shows the need for top management to set policies that define responsibilities for developing people and that assure participation by all levels of management in organizing specific programs.

Attitudes

There is a need to change some attitudes to make real progress in management improvement. It has been said that management improvement is all right for big private business, but the task is to get on with building roads and that there has not been time, as yet, to do any work-scheduling. This is not to imply that these are attitudes held by all top administrators in highway departments; however, such remarks are typical of comments that make clear that there are some who still look on management improvement as an extracurricular activity. Policy statements and examples set by the top people in the department are the best ways to change these attitudes. Quite a different, but equally frustrating, attitude is the one that implies there is no opportunity to make management improvements because of the obstacles imposed by civil service or political influences. A less specific but equally defeatist type of expression says simply that highways departments are different.

Actually, much of what might be called the art of management is directed toward overcoming the obstacles or reducing the impact of undesirable influences. In work with highway department representatives, situations have been seen where the civil service agency turned out to be a true help rather than an inflexible obstacle, but it was in response to a clear defining of the needs of the highway departments and an effective presentation to the civil service people. Also observed are situations where political highway commissioners showed a surprising understanding of the political values associated with good business practices.

Such things do not happen with a defeatist attitude. They do happen where the highway department has defined its objectives, established a plan, and then worked with civil service or the politicians to obtain their understanding, support, and assistance.

Reading Management Literature

Most highway engineer-administrators read the construction and engineering trade publications and professional journals. There are few, however, who give equal attention to current writings in the field of management. Yet the latter writings probably have more application to the highway administrator's job than do the technical publications.

There are two highway departments where an effort is made to refer to top administrators selected periodicals dealing with management topics. The response has been quite enthusiastic. Another highway department has started a management newsletter which will circulate interesting and helpful items dealing with management problems.

At the risk of being accused of conducting a membership drive, it seems as a minimum activity in this area every highway department should join the American Management Association and take advantage of the vast amount of material developed and circulated to its membership by that Association. An appropriate individual in the highway department should review the AMA publications as they appear and see that significant material is widely circulated to administrative personnel.

Changes Must Be Managed

One critical need in management improvement is the adequate management of "change." There are numerous instances where highway departments have been reorganized but the results have not reflected the intent. For example, despite the intent in some cases to decentralize authority to the field, all too many decisions are still being made at headquarters. Despite the intent to create a line and staff organization with the staff providing a planning, advisory, and service function, the staff is actually a group of functional-operating heads each holding authority for his individual function. This is not saying a department may not be effectively operated on a centralized functional basis; rather, if it is intended to decentralize authority and create a line and staff operation, where previously authority was centralized, such a change must be carefully managed. It will not just happen because a new organization chart was prepared. It is necessary to supplement the chart with some specific definitions of the function to be delegated to key jobs, to spell out clearly the responsibilities of the key jobs, and to indicate the relationship between them. It is necessary, too, to work with the individuals who are primarily involved in adjusting to the change.

In making this kind of change, there are generally two primary problem areas. One is to get the district engineer to accept and use the authority he is presumed to have been delegated; the other, to get staff men to recognize the potentiality for important contribution in the planning field in contrast with the operating phase of the business.

The need then is to recognize the problems associated with change and to work with the people involved. This is to a considerable extent the problem of the chief administrator. He must define what he wants and guide his organization toward the objective. He may utilize the assistance of an organizational planning unit if he has one. He may engage outside counsel to provide guidance; but, most important, he must recognize that changes must be managed and be prepared to provide the management required. Otherwise, the change intended to effect a management improvement may be an empty gesture—a change on paper but not in fact.

Statistical Data for Management

All highway departments maintain vast amounts of statistical data. In some instances these data are being used for management decisions.

However, there is a vast potential for analyzing and using the statistical data way beyond the current practice. One State, during the past year, used its cost records for all equipment maintenance throughout the State to establish standards for labor requirements for maintaining each class of equipment. Manpower budgets for each repair garage are now set on the basis of the standards. They provide a completely defensible total equipment maintenance budget requirement for the highway department. In this case they promise, also, an early savings of considerable money as adjustments are made to meet the standards. Yet the standards are quite conservative.

In all of the highway maintenance activities there appear to be tremendous potentialities for more use of statistical data analyses. Comparisons of unit costs from year to year and from district to district should be regular control practices. Production standards should be established for comparable work to provide the basis for performance budgeting and control.

The results of different work methods and practices should be analyzed to take advantage of improved practices. This is an area of management improvement in which much needs to be done—and can be done—with effective utilization of data now generally being accumulated.

Development of People

It can be truthfully said that management improvement will take care of itself if an adequate job is done in developing people. In fact, there is substantial evidence that a successful enterprise of any kind will be largely determined by its manpower development program. In the highway field particularly, the accomplishments of a department over a period of time are directly related to the caliber of its personnel, and the caliber of personnel will be a direct reflection of the total development of people effort.

What needs to be done that is not being done to develop people in highway departments begins with a good manpower inventory. Many highway departments have records of employment dates, salary, and civil service changes, but little else. There is need to have a complete record for each manager or potential manager, with his educational and experience record presented in easily-appraisable form. There is need, too, to have an appraisal of his job performance and his potential. Finally, it is necessary to know what the individual should be doing to develop himself on and off the job, and what the department and his supervisor should be doing to assist him.

This is a fairly complete package. It cannot be done all at once, but it provides the goal toward which the inventory should be directed. In any case, the beginning inventory record can be developed very quickly with much of the basic information needed.

In addition to the inventory, the project to develop people should encourage more effective superior-subordinate relationships built around performance appraisals and coaching and counseling by all supervisors. This is the most critical area of the entire development of people activity. It is critical because development occurs primarily on the job, so it is the superior-subordinate job relationship that determines whether the development will be effective or ineffective.

There has been a great deal of study and much writing on how to attain an effective superior-subordinate relationship for appraisal and coaching and counseling. There are two points to emphasize: (a) performance appraising and follow-up interviews should be made a requirement of supervisors and (b) the superior-subordinate appraising and coaching activity should be job-oriented.

There are a variety of related activities that need to be associated with the development of people effort: (a) forecasts of management manpower requirements in the years ahead, (b) replacement schedules to provide assurance that there are competent people to fill positions when they become vacant, (c) lists of promotables from which to develop the replacement schedules, and (d) organized programs for training and development to prepare people for advancement and, equally important, to do a better job where they are. In few State highway departments, if any, are all these things being done now. Some States have made beginnings of an encouraging nature. Many States have shown a recognition of the needs in these areas and soon, no doubt, will be undertaking organized programs.

Although much needs to be done in management manpower development in highway departments, big private industry is also faced with similar problems and finds itself equally challenged. Illustrative of this are some comments by the personnel director of a large industrial organization in discussing the need for developing supervisory employees.

> We know you don't hire managers, you have to make them.
>
> Each of you, during the next ten years, must plan on replacing between 30 percent and 40 percent of the men under your supervision. Some of the jobs that will have to be filled require years of background and experience, which means that replacements must be selected well in advance of the anticipated need.
>
> Too often still, when an unexpected managerial job opens up, we have to sit down, scratch our heads, shuffle personnel cards, and make a curbstone decision between Tom, Dick, and Harry—and then sit back and hope that we've picked the right guy. I think the future will not be too tolerant of industrial organizations which rely on such haphazard methods.
>
> We've got to learn to appraise a man's potential more quickly . . . diversify his experience more widely . . . and test his judgment more thoroughly . . . and well in advance of the need for him. When you need him, it's too late to train him.
>
> What this company is ten years from now is going to depend largely on how good a job we are doing right now in selecting, promoting, and developing our people.

The author does not presume to have made a comprehensive appraisal of all management improvement activities being carried on by State highway departments. From the foregoing, however, it must be apparent that there is a great deal of interest and much effort being directed toward improved management.

In an effort to summarize the activities that have the greatest potential and toward which almost all highway departments could profitably devote increased attention are the following:

1. The conducting of management seminars in the States on a carefully scheduled basis with the ultimate objective of having all supervisors participate.

2. The development of a management guide that will define the functions, responsibilities, authorities, and principal relationships of all key positions.

3. The initiation of a development of people project to include a manpower inventory, performance appraisals, and a training and development program including a planned superior-subordinate coaching and counseling relationship.

4. Planning and scheduling highway and equipment maintenance by making better use of statistical data now available and by introducing

new management methods and establishing performance standards and training programs for maintenance supervisory personnel.

The degree to which these things may be expected to be successfully advanced in any highway department will be dependent, first, on the establishment of clear-cut policies and objectives by top management and, second, on the assignment of competent people to provide guidance in carrying the activities forward.

Technical Institute Training for Highway Engineering Technicians

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In this paper the curricula of approximately 30 junior colleges and technical institutes are analyzed to determine the types of educational programs being offered that would be of benefit to the prospective employees of a State highway department in a subprofessional or technician classification. Curricula are evaluated on the basis of courses included, course contents, and field and laboratory applications to actual practice.

The advantages of junior college and technical institute training to the employee and to the highway department are discussed. The place of an employee with this training in the engineering organization of a State highway department for his initial assignment and his opportunities for a career in highway engineering and eventual advancement from a subprofessional to a professional classification are explored.

Finally, a recommended two-year terminal curriculum is developed for those persons who are specifically interested in going into highway engineering at the subprofessional level.

• AT THE 40th Annual Meeting of the Highway Research Board, the Committee on Education and Training of Highway Engineering Personnel discussed at some length the training and education the growing number of technical institutes in the United States were offering in the field of Civil Technology that would be of particular interest and benefit to the high school graduate who wished to prepare himself for a career in engineering in one of the State highway departments but did not feel that he could take a formal four-year course in Civil Engineering. Of those in attendance only a few had the vaguest idea of what was being offered at any schools other than possibly one in their own immediate localities. It was decided that it would be of interest to the entire group for a survey to be made of all the known technical institutes in the United States and determine the over-all picture of their curriculum offerings as they related to highway engineering.

This was a matter of keen interest to the author not only because of his responsibility for training engineering employees for the State Road Department of Florida but also because of his co-chairmanship of a State advisory committee for technical education for the Florida Department of Education which is rapidly expanding the State junior college system and technical education programs within the junior colleges. Subsequently, M. L. Archer, Kentucky Highway Department; R.

J. Paquette, Georgia Institute of Technology; and L. Csanyi, Iowa State College, volunteered their assistance. Also, L. L. Smith, Assistant Research Engineer, State Road Department of Florida, was pressed into service to assist in analyzing the information.

The first step in making the survey was to secure a list of all technical institutes and junior colleges in the United States. This was obtained from the Florida State Department of Education with the cooperation of the U. S. Department of Education.

The list included 56 schools. A letter was prepared asking each school to furnish a copy of its latest catalogue with course description, if it offered a curriculum in Civil Technology or closely related to Civil Technology. Thirty-eight schools complied with the request and supplied the requested materials. Several others replied saying that their new catalogues would be out shortly and they would send one immediately, though they never did. Because the response was 66 percent, it was thought that this was sufficient to get a representative sampling and no follow-up was made for those who promised catalogues but did not send them.

The distribution of schools by State was California, eight; Florida, six; Oregon, five; North Carolina, four; Wisconsin, three; New York and Colorado, two each; and North Dakota, Indiana, Pennsylvania, Massachusetts, Missouri, Mississippi, Georgia, and Michigan, one each. It was felt that this was good geographic distribution.

Three of the 38 schools had a one-year terminal curriculum and the other 35 offered a two-year terminal curriculum.

The titles of the curricula varied among the schools. In 3 schools they were designated as Pre-Engineering; in 3, Engineering Aide; in 3, Engineering Technology; in 4, Surveying Technology; in 8, Highway Technology; in 1, Construction Technology; and in 16, Civil Technology.

In reviewing the curricula of these schools it must be borne in mind that the only information available was the catalogues. In these the course titles can be misleading and the course description so general that it is difficult to pinpoint what is actually being taught. However, an attempt was made to group the courses in general areas and the following discussion of the curricula is on the basis of this information.

The three schools offering a one-year terminal curriculum were similar in that one required 32 semester hours credit for graduation, the second 33 semester hours, and the third 45 quarter hours, which is equivalent to 30 semester hours. In all three cases, none of the courses was intended to be transferable for college credit.

As to the title of the curriculum, one was called Civil Engineering Aide and the other two Highway Technology.

A further look at the curricula showed that the school with the Engineering Aide curriculum actually required more courses to be taken in the highway field than did either of those designated Highway Technology (Table 1). In the Engineering Aide curriculum 7 semester hours of highway engineering were required but none in the other two. In other subjects, the three schools have relatively equivalent requirements except the one with the Engineering Aide curriculum is low on its requirements in English and physics which allows the 7 hours of highway engineering to be included without increasing the over-all total of required number of hours.

It can be concluded that all three of these schools had as their objectives for these curricula very concentrated training for technicians in-

TABLE 1
CURRICULUM, ONE-YEAR SCHOOLS

Subjects	Engineering Aide (semester hr)	Highway Technology (quarter hr)	Highway Technology (semester hr)
General education	3	0	4
English	2	9	4
Algebra and trigonometry	6	9	4
Physics	2	8	6
Drawing	5	6	4
Plane surveying	2	9	4
Route surveying	3	0	4
Highway engineering	7	0	0
Topographic mapping	0	2	0
Testing and inspection	2	0	2
Construction methods	0	1	1
Total	32	45	33

terested in entering the highway engineering field in either surveying, roadway design, or possibly construction inspection.

A detailed study of the curricula for the other 35 schools, all of which offer two-year terminal programs, reveals again that the title of the curriculum is not in keeping with the types of courses included. On the other hand, the curricula of all 35 do have a number of features in common (Table 2).

For example, 33 of the 35 schools include English as a requirement for a degree. The number of hours required in English varies from a maximum of 10 semester hours to a minimum of 3 semester hours. One school which does not require any English requires 1 semester hour of report writing. Two schools require 6 hours

TABLE 2
CREDIT HOURS,[1] TYPICAL COURSES
TWO-YEAR CURRICULA

School Number	Curriculum Title	English	Report Writing	Mathematics	Chemistry	Physics	Drawing	Surveying	Highway Engineering	Soil Mechanics	Properties of Materials	Testing and Inspection	Construction Methods	Engineering Mechanics	Structural Design
1	Pre-Engr.	3	—	17	10	12	4	3	—	—	3	—	—	3	—
2	Pre-Engr.	6	—	16	8	10	6	—	—	—	—	—	—	3	—
3	Pre-Engr.	6	—	12	10	12	4	6	—	—	2	—	—	3	—
4	Engr. Aide	6	—	20	—	8	9	—	—	—	3	2	—	6	—
5	Engr. Aide	—	1	5	—	3	6	18	—	2	—	—	—	—	2
6	Engr. Tech.	6	—	3	—	8	8	6	—	—	—	—	—	—	—
7	Engr. Tech.	6	—	6	—	—	4	6	—	—	3	—	—	6	—
8	Engr. Tech.	6	—	9	8	8	6	—	—	—	—	—	—	—	5
9	Surv. Tech.	6	3	13	—	3	2	23	—	—	—	—	—	3	—
10	Surv. Tech.	3	—	—	—	6	2	20	—	4	—	—	—	—	—
11	Surv. Tech.	3	—	12	—	—	13	16	—	—	2	—	—	—	—
12	Surv. Tech.	10Q	3Q	10Q	—	12Q	10Q	27Q	3Q	—	2Q	2Q	—	7Q	—
13	Hwy. Tech.	9Q	—	8Q	—	6Q	11Q	12Q	2Q	—	—	—	8Q	7Q	11Q
14	Hwy. Tech.	12Q	—	15Q	—	8Q	9Q	16Q	8Q	3.5Q	—	—	5Q	16Q	9.5Q
15	Hwy. Tech.	6Q	3Q	11Q	—	8Q	8Q	35Q	—	4Q	—	—	—	8Q	4Q
16	Hwy. Tech.	6	—	6	—	4	6	6	6	—	2	3	6	3	4
17	Hwy. Tech.	6Q	3Q	9Q	—	8Q	6Q	23Q	4Q	6Q	—	—	—	6Q	2Q
18	Const. Tech.	10Q	3Q	10Q	10Q	12Q	10Q	9Q	3Q	—	2Q	2Q	3Q	7Q	8Q
19	Civil Tech.	6	—	4	—	3	8	15	—	—	3	—	—	3	—
20	Civil Tech.	5	2	6	3	8	5	7	2	—	2	2	—	5	3
21	Civil Tech.	9Q	—	9Q	—	12Q	21Q	9Q	—	—	—	—	—	6Q	—
22	Civil Tech.	6	3	9	4	8	6	6	4	—	—	—	—	7	—
23	Civil Tech.	6	—	12	4	8	4	5	3	—	—	—	—	7	—
24	Civil Tech.	9Q	—	10Q	10Q	15Q	18Q	7Q	—	—	—	—	—	—	—
25	Civil Tech.	9	—	14	—	6	13	4	—	—	1	—	—	12	14
26	Civil Tech.	6	—	9	—	6	4	6	—	—	—	—	—	—	—
27	Civil Tech.	6	2	10	5	5	11	9	—	—	2	—	—	3	—
28	Civil Tech.	6	3	6	8	8	12	9	3	—	—	—	—	3	—
29	Civil Tech.	6	—	6	—	4	7	11	—	—	—	—	—	3	—
30	Civil Tech.	6Q	3Q	10Q	5Q	16Q	6Q	22Q	3Q	—	3Q	—	4Q	7Q	10Q
31	Civil Tech.	—	—	9	—	15	18	21	—	—	—	—	—	8	21
32	Civil Tech.	6Q	—	11Q	—	8Q	6Q	24Q	3Q	6Q	4Q	1Q	—	8Q	2Q
33	Civil Tech.	6	3	8	—	8	6	3	3	—	—	—	—	6	3
34	Civil Tech.	9Q	—	27Q	—	12Q	9Q	19Q	5Q	—	—	—	2Q	3Q	3Q
35	Civil Tech.	10	2	10	—	12	6	10	2	—	—	6	3	—	—

[1] Q denotes quarter hours; all others are semester hours.

of English plus 3 hours of report writing and one requires 10 semester hours of English plus 2 of report writing. The average amount of English required is 6 semester hours.

In the field of mathematics, 7 of the 35 schools offer a course entitled Technical Mathematics which in general is a review of high school algebra but also goes into college algebra and trigonometry. The number of semester hours required varies from a maximum of 9 to a minimum of 2, with 6 being the most common requirement.

Ten of the two-year schools require from 4 to 10 hours of a course called General College Mathematics, with 8 hours being the predominant requirement. This course is more advanced than the Technical Mathematics course and has an introduction to calculus.

In trigonometry, 17 schools require the almost unanimous 3 semester hours. One school does not require any trigonometry but requires it for admission. Sixteen other schools teach trigonometry through a course in either Technical Mathematics or General College Mathematics.

Five schools require from 3 to 10 semester hours of calculus. One school requires 3 semester hours of differential equations.

The curricula for all schools requiring calculus are made up entirely of courses that are acceptable for college credit or a large number of courses that are transferable for college credit.

The total amount of mathematics required for a degree in the two-year schools surveyed varies from 3 to 20 semester hours. The curricula that include calculus vary in their mathematics requirements from 14 to 18 semester hours.

Thirty-three 2-year schools require physics for an associate degree. The number of hours required varies from 3 to 15 semester hours. The median requirement is 8 semester hours.

Chemistry is required by 12 schools with 3 to 10 semester hours being required for an associate degree. In general, those requiring 8 to 10 semester hours have curricula composed of courses the majority of which are transferable for college credit.

Approximately 50 percent of the 35 two-year curricula surveyed are made up of courses that are transferable for college credit. By curricula titles, one is called Engineering Aide, one Engineering Technician, three Pre-Engineering, three Surveying Technology, one Construction Technology, and the balance Civil Technology.

Engineering drawing is required in all the schools surveyed, and 15 of the two-year schools require descriptive geometry. Fourteen schools require a course in structural drawing. Combining engineering drawing, descriptive geometry, and structural drawing, the total amount of drawing required at all schools varies from 2 to 14 semester hours. The median is 8 semester hours.

Thirty-two schools require from 3 to 21 semester hours of plane surveying and 23 require from 2 to 6 semester hours of route surveying. The median is approximately 6 semester hours of combined plane and route surveying. In many instances both are taught under the course title of Plane Surveying.

Geodetic surveying is taught in 3 two-year schools and photogrammetry in 4. Topographic surveying is required in 17 schools.

The total of semester hours of combined surveying courses required in all two-year schools varies from 3 to 23. The maximum of 23 hours is in a curriculum entitled Surveying Technology and the other three schools surveyed with the same curriculum title require 20, 16, and 18 semester hours. On the other hand, one curriculum designated as Civil Technology requires 21 semester hours and one in Highway Technology requires 24 hours.

Sixteen schools teach highway engineering. No two courses appeared to be the same. They varied from highway construction, highway surveying, highway drafting, to traffic engineering. One was devoted to drainage, and several were primarily route surveying.

Approximately 50 percent of the schools with curricula in Civil Technology had requirements in highway engineering, and 3 of the 7 with curricula designated as Highway Technology had no requirement in highway engineering as a designated course.

Six schools required instruction in geology, 6 soil mechanics, 13 fluid mechanics or hydraulics, 3 water supply and sewage, 5 shops, 14 properties of materials, 7 testing and inspection procedures, and 7 construction methods.

Seventeen of the two-year schools require a course in statics, 15 require strength of materials, 2 require dynamics, and 8 require engineering mechanics. Because there is an overlapping of subject matter taught under these titles in most of the schools it would be more realistic to analyze the offerings of the three courses as one group and designate it as Engineering Mechanics.

On this basis, 27 of the 35 schools offer either single or a combination of courses in the field of engineering mechanics. On the basis of semester hours required, they vary from 3 to 12, with 6 being the possible median.

In the field of structural design 6 schools require courses in structural analysis, 7 in steel and timber design, 7 in reinforced concrete design, and 6 in plane concrete design.

Again, grouping these courses under a general title of Structural Design, 15 of the 35 schools have offerings in this area with the requirement ranging from 2 quarter hours to 21 semester hours with actually no real median being common to the majority of schools.

This discussion has purposely omitted the nontechnical or cultural courses. Most schools require varying amounts of these and they include physical education, health and hygiene, history, humanities, economics, American institutions, etc.

Before closing, it probably would be of interest to point out that the total number of hours on a semester basis required for a degree from the two-year colleges varies from a minimum of 60 to a maximum of 82. In some cases physical education is included in the totals and in others it is excluded. The school requiring 82 semester hours is on a quarter system and the total of quarter hours is 124. To complete the requirements for a degree in Highway Technology at this school in six quarters a student must complete 22 hours per quarter for two quarters and 21½ hours in another quarter, and these totals do not include physical education. This is a difficult schedule for any student. In fact, a student completing this curriculum would have two-thirds of the technical course hours normally required for a four-year college degree.

A graduate of a two-year technical education program such as those just discussed who plans to go into highway work should have a well-rounded subprofessional knowledge of highway engineering. He should be able with a minimum of instruction at the very beginning to direct the work of a surveying party either on location or construction under the general supervision of a professional engineer. (Here a professional engineer is considered an engineer who by experience and education is qualified to direct the work of several engineering subordinates, is in charge of an engineering project of a rather complex nature either on construction or design, and may or may not be a registered professional engineer.) He should be able to handle assignments in a drafting room (either on roadway or on structural design) or in a materials testing laboratory, and as an inspector on construction directing the work of one or more subordinates and under the general supervision of a professional engineer.

The principal objective of such a program should be to teach the student why he does certain things in the way he does them, the possible causes and effects of errors, as well as ways of correcting them, and to a lesser degree the skill and efficiency with which he should be able to carry out his assignments. Skills and efficiency are developed in actual practice and cannot be taught in a two-year curriculum. If a man is given the academic background to understand the fundamental principles involved in the engineering assignments he will get in practice, he will develop proficiency through on-the-job practice.

If the prospective employee is to have a well-rounded training for these subprofessional assignments, he must have instruction in the following subjects:

1. Plane surveying.
2. Route surveying.
3. Algebra.
4. Analytic geometry.
5. Trigonometry.
6. Drawing.
7. Descriptive geometry.
8. Physics.
9. Engineering mechanics.
10. Strength of materials.
11. Properties of materials.
12. Elementary soil mechanics.
13. Highway engineering.
14. Structural drawing.

These are all fundamental courses on which the technician can build his knowledge by actual practice and further study. In addition, such courses as economics, history, and humanities will add to his mental training and over-all knowledge.

English is a must if a technician has any ambition at all. He must be able to organize his thoughts, write using correct grammar, and also communicate verbally. In addition, adequate training in English will add to his ability to read and properly interpret specifications, written instructions, and other documents. There is nothing more discouraging than to have a technically trained employee who cannot effectively communicate with his superiors either verbally or in writing.

From the student's standpoint it is highly desirable for the basic courses, at least, mathematics, surveying, physics, English and possibly engineering mechanics, to be taught at the college level so that they can be used for college credit if desired. The author has known several highway employees who have graduated from a technical institute and then decided to go to an accredited four-year engineering college where none of their technical institute credits were acceptable. There are others who would have liked to have gone to a four-year college but could not afford the six to seven years it would have required, counting the two years they had already spent on a two-year curriculum for which they could not get credit in a four-year college.

A curriculum meeting the stated requirements would at least be similar to the one in Table 3. A person having completed this curriculum would be qualified for a position generally classified in most highway departments as an Engineering Aide III. He ought to feel equally at home on an initial assignment to materials testing and inspection, drafting and computations either in roadway design or structural design; construction inspection; office computations on construction; or surveying either on construction or location.

The question arises as to what the future would have to offer a person

TABLE 3
RECOMMENDED CURRICULUM

Semester One	Hours	Semester Two	Hours
(a) First Year			
English	3	English	3
Algebra	3	Trigonometry	3
American government	3	Physics	4
Engineering drawing	3	Descriptive geometry	3
Chemistry	4	Economics	3
Total	16		17
(b) Second Year			
Report writing	3	Soil mechanics	3
Analytic geometry	4	Route surveying	3
Physics	4	Strength of materials	4
Plane surveying	3	Highway engineering	3
Engineering mechanics	3	Structural drawing	2
		Properties of materials	2
Total	17		17

in a highway department with this type of training. Under present classification systems in some highway departments, he would be stymied after two or three years. In others, he could advance to possibly a resident engineer position on construction or a district materials engineer position and may be a squad leader's position in a drafting room in a reasonable time.

The growth of the number of schools offering a two-year terminal curriculum in Civil Technology or its equivalent is rapid, and it would be to the advantage of all highway departments to establish career ladders for technicians (separate and distinct from those for the engineer) that would lead to a position requiring registration as a professional engineer. At this point a technician should be able to attain an engineer title on the assumption that as a registered professional engineer it is immaterial whether his background includes a degree from a four-year accredited engineering college or a two-year accredited technical institute or junior college.

As a technician advances up the technician career ladder he would receive compensation in some instances comparable to an engineer classification. However, as a general rule he would be a specialist—his administrative responsibilities would not be as great as those of a person in a financially comparable engineer position but his responsibility for the technical adequacy of this work would be equivalent to or even greater than that of the engineer.

If the suggested model curriculum for a two-year program is compared with the 35 programs surveyed, only 21 offer courses in structural drawing or some type of structural design that undoubtedly includes structural drawing. Therefore, on this score 14 would not comply with the model curriculum, and in line with the philosophy expressed in this paper these schools are not particularly concerned with training technicians who might be interested in working in the structural design section of a highway department.

Twenty-nine schools do not offer soil mechanics as a separate course, although some may have a minimum amount of instruction in this subject included in some other course. Twenty-one do not offer a course designated as Properties of Materials. It appeared that the training of technicians for materials and testing is not a primary objective of a majority of the schools surveyed, but here again, some of the subject matter normally taught under this title may be included in another course or other courses. Construction methods is not included in 28 curricula.

Turning now to what might be termed the basic courses in Civil Engineering, three junior colleges, all in Florida, do not include surveying in their curriculum. One which has a program in surveying technology does not offer any course in mathematics. Eight do not offer instruction in engineering mechanics. One school does not require any English or report writing.

From this survey it must be concluded that approximately 20 of the 35 two-year curricula are designated to train technicians who are interested in going into either highway construction, roadway design, materials testing and inspection, or structural design. The use of the word "approximately" is necessary because the survey is based on course titles and catalogue descriptions which are inadequate for a detailed analysis. Approximately 26 have curricula designed to train a person for a technician's position in a highway department either on construction, location, or roadway design.

Unfortunately, the number of graduates estimated for each of the curricula annually was not available for this study. However, it can be seen that the technical institutes and junior colleges are a source of supply for technicians which are badly needed by most highway departments. The number of schools and the number of students are both increasing rapidly so that there will apparently by a substantial supply of academically trained technicians for highway work in the not too distant future. As this day approaches some highway departments may be able to curtail or terminate some of the training that they are now having to do with their own personnel. Also, it will relieve the professional engineers of some of their routine tasks and make them truly professional engineers.

Fiscal Management and Control—A Symposium

I. The Place of Financial Management in the State Highway Department

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• THE PLACE of financial administration and of the financial manager in the modern state highway department is far different from that implicitly suggested by one state highway official in the 1920's who said, "The finance man is all right. I do wish, however, that he would stick to his bookkeeping and stay out of the engineers' hair while they build and maintain highways." This view of the fiscal management job as that of a bookkeeper was somewhat antiquated in the 1920's; it would be far more archaic if held today. One would err (a) to infer that the accounts can be fully meaningful in the absence of adequate reports from those persons who "build and maintain highways"; (b) to assume that records are kept primarily for someone other than the people who see to the construction and upkeep of highways; and (c) to conclude that the only function of fiscal management—or even a principal one—is bookkeeping. What then is the place of fiscal administration and of the finance officer in a state highway department?

The present report seeks to answer this query in two distinct ways. The first part inquires into the basic fiscal administration assignment and the position of the principal finance officer among other administrators. In search of satisfactory answers, business experience generally, and highway administration experience in particular, contribute.

The second part of the report seeks an answer in terms of the interrelationships involved in financial administration as projected in the literature and as practiced in the states. The principal relationships referred to are those tying financial management to highway planning, however administered; those providing the nexus with construction-oriented and maintenance personnel; those assisting top management; and those relating highway finances to the total finances of the state and to state (as distinguished from departmental) policy. The analysis in the second part, although it includes those principal relationships that are the sole concern of the highway department, omits some highly significant finance administration interconnections.

From these two approaches one may secure sufficient evidence to justify a firm hypothesis as to the place of fiscal management in total state highway administration. The analysis may also tell something of the role of the highway finance administrator in the operating department of the future.

FINANCE ADMINISTRATION IN PRACTICE

Administrative arrangements in business and changes in them may suggest analogies with those in state highway departments. The parallel seems especially close in the instance of businesses in which engineering functions bulk large.

The Business Analogy

In business circles there has been recent pointed comment (*1*) on the choice of finance specialists as chief executive of each of several large, engineering-type corporations. For instance, Gerald L. Phillippe was named president of General Electric Company; Lynn A. Townsend, of Chrysler Corporation; Frederick G. Donner, chairman and chief executive officer of General Motors; and Ernest R. Breech, former board chairman of Ford Motor Company, board chairman of Trans World Air Lines. Developments such as these raise question as to what in the finance man's background renders him especially eligible for such preferment.

Recently, some careful statistical studies (*2-6*) of corporations generally indicate (a) that, despite the comparatively few new recruits in finance, more corporate executives have begun their work in that field than in any other (*3*) and (b) that corporate top executives are increasingly drawn from the financial management field (*2, 7*). Each of these phases of the situation needs to be examined briefly for its analogy with the highway department placement of financial management.

A recent comprehensive survey (*5*) shows a great deal about nearly 2,800 finance officers of corporations in the United States and Canada. Of those identified by line of work 1,471 were in manufacturing; 158 in banking, finance, and insurance; 197 in public service businesses; 180 in construction, extractive, and related refining; 183 in distribution; and 72 in all other fields reported. Seventy-one percent were employed by the corporation board of directors rather than by a corporate executive, though over half reported directly to the president or board chairman and a fourth to the board itself. Typically, accounting, office management, corporate management planning (forecasts, long-range planning, budgeting), property administration, insurance, contract negotiations (but not ordinary purchasing), and often aspects of traffic management are within the purview of the fiscal officer. The finance official in over 80 percent of the cases serves on a long-range planning or kindred committee. It is significant that 70 percent of the finance officers studied are college graduates and that 40 percent of these have completed one or more years of graduate study. Of those who did graduate work, two-thirds completed at least two years.

There are numerous lines of evidence indicating that the corporate finance men's positions in many, perhaps most, of the sizable corporations have been strengthened in recent years. An examination of one group of such personnel shows, for instance, that from 1956 to 1960 the number of corporations assigning finance officers vice presidential rank advanced from 74 to 186 (*2*). Of the 260 corporations Stiller (*6*) studied, 228 had by 1960 chief fiscal people who held positions as corporate officials other than the remaining 32 "controllers." Perhaps the most convincing indication of the altered position of finance management is the recent emphatic tendency toward corporate promotion of finance people to chief executive officer (for example, Browne (*2*) shows that, although only 75 members of the Controller's Institute had been so promoted ten years ago, 235 held positions as chief executive in 1960) and the frequent board of directors' insistence on personnel experienced in finance as a prerequisite for consideration as such chief executive (*7*).

The widespread formal recognition of finance personnel in corporation practice and especially the recent upsurge in the extent of their acceptance pose important questions. The explanation of the emphasis on the fiscal aspect of administration is apparent in fields like banking and insurance; it is not so apparent in the case of enterprises manufacturing appliances and electronic equipment, in public utilities, and in corporate producers of transportation equipment—all fields in which engineering functions bulk large. One possible explanation lies in the emphasis since the mid-1920's on the management functions of corporate finance personnel (*8-10*) which emerged in the 1950's into the pattern that has been suggested already. This stress on management accompanied corporate executive emphasis on the interpretation of the administrative process as a unified whole, whether directly involving production, marketing, personnel, or finance. The interpretation and its administrative consequences require the distinctly management functions of finance in all aspects of the individual corporation's activity. Cost control is important in Ford Motor Company engineering, production, and marketing; thus, throughout all these and other aspects of the corporate work, financial management skill is essential.

Corporate development of designs for the future following the 1920's meant new stress on long-term planning. There are, in one sense, two facets of the farther look ahead. One involves the economic functions of the corporation. An industrial or marketing enterprise plans its future products and the methods of selling them (*10-12*). Working out such programs requires the participation of finance administration personnel. More particularly, as the cost of each undertaking must be met, execution of the plan requires even stronger accent on fiscal management.

The other form of this long-range planning development expresses itself in the enlarged use of budgeting as a sophisticated vehicle for administration (*13-15*). The essence of budgeting is the forward outlook and the controlled execution. For example, when a corporation considers acquisition of the equipment necessary to turn out a new product, the action inevitably involves not only a view toward the future in terms of the productive processing and the marketing of the product but also a cooperative examination of the possibility and the alternative means of financing. For instance, Miller (*16*) shows that of 127 "well-managed" corporations all but 6 indicate that they use specified formal and established budget tests in planning capital investment. About one-half use two or more such tests. A large minority of the concerns employ highly sophisticated budget techniques.

The typical finance man in current corporate management occupies a position second only to the chief executive. In some cases, the function is of such moment that the latter performs it himself. And the dignity of fiscal administration is decidedly enhanced in recent years. The trend may perhaps be explained by the enlarged place of forward planning and by new emphasis on efficient management as illustrated by cost reduction programing.

Financial Management in Highway Departments

The place of financial administration in state highway departments is roughly parallel to that in private business except that (a) there are adaptations to their public character, such as the service objective, that in a degree displace the profit-making functions of the business enterprise, (b) the differences among the various highway departments seem to be greater than those among corporations, partly because of the former's political relationships, and (c) there appears to be a greater proportion of highway departments in which management emphasis lags.

There is a corporate and highway department analogy in three ways: (a) long-term departmental programing based on highway needs studies (*17*) is functionally much the same as corporate product and market planning; (b) although construction and current budgetary practices in state highway departments differ procedurally and in some degree functionally from corporate counterparts, the parallel is certainly close (*18*, *19*); and (c) generalization about similarities in the use of budgeting in highway and in corporate management is also broadly applicable to accounting, internal auditing, purchasing, property control, and debt management (*20*).

Trends.—Although highway needs studies have been made in the states for many years, they have become the widespread basis for program formulation in the present-day sense only recently. One may perhaps assume that the increasing stress on such a long-term program is related to the growth of planning and of budgeting staffs—or, more broadly, to the currently increased sophistication in highway management. Even now, maximum employment of needs studies for policy definition is apparently a minority practice among the states; *e.g.*, in Tennessee (*21*).

The comprehensive use of modern program budgeting in state highway administration was extremely rare prior to the close of World War II. With growing administration know-how, signaled by the increasing recognition of fiscal techniques, postwar financial management has matured reasonably up-to-date budgeting apparatus in a number of public works agencies. This maturity appears to be reflected in the trend toward the integration of fiscal management in many state highway departments (*22*). By 1959 at least 21 state departments had an organization that encouraged integrated financial management equivalent to that employed under the typical vice-president finance in the private corporation of the 1960's.

Present Situation.—The policy as to fiscal functions and its actual execution are far more meaningful than the structural arrangements. Conference and correspondence with the top officials of about half the state highway departments indicate that a large proportion regard budget-oriented functions as the most critical financial management tasks of the department. Eighteen of the 21 departments giving categorical information regard such issues as being of first importance. Even though some of these 18 departments clearly do not recognize the full potential of budget management for over-all administration, the extent to which the department heads do envision their opportunity presages future emphasis on modern program budgeting. Even so, other state highway executives seem to assume that the budget task is finished when the year's plan is approved.

As to other fiscal functions, each state seems to distribute emphasis differently. There is heavy stress on accounting, which in some cases is apparently not completely distinguished from budgeting. In rating financial management functions, highway department heads assign an important place to procurement, property control, and related functions. A number of states accord marked emphasis to debt and treasury management even though in many states these functions are administered outside the department of highways. It appears that in most states insurance problems (fidelity, workmen's compensation, fire, and tornado) are regarded as relatively minor or are looked after by some agency outside the highway department.

Certain state highway administrators emphasize the interdependence among fiscal functions and the interrelationships of these with engineering and other operating activities. These complexities suggest to a number of correspondents the wisdom of integrated responsibility under a single subordinate for exercising or supervising financial management. It appears that 16 of the 22 departmental executive correspondents accord general fiscal management, subject to policy direction, to one principal financial official. Some of them include management utilization of planning results for physical-financial construction program planning or execution or both. Fringe subject matter such as public relations and personnel functions are sometimes lodged with finance but more frequently not. One correspondent indicates that fiscal management is subordinate to, rather than coordinate with, engineering management.

Highway executives report surprising satisfaction with diverse financial management structures. The three who would be disposed to make considerable changes, if beginning anew, each independently concurred in desiring a financial management comprehending all fiscal functions and reporting directly to the department head.

There appears to be a considerable area of agreement among officials who express a view to the effect that state highway departments should lodge electronic data processing in the hands of the finance officer. It was reported in 1959 that in most cases computer centers were independent of technical engineering and finance divisions of the departments (*23*). In Kentucky, where such equipment is used mainly for accounting and closely-related purposes, this viewpoint is to be expected. The conclusion is more interesting in Illinois where engineering use of the computer, relatively speaking, bulks larger.

Many progressive state highway departments have assigned major management responsibility to fiscal administration personnel, subject to policy supervision, but the extent of the assignment falls far short of the practice in private corporations. The prevailing policies and trends suggest a far larger place for professional management people in the future highway department—a position not independent of, or superior to, planning or engineering, but closely integrated with and helpful to such functional administration. The widespread state highway department development of program budgeting and the top-level recognition of opportunities for management effectiveness in this direction could greatly accelerate the current trend.

FINANCIAL ADMINISTRATION INTERRELATIONSHIPS

One of the major problems of highway administration is that of producing and maintaining a general awareness throughout the ranks of supervisory personnel of the operating assistance that financial management can provide. (Prerequisite, of course, is the capacity to make such aid available.) Similarly, there is necessity for pervasive understanding that the correctness and serviceability of records depend on accurate and punctual reports from engineering and other operating personnel. This complex of issues is merely a special case of the fundamental truth that a highway department is a unified operation in which each basis for conclusion typically depends on performance organizationally and geographically separated from the particular action, report, or decision (*18, 24, 25*).

The truism that a state highway department, in order to be effective, must be operated as a unified whole sometimes presents a real departmental problem. So a careful examination of interrelationships within the agency, especially between financial and engineering groups, seems essential to understanding the place of financial management in highway administration. This section will be devoted to such an analysis, using budgeting as a particular case. (Revenue earmarking for highways is assumed in the case analysis. For states whose highway programs are supported by periodic appropriations slight adaptations are essential.) Budget management is chosen as the vehicle for presentation of the basic idea not because it is generally typical but because the interrelationships are easier to see than are those affecting procurement, warehousing, debt administration, property control, etc. Stress on the pervasive interdependence of all fiscal management with other departmental activities, as in the case of budgeting, is appropriate.

For the purpose of steering a straight course, a clear definition of the basic concept of budgeting is important. Informally put, the following statement represents an approximate consensus among finance and other management-oriented students. Highway department budgeting involves the development of a documented work program (including a priority-based list of construction projects), the careful linkage of that program with a formal comprehensive financial plan, and the carrying out of the entire scheme through an administrative arrangement which includes a definite work schedule. Thus, budgeting embodies both maintenance and construction-oriented planning (in the latter case, planning for location, design, right-of-way work, and construction proper) as well as financial planning. It includes plan execution as well as plan formulation (*26*, p. 2). Some highway departments have aspects of budgeting without capitalizing on the entire process. It is usual in such agencies to refer to whatever elements of the budgetary process the department has as its budgeting operation. More precise language would reserve the term "budgeting" for the comprehensive plan, including apparatus for submission, approval, and execution as well as for the preparation of the plan. For many purposes it is important to distinguish the "capital" or "construction" budget from the "current budget." The former characteristically involves advance planning for several years. The latter, which includes the first year of the capital budget as well as the program for all maintenance and other current activities, is equally based on a program conception. For the objectives of the present discussion, "budgeting" obviously refers to both capital and current aspects.

In this section, the comprehensive concept of budgeting or the budgetary process is quite freely assumed even though some elements are missing from most of the state highway departments. (Thus, the interrelationships analyzed in the rest of this paper are only in part those that actually exist in any one state department.) A kindred concept of other fiscal functions is also assumed —oddly enough, with less sweeping departures from existing practice.

Interrelationships in Planning

Planning provides the basic foundation on which budgeting rests even though the budgetary process itself does not include all phases. For example, sound management appears to require a long-range definition of objectives such as may well emerge from a comprehensive highway needs study. The focus of such a determination of purposes may include a period of 15 to 30 years (*27*). In keeping with the specific goals set up officially, the budgetary process involves the preparation, submission, approval, and execution of a priority-based program concerned with construction, operation, and maintenance of the highway system. Thus, the budget is a product, as well as a servant, of management activity looking toward implementing the long-term plan.

Highway administration circles understand the planning function sufficiently that one need make only general reference to the necessity that the work can be carried out with active collaboration from all branches of the engineering staff. For instance, economy studies to lay the groundwork for alternative route selection could scarcely be successful without the investigations of real estate costs that the regular right-of-way staff people prepare for the purpose.

Both engineering and planning contributions are essential to the largely general management decisions that the financial administrator reflects in the actual budget. Indeed, program details of performance regarding each engineering function are likely to be directly or through the planning staff the joint product of the financial and the particular engineering management concerned. Thus, finance, planning, and maintenance directors may jointly see to fixing the program—subject, of course, to official general approval. In such a three-way conference, for example, the finance personnel can provide from the record the unit cost data classified by maintenance activity for each class of highway (*28,* pp. 27-29). Planning staff members can supply information regarding physical and traffic interrelationships with other departmental functions. And maintenance people, of course, in view of these kinds of intelligence coupled with information from their own files can make basic maintenance budget estimates. The necessity for collaboration is summarized effectively in Smithies (*19*). The military poses many of the same problems as a highway agency (see Mosher (*29*), especially pp. 57-70).

Although the position of the planning staff is a dominant one in aiding toward decision as to estimates, operating divisions and top management of the highway department depend more heavily on the financial administration for budget-implementation counsel. This whole situation will be clarified later.

As in private business, the effective use of planning results contributes heavily to both construction and maintenance director decisions conforming with departmental policy. To put the point differently: The planning staff and fiscal management provide, in the light of prior cooperation from directors of engineering operations, significant data and analyses necessary for wise budget estimates by those directors and their top supervision. In the use of planning results, as in the development of planning data, the department functions as a single unit. In a large establishment, such unity is feasible only with a planned program uniformly interpreted among persons concerned.

Interrelationships in Budget Administration

One facet of the fiscal management relationship with other branches of the highway department has emerged from the discussion of the employment of the planning results to make budget estimates. As already implied, the formulation of the construction budget is a collaborative process between planning and financial management staffs based largely on decisions by top management and by the engineering line administrators (*29*). The submission for final approval, whether to the head of the department, the governor, or the legislature, as the law requires in a particular state, may result in program adaptations or alterations which also necessitate similar cooperation. Highway construction budgeting in Great Britain (*30*, pp. 202-205 and Ch. 9) suggests kindred interrelationships.

On the face of the situation, the top departmental officials and the engineering administrators must base certain construction decisions almost exclusively on information and analysis from the financial management (*24*, *26*). The whole truth is more complex. For example, the integrity of the data regarding progress on particular projects, whether in administrative, physical, or financial respects, is completely dependent on engineering and other field reports. This fact usually presents little or no difficulty with respect to contract performance; but, in some states, the situation as to force account engineering, right-of-way, and construction activity, and maintenance work is such that the finance administration reports based on project data from the field staff falls short of desirable standards of accuracy. Reports based on field-provided information, whether as to work results or costs, cannot be more precise than the basic data in the reports. This problem has been inadequately reported in the literature; but in practice serious weaknesses have been detected, for example, in the allocation of engineering and right-of-way staff time among projects.

Assuming conditions of good reporting are satisfactorily met, it is important to sketch the position of financial management in the process of construction budget execution. As this function is largely meshed with current program administration, however, the analysis may be deferred momentarily. There is a basic difference in that construction reports must show project financial and action data as well as over-all information similar to that reported for operation and maintenance.

Procedure in different departments in preparing current budget estimates varies widely. Thus, no one assumption as to the formalities can have general validity. For the purpose of showing the position of financial management in the process, however, the following reasonably typical but abbreviated steps in this process in agencies that do program budgeting can be used.

1. The department head and the finance chief confer to assure that the latter adopts a budget-estimates preparation outlook consistent with departmental policy.
2. The finance chief meets with all line administrators who participate in program definition, whether in the engineering staff or not, for the purpose of explaining forms and procedures incident to program summaries and estimating tabulations.
3. The department head, in the light of budget staff work, including revenue estimates, meets with all these administrators and the entire professional budget staff to discuss policy, including any issues of program balance or financial limitations that he finds desirable or necessary. (In strong-governor states this conference is preceded by appropriate discussions with the state's chief executive.)
4. Continually aided individually by budget and planning staffs, the maintenance, right-of-way, design, construction, and other administrators prepare work program summaries and estimates that are final when legally approved.
5. Financial management reviews work program summaries and estimates for consistency with prescribed form and with policy and formulates

the budget document for official approval. (In any instance in which subagency documentation is incorrect or inadequate, the budget staff seeks with the administrator concerned to eliminate the imperfections discovered. When the estimates are ready for approval, they are submitted to the department head with budget staff recommendations based on prior interpretation of policy.)

6. The department head approves or secures the executive or legislative approval established by law as necessary to make the budget official. However, if a department is headed by a commission, rather than an individual executive, the members participate. (Of course, in the event of unresolved disagreements between budget staff and a chief administrator as to the interpretation of policy, it is necessary that financial management report the facts and arrange for conference so that the department head may have full explanation and render a decision to be incorporated in the budget document before it is placed in form for final approval. If the background discussions among the planning staff and all administrators and the earlier steps just specified have been successful, such disagreements should be rare after comprehensive program budgeting has been in use for a while.)

Following budget approval, the procedures again depend on accounting and other formal arrangements. A usual practice is that the budget staff, in keeping with top-level administrative directives, makes allotments (that is, makes appropriated money available in keeping with the accepted work program) on a quarterly basis. It then becomes the responsibility of each operating administrator, aided as desired by the budget staff, to execute the share of the total expenditure plan represented by the program for his branch of the agency.

The budget staff assists the top-level management and the heads of operating units in numerous ways to make each dollar available go as far as possible. One of the most obvious services is monthly analysis of financial reports in the light of program development to show the heads of traffic, maintenance, and other administrative units exactly where their expenditure and work programs stand as compared with the budget estimates. In the case of construction-oriented units, this written and oral analysis extends to individual projects. As for construction budgeting, see Martin (*18*) especially pp. 11-14; for total budgeting generally, see Elkins (*31*), Mosher (*29*), Royal Institute of Public Administration (*30*), and Smithies (*19*); for certain dimensions not fully developed in any of these works, see Martin and Cush (*32*, Ch. 5). In the instance of current activities, policy on this score depends on local procedures. In program budgeting in all cases, the relationship involves budget staff aid to department head, chief engineer, design administrator, or other supervisor in utilization of budget information or other professional findings to achieve the most effective and economical management of his sphere of responsibility. Thus, in program execution, the budget staff is management consultant to line administrators. The necessity for this was recently stated in one context as follows (*33*):

> The engineer's formal preparation for making capital investment decisions consists of a course in engineering economy. . . . As taught to the engineer, engineering economy is usually a one-semester course based on economics, cost accounting, simple mathematics, statistics, interest rate theory and a heavy dose of questionable depreciation theory. This is only a part of the subject matter that the average management student absorbs in four years, yet the engineer is expected to master the field in one semester.

This comment on one phase of management applies to other facets. Incidentally, the management people actually employed in program budget staffs more and more are individuals who after four years of college have had one to four years of graduate study. The mounting level of professional management education is consistent with complexity of demands of state budget administration.

Interrelationships in Other Phases of Financial Management

Carl Fritts (*34*, p. 25) of the Automotive Safety Foundation has pointed out the necessity of balancing departmental emphasis on "business functions" (including budgeting, accounting and auditing, records, purchasing, personnel management, property control, office operation, and in some states planning and aspects of treasury and debt administration) with that on "engineering functions." Some other highway administration specialists would associate with "business functions" the more general administrative management facilities necessary to a well-directed state highway department, including planning, public relations, and miscellaneous service facilities such as multilithing, blue printing, and even map making (if the last is not separately provided by planning) (*35*, pp. 156-157). Numerous other reports and surveys in the past dozen years would seek kindred, but usually less comprehensive, integration of the "general administrative" machinery.

But, whether the "business functions" are or are not organizationally tied in with other general administrative tasks, it is essential to efficiency in highway administration that the structural arrangement imply no separateness in the actual conduct of business. Indeed, a condition of effective over-all operation seems to be a close functional tie between engineering and finance management. The business function personnel may administer property control; if so, the purpose is to contribute to highway maintenance and construction. It may provide for procurement. In some states materials and equipment purchasing, as well as contract procurement, are handled by the highway department; in other states by a general procurement agency—wholly or in part. In the latter instance "provide for procurement" typically means merely see that purchase requisitions are in order, that proper records are made, and that requisitions are promptly forwarded to the state's procurement staff. There is also follow-up activity. If procurement is provided, the equipment and other things purchased are mainly to build or maintain roads and streets. Thus, the "business functions" are separately administered in close cooperation with all the other branches of the department. They are split off in order economically to secure needed business skills and to free highway technicians, especially engineers, of the diversion from their own work.

As in budgeting, but according to techniques characteristic of each function, finance management is successful only as it is integrated with, and helps, technical highway manpower. No highway department exists to keep accounts or to manage debts or to procure commodities or contracts. On the contrary, these activities are carried out in highway administration agencies to facilitate road and street operation, maintenance, and construction. Conditions of departmental success include (a) financial management geared in attitude and performance to the helping function, (b) provision of accurate reports from the operating front that will enable fiscal personnel to function with integrity, and (c) maintenance of an atmosphere of mutual respect and cooperation between persons performing "business functions" and those carrying on "engineering functions."

CONCLUSIONS

Several conclusions from the analysis seem to be clear:

1. Budgeting, accounting, internal auditing, other records, procurement, property administration, general office operation, and, to the extent the functions are not allocated by law to a different agency, treasury and debt administration are highway department activities that involve financial management skills in varying degrees. Finance administrators also collaborate with outside auditors. Budgeting and some of the other enumerated functions utilize the most general professional management abilities. In addition, personnel, planning, aspects of public relations, and some other functions of the typical state highway departments challenge administrative skills and present fiscal angles.

2. There is a strong tendency—following the even more developed practice of private business—toward integration of all or most of these functions under one individual reporting directly to the department head. This design may or may not include personnel, planning, and kindred tasks, depending on departmental policy as to the orientation of these activities. The indicated trend toward substantial management integration, for the most part, is supported by prescriptive management studies.

3. If any of the functions referred to in Conclusion 1 are administered outside the supervision of the head of the "business functions" generally, they are more and more generally managed by individuals coordinate with the chief fiscal officer and with the chief engineer to the extent that they are not lodged directly in the office of the departmental executive.

4. There is a pronounced movement—again slower than in private business—to recognize the functions enumerated in Conclusion 1 as distinctly management assignments and to man them largely with people professionally trained in that area.

5. The personnel engaged in the performance of "business functions," especially individuals having management responsibility, must recognize their tasks as involving facilitating activity, not as turning out end products. Their assignments are important to the extent that they cooperatively help toward the performance of the "engineering functions."

6. As a corollary of Conclusion 5, one measure of the success of financial management in a particular state highway department is the extent to which the engineering staff capitalizes on consultation with fiscal management personnel in much the same sense that the nontechnical department head makes use of engineering skills.

7. The generalizations in Conclusions 1-6, in the absence of unanticipated developments in administration, are more likely to be fully applicable to the state highway department of the future than to that of today.

ACKNOWLEDGMENTS

The author has profited from a critical reading of this paper in an earlier draft by E. H. Holmes and F. C. Turner, Bureau of Public Roads; Wendell Beals, Utha Conrad, Ken Cook, Betty Rae King, Joe Massie, and Ralph Pickett of the University of Kentucky; W. F. Babcock, North Carolina State Highway Commission; Dwight H. Bray and Leslie Dawson, Kentucky Department of Highways; W. L. Haas, Highway Management Consultant; Eugene C.

Holshouser, University of Georgia; and Charles M. Noble, Consulting Engineer. The author is especially indebted to the highway department executives throughout the country who have written observations on their own fiscal management.

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II. A Modern Look at Financial Administration in State Highway Departments

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• THIS paper attempts to stimulate thinking in the area of financial administration in State highway departments by discussing the interrelationships of that area with over-all State government, the U. S. Bureau of Public Roads, and other aspects of management within a highway department.

Specific examples are cited based on both industrial and State highway department experiences to illustrate and emphasize various points, particularly in areas of the need for another look at the training required of those placed in positions discharging day-to-day heavy management responsibilities. The value to over-all management of coordinated efforts of all aspects of management and the part financial administration might play in the management of a modern highway department are also illustrated. Reference is made to the approaches in certain specific areas that highway departments' industrial counterparts have effectively used in an effort to stimulate thinking on how accounting systems might be developed to generate reports to assist management at all levels in the discharge of its responsibilities.

As a point of departure and certainly as an oversimplification, sound financial management in any business, including the highway department, depends on three fundamental processes: planning, organizing, and executing. Planning is setting objectives, forecasting future conditions, and determining the future course of action and policies required to attain objectives in the light of forecasts. Organizing is determining and arranging the resources of materials, manpower, and money by function and in relation to the whole to meet the planned objectives. Executing is the carrying out of the approved plan to attain desired objectives. It involves the determination of actual results as compared with predicted performance.

PLANNING

Planning is no different in the highway business than in private business. It involves the establishment of an over-all long-range objective that can logically be translated into a short-range program resulting in a comprehensive plan of operation for the department. It includes, first of all, a determination of the level of automotive transportation service to be provided over a 15- or 20-yr period, with appropriate forecasting of future population and traffic demands to assess probable future maintenance and construction needs. Unless these long-range objectives can be set forth, it is difficult to organize for and logically execute the operation. Too often, the setting of this long-range goal is the basic financial management stumbling block.

Historically, many highway departments have operated merely by making use of the highway funds made available annually or biannually by the State legislature. Too often the amounts of these funds have been based on historical structures of taxation without a realistic appraisal by the highway department or the legislature of the desirable levels of service to be provided or the funds necessary to maintain a reasonably adequate highway system. It might be concluded that this fixed taxation sets a series of goals of achievement for the highway agency. Actually, the reverse takes place in that the goal is merely the expenditure of these funds without a realistic appraisal as to whether the funds being spent are providing too high or too low a level of service consistent with the probable future needs.

The Interstate System, although having passed through several stormy years, represents the opposite approach—that of designating a goal to be achieved and then providing funds to permit its attainment. Admittedly, this great system of highways will solve only a part of the over-all highway problem. If the total highway transportation problem is to be solved, a similar approach is needed within the State highway departments for the primary highways and, of equal importance, the secondary roads and the urban system.

As an illustration of what steps might be taken so that highway departments may achieve this over-all long-range objective, some of the actions taken by the North Carolina State Highway Commission may be of interest. It is important, however, to understand that in North Carolina the highway commission has responsibility for over 70,000 mi of roads, including all of the primary roads, all of the secondary roads, and all of the major thoroughfares in urban areas. Thus, in this State it may be easier to work towards a long-range goal for all highway systems than in the majority of the States where highway activities are under many separate governmental units.

As a first step, detailed studies were made to determine what might be considered the appropriate levels of service to be provided on all highway systems for the future. This first assumption of future levels of service to be provided sets the long-range objective or goal. With the goal established, the regular population and traffic projections were made for all primary, secondary, and urban thoroughfares. Cooperative thoroughfares were developed in all major urban areas. Relating future traffic projections to the primary, secondary, and urban system as it related to the capacity of the existing systems as defined by the level of service, it was then possible to determine the future needs.

As a result of this study, it was possible to estimate the cost of thousands of improvements and to establish a period when such improvements might be reasonably needed. These needs when related to probable future revenue indicated that a minimum of approximately $500,000,000 in additional funds would be needed over the next 15 years to provide a reasonable level of service.

This report, given rather wide publicity and made available to the Governor and the legislature, indicated that North Carolina was indeed falling behind in coping with its highway transportation problem. Taking cognizance of this fact, the Governor and the members of the last legislature took positive steps to provide more funds for highway operations, which is closing the gap between highway needs and highway revenues.

Admittedly this long-range plan is not unique and many highway departments have been developing such plans. However, it is considered fundamental that any logical fiscal management depends on such a plan in considerable detail as an over-all objective. Without it, legislative bodies have no basis for evaluating highway needs as they relate to over-all governmental needs. Without it there is no equitable basis for determining the appropriate distribution of the total tax dollar.

Without such an objective, highway management tends to be lackadaisical at all levels of State government. As an aside, such a plan brings home forcefully to the people that the highway department is not necessarily "the rich uncle" that many believe it to be.

Finally, without such a plan setting forth objectives to be attained, the problem of organizing to do the job is most difficult and it is virtually impossible to have efficient management within the various organization units. In short, it is management by crisis.

Short-Range Plan

Before manpower, equipment or other organization requirements can be effectively mobilized, long-range goals must be reduced to immediate short-range objectives. In the highway business this is most difficult.

On the one hand, the time required to plan, locate, design, and acquire rights-of-way for construction projects often takes from three to five years. Unless such a complete and detailed 3- to 5-yr program exists, it is virtually impossible to mobilize the manpower in the operating departments to produce a balanced construction program.

On the other hand, budgeting for such specific projects and functional performance within the specific project area must be tied to annual or biannual legislative budgets. Such overlapping is often confusing to highway commissions, members of the legislature, and the public at large because most agencies of government are spending their allocated funds on an annual basis whereas the highway departments are today working on projects that will go under construction three to five years later based upon anticipated revenues. The 3- to 5-yr short-range plan is, however, absolutely fundamental for any intelligent management of the highway operation.

As a further illustration of the importance of long-range planning, when such a detailed plan exists, it is then possible for the policy-making body of the highway department to set up a 3- to 5-yr work program and make rational decisions as to the amount of total funds that are needed for each given portion of the highway system (primary, secondary, and urban) after first determining what must logically be spent for the maintenance of the existing system. Similarly with such a long-range plan, it is possible for the policy-making body to select construction or improvement projects in a logical manner, based on the relative needs on a statewide basis within any given system.

It is therefore axiomatic that the second step that must be taken in the management process is the development of the short-range plan that will permit the highway administration to make rational determinations of how its manpower and materials should be organized to accomplish the work program.

ORGANIZING

Personnel organization considerations are basic for effective management. Often it is difficult for engineering departments to realize this. However, unless major functional activities are logically grouped and specifically pinpointed to the organization plan and, conversely, the functions of each organizational unit clearly defined so that responsibility and accountability for each major function can be fixed, it is impossible to set up modern accounting systems to generate reports that will reflect the status of a given activity in actual performance as it relates to any planned objectives or course of action. It would logically follow that if the work program is not on schedule, corrective action would be difficult because of the inability of determining exactly what is wrong and who is responsible. Thus, organization considerations are basic and fundamental to the development of accounting systems that will provide highway management with reports vital and essential for efficient operation.

The first step to be taken in bringing the personnel organization into harmony with the short-range plan is a critical examination of the existing functional and departmental structure. In making such a study, it has often been found that departmental functions are not clearly defined, that often there is overlapping responsibility, and that within the total structure itself, there may be an excessive number of departments reporting to a single administrative head. An analysis of one organization chart shows over 30 departments reporting to such an administrative head. In another case, the entire planning function was found to be subordinated and almost lost within the actual engineering operational activities. Therefore, the establishment of the functional activities, definitions of responsibility, and adequate lines of communications with proper levels of intermediate supervision is the first step in organizing for effective financial management.

Once the functional activities have been determined and properly oriented, the second step should be to organize the functional department itself so that it may carry out its duties and responsibilities most efficiently. Such departmental analysis may show an excessive number of supervisors on the one hand, and on the other, a complete lack of necessary supervision. Overlapping jurisdictions may be found that should be clarified. Improper emphasis in certain sections of the functional department may also be apparent. Such functional departmental analysis as related to the short-range plan often brings to light bottlenecks in the production process because of inadequate staffing, inadequately trained personnel, or a lack of understanding of the objectives to be achieved. As an example, quite often it has been found that roadway design departments have historically grown to cope with secondary and rural primary projects, whereas the short-range program indicates a definite shifting of emphasis to urban expressways.

Those charged with the responsibility for administrative management in the North Carolina Highway Department have found that their most valuable tool in assuring coordinated action and proper orientation to achieve its short-range plan is the development of a specific organizational complement of personnel by specific job function for each department.

It should be mentioned that there was initially somewhat of a resistance to the development of such a detailed departmental organization by many line department heads because it appeared to them to be an excessive control over their activities. It is believed, however, that after such critical analyses were completed and many departments reorganized, each department head now has a better understanding of the relationship of his function to the total operation, that he has a better understanding of the strength and weaknesses of his department, where additional in-service training is to be required and because of this study, the department head has become a more effective manager of his activities. It has often been said that the technically-trained engineer has not had adequate experience to discharge heavy management responsibilities. Such functional and departmental self-analyses provide the department head with a much better understanding of his administrative responsibilities and considerably enhance his effectiveness.

It is possible that the matter of continuing self-analysis of the personnel organizations and complements may be overemphasized as far as some State highway departments are concerned. It is considered, however, to be a most important management consideration in North Carolina with its 70,000-mi highway system to be maintained with an annual payroll exceeding $40,000,000.

Manpower mobilization is only one phase of organizing for the short-range plan. With an equipment rental business in excess of $15,000,000 per year in North Carolina, the same close scrutiny must be given to the development of a table of equipment for each field maintenance and force account construction operation. Without this, it is impossible for the equipment department to determine the amount of equipment that will be necessary, the proper distribution of the equipment, and proper replacement tables and preventive maintenance programs.

In summary, effective financial management at all levels dictates a continuing need for manpower, equipment, and material review as objectives and goals of the short-range plans are changed or modified. Technological advances and improved techniques similarly demand critical analysis of organizational structure to assure coordinated action towards planned goals at the most efficient cost.

EXECUTING

The execution phase of management can be considered as "controlled performance," "management controls," or "operational control." Execution refers to controls for use by management to assure that objectives and plans are being met in actual performance within the various organizational units. It implies that areas within the operation that must be controlled be identified; that adequate plans and standards are established in each area; that the accounting system provides adequate records of actual performance and results; and that results compared to the plan are reported in such a way that responsibility is identified and exceptions can be made the subject of corrective action.

For many years finance departments were primarily concerned with routine record-keeping, limited to matters pertaining to receipt, deposit, and disbursement of funds. Recent technological advances, the development of accounting machines and high-speed electronic computers, today permit the finance department to play a key role in management control. Status reports may now be regularly produced on any and all matters informing management at all levels of the status of any given activity.

Without attempting to dwell on this point, it is considered of extreme importance, however, that the engineering organization in the highway department understand that all that modern accounting equipment can do is to receive, arrange, and report information in a specified manner. The accuracy of such reports is dependent entirely on the source information received from the various engineering operations. It appears, therefore, that one of the weaknesses in financial management is personnel not understanding the importance of providing accurate information.

Finally, such modern-day reporting systems are of little or no value if corrective action indicated by the report is not taken by the appropriate department head. The controller, making the reports, cannot be expected to take corrective action, which is the responsibility of top-level management and the line department head.

Although the highway business is essentially a production business, the actual control of construction and maintenance activities differs materially in many instances from that of private business, which is essentially a production operation. In most such private businesses information valuable for management control purposes can be reported in percentages of labor and materials consumed in the activity. In the highway business this same approach can be taken in some of the functions. The routine marking and signing of the highways, retreatment and resurfacing maintenance activities, routine unpaved road stabilization, mowing operations, and certain force account construction activities on secondary roads can be analyzed similarly to production in private business.

The matter of controlling the production of a highway construction project from its initial planning stage to the day of contract letting presents, however, a problem unique to this industry. It is difficult, if not impossible, for example, to set forth that the initial planning on a highway project should cost a given number of dollars or take a specific amount of manpower a given time to complete the job. Similarly, any report reflecting a percentage of total anticipated costs having been expended may be meaningless insofar as percentage of progress is concerned. This is also true in part of field location, of preliminary design, and in right-of-way acquisition. Previous statements should not be interpreted as implying that controls should not be exerted over these activities. Rather, they are made to establish the point that within the creative framework of the highway business for management to determine that it should take, for example, $10,000 to do the preliminary design on a job, would be a completely fallacious approach in that a designer's goal would be to complete his phase of the work within a given framework of money rather than taking the time and effort necessary to develop the most economical and practical design for the stipulated conditions. Quite often this point has been misunderstood and poor highway designs have resulted when a given field party was told to have a specific highway job located in a given number of days.

It follows therefore that certain other devices and techniques should be used to control these activities. Such techniques may in part be susceptible to cost accounting whereas others will not.

As previously explained, the length of time or the cost to carry out any given planning, location, design, or right-of-way acquisition on any given project may be difficult to determine. This is, however, only part of the problem. At any given time in a large highway department there may be several hundred projects in any one of the previously mentioned stages. Therefore, the problem is further compounded by having to keep not just one project on a given schedule but rather the total activities of a vast group of projects in a schedule that permits each department to operate effectively with each activity being coordinated with each other activity to minimize overlapping and lost motion. This problem is then one of "logistics."

This over-all problem can best be handled by a "project control center" that is responsible for scheduling the activities on all highway projects in cooperation with operating personnel; for keeping up-to-date reports on the progress of each activity on each project; and for informing top-level management where projects are getting completely out of a predetermined schedule. As projects become stalled for one reason or another (such as the need for a line revision, difficulties in right-of-way acquisition), then periodically it is necessary to make revisions in the operating schedules of some projects, moving them back in terms of contract letting and at the same time rearranging other projects such that the available manpower can still be used effectively.

The importance of this project control operation is far more significant today than it was two years ago. With the bulk of highway construction projects being Federal-aid projects and with the so-called reimbursement planning, or as it is more commonly known, "contract control," which specifies the amount of Federal funds that may be placed under contract in any one given quarter, the actual planning and design and scheduling of projects must be kept in close harmony with Federal funds available for contract lettings.

In North Carolina, this project control operation has worked very successfully. Proposed construction schedules for each department are shown on a master board for approximately a five-year period with approximate dates of authorization for preliminary engineering, right-of-way acquisition, and contract letting. These schedules are then tied to the accounting records through control accounts and related to reimbursement planning schedules as anticipated for the next several years. Such a device as this project control operation not only is valuable as a financial control for Federal-aid operations but, of greater importance, it also alerts all operating departments to the schedules of projects that they will be working on for several years. Naturally one of the most important parts of such a project control division is also the balancing of the projects such that excessive workloads for both designers and construction personnel will not take place in any given part of the State for a prolonged period. It might be mentioned also that although there has been a great deal of criticism concerning control, it is actually a good management device and allows an orderly production of work. Finally, in the peculiar operation in North Carolina, the project control operation is a must because of the two thousand or more secondary road projects that may be under design, force account, or contract construction during a given period of time.

Much more could be said about the problems of execution in the highway department. The previous discussion has merely been set forth to illustrate some of the problems.

SUMMARY AND CONCLUSIONS

This paper has dealt with certain problems in planning, organizing, and executing the highway program. The question might be raised as to what all this has to do with financial management. Too often financial management is considered as bookkeeping. It is therefore the thesis of this presentation that financial management encompasses these three processes in their entirety.

The effectiveness with which each department operates, a clear definition of functional responsibility, control of production, the utilization of equipment, personnel, and the efficiency of each of these activities determine the effectiveness of financial management in the highway department.

No mention has been made of budgeting in this discussion of financial management. Such an approach was premeditated. Too often to the engineering employee and in State government, the concept of budgeting is thought of as the amount of funds for routine supplies, travel expense, telephone and telegraph, office equipment, and other incidentals that are insignificant in the total budgeting process. Actually a budget must reflect the considered judgment of top-level management as to its goals and objectives, which is planning. Similarly, the budget must reflect the logical arrangement of manpower, materials, and equipment to achieve its objectives, which is organizing. Similarly, it must guide and assist top-level and lower-echelon management in the daily discharge of its assigned task, which is executing.

It is concluded, therefore, that modern-day financial management in the highway departments can only be effective if this total concept of budgeting is understood as contrasted with the too often accepted concept of budgeting used merely as a control device for routine departmental expenditures. Finally, it is believed that highway departments should give serious consideration to the establishment of a top-level department charged with the responsibility of continuously analyzing and coordinating the planning, organizing, and executing processes.

III. Use of Fiscal Management in the Michigan Highway Department

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• FROM COAST to coast, State highway departments are engaged in a massive roadbuilding program to meet the insatiable demands of a Twentieth Century civilization on wheels. Social and economic patterns are being reshaped, and everywhere the emphasis is on better, more rapid, and safer transportation. So far these ultimate goals are still to be achieved. In many areas, roads are becoming obsolete almost as fast as they are built. Inflation chips away at projected mileage, and additional finances to provide for the necessities and the expanding needs present problems of major proportion. Highway administrators are faced with momentous fiscal decisions. The odds are weighted against them. But they are applying new techniques to their problems.

One solution has emerged that highway administrators can use with surprising results: fiscal management.

How well each administrator uses the resources of men, machines, facilities, materials, and time will determine their progress because all involve money. This imposes the responsibility to plan effectively, budget properly, spend judiciously, and stretch financial resources. This paper discusses how the Michigan Highway Department is approaching the problem.

To provide a basis for advance fiscal planning as well as other department activities, Michigan conducted a highway study projecting needs for the next 20 years. Every mile of State highway, county road, and city and village street was evaluated to determine its ability to carry the traffic load from 1960 to 1980.

The study revealed that traffic volume on highways, roads, and streets will increase 91 percent. Only a tenth of the State's 110,000 mi of roads and streets would be adequate for the anticipated traffic load, and financial needs are at least $11,000,000,000 while estimated income during the same period's only $8,000,000,000.

People and their economic activities generate highway needs, but new highways and money expended in their building can stimulate economic growth, increase productive efficiency, and improve the competitive position of every economic activity in Michigan.

The needs study is to a highway department what the market forecast is to business—a set of studied predictions and indications to be used for an essential element of fiscal management, advanced planning. It involves two principal types of cash projections: future expenditures and future income. Their importance is obvious, but the procedures necessary to secure figures with a relatively high degree of accuracy are not so obvious.

To explain how the highway department establishes funds and how they are disbursed by the finance division, Michigan highway construction is financed through 14 different funds: ten are established in cooperation with other governmental subdivisions (counties and municipalities); three are bond funds supported entirely by the State highway department, and the last, and largest, is the State Trunkline Fund, composed of fuel and weight tax revenues, payments by municipalities and counties, and Federal-aid reimbursements. Expenditures from this fund include payment of all bond interest and retirements, the department's administrative and operating costs (including maintenance of roads and structures), and right-of-way and construction costs not included under the other funds.

It must be remembered that there are various fluctuations, especially seasonal, in total revenues and expenditures. For instance, motor fuel taxes account for approximately two-thirds of the tax revenue of the State Trunkline Fund, and there is no great variation in tax receipts from one month to another. However, motor vehicle weight taxes, which constitute the remaining third, are collected primarily in January, February and March, and result in a peak in cash available in April and May each year. The peak in progress payments occurs during July through October.

To facilitate cash expenditure projections, payment expectancy factor tables based on an analysis of about 1,400 recently completed contracts is computed. Separate factors were determined for each of fourteen different average lengths of contracts and for awards in each of the four quarters of the year. The appropriate factors were applied to the unpaid balance of each contract at each projection date to estimate the balances expected to be paid each month until completion of the contract. The balances were added for each month to determine total cash to be expended on contracts in that month.

This same procedure is applied to future projects yet to be awarded. A similar factor table and procedure was established for right-of-way acquisition projects and expected payments. Expected Federal-aid receipts were also studied and factors established.

It is easily seen that considerable data and extensive computations are required to establish and keep current cash projections. Reports and memoranda from various divisions and sections of the highway department are forwarded currently to the financial planning section where the data is transferred to punched cards. Computations and other operations are made through electronic and mechanical data processing equipment.

Monthly tabulations provide an analysis of payments and a statement of anticipated Federal-aid receipts. Subsequent projections are made showing the balance of funds available. Net cash requirements for each month are ascertained by deducting total projected receipts from the total projected expenditures.

Because certain contracts do not follow a factor table pattern and are usually paid in a lump sum or after road construction is completed, the data for these contracts are not recorded on punch cards. The projections are worked out manually. Included in this category of contracts are those with railroads, municipalities and utilities, steel vendors and contracts dealing with demolition, removal of structures, signs, signals, seeding, sodding, fencing, and guardrails.

Because bond sales must be prepared four months in advance, cash projections have been of great value in determining cash availability for such sales and whether they should be deferred because of unfavorable market conditions.

Projections may indicate sufficient funds to complete a planned project but it must be remembered the fund may be operating at a deficit because payments must be made to contractors and landowners before Federal aid may be claimed and received. Projections make it possible to arrange temporary advances and loans to a particular fund to carry it through the low period and avoid sale of additional bonds.

When a State is making rapid highway building progress, it is likely to outrun Federal-aid authorizations and allotments. Thus with construction costs and related Federal-aid earnings computed far in advance, which Federal-aid projects must be deferred or delayed can be determined and a prudent selection of individual projects made. It is also easier to determine well in advance when such aid may be claimed, and cash projections are commended as an important fiscal tool of management.

In the Michigan State Highway Department, project programing involves correlation of traffic and condition information pertaining to the entire trunkline system. The programing also involves selection of portions of the system to be improved and determination of the work and cost estimates. The final selection of the program, in turn, determines work schedules for the route location, road and bridge design, and right-of-way divisions.

An invaluable aid in programing has been the establishment of a sufficiency rating system with data compiled annually. The condition of the entire highway system is shown each year, thus providing a basis of comparison from year to year. This is especially helpful to the planning, engineering, design, and maintenance divisions.

A minimum lead time of eighteen months, and preferably two years, is needed to establish a program prior to actual implementation. Costs are estimated by the programing division and a proposed budget is developed for each project, concurrent with establishment of the program. The lead time period is necessary for route location studies, reports, surveys, preliminary plan preparation, and right-of-way acquisition.

When a program has been established, control systems are mandatory to keep within estimated receipts and costs. Additional controls are established for monthly or periodic reviews on each project programed and cost estimates are assembled by the route location, engineering, design, and right-of-way divisions.

Obviously, the volume of program data accumulated requires fast and accurate computations. The entire program is put on tab cards and data processing equipment is used until the program is finalized and projects let. The programed projects are scheduled by quarters for each year, thus allowing every division concerned to re-evaluate the initial schedule in relation to their current operations. How this becomes a fiscal tool is by an evenly divided program considering availability of funds, urgency of particular projects, ability of contractors to accept work, and ability to secure adequate seasonal employees.

The department's first 5-yr program was adopted in 1957. It was based on a sound statistical projection of population trends, vehicle registrations, vehicle miles of travel, adequacy of present facilities, Federal interstate roads, and State trunklines. Fiscal implications involved were the following:

1. The department's ability to finance.

2. The ability to secure adequate and competent personnel, facilities, and equipment within the required limitations to avoid delays.

3. The ability of the road building industry to accept and complete adequately a program of such magnitude within the limitations.

Other considerations included scheduling of construction in respect to the resources available, by fiscal year, for the 5-yr period while retaining sufficient flexibility to compensate for delays in obtaining preliminary plans, specifications, design, right-of-way, etc.

Other prime objectives included the following provisions:

1. Allowing local units of government to start budgeting for their share of the improvement and planning other improvements in conjunction with the building program.
2. Allowing property owners along the proposed routes, as well as business and industry, to plan for the future.
3. Giving the road building industry a guide to the program demands.
4. Giving employees a definite, long-range goal and target dates for various steps that must be taken before a project is completed.

The end product consisted of a 5-yr program costing $1,250,000,000 to construct 900 mi of freeway and modernize more than 3,000 mi of other trunklines.

Nearly four-fifths of the $1,250,000,000 of construction work is either completed or under contract. With few exceptions the established contract award dates have been met on schedule.

The success of this program can be measured in terms of public acceptance and the tribute paid by the voters of Michigan in April 1961 to the man who directed it—Commissioner John C. Mackie.

The development and results of the first 5-yr construction program have been so successful a second 5-yr program has been developed from 1962-1967, calling for an additional 175 mi of freeway, mostly in urban areas, and modernization of 1,400 mi of other trunkline highways at an estimated cost of $750,000,000.

Federal Highway Administrator Rex M. Whitton, in an address at the recent AASHO Convention, urged all States to develop similar 5-yr programs, saying, "It gives the valuable needed time in which to properly locate, design and secure right-of-way for proposed improvements."

There are other advantages to a long term program:

1. Improvements can be arranged in order of need.
2. Goals can be set and the taxpayer shown what his tax dollar is buying.
3. Economies can be effected in capital improvements.
4. Availability of funds can be assured for needed improvements.
5. Too large an accumulation of debt can be prevented.
6. Undue expansion of operating and maintenance costs can be prevented.
7. The effect of fixed charges on the remainder of the operating budget can be determined.
8. Public confidence is inspired in the goals of the administration.
9. The effectiveness of pressure groups is reduced.
10. The economy and the construction industry is stabilized to a great extent.

In July 1961, the Michigan State Highway Department established an "Engineering Development Committee." This committee is to the department what a product development unit is to an industrial corporation. It is the "idea" group, and is composed of individuals qualified in the fields of construction, materials, traffic and traffic devices, methods and equipment, design, objective planning, and finance. The committee's prime objective is to study and develop highways, transportation networks, long-range methods of financing, and future requirements of the motoring public and motor transportation.

The Michigan State Highway Department utilizes budgets to establish definite goals, outline fiscal policy, promote cooperation between divisions and indicate when changes in established goals are desired. However, budgets have limitations. They are not self-executing, nor do they ever replace the need for sound judgment and good administration. To be fully effective, budgets require constant vigilance and complete departmental cooperation.

To illustrate how the three phases of budgeting—preparation, execution and control—serve as fiscal management tools in the department, the preparation process starts at the office or division level, which allows all levels to evaluate their operations periodically and request changes as desired. Then budget hearings are held between management and each office or division. This permits management periodic evaluation of programs and operations and provides the material necessary for decisions on many fiscal policies. At the same time, the budget process permits exchange of program information from bottom to top and top to bottom.

During the approval phase, the legislature is provided with the proposed budget, which contains information relative to the department's past activities, current operations, and future programs. They evaluate past performance and set the fiscal policy for future operations. The end result is an appropriation bill that is the legal framework for the fiscal operations during the ensuing fiscal period. With this legislative guide line, the highway department develops quarterly allotments of the line item appropriations.

At this point it applies the final phase of budgeting, budget control. (The department "controls" both before and after the expenditure.) Because the operating budget itemizes allotted personnel and equipment, requests for new employees or new equipment are checked against budget authorization. If they were not included, the office or division must propose a substitution for approval by management. This process allows for flexibility of operations while maintaining sound fiscal control.

Post-expenditure controls consist of accurate accounting information, promptly reported. The information compiled as a monthly statement tells how well the department is operating within its allotments and serves to indicate trouble spots before they become acute.

Internal auditing has been defined as the independent activity within an agency to ascertain, for management, whether its policies and procedures are adequate and properly adhered to; to provide management with systematic and objective appraisals of internal controls, operating procedures and practices; and to verify the accuracy and reliability of the financial records and reports. However, in the Michigan State Highway Department, as in all fiscal operations, the actual implementation of internal audit does not conform to any single classic definition of the subject.

The organization and methods impose requirements on the internal audit staff, which are undoubtedly characteristic only of this department. This can best be demonstrated by a review of the contractual relationship with counties and municipalities for the performance of maintenance on trunklines. For this purpose, it has established cost contracts with both counties and municipalities. The basic philosophy of this contract is allowance of charges to the department that represent actual cost as established by post-audit and adjustment of the charges according to this audit at the year's end.

This requires extensive audits and includes the development of a schedule of standard hourly equipment rental rates, by accumulating equipment cost and utilization information from all counties in the State, and conversion of this information to standard hourly rates for use in the subsequent year. Material charges require the cost audit of gravel and other aggregate production operations, and the establishment of unit costs for this material within the agency being audited. This function alone requires a large portion of available audit manhours. However, it has given a further degree of fiscal control over a large part of the operations. Its effectiveness is evidenced by the total savings through adjustment to the contract charges for the past year in the amount of $236,850.

Each transaction in the right-of-way division is audited to insure conformity with Federal requirements, and the department's stated policies. In addition, selected right-of-way transactions are reviewed by identification on maps, verification of the proper accounting of surplus properties, and a review of procedures and systems.

Much work in the construction area is performed under force account conditions and terms. One of the force account activities is the relocation costs of utilities, pipelines, and railroads. Audits of charges for this item have resulted in savings of $46,000 in the past year. Other audit functions performed regularly include:

1. Annual verification of physical inventories and a review of stores and inventory operations.
2. Periodic reconciliations of cash and cash accounts.
3. Periodic review of accounts receivable and the write-off of uncollectable accounts.

Internal audit functions both as a line and staff activity. It represents a necessary and efficient form of fiscal control for management of the many facets of the organization. The theory, principle, and application of fiscal management have been accepted at all levels within the department. This may be attributable to programs, seminars, and management conferences, all designed to effect a better understanding of fiscal management through the mutual exchange of ideas and practical applications. Immeasurable assistance has come from the American Association of State Highway Officials and the National Highway Users Conference through their management seminars.

With the rapid developments in electronics and changing needs and conditions, the ultimate in fiscal management continues to challenge the most able administrator. This challenge can be met by a firm dedication to the principles of fiscal management, the continued application of the varied tools available, and an open-minded approach to new management concepts.

DISCUSSION

GUILFORD P. ST. CLAIR, *Bureau of Public Roads.*—Although varied in approach and each a distinctive contribution to the literature of highway management, the three papers of this symposium convey among other things a common message to the effect that highway engineers had better watch out because the financial and management experts are moving in. No longer can the engineer hope to control the operation of State highway departments without challenge.

Although there is an element of wry humor and perhaps not always friendly rivalry in the contest for highway management control, the struggle is a healthy one and the engineers should accept the challenge by moving to increase the skill and vision of their planning and the effectiveness of their management. There is a tendency for the engineer, and particularly for the oldtimer at the game, to use the words "bookkeeper" and "clerk" in an effort to disparage the ability of people from the accounting and management fields to handle some of the tasks that highway engineers have handled in the past. This is dangerous. One needs only to have listened to the papers of this session to be assured that the day of the green eye-shade, the alpaca coat, and the high stool is long gone. The administrative personnel of today are very sharp fellows. They are familiar with many other matters besides double-entry bookkeeping; they show an annoying capacity to understand engineering principles as well as engineering processes; and they are often able to challenge the engineer on points of general education. To sell them short is to court disaster.

Having acknowledged the prowess and knightly posture of the challengers, the writer would still like, as an oldtimer, to claim the privilege of recalling some features in highway history of the last 30 years that may reflect some little credit upon the highway engineers of that era. Whether the record is sufficiently good to indicate that the engineer is worthy of remaining in the front rank of highway planning and management, only the event will show. One can only relate past happenings, "All of which I saw and part of which I was," as Aeneas said of some ancient goings-on.

The history of highway engineering up to the early 1930's was primarily that of building roads where they were obviously needed and developing structural techniques in the art of highway building. Gradually the importance of studying and regulating traffic was realized and the problems of highway taxation and finance thrust themselves upon the highway executives. The integrated concept of highway planning was crystallized out of these early efforts by Herbert S. Fairbank of the Bureau of Public Roads, who conceived and set in motion the highway planning survey operation that was initiated in 1935 and rather rapidly adopted, wholly or in part, by all States. The concept of the highway planning survey was that of gathering facts about the highway plant, about the volume and composition of highway traffic, about the life characteristics of highways and highway elements, and about highway taxation and finance; and on the basis of this fact-gathering to project plans for the future development of the highway network.

Concurrently the art of traffic engineering developed rapidly in cities as well as in State highway departments. University instruction and research were enlisted in the attack on these emerging problems of highway planning and management. Thomas H. MacDonald and others foresaw the need for an interregional or interstate system of highways linking the principal centers of population and industry, and planted the seeds that are now coming to fruition.

World War II put a temporary stop order on ordinary highway planning activity and at the same time strained the capacity of the highway plant so that at the war's end there was a backlog of sadly needed highway improvements all over the country. During and shortly after the war the necessities of urban as well as rural highways claimed the attention of highway engineers, and from these efforts sprang the early urban origin-destination studies, chiefly by the home-interview method. Shortly after the war the Automotive Safety Foundation created an engineering staff to aid the States and the Bureau of Public Roads in attacking the problem of the backlog of highway needs and exploring the means of satisfying these needs on an accelerated basis. This triumvirate developed the original techniques of the highway needs study. The tremendous and unforeseen development in the numbers and travel of motor vehicles made the needs studies doubly urgent and soon proved that the early estimates, thought by many to be extravagant, were only too conservative. This has only sketched the bare outlines of an exciting story. What it is derived to emphasize is that most of this work was the work of engineers, some of whom surely brought to the task a commendable vision and integrity of purpose.

Even during the early planning survey era, it was recognized that other disciplines than that of engineering were needed to solve the increasingly complex problems of highway transportation. Economists were needed to aid in the study of highway finance and related subjects. Psychologists were needed in the field of driver behavior. More recently geographers, social scientists and public health specialists have been called in to counsel and assist in the solution of numerous problems. In the allied fields of planning, programing and budgeting, the skills of accountants, management experts, economists, and political scientists are needed and should be welcomed. This is a big job ahead, and needed in the effort are all the brains, all the skillful hands, and all the strong backs that can be enlisted.

In short, the highway business is a cooperative undertaking in which each can make his best contribution unselfishly and without recrimination. As to the future of the highway engineer, it is not probable that the management group is going to take over and make the engineers into a group of worker bees, expert at their narrow tasks but otherwise witless. For there is another development going on in the engineering colleges. Increasingly science and the humanities are being emphasized at these schools, and training in the routine tasks is being given to the engineering aid or technician. A crop of young engineers is being raised in the Bureau of Public Roads and the State highway departments who can be relied on to give the boys from the administrative field a run for the front-office jobs.

Highway Fund Distribution Policy

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Geographic apportionments of highway funds by the Federal and State governments have greatly increased in importance in recent years. The trend toward centralization in highway finance, especially as a result of Federal decisions, has occurred together with increased highway expenditures on behalf of local travel within geographic regions, particularly in urban areas. The channeling of the enormous central pools of highway revenues to separate regions and road systems is a distinct part of public policy for highway financing.

An economic view is suggested of the apportionment process. The geographic division of highway user tax proceeds links the collection of the revenues with their expenditure for specific investment purposes. Therefore, distribution policy may be analyzed both in terms of the economic objectives of user taxation, and the economic principles applicable to highway expenditures. Both approaches are considered, as well as the problem of drawing them together.

The distribution policies of the Federal and State governments have been influential in shaping the direction of transportation developments among regions. At the time before World War II when nearly all tax collections were expended upon rural roads, the shift of money from cities to rural regions gave intercity and rural motorists the benefit of support from the broad urban highway tax base. In the postwar period, the decision to finance urban freeways from user taxes has been an important factor in the course of metropolitan transport planning. Various economic issues raised by the rural-urban division of highway funds are discussed briefly.

• THIS PAPER discusses the apportionment of highway funds as a question of highway fiscal policy. In highway financing, more attention has been centered on the imposition of taxes, and the collection of revenues, than on the distribution of the proceeds. It is desirable, in view of recent trends, that highway finance analysis be advanced beyond these customary limits to include the allocation of revenues to particular regions and road systems within a taxing area, as well as the total supply of funds.

What is involved in apportionment policy? Highway user taxes are nearly all collected by the Federal Government and the States, and the apportionment process may be said to begin with the division of the revenues between governmental road systems. For example, a fuel tax of $0.10 per gallon within a State is paid as a unit by the motorist, but it might be composed of the following divisions:

a $0.04 Federal tax to pay for the Interstate Highway System and other Federal-aid roads, $0.015 imposed by the State for distribution to local rural governments, $0.005 for allocation to municipalities, and the balance of $0.04 retained by the State for State system highways. The next step is a geographic division of funds for each class of road. This stage may be accomplished by formula or by administrative discretion; a formula is almost a necessity if there is a transfer of money from the tax-collecting government to the spending unit. Even in the absence of intergovernmental transfers, formulas are often used to apportion funds among regions within the taxing jurisdiction, most notably in California. When the distribution of funds is decided entirely by administrative decision, the process may still be controlled by standards and directives set forth in laws governing the use of highway money.

The final apportionment to specific projects is largely an executive procedure, based upon some kind of short-run capital budget which is meant to direct funds to their most productive uses. A central tax-collecting government fully responsible for the planning and administration of all roads and streets within its boundaries might choose to treat the entire division of revenues, from start to finish, as a programing procedure, deciding the direction of funds by a priority-rating system. In this case, apportionment would involve no substantive policy decisions connected with highway financing. At the other extreme, the central government might be merely a collecting agency for highway user taxes, returning funds to the area of their origin without strings attached. Again, no apportionment decision, other than a purely mechanical one, would be required.

It is in the range between the collection of highway user taxes and the final capital budgeting of available funds that apportionment may be regarded as a separate, distinct chapter in highway finance policy and government decision making. The allocation of funds from the central authorities is generally intended to promote broad transport objectives, and to further their accomplishment the distributions are usually accompanied by stringent controls over the use of the funds.

The gap between the payment of highway user taxes and their ultimate use has been widening in recent years, the result of growing central control over finances and increasing expenditures primarily for the benefit of local travel purposes, especially in urbanized regions. Today, the Federal Interstate Highway program, toll roads, and State efforts have brought many States to the point where an actual completion of major intercity, interregional, and interstate road networks can be confidently foreseen. Their completion, in fact, would require only a few years if all highway user tax collections, State and Federal, were expended exclusively for such highways. However, the central governments are now deeply engaged in the sphere of local transport, providing highways to serve predominantly short-distance vehicle trips, and no end to the need for highway funds for this purpose is yet in sight.

These trends have added greatly to the economic significance of apportionment policy, which channels the flow of funds from the enormous central pool of revenues to the separate regions and road systems. In metropolitan communities, much highway planning is now done in the course of comprehensive transportation studies of local transport needs; often, the planning is carried on independently of State highway agencies, or in cooperation with them. These regional plans develop highway needs estimates from predictions of economic and traffic growth within their study area. But an important planning consideration is beyond local control: the supply of highway funds depends upon taxation and apportionment decisions at the State and Federal levels of government, and fiscal policies must be seen as external circumstances in the area plans.

The purpose of this paper is to suggest some economic considerations by which the apportionment process may be approached and analyzed. The ideas presented here are in part the outgrowth of a research inquiry into California's long experience with dividing highway funds among regions (*1*), but it is believed that they have general application. The theme to be developed is that an economic analysis of highway tax apportionment brings together two different sets of principles: (a) those which are associated with assessing the financial feasibility of an entire highway program in order to devise equitable taxation methods, and (b) those associated with the evaluation of specific highway expenditures in order to establish the "need" for investment in roads. To unite these different branches of economic analysis, it is necessary to examine how taxation objectives and concepts may influence the allocation of funds for specific purposes, and how the monetary factor affects highway investment planning.

METHODS OF HIGHWAY FUND APPORTIONMENT

The division of funds takes place in two ways: (a) by the direct expenditures of the tax-collecting governments on their own road systems, and (b) by intergovernmental aids. Nearly all Federal money is distributed by the second method, but the States spend the majority of their own user tax revenues directly on State-administered highways. The quantity of highway user money that ultimately filters through to a region thus depends on three decisions in public policy: (a) the distribution of Federal funds among the States, (b) the locality of direct State highway expenditures, and (c) the allocation of State tax funds to local governments. These highway revenues may then be supplemented by city and county tax sources, or by tolls.

Apportionment of Federal Government Funds Among the States

The two varieties of Federal apportionments to States, the grants-in-aid to "ABC" roads and the funds for constructing the Interstate Highway System, are allocated according to different principles of distribution which are reflected in the nature of the distribution formulas.

Monies for "ABC" highways—the primary, secondary, and urban Federal-aid road systems—must be matched dollar-for-dollar by the States. In the distribution formula for primary roads, area, population, and the mileage of "rural delivery and star routes" have equal weight. The formula is the same for secondary roads, except that rural rather than total population is used. The entire spread of urban road aids is based on the population of "municipalities and other urban places."

The primary formula has endured since its enactment in 1921. This, perhaps, is less a tribute to the wisdom of the law than an illustration of one characteristic of legislative formulas: once in existence, they are not easily altered, as interests are built up for their continuation. As for its basic intention, Dearing (*2*) wrote of the primary formula that it was "designed to satisfy the national interest" in highways. "It would have been entirely appropriate," he said, "for the government to have pursued [its] declared objectives through the designation, construction, and maintenance of its own system of national roads." Instead, the national government chose to supplement and stimulate State efforts by working through State highway organization, and by prescribing certain managerial and construction standards for the use of the funds. "It is also worth noting," he continued, "that in the development of Federal policy no significant effort has been made to distribute funds in accordance with the benefits derived by direct users of the highways." The formula was not framed to approximate the volume of traffic, or the needs of highways, in each of the States. Area, population, and road mileage are broad indications of the relative importance of each State for interstate commerce, national defense, postal service, etc. It was not intended that the funds be used as a public subsidy to motorists, replacing tax revenues that might otherwise have been obtained from highway users, but as an incentive for State action which would further national objectives. This orientation of Federal policy, applies as well to the secondary and urban road aids that are directed specifically to local rural and local urban transport; there is no attempt in the formulas to proportion the size and distribution of the grants to the volume of each class of traffic.

By 1956 highway activities had attained a level of maturity in most of the States sufficient to raise a question as to whether the original purpose of stimulating State activity to further certain national purposes had not been accomplished, so that the continuation of aids could only be interpreted as a deliberate subsidy to motorists. In that year, however, Federal user taxes on motorists became the source of "ABC" funds, which theoretically disposed of the subsidy question. However, no change was made in the distribution formulas to parallel this action. The formulas thus became the means by which motorists in some States now provide large sums of money for the benefit of motorists in other States. It might have been appropriate to reconsider the formulas in the light of the new Federal fiscal policy, but no detailed appraisal appears to have been made at the time, nor has there been one to date.

Funds for the Interstate Highway program, which commenced on a large scale in 1956, are allocated among States in proportion to the remaining cost of completing the system in each State. The intention of Congress was declared in 1956 to be that cost should become the basis of distribution when suitable estimates of needs had been prepared. The need basis was formally adopted in 1958 after an extensive study of system costs. Therefore, legislative power over the spread of funds was limited to a periodic review of the cost estimates, the prescription of certain standards for calculating costs, and the inclusion of new routes in the system. Actual distribution decisions have depended mainly on the determination of costs.

The process of cost ascertainment is not restricted solely to the highway needs of vehicles traveling in interstate commerce but covers all vehicles that use the routes that comprise the Interstate System network. This is logical, inasmuch as the Federal government is charged with 90 percent of the total cost of Interstate highways and can invite only a small amount of State cooperation. It is not feasible, as a rule, for plans to be devised to serve only a portion of the total traffic flow along a route, unless there is some reason and technique for segregating traffic. To remove doubts about the matter, Congress directed in 1956 that local traffic be given "equal consideration" with interstate vehicles "to the extent that is practicable, suitable, and feasible," a proviso that has favored the distribution of funds to States with large volumes of local traffic relative to through vehicle movements—the States, that is, with large urban concentrations. Adding to the force of the foregoing directive was an addition of 2,300 mi of routes to the Interstate System, the entire increase possible under the law, around and through urban areas.

It is not surprising, therefore, that the apportionment of Interstate highway funds is much more closely correlated with the total volume of vehicle travel in each State than with the Federal-aid primary formula, whose factors presumably expressed the national interest in interstate commerce and other national objectives.

(The section 108d formula's percentages, which represent nearly the entire cost of the Interstate System in each State as estimated in 1958, have these simple correlation coefficients: (a) with the Federal-aid primary formula, 0.75; (b) with the 1956 total traffic volume on all roads in each State, 0.93; and (c) with the 1976 total traffic volume predicted in each State, 0.93. The traffic data were reported in *Public Roads,* February 1960.)

Again, the source of the revenue for the system does much to explain the basis of apportionment. The Federal decision to set up the Interstate System program was coincident with, and probably contingent on, the conversion of Federal finances to user taxation. Although the system is expected to handle about one-fifth of the nation's vehicle movement when completed, the Federal user taxes are paid by all motor vehicle travel, and there is an understandable desire that the funds be returned in rough proportion to the State and area of their origin. Notice may be taken of the many recent references to the geographic origin of revenues made to counteract the suggestion that certain segments of the system having the least "national interest" because of a predominance of local vehicles might be deleted from the system.

A modest amount of Federal assistance is extended to forest roads and miscellaneous other purposes but need not be considered here.

Apportionment of State Funds for State Highways

Highway funds received by the States from Federal grants or from State-levied taxes, and used for highway purposes, are either distributed to local governments or spent directly on State-controlled highways.

The policy of each State for distributing State system expenditures among localities within its borders is determined in part by the destination of Federal funds. In most States, nearly all Federal money is retained by the State highway agency, rather than passed on to cities and counties, and it must be matched with State funds. Interstate System money must be used on a specific, limited mileage of roads. This is true also of primary and secondary aids and the matching funds, but the mileage of "ABC" highways, with a few exceptions, includes the majority of State system highways in each State, giving a wide latitude to the place of its investment. After matching the Federal grants and meeting the expenses of maintaining and operating the highway plant, a State may use its remaining construction funds to offset the effect of the Federal constraints. If, for example, Federal money must be used largely in Region "A" because an Interstate route passes through that locality, a State may allocate its own funds largely to Region "B". Of course, a State's ability to do this is no greater than its own highway revenues permit. In California, highway money raised from State taxes is sufficiently ample to overcome virtually any influence that Federal funds might have on the total geographic apportionment of State system expenditures, but few States are able to exercise this much discretion.

A separate, distinct policy for the distribution of State system funds is not often defined by a State. The decisions that govern the geographic allocation are reached when roads are classified as State highway routes (making them eligible for the expenditure of State money), when legislative directives are enacted for the preparation of long-range highway plans or annual budgets, and when provision is made for the administration of the State highway function. States approach the problems in many and varied ways, making general statements about the procedures difficult. (Questionnaires received from 36 States recently showed that 6 States apportioned funds among two or three major regions of the State; 23 States, by districts; and 11 States, to lesser jurisdictions. Of course funds are also split among the several systems. Apportionments provide for a spread of projects, that has not necessarily been made in accordance with relative need, or equity.) (*3*) To the extent that generalization is possible, it appears that a State legislature typically gives formal approval to a comprehensive plan of some kind for constructing a basic road system. Short-term programs designed to accomplish the long-range objectives are prepared by the State's highway administrative agency and are reviewed and approved by a highway advisory commission, the State legislature, or both. In a few instances, California for example, the State legislatures enact apportionment formulas to guide the flow of State system funds among regions within the State, even though there is no transfer of money between governments. Another type of legislative formula gives priority to one class of State highway over another. Such formulas fix limits to the scope of administrative decisions.

One idea has typically dominated past apportionments, that the State is engaged in constructing a single "Statewide" network of roads for connecting major population centers and providing access to all places where there is sufficient Statewide "interest" in good highways. The corollary to this concept is that the proper basis for apportionment is the need of each part of the system for funds to permit its completion, with needs being determined through engineering evaluation. In view of the broadening Federal-State highway activities in local transport during the past decade, this concept might well be reconsidered as a guide to the distribution of funds.

The geographic division of State highway funds is one of the least studied areas of public decision-making in transport today. The effort of the Bureau of Public Roads to provide a compilation of State laws relating to the programing of highway funds is most commendable, and ambitious in view of the variations in practice discovered for a relatively few States (*4*). It is necessary, also, to focus attention on the distributive process itself, on the criteria used and the standards followed, as well as the legal and administrative mechanics of the process.

State Apportionments to Local Governments

The geographic apportionment of State highway aids to local rural and urban governments is accomplished entirely or in part by legislative formulas in all but 4 of the 48 continental States. The funds are technically apportioned as grants in aid, often with certain conditions and controls established over their use.

One study of the subject (*5*) reveals that area, population, road mileage, vehicle registrations, user tax collections (primarily vehicle rather than fuel taxes), and equal portions are the most popular factors included in the formulas, but this list is by no means exhaustive. The opportunities for improvisation, grafting, and experimentation with different combinations are endless. If a pattern can be discerned, however, it shows that the rural aids are most often distributed according to a combination that includes some measure of over-all need or "interest," such as area or road mileage, and an approximation of local travel volumes, such as vehicle registrations and tax collections. This combination brings together both the characteristics of the grant-in-aid and the shared revenue. For urban apportionments, population is the factor chosen most often. But one hesitates to attribute a high degree of rationality to these formulas. "As a general rule," the study concludes, "it appears the formulas in use have been developed over time by trial and error conditioned by a generous amount of pulling and hauling by opposing local interests."

One explanation of State aids to local roads arises from the very evident need for a system of distributor, feeder, and access streets, without which major arterial highways built by the States with Federal assistance could not function effectively. It might, therefore, be argued that the most appropriate basis of fund allocation is the cost of constructing and maintaining these roads, rather than a return of funds to their source. However, certain objections could be advanced to an apportionment based solely on need. The most important of these is the lack of information: the extensive mileage of rural roads and city streets cannot be surveyed without a huge expenditure of technical manpower which has not been available for this task in most States. Uniform standards for review by State authorities also require considerable improvement. When attempted, local road studies have revealed a large range for the exercise of local discretion about the desirable extent of highway improvement in each area, which does not always admit of a factual solution. Because it is difficult, if not impossible, to harmonize these differences by engineering estimates, it is felt by many that local choice might better be left to local financing; but the difficulty here is that local governments do not have the freedom to impose taxes on road users enjoyed by the States.

Present policies for State distributions to local roads can be summed up by saying that in most instances the States extend a measure of assistance to the cities and rural governments, leaving the balance of the problem to a local solution. The trend is toward larger aids, but definitive apportionment standards based on needs must wait for more reliable appraisals of local highway requirements.

Summary of All Monies

The total division of highway funds is summarized in Table 1 for two years a decade apart. Over this span, the trend toward centralized highway financing is clearly seen. By much the largest increase as a source of funds has been the Federal government. Direct State spending of State-collected revenues has accounted for the next largest growth.

TABLE 1
TOTAL FUNDS MADE AVAILABLE FOR HIGHWAYS IN THE UNITED STATES, 1949 and 1959 [1,2]

Financing Medium	Funds		
	1949 ($\$ \times 10^6$)	1959 ($\$ \times 10^6$)	Increase (%)
Intergovernmental apportionment:			
Federal to State	429	3,035	607
State to local	735	1,513	106
Total	1,164	4,548	291
Intragovernmental funds:			
State	1,577	3,833	143
Local	1,069	1,800	68
Total	2,646	5,633	112
Total all funds	3,810	10,181	167

[1] Source: U.S. Bureau of Public Roads, *Highway Statistics* (1949 and 1959).

[2] Excludes State toll financing and proceeds from sale of bonds.

Small increases have been registered by State transfers of user funds to local governments and local fiscal sources, mostly non-user.

Combined Federal-State spending for highways rose about 250 percent between 1949 and 1959. The spending of local governments, including the aids received from the States, rose by 84 percent in the same period of time.

By 1959, almost one-half of all highway funds (and considerably more than half of all construction money) was being transferred between governments and thus guided by legislative formula to its destination.

ECONOMIC BASIS OF APPORTIONMENT

An examination of the division of highway funds in any State reveals a complex interplay of forces which are pertinent to the public welfare. An economic view of the process deals with but one of its many aspects. It implies an abstraction from other factors, chiefly political, which influence actual policies. "Our frequent naive assumption that the government is a monolithic entity devoted only to the public welfare and knowledgeable about how to attain it has had several most unfortunate consequences," Hitch has observed, speaking for economists; ". . . it has closed some promising fields to economic analysis, *e.g.*, government expenditure and government organization." (*6*)

It seems appropriate, however, to assume that the public interest regarding expenditures for highway purposes is definable by economic standards, and that government activity in this field may be guided by economic principles. The highway is one of the essential elements of motor transport, along with vehicles, drivers, fuel, etc., and in fact highways are a rather small, but nevertheless, strategic, item in the total motor transport budget. Highway cost is an expense complementary to the other costs of travel by motor vehicle, and if highway space is not available in sufficient amount, the motorist suffers the consequences of congestion, delays, and accidents. To use the language of those who inform the American populace as to the urgency of making road expenditures, "we pay for good roads whether we have them or not." The broad purpose of highway policy of all kinds, including financing, is to maintain a balance between the total number of vehicles within an area and the space for them to move about.

Most of the policy issues connected with highway fund apportionment turn on the question of whether the revenues earned from highway user taxes should be returned to the area of their origin or apportioned on some other basis. It is generally claimed by those favoring a return of funds to their geographic source that the failure to follow this policy upsets the desirable balance between the amount of vehicle usage and the provision of road facilities; those arguing for redistribution of user payments, on the other hand usually hold that the sources of earnings do not indicate places of the need for expenditures. There are both size and time dimensions to this controversy. If enough money were available to meet all highway requirements created by motor vehicle travel in all localities, it is quite possible that funds returned to source would also be brought to the places of need. However, apportionments usually must be decided within limited budgets and time periods, forcing consideration of road needs which have the greatest claim on funds currently available. For policy purposes, the more that the size and time horizons can be enlarged, the more opportunity there is to weight both the origin and destination of the funds.

It is the peculiar character of highway user taxation among public finances which gives significance to the apportionment problem beyond the scope of direct capital budgeting and programing. The geographic distribution of a lump sum of money, allotted for highway purposes from a general public budget, would raise no immediate issues regarding the origin of the funds. Highway needs, established on economic grounds and in other ways, would constitute a sufficient guide. However, the payment of user taxes is one element in the concept of highway need, since evidence of a willingness to pay for roads is taken as a measure of their value to the motorist.

Economic Principles of Highway User Financing.

The economic basis of highway user taxation is a pricing philosophy, with motor fuel and vehicle taxes on highway use performing the functions of prices. The theory views user taxes as representing to motorists the cost of supplying them with roads. If people travel in cars, they use highways and pay for them; if they do not travel over the roads, they do not pay. There is a presumption in this doctrine that the money contributed by each motorist will be spent for his benefit; otherwise, some drivers would be required to pay for roads they did not use, and other users would enjoy the services of roads for which they made no payment. The implication of this thinking for the apportionment of user revenues is this: over the long run, money should be directed to those areas and roads from whence it originated, so that there will be equal payments made for benefits received. Brownlee and Heller have suggested that if highway user charges are based on production costs "when demand and supply are equal" (*i.e.*, when congestion or excess capacity are absent), the revenues be imputed to each separate portion of the highway system "equivalent to the payment that would have been made for the use of that portion if tolls equal to the established prices had been charged" (*7*).

Fundamental objectives of highway user taxation were presented in 1954 by Richard Zettel (*8*). The list, which seems comprehensive, includes (a) tax equity, (b) tax neutrality, and (c) standards for highway investment. Equity is an objective because highway services are distributed unevenly through the society, and society seeks compensation from each user of the services in proportion to benefits received. User taxation is intended to make tax policy neutral in the choice among transport alternatives, by requiring compensation for the cost of providing services to those who benefit, while not forcing taxpayers to pay for services that they do not use. As an investment standard, user taxation establishes a connection between the expenditure of funds and those who provide them, thus offering a test of how users evaluate the investment of resources on their behalf. The economic virtues of this service are two: it opens a channel for making the demands of users effective, and it supplies a restraint on spending beyond the point of economic gain. Indirectly, user taxation contributes to the intelligent investment of highway funds by setting up a dependable source of revenue as a basis for highway planning, a revenue source related to the principal factors used in estimating highway needs.

These objectives are those of ideal user taxes, the purposes to be sought in practice. This list of advantages is founded on "a conception of the highway function as fundamentally different from other functions of government," as Zettel says; a conception of road policy as a form of public enterprise, operated in a manner analogous to a public utility business, and financed by the specific beneficiaries rather than the general public. Although the terms "equity" and "neutrality" refer to public financing rather than the market economy (because highway money is raised by taxation rather than by pricing, excepting road tolls), the similarity between user taxes and transportation prices has been often noted, and with justification. The public regulation of transport rates and fares has as its objective that charges afford a fair competitive choice among routes and services but not be unduly preferential or prejudicial to particular buyers. Further, transport prices, like other prices, are meant to "effectively discourage the urgency of all those whose demand for the goods or services in question is less than their disinclination to pay," which confines resources to satisfying wants of the greatest urgency (*9*).

In considering the practical application of these concepts, Zettel discovers such imposing concessions to realism which must be made to accommodate the total philosophy that he concludes:

> For the present, at least, user-tax analysis provides no more than a rough guide to the economic justification of any proposed future highway program. Its principal merit, as we have suggested, is that it incites the active interest and participation of users themselves in the highway function.

Nevertheless, the broad idea that "users pay" for roads has tended to sweep aside a great many other misgivings about the wisdom of highway expenditures which might be expressed if funds had to be voted annually as general appropriations. The commercial principle, or the special benefit basis, has been the underlying economic rationale for the supply of highway money. It has been advanced to support the main pillars of highway finance: taxation in proportion to vehicle ownership and use, earmarking of the proceeds for highways alone, and assignment of main highway costs to users rather than general taxpayers.

These policies are seen as equitable because motorists are required to pay according to vehicle size and distance traveled, the principal elements of highway use. Because payment is not demanded through general taxes if alternative means of transport are preferred, the method of raising revenues is considered neutral. And earmarking of the proceeds requires that the funds be expended only on highways, so that the payments reflect the cost of providing roads, and not the cost of any other public activity. Thus, the link between payment and expenditure is sealed. If one were to judge by the frequent references made to commercial principles, their influence in highway finance policy would seem to be substantial. C. D. Martin, for example, Undersecretary of Commerce for Transportation, spoke to the Congress in support of proposed Federal user tax increases as follows:

> It is a sound business principle that such a highway plant be paid for by charges to its customers at rates which are related as closely as possible to the amount and kind of service received by the user and the cost of providing that service. In the case of our highway system, the State and Federal governments which provide the service must assess and collect charges for use of the facility just exactly like any other business concern.
>
> In this sense, what we are talking about at this hearing is not taxes as such, but rather a schedule of rates to be charged the various users who receive varying amounts and kinds of service. (*10*)

The rational motorist, it may be presumed, would prefer to pay for improved roads rather than their absence, and this idea briefly sums up the purpose of highway user taxation —to allow the motoring public to "vote" on the size of the investment in roads it desires through tax rates that reflect the reasonable expenses of road building and all other relevant costs. In the public enterprise situation, market demand ("voting" with dollars) is apt to be a more exact measure of popular choice than political balloting, if the institutional setting is properly arranged to foster fair competition and economic efficiency. Such, at least, is the theory summoned in support of taxing for highways in proportion to use, tax earmarking, and reliance on users as the primary source of revenue.

Highway finance theory is only half-satisfied by the appropriate form of taxation; the tax objectives must also be served by the manner in which the money is used.

Zettel points out the fiscal question inherent in the distribution of user revenues among regions and road systems: "Perhaps the greatest weakness of user taxation is that it cannot be adapted to the variability of highway costs in terms of service units." To the extent that there is variation in costs and uniformity in price, the expenditure of funds cannot be strictly in accordance with relative road use, if needs are to be met. Instead, surplus earnings on some roads will have to be shifted to meet deficits on other roads. "The essential public decision to be made is the point at which the disparity between costs and earnings on particular facilities is so great that it is unreasonable to draw earnings from the rest of the highway system to make up the entire difference." (*8*).

User taxes, in other words, are not prices in a competitive market that allows the buyer to value the services of roads in relation to other spending choices. Imposed uniformly over a wide taxing jurisdiction, they cannot be adjusted to the costs of individual roads, so that the motorists does not always pay precisely according to the cost of service rendered. His payment, if it does coincide with cost, indicates a preference for what is offered to him rather than not traveling, but it does not indicate his desire for road service of a certain quality and may only imperfectly reflect his choice of highway transport over other means of movement. These factors can be accounted for in a determination of highway needs, but the transfer of surplus earnings to meet deficits—a process known as "cross-subsidization"—may have a strong influence on how road requirements are evaluated.

It has not been the custom of highway finance analysts to delve deeply into the ultimate destiny of highway revenues, once they have been accumulated. But given today's highly centralized financing and the ability to exercise economic power by shifting funds among regions and road system, it seems important that inquiry be made as to whether policies for distributing user funds are equitable to motorists, impartial to competition, and conducive to rational investment decision making.

To do this, the effect of apportionment decisions on the calculation of needs must be considered. First, it is necessary to examine some of the techniques and assumptions of highway investment evaluation, particularly as they are affected by the availability of money.

Economic Principles of Highway Investment Analysis

In a very rudimentary sense, the principle of highway capital expenditure evaluation is that no investment project is economic unless the benefits it provides are equal to, or in excess of, its costs. At any point in time, those projects have first claim on funds that display the greatest excess of benefits over costs, but over the long run the purpose is to complete all projects for which an economic "need," evidenced by a positive benefit-cost ratio, can be shown, if funds for doing so are available.

The Savings Concept of Benefits.—The rating of existing roads for their adequacy to serve present traffic is the initial step in assessing the need for highway improvements. The improvement warrant, Campbell explains in an exposition of the engineering evaluation of highways (*3*), is a diagnosis that shows, after a rating has been completed, that a change is desirable. The warrant for action does not prescribe the cure; it merely says that action should be taken because the present facility has worn out or is otherwise inadequate.

The ratings lead to the formation of short-term improvement programs. Their usefulness is greater if preliminary assumptions can be made about some of the broader aspects of highway policy (about improvement standards, over-all highway need objectives, the financial capability to meet needs, etc.) for with these goals and limits given, short-term programs can be framed to meet long-term objectives. But this is a two-way avenue: the rating of roads is a necessary first stage in the preparation of long-range need estimates.

It is also suggested that the sufficiency rating for a road is not an economic rating, but this concept needs clarification. It is difficult to imagine a road on which "action" should be taken according to the rating standards, but for which there is insufficient economic justification for the action. Granted that the rating itself does not show exactly what improvement should be made when there is choice among alternatives, some of which might be economically justified and others not, a highway rated as "deficient" would appear to need at least a minimum correction. Yet, there is an important economic judgment in this idea; namely, that traffic demand will continue in the future. In the case of a deficiency due to insufficient road capacity rather than natural depreciation or obsolescence, the economic assumption is that traffic will continue at least at its present volume. No deficiency could be shown if traffic were expected to decline abruptly in the immediate future.

An illustration will help explain these points. At intersections, the right-of-way must be assigned or apportioned among cars entering on the feeding streets unless vehicle volumes are low. A stop sign is a simple means of assignment. Traffic signals require more expense. Warrants for installing traffic signals are generally related to the volume of vehicles (or pedestrians)—when traffic flows become sufficiently large to create undue delays for vehicles, on the main street or the side street, or cause frequent accidents, signals are considered justifiable. A far more elaborate improvement for intersection control is the grade separation. It is possible that consideration of future growth of traffic might suggest that an expenditure for this purpose would have greater justification than for a signal which could become outdated within a few years. Thus, the minimum improvement of a signal may be warranted because traffic volume has reached a certain level, but a more ambitious improvement could be proposed to serve the level that future traffic flows will eventually attain.

The economic evaluation implicit in the assumption of a given traffic volume is that the "savings" to motorists in vehicle operating costs, time, risks, and other factors that could be credited with monetary value are larger than the cost of improvement. Therefore, an expenditure upon a road intended to provide savings in excess of cost will bring about a reduction in the total cost of movement to motorists and would be accepted by users as an economic gain. This is true of the minimal type of road improvement needed to eliminate a deficiency.

A more complex decision is required when there is a choice between alternative actions, a choice which depends upon the growth of traffic expected in the future. What value will future traffic place upon an improvement? It cannot be said that the value is necessarily measured by the savings possible to present users from the improvement. Savings cannot be credited to non-existent vehicle use, except by an assumption. This is the future traffic would appear even in the absence of improvement, or at least no more improvement than necesary to maintain the existing level of traffic service.

The Value of Travel.—About the existing, measurable flow of cars, it is known that all drivers value their travel at least enough to accept the time, operating, and other costs prevailing with the present facility, in preference to not traveling, so that any reduction in these costs, if greater than the expenditure needed to effect them, would be counted as a gain by the average users. (It must be conveniently overlooked that not all users are average—not all, for instance, place the same value on saving a minute.) By using the savings to future users as an economic justification for improvement, they are automatically credited with the same value on their travel. There is an identity assumed between the present flow of vehicles, which can be measured, and the increased flow in the future, which must be forecast.

In most instances of correcting road deficiencies, this assumption is not unreasonable. Existing traffic flows originate with the basic causes of traffic generation and interchange (population, land use activities, etc.). If the basic producers of traffic increase in size, a similar rise in travel demand can be anticipated, and if the relationship between traffic and land use remains roughly the same, there is every reason to assume that new users would look on savings from road improvements in much the same way as present users. Nevertheless, the more that predicted traffic becomes a variable factor in the planning of highway improvements, the more economic significance is acquired by the assumption concerning the value of travel to the motorist.

There is usually a large quantity of savings to users along a heavily congested route due to highway improvement. But the rigors of driving prior to such an improvement may also discourage a certain amount of road use. New highways attract new traffic, as well as benefit existing users. It would be an error to credit the full value of savings to those motorists who are specifically induced to travel by the road improvement itself. Previous to the improvement, the cost of travel was higher to the induced motorists than their value upon taking a trip via that route. Therefore, savings would overstate the actual size of the benefit.

To look at the matter another way, the objective of much long-range highway planning is plainly to prevent a chosen standard of traffic service from deteriorating, rather than to improve it. For example, a mainline two-lane highway may become inadequate to carry traffic if normal traffic growth continues, and hence requires conversion to a multi-lane facility. The requisite multi-lane road may be divided or undivided, may have partial or full access controls—there is a range for choice, depending on anticipations about future traffic growth. The problem is to proportion the number of lanes and other design features of the road to the volume of traffic that is forecast. Now there are no "savings" to be shown over the existing road; the optimum road improvement permits vehicle users in the future to enjoy, at minimum reasonable cost, the same quality of movement as was provided by the existing two-lane facility. Consequently, the optimum improvement by allowing sufficiently for traffic growth does provide a substantial benefit in road service to motor transport, the value of which is realized to its fullest by forestalling a diminution in traffic service values.

Perhaps this is most apparent in the urban context. The conversion of routes from city street to freeway travel standards offers an unquestioned quantity of savings to the users of surface streets with which to offset investment costs. But large-scale urban highway planning is now often involved with determining the amount of freeway capacity that must be built to preserve the ability of the freeway network to provide unobstructed services to motorists. The justification, especially as the mileage of freeway increases, lies less in the benefits to motorists over surface street movement and more in the value of keeping a balance between the number of freeway lanes and the demands of motor vehicles.

The transition in thought, from determining the best facility to serve a given traffic volume to proportioning the total investment in roads to a variable traffic volume, does not always occur in engineering studies. (The economic model portraying the difference between minimizing cost for a given output of highway transport services and adjusting plant to a variable output is excellently presented by Nicholson (*11*).) It is the most conservative type of highway planning to calculate investment needs only to the extent necessary to bring roads up to tolerable service standards for present traffic volumes. Intelligent planning requires consideration of the economic life of an improvement, at least to the extent that predictions of traffic can be made with confidence.

Recognition that the benefits of direct "savings" are insufficient to evaluate the return on forward-looking highway investment has stimulated efforts in highway analysis to enlarge benefit concepts. The total value of the transport service of highways is revealed by the benefits conferred on the economy and the community at large—by the "distributive" and "spillover" effects of highway spending. Recently, the encouragement given by highway improvement to productive enterprises and commercial activities, along with the stimulation of roads to land values, has been given much attention. An enlarged scope for social activities and other "nonpecuniary" effects has also been mentioned. This widening view of highway transport values has opened a fertile field for economic research and theory which will be increasingly exploited by analysts in coming years. However, there is a danger in the broad approach: it requires a vast knowledge of interrelationships in the economy not now possessed, and the quest for certainty may, as Hitch observes, lead to such excursions into remote secondary effects that "perfectionism can stultify otherwise good economics" (*5*). There is much to be said for relying on "expected values" in the face of uncertainty.

More to the immediate point, it is important that these newer benefit concepts not be misrepresented. To show the total return on a highway investment, there is a tendency to add the distributive and spillover values to the "savings" realized by highway users, but this is a most unwarranted combining of two different ideas of value. It confuses the value of savings with the value of the travel to the users. The indirect effects of a highway improvement, if they can be confidently traced to the fact of an improvement in service, suggest that new users have found the total cost of vehicle movement, including the expenditure on the road, sufficiently lowered to make worthwhile travel that otherwise would not have taken place. It would be a distinct error to add savings for nonexistent traffic to those values that explain why new traffic growth occurs; this is "double-counting," as some writers have pointed out. It would be legitimate to add direct savings accruing to present users to secondary values, as long as all indirect benefits could be associated with new users.

In economic terms, the excess of travel valuation over the highway expense needed to make savings available and other costs of highway movement is a "consumer surplus." For all present users, consumer surplus is increased by the amount of savings they realize from a project. For induced users, the surplus indicates a value of travel in excess of the total cost of movement.

The Role of Money.—One reason for a broader focus on highway benefits has been a growing dissatisfaction about the role of money in investment evaluation. Money is usually considered as an external fact: its availability, or lack of it, indicates whether an investment project is financially feasible, or to what extent there is "fiscal capability" for carrying out a highway program. But as an index of value—as a guide to economic desirability, which is its normal function in economic theory—money does not often enter directly into engineering evaluation.

Nevertheless, some assumption about it cannot be avoided. The customary benefit-cost evaluation of highway needs is framed to show how a given sum of money may be expended most efficiently among alternative spending projects. The starting point is a budgetary constraint of some kind. The size of the constraint may directly influence the standards used for calculating needs; there is a clear disposition among engineering planners to equate an increase in the amount of funds available with a decision to upgrade the quality of the service desired by highway users. On a more elaborate scale, if highways are being planned as a unified system of roads, the needs of any single project cannot be definitely stated until it is known how large a system can be built with the money supply in prospect. As a practical matter, it is desirable as a basis for intelligent highway investment planning that there be a predictable supply of funds from established final policies.

The budgetary constraint is equivalent to the assumption of a fixed traffic volume for highway benefit evaluation; indeed, if a user charge philosophy is accepted for finance, the two come to much the same thing. A fixed flow of funds indicates a public preference to spend at least that sum of money upon highways. Since the source of money is highway travel, the amount available could be interpreted as a measure of the minimum value upon highway service of users, in preference to not traveling by motor vehicle. If this were correct, then any improvement in the quality of service—faster speeds, reduced accident risk, etc.—which produced "savings" in excess of the expense would leave users better off.

In economic terms, the demand for travel would be considered inelastic, to the extent that savings were submitted as a justification for expenditure upon highways. If there were some doubt about the willingness of users to pay highway taxes, the budgetary constraint could not be fixed until the elasticity of the demand had been determined.

For a region within a taxing area, the budgetary constraint is the result of apportionment policy among governments and road systems. It reflects not only what users are willing to pay en masse, but also the decisions about how to employ the revenues. These decisions may be based upon a blend of considerations about how much users would be willing to pay for highway travel if given the opportunity to do so, and the opportunities for achieving greater efficiency in vehicle movement by reducing total travel costs to users.

Actually, neither economists nor engineers have been particularly eager to take on the task of evaluating the necessity of road expenditures, preferring instead to let their own predictions be guided by the standards used in the other field. Thus, the results of engineering determinations have been viewed from the fiscal side of the fence as evidence of the extent that motorists need roads. The engineering results serve as the basis for setting user tax rates, dividing tax responsibility, and arranging that the total supply of funds be guided to the point of need. Engineers have seen the flow of funds as evidence that users want highways, the main object of engineering valuation being to assist in providing the optimum facilities at a minimum reasonable cost, thereby securing the most benefits for the money available. In this peculiar impasse, it is necessary at some point that, as Campbell says, "fiscal and priority planning must lock step."

Position of Fund Distribution in Highway Planning

The appropriate middle ground is being sought in the increased attention given to capital programing of highway funds—to a determination of how large sums of money should be placed to yield the highest return on investment. New techniques of systems analysis, borrowed from fields other than transport, appear to have a profitable application to the problems of programing; *e.g.*, (*12*). The programing approach requires dividing long-range plans into time periods, road systems, and travel purposes to facilitate analysis.

A further division of planning is found in the growing popularity of regional master plans for transport, which take account of all travel purposes and means of movement within a limited region. The "integrated" plans begin with a broad pattern of land usage, continue by predicting the generation and interchange of traffic from this pattern, proceed to determine the transport facilities required for handling the expected flows of vehicles and people, and end by estimating the cost of the proposed transport system, including highways. Eventually the planning mill grinds out an estimate of highway need that, if complete, embraces all requests for the highway tax dollar.

Highway apportionment policy occupies a strategic position with respect to these efforts at better investment planning. Fiscal policy has the power to establish the financial feasibility of the plans. Unfortunately, shelving fiscal considerations to the late stages of system planning for a region has sometimes revealed a wide disparity between total needs and revenues, and unless the difference is made up by a tax rise, all that can be said about financial feasibility is that a certain percentage of the total need may be met. Moreover the percentage may not be the same for different elements of the plan: funds may be overly concentrated on arterial freeway facilities, which could upset the desired balance between freeways and local roads, parking facilities, and public transit.

The tendency is for road planning which is carried on independent of the financial factor, as is true of regional road planning, also avoids a stern analysis of the economic basis of highway need. Instead, this approach invites a direct transition from technically-estimated costs to a finding of financial feasibility, depending on whatever sources of revenues happen to be available. It is natural that highway agencies would be somewhat compelled to develop need estimates sufficient to absorb the anticipated flow of Federal-State funds to their locality. If it is believed that the statement of highway requirements will directly influence the proportion of the total distribution going to each region, quite liberal estimates of need may be submitted to the central government.

To place highway fund apportionments in their proper economic perspective, the concept of financial feasibility must be broadly interpreted. What the financial feasibility of highway plans may indicate about their economic feasibility should be determined. A study in the water-resource field (*13*) says that the financial feasibility of a water project depends upon whether the project "generates revenues that suffice to cover all costs, including interest on funds borrowed to finance the project." Its "economic feasibility" is determined by whether "the economic valuation of the 'benefits,' to whomever they accrue, exceeds the economic valuation of the 'costs,' to whomever they accrue," when both costs and benefits are discounted to a given year. If there is a divergence found between economic and financial feasibility—between the evaluation based upon costs versus benefits, on the one hand, and actual outlays and receipts, on the other—further consideration would have to be given to the reason for the difference. An investment project which was not financially feasible, could be acceptable economically if government in its decisions took account of costs and benefits, real or alleged, which would not enter into the calculations of private investors.

Following this line of thought, the financial feasibility of a highway investment, or group of projects, would depend upon whether the user revenues earned from travel on the facilities covered their costs. The obvious defects of highway user charges as prices would not ordinarily offer a private investor the test of financial feasibility which would be provided by road tolls on single projects. But the total budget for highways can be as large as the revenues generated collectively by all projects, past and present. A project which does not produce enough revenue to cover its costs can be made financially feasible by a judicious transfer of surpluses on other roads to meet the deficit. When funds are thus distributed by apportionment methods to establish the fiscal feasibility of highway plans, the government is using cross-subsidization among highway users. This course may be justified on grounds of economic feasibility or social necessity, but it should also be asked whether the policy is consistent with the fiscal objectives of highway user taxation.

The Rural-Urban Apportionment Problem.—For example, before World War II, nearly all user tax collections were channeled to rural regions, despite evidence that almost one-half the revenue originated within cities. Although much usage of main rural highways represented travel by urban residents between cities, and although a strong case could be shown for giving priority to the improvement of the main intercity highway network, a good share of the urban-produced funds were expended on local rural roads, whose initial benefits were received primarily by farmers and other rural residents. This type of geographic apportionment also seems to have violated the neutrality standard of fair transport competition. For in the torrid rivalry generated between railroads and intercity trucks, the latter apparently enjoyed the benefits of support from the broad urban highway tax base. Even though the redistribution of urban monies might be interpreted as providing mostly for local rural roads, the motor trucking industry, along with other intercity motorists, was thereby relieved of the costly "branch" and "feeder" routes which plague large transport systems in all lines of transport. Finally, it is doubtful whether the rural allocation represented the most rational investment of funds.

A contemporary observer of the prewar scene noted that for a number of years there was no dispute over the diversion of urban funds to rural areas. Urban residents, in fact, were leaders in promoting the improvement of intercity rural highways; however, the growing inadequacies of city streets to carry traffic eventually led municipal interests to urge strongly that urban earnings be retained for use within cities (*14*). At that time, Owen commented that the distribution of user funds "is more often a function of the loudness of the demand for funds rather than of any economic consideration of where the money should be spent. Many states grant little or no part of the vehicle-tax fund to cities where the greatest needs of the motorist are generally concentrated, while on the other hand there is often an over-generous contribution to local rural units and a corresponding incentive to wasteful expenditures." (*15*)

Much of the geographic imbalance between rural and urban areas in the division of highway user revenues has now been corrected. This change was accomplished in part by increases in State tax allocations to cities, even more by direct spending upon State system highways in cities. Most important was the action of the Federal Government in levying taxes upon vehicles and motor fuel and apportioning the revenues on the basis of needs, a large percentage of which were calculated to be within urban limits. Thus, public policy responded to the opinion that the larger economic investment needs for highways were concentrated in the metropolitan regions.

There is adequate cause for considering that this redistribution advanced the objectives of highway user taxation, as well as the benefit-cost principle. The alleged inequity to urban motorists has been reduced as the allocation of funds to urban regions has been brought more into line with urban earnings.

But the case is far from clear, when basic differences are observed in the road system of rural and urban areas. The local rural road network is usually quite costly to construct and maintain, in relation to the number of vehicle-miles it handles. The unit costs of the main rural arterial system are lower. Consequently, if there is cross-subsidization between road users within the rural region, it occurs when surpluses from main highways are transferred to the support of high-cost local roads. In contrast, some of the most expensive highway facilities now being built, both in total cost and cost per vehicle-mile, are the freeways in large metropolitan areas; whereas the urban surface street network, having the primary function of giving block-by-block access to properties rather than facilitating vehicle mobility, generally produces surpluses in highway user earnings. To accentuate this difference, the municipal surface streets have a far larger percentage of their total cost met from non-user tax sources than local roads in rural regions.

There is much evidence (even though the facts are difficult to establish) that the large quantity of surplus earnings from metropolitan city streets are now being directed to the support of urban freeways. Assuming that the total earnings of a metropolitan area are retained within the region, does the use of money primarily for freeway building accord with economic investment principles? It is true that arterial highway service in large cities, if it is to be efficient, must be fairly unobstructed. But the transfer of funds between road systems has also favored a highway solution to the handling of peak period traffic loads. Certain questions may be raised about this policy.

Is it equitable that the costs of commuting by highways be distributed among all motorists? At least one class of highway user (motor truckers) are not a major factor in the urban peak increment; indeed, it appears that they make special efforts to avoid the worst periods of the rush hour. Then, too, other vehicle owners who do their commuting by public transit might find the situation unsatisfactory.

From the standpoint of economic investment, the construction of freeways whose prime justification lies in relieving peak hour congestion on other freeways requires large expenditures for a travel purpose with extremely high marginal costs. Also, the support to peak users from off-peak travel and surface street usage is manifestly not neutral with respect to public transit, which must derive its main economic support from the commuter demand.

CONCLUDING REMARKS

The basic problem of geographic apportionment is to devise consistent rules and criteria for allocating funds among separate regions and districts. Highway finance analysts have felt disposed to leave the question of apportionment largely to the determination of needs because the calculation of road requirements can allow for a certain degree of diversity among regions, whereas tying the allocation of funds to a specific (such as vehicle registrations) leaves highway policy inflexible for meeting different conditions. However, there are distinct difficulties in attempting to estimate and evaluate highway needs completely independent of the sources of finance.

It would be an immense simplification for all concerned with highway planning if the highway-vehicle relationship were in perfectly fixed proportions—if each unit of highway use required exactly the same expenditure for road space. A distribution of funds in proportion to relative highway use in each region would then meet the needs of motorists and would return user tax earnings to their source for the benefit of those who paid them; over time, both taxation principles and the standards of investment efficiency would be satisfied, and the difficult and sensitive policy issues raised by transferring tax funds from one place to another would be overcome. The budgetary process for highway money would still demand intelligent management, but it would be mostly mechanical, without presenting any substantive policy questions for legislative decision.

Because this is not the case, a central highway authority must face the fact that the same engineering standards, fiscal methods, and other highway practices have to be arranged to meet the extremely diverse conditions of transport among regions. The fiscal question in distributing money among regions is whether there are differences that would cause a conflict in standards of investment need and finance—whether economic feasibility, as understood in terms of benefit-cost concepts, agrees with financial feasibility as based on the objectives sought through highway user taxation.

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Collecting Statistics on Vehicles in Use

FREDERICK E. MAY, *Assistant Professor, School of Business and Public Administration, University of Missouri, Columbia*

• IN EARLY 1960 the Bureau of Business and Economic Research at the University of Missouri was asked by the State Highway Commission to prepare a report to the State Legislature on future financing of Missouri roads. The author made the 20-yr projections of indicators of highway use. These forecasts of the behavior of three major indicators—motor vehicle registrations, fuel consumption, and motor vehicle travel—became the basis for financial projections of registration fees and motor fuel taxes.

During the preparation of the forecasts, the unavailability of inadequacy of certain data series that were requested from the Missouri State Highway Department became apparent. Some examples of major questions needing more adequate data were the following:

1. What effect would compact cars have on registration fees, fuel consumption, and travel?
2. What impact would multiple car ownership have on travel mileage and fuel consumption of second and third cars?
3. What effect would piggy-back transportation by railroads have on the number of trucks, their weight, and their zone of registration—local or beyond local?
4. How much travel in Missouri was done by out-of-state vehicles and what was their impact on fuel consumption and travel mileage? Was the trend for out-of-state travel different from domestic travel?
5. What effect did the present tax differential between Missouri and surrounding States have on fuel consumption by out-of-state vehicles and domestic consumption?
6. What were the trends in fuel usage of diesel gasoline motors?

These are some of the more complex questions for which answers were desired for making projections and they would have required either cross-tabulations of data that were simply not available or new data that had never been gathered. However, it was apparent that even the major series of data to be projected (that is, motor vehicle registrations and travel mileage) might need improved accuracy and reliability.

As a result, two studies were made for the Missouri State Highway Department: (a) a survey of the present methods of collecting statistics on highway use; and (b) a design of a procedure for recording vehicles in actual use in Missouri.

In considering the general applicability of the findings reported, it must be kept in mind that this is a case study of Missouri conditions. However, some of the conclusions may have more general applicability insofar as some of the problems encountered are similar to those in other States.

The first general finding to report is that registration data that the State Highway Department receives from the State Division of Revenue are dominated by that Division's concern with immediate availability for use by police and sheriff departments, field collection, and size of the registration form to fit existing file and photostatic facilities. Relatively little concern is shown for the data needs of highway engineers and planners.

To be more specific, Missouri adopted the "continuous registration" procedure for passenger cars a few years ago. This method spreads the actual registration of passenger cars as evenly as possible over twelve months of the year. The method is relatively new and has been adopted in several States in the last few years. Although it seems to be an excellent procedure from the standpoint of the registration authorities, it appears to present some rather difficult problems of interpretation and analysis to those who would use the information for statistical purposes.

There is, for example, the problem of distinguishing between the concept of a "flow" or "volume" and a "stock" or "inventory." The number of vehicles in use (or registered) is a stock. A certain number of vehicles exists on any given day and if technically feasible, that number could be counted on any one selected day. The size of this stock of vehicles in use changes, of course, from day to day. Newly manufactured vehicles enter the stock of vehicles in use through automobile dealers, and for any particular State, used vehicles come in from other States. Vehicles also disappear daily by out-migration from the State to other States and by scrappage. However, vehicles in use (or registered) are a stock that, like any population of human beings or animals, is increased by births (new manufactures and in-migration) and decreased by deaths (out-migration and scrappage). "Vehicles in use during a given year" means the average level of the inventory of vehicles as it was composed of 365 individual daily stocks of vehicles in use.

The number of vehicles registered at the end of the year includes scrapped vehicles as well as out-migrations during the year and is, therefore, likely to exceed the number of vehicles in use at the end of the year. If it were technically feasible, one would like to be able to take an inventory or count of all vehicles in use on any one day during the year. The more commonly used registration methods come closer to this type of cross-section count by requiring owners to register their vehicles by a certain date, usually early in each year. R. L. Polk and Company stops counting registrations about midyear and thus probably avoids some of the counting of scrappage and out-migration.

With the "continuous registration" procedure, there exists a danger that registered vehicles are regarded as a "flow" or "volume" rather than a "stock" that exists on any one selected date.

This is the problem in Missouri where the Department of Revenue (Motor Vehicle Registration Division) reports only the renewal notices for passenger cars to the State Highway Department. More precisely, the Motor Vehicle Division files the registration of passenger cars by the month in which the car is registered. One month before the renewal of an owner's registration is due the department sends out a notice on an IBM card which serves both as a reminder and as an application for re-registration. It is a copy of this renewal notice which the State Highway Department receives.

Thus, the State Highway Department is not notified at the time of original registration but only eleven months later that a car has been registered that apparently is still in the same owner's hands and has come up for re-registration or renewal. For example, a passenger car is registered by its owner in December 1960. The State Highway Department is notified in November 1961 that the car is ready for re-registration. The statistician then counts this car as part of the 1960 registration. No particular confusion of stock and volume exists in this example. However, the Highway Department statistician has had to wait eleven months beyond the end of 1960 to receive a complete record of 1960 registrations.

As an example where confusion over flow and inventory concepts arises, the following illustration will serve to point up a common statistical difficulty.

Sometimes the owner of the car registered in December 1960 buys another one during 1961. For example, he may have bought a new 1962 model in September 1961. This transaction was then handled as a transfer of license plate to the new car, for which the owner paid a transfer fee. His renewal again comes up in December 1961. By November 1961, the Motor Vehicle Division has moved the owner's card, showing the vehicle registered in December 1960, from its primary place in the file and has placed the new 1962 model car in the primary position. The owner and the State Highway Department now receive a copy of the renewal notice which identifies the 1962 model car bought in September 1961 as the owner's car. The State Highway Department statistician, however, must now make one of several alternative decisions:

1. Assume that all renewal notices mailed out in November 1961 represent the December 1960 registrations, and thus place a 1962 model car into the 1960 registration year.
2. Trace back all transactions that took place during the year to avoid the previous type of error; or
3. Assume that December 1961 renewal notices are part of the December 1960 to December 1961 registrations, and thus fall into the flow vs stock error.

A somewhat extreme example has been chosen, using a new model car (1962) that ends up in the 1960 registration, a year during which it obviously had not been manufactured as yet, to make clear the basic difficulty encountered by a system of using renewal notices to accumulate detail registration records at the Missouri State Highway Department.

Even without the special difficulties encountered in Missouri because renewal applications rather than original registrations are used for detailed analysis by the Highway Department, the "continuous registration" procedure presents problems to the statistical analyst. If an original registration file is accumulated by the State Highway Department, then this file must be purged either monthly or annually for duplications that enter because of changes in ownerships. That is, the February file must be compared to the January file and all ownership changes that occurred in February but also registered in January pulled out. In March, the entire procedure must be repeated to select all ownership changes in March who also registered in January or February. This procedure, or a similar one, would have to be used every month through December each year. Each time the files are compared, the number of individual items increases, of course, until by December about 1,400,000 individual items must be run through in Missouri. The Missouri State Highway Department apparently adopted the renewal notice procedure primarily because it would have been faced with this large and costly collating task.

It has been estimated that only 75 percent of the registration data represent currently accurate information. The other 25 percent are duplications due to the practice of

TABLE 1

A COMPARISON OF ANNUAL U.S. PASSENGER CAR REGISTRATIONS[1] AS COMPILED BY U.S. BUREAU OF PUBLIC ROADS TO ESTIMATES OF PASSENGER CAR REGISTRATIONS AND CARS IN USE BASED ON R. L. POLK AND COMPANY DATA, 1954-1960

Year	U.S. Bureau of Public Roads December 31	R. L. Polk and Company			Difference Between Estimates		Ratio of B.P.R. to Polk	
		Cars Reported Registrations July 1	Registration Estimate to End of Year Adjustment 1[2]	Cars in Use Estimate at End of Year Adjustment 2[3]	Col. 1 — Col. 3	Col. 1 — Col. 4	Col. 1 ÷ Col. 3	Col. 1 ÷ Col. 4
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1960	—	57.1	—	—	—	—	—	—
1959	59.6	55.1	58.0	53.7	1.6	5.9	1.03	1.11
1958	56.9	52.5	54.8	52.0	2.1	4.9	1.04	1.09
1957	55.9	51.4	54.3	50.1	1.6	5.8	1.03	1.12
1956	54.2	49.8	52.7	48.3	1.5	5.9	1.03	1.12
1955	52.1	47.4	51.0	46.7	1.1	5.4	1.02	1.12
1954	48.5	—	—	43.9	—	4.6	—	1.10

[1] Source: "Automobile Facts and Figures," 1958, 1959-60, 1961 Editions, Automobile Manufacturers Association. Number of passenger cars in millions.

[2] Polk reports registrations as of July 1, whereas BPR registrations are reported as of December 31. To reconcile these registration estimates, the number of new passenger cars registered during the first six months of each year was added to the Polk midyear registration data.

[3] The number of new passenger cars registered during the first six months of each year was subtracted from the Polk registration for July 1 of that year to provide a comparison of estimated cars in use at the previous December 31 to annual registrations reported by BPR for the previous year.

pro-rating or to changes that have not as yet entered the registration file or registrations that come in after listings have been prepared.

As to commercial vehicles, the State Highway Department receives the complete original registration record in the form of a deck of IBM cards sometime during February or March the year following the registration year. Because all commercial vehicle owners must register in January of each year, these data for most commercial vehicle registrations are again over a year old. Of course, the truckers have made numerous changes from the original registration because of changing needs for their operation from "local to beyond local" or from one type and weight combination of tractor trailer to another. At least these changes have been taken care of by the time the State Highway Department receives the data. However, a specific type of vehicle originally registered may be in use for only a part of the year. At present only the last registered vehicle is left in the IBM deck under the vehicle number. It then becomes a huge task to try to match the various license numbers and to bring together a composite picture of the various ways in which the particular owner has used the vehicle in question during the year. In this case it would seem to be desirable to prorate the type of vehicle in use on a time basis and in fact to establish a weighted composite that would reflect the actual use made of the various combination units.

A series of vehicles in use seems a much better base with which to correlate fuel consumption and travel mileage of vehicles domestic to the state than are registration figures. Registration figures are good for financial planning of registration fees. The number of reported vehicles as registered in a 1-yr period contains, however, a substantial number of scrapped vehicles. This scrappage may run as high as 8 percent and duplicate registrations which slip in for various reasons may raise the difference between vehicles in use and registrations to perhaps as much as 20 percent.

Table 1 compares U.S. passenger car registrations as compiled by the U.S. Bureau of Public Roads to a reasonable estimate of the number of vehicles in use based on adjusted R. L. Polk and Company figures. Because Polk runs its registrations only to midyear and scrappage is relatively small during the first half of the registration year, the Polk data are assumed to be closer to "vehicles in use" at midyear than the year-end registration total reported by the Bureau of Public Roads. However, because the midyear closing date used by Polk is not comparable to the end-of-year date used by the Bureau of Public Roads, the actual Polk registration data have been rolled back to the beginning of the year by a subtraction of new car registrations during the first 6 months of the year. It must be borne in mind that the adjusted Polk data still include the first 6 months of scrappage. Therefore, the difference between adjusted Polk data and Bureau of Public Roads data allows only a rough and minimum estimate of the difference between vehicles in use and vehicles registered. For the years 1954 through 1959 the registration data compiled by the Bureau exceeded the adjusted Polk data between 9 and 12 percent each year.

The second major finding concerns the traffic count data that serve in part at least to establish travel figures. There exists a natural conflict between the needs of the design engineers and the needs of the economic and financial planner. Design engineers need information on specific segments of the road network and because design of roads is a day-to-day job it is easy to understand that the planning division concentrates on traffic counts suitable for this type of work. Travel information is needed less frequently and has less immediate visible value. It is useful for longer range planning studies.

Traffic counts designed to develop travel information do not have to be made on each and every specific segment of the road network. Travel information could easily be developed from a sampling of a small part of the entire network. A carefully designed sample can develop more accurate information than the present setup whereby travel is counted very frequently on some roads and not at all on others. In the past, Missouri has concentrated traffic counts on the rural State network. Relatively little work has been done on the county roads but it may be considered reasonably adequate in view of the relatively small contribution that the county roads make to total travel mileage. However, the program in the cities needs to be augmented. It is totally inadequate for developing travel mileage figures. They are derived from the difference between statewide estimates and the total of separate estimates for the rural State and county systems.

Conclusions and recommendations for policy on collecting statistics on indicators of highway use are the following:

1. In an over-all evaluation of the traffic counting program and the motor vehicle registration data, one should theoretically be comparing the value of the statistics to the users; that is, their utility to the cost of producing these data. It is most difficult to assess the specific value of the data to their individual users and only slightly less difficult to rank order these in terms of value to their users.

2. As the program functions at present, it appears that the highest value is placed on individual design requirements and the service rating assigned on the rural State highway system and on the day-to-day requests for motor vehicle registration information. Travel mileage data insofar as they are obtained come as a by-product of the operation on the State's rural highway system. For the county roads traffic program, travel data contribute the major part of the value obtained but they are not collected in the most efficient and representative manner. In the cities, the traffic counting programs contribute primarily to design of individual State highways. Detail motor vehicle registration data are inaccurate for passenger cars. They do not even check out with the totals reported. They are reasonably accurate but not current for commercial vehicles.

3. Whether the values or utilities presently obtained are the most desirable and whether the amounts spent are balanced to obtain the largest utilities is a difficult question to answer. If, however, a higher value is put on long-range planning and over-all planning for all roads and streets in the State, then the present operation appears to be out of balance with respect to amounts spent on specific areas and values obtained. A better balanced program would call for substantially more and better quality motor vehicle registration, fuel consumption, and travel estimates and more attention to the coordination of the design problems of the entire road network of Missouri.

DEPARTMENT OF DESIGN

Influence of Vehicle Speed on Pavement Deflections

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• AS A PART of the series of tests performed on the AASHO Road Test at Ottawa, Ill., measurements were obtained of the actual pavement deflections for a range of vehicle types and vehicular speeds. In all cases it was found that the deflections decreased as the speed of the vehicles increased. Similar results have been reported on tests at Road Test One-MD (*1*) and in a study of the subgrade support characteristics on US 52 in Indiana (*2*). Of interest in the present paper is the explanation of the mechanism responsible for this phenomenon.

FORMULATION OF PROBLEM

Regardless of the simplifications made in the analysis of a pavement system subjected to transient loads, at least the following factors must be considered: (a) the nature of the forcing function, (b) the inertia of the pavement and its support, (c) elastic deformations (restoring force), and (d) time-dependent deformations. As the last three factors represent basic parameters of viscoelastic theory (mass m, spring function k, and damping function c, respectively), it is natural that a solution be sought within the framework of this theory. The particular model chosen for the pavement-support system is the simple Voigt element shown in Figure 1. Although more complicated models could be postulated (*3*, *4*, and *5*) the resulting solutions tend to obscure the controlling mechanism of the phenomenon under consideration.

The loading function will be taken as a rectangular pulse of intensity P

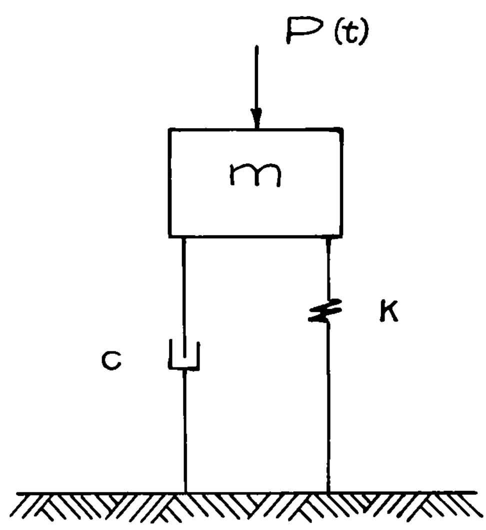

Figure 1. Simple Voigt element.

and of duration t_1 as shown in Figure 2. Here it is assumed that a point in the pavement system is subjected to a load of uniform intensity P during the t_1 seconds required for the passage of the vehicle.

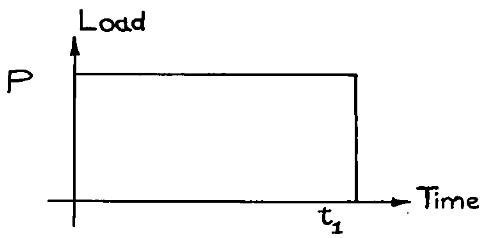

Figure 2. Loading function.

SOLUTION

Taking $y(t)$ as the dynamic deflection, the differential equation of motion is

$$m\frac{d^2y}{dt^2} + c\frac{dy}{dt} + ky = P\,[u(t) - u(t-t_1)] \tag{1}$$

which can be rewritten as

$$\frac{d^2y}{dt^2} + 2aw\frac{dy}{dt} + w^2y = \frac{p}{m}\,[u(t) - u(t-t_1)] \tag{2}$$

in which

$w = \sqrt{k/m} =$ undamped natural frequency;

$a = c/2mw = c/c_{cr} =$ damping factor;

$u(t-t_1) =$ unit step function $= 0$ when $t < t_1$ and 1 when $t > t_1$.

By Laplace transforms the solution of Eq. 2 is found to be

$$y(t) = \Delta_{st}\left[1 + \frac{e^{-awt}}{\sqrt{1-a^2}}\sin\left(wt\sqrt{1-a^2} - \psi\right)\right] \tag{3a}$$

for $t < t_1$

$$y(t) = \Delta_{st}\left\{\frac{e^{-awt}}{\sqrt{1-a^2}}\sin\left(wt\sqrt{1-a^2} - \psi\right) - \frac{e^{-aw(t-t_1)}}{\sqrt{1-a^2}}\sin\left[w\sqrt{1-a^2}\,(t-t_1) - \psi\right]\right\}$$

for $t > t_1$ (3b)

in which

$\Delta_{st} = P/k =$ static deflection

$$\psi = \tan^{-1}\frac{\sqrt{1-a^2}}{-a}$$

Deflection curves of actual highway pavements under loads show that the motion is not oscillatory but rather a creeping back to the equilibrium position. Such systems are said to be "overdamped"; that is, the damping factor a is greater than unity. Taking $a > 1$ in Eqs. 3a and 3b, it follows that

$$D = \frac{y(t)}{\Delta_{st}} = 1 - \frac{e^{-awt}}{\sqrt{a^2-1}}\sinh\left(wt\sqrt{a^2-1} + \phi\right)$$

for $t < t_1$ (4a)

$$D = \frac{c^{-aw(t-t_1)}}{\sqrt{a^2-1}} \sinh [w\sqrt{a^2-1}\,(t-t_1) + \phi] - \frac{e^{-awt}}{\sqrt{a^2-1}} \sinh (wt\sqrt{a^2-1} + \phi)$$

for $t > t_1$ (4b)

in which

$D = y(t)/\Delta_{st} =$ dynamic deflection factor

$\phi = \tanh^{-1} \sqrt{a^2-1}/a$

CONCLUSION

A plot of the dynamic deflection factor as a function of time ($w = 20$ cycles per sec) for loading durations of $t_1 = 0.078$ sec and 0.314 sec with $a = 2$ is shown in Figure 3.

Noting that the time t_1 for the passage of the vehicle varies inversely with the velocity, as say

$$v = \frac{L}{t_1} \tag{5}$$

in which L is the effective length of the loading pulse, Eqs. 4a and 4b can be solved to yield the ratio of the maximum deflection factor, $D_{max} = y(t)_{max}/\Delta_{st}$ as a function of v/wL for any damping factor a. A plot of this relationship is given in Figure 4 for a range of a-values. It is immediately apparent from this plot that deflections decrease as vehicular speeds increase.

A search of the literature was made to obtain a measure of L and w for highway pavements. Deflection patterns in the Road Test One-MD report (*1*, Figs. 99 and 103) indicate that an effective length of 20 ft is not unreasonable. The reduction in length (24 to 28 ft in the report) to 20 ft is to account for the difference between the assumed rectangular pulse and the actual sinusoidal type of loading of real vehicles. Estimates of w were more difficult to obtain. The DEGEBO studies (*6*, Table 18-1) of the natural frequency of various soils (from peat to sandstone) indicated a range of frequencies of 12.5 to 34.0 cycles per sec. Later studies by Nijboer and Van der Poel (*7*) on

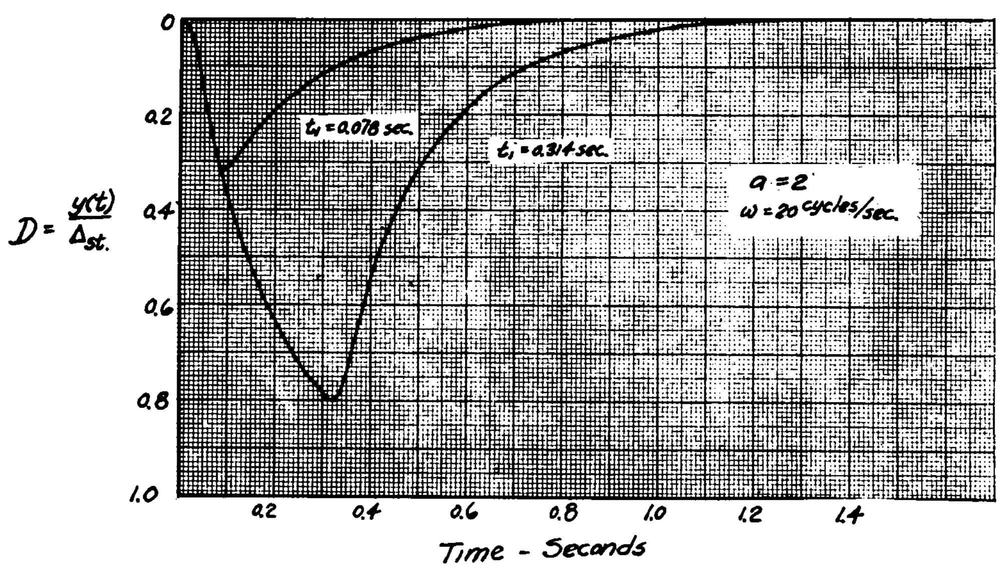

Figure 3. Dynamic deflection factor as function of time.

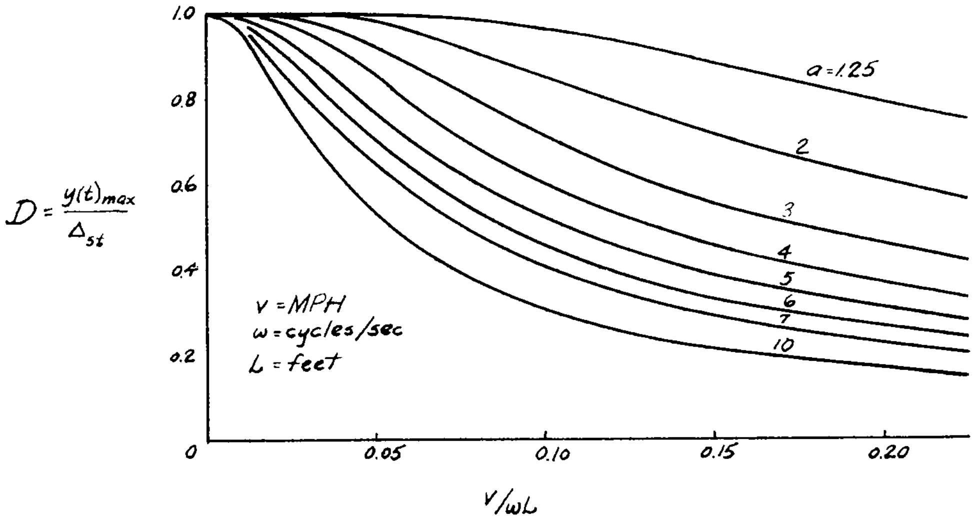

Figure 4. Maximum deflection factor as function of $v/w\ L$.

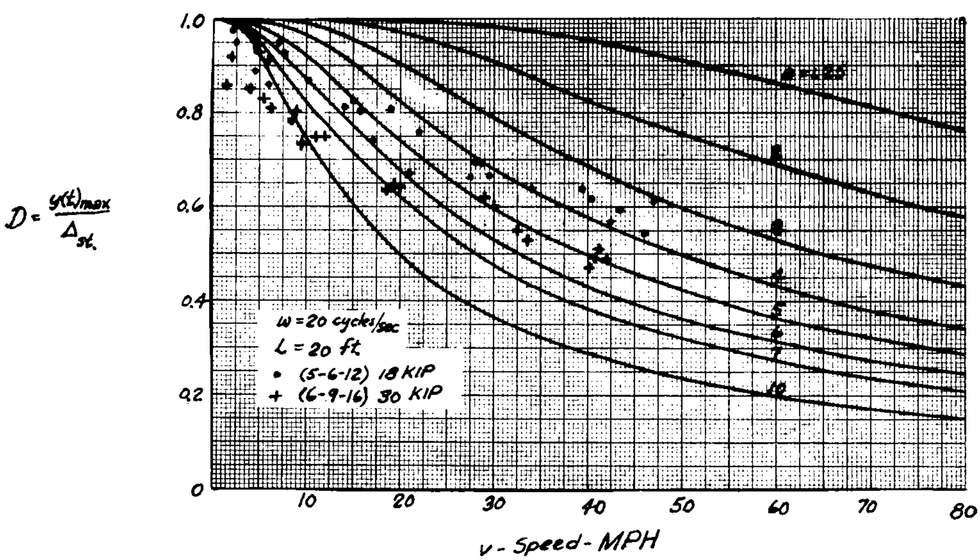

Figure 5. Maximum deflection factor as function of velocity.

the dynamic characteristics of asphaltic pavements gave approximately the same range in frequencies for these pavements. Hence, as an average value the frequency was taken as 20 cycles per sec. Making these assumptions, the abscissa scale in Figure 5 was established.

The crosses and dots in Figure 5 represent some deflection ratios obtained from the AASHO Road Test data at Ottawa, Ill. (Fig. 6). Por-

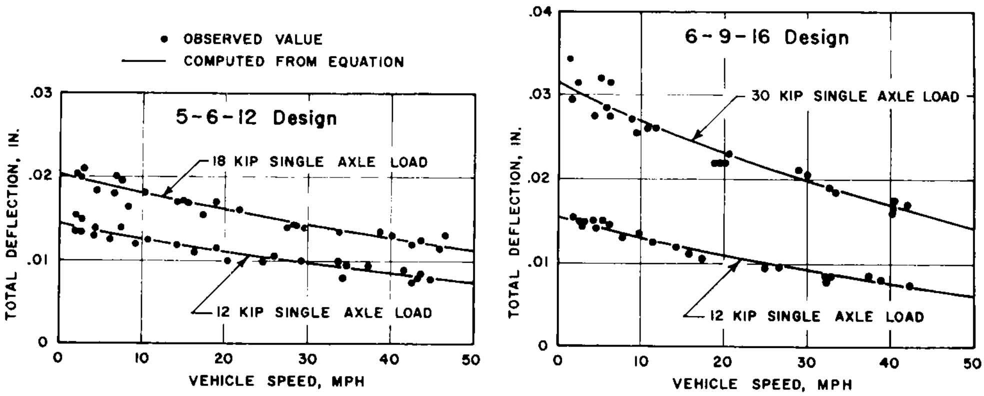

Figure 6. Variable speed study, effect of vehicle speed on total deflection (courtesy of AASHO Road Test).

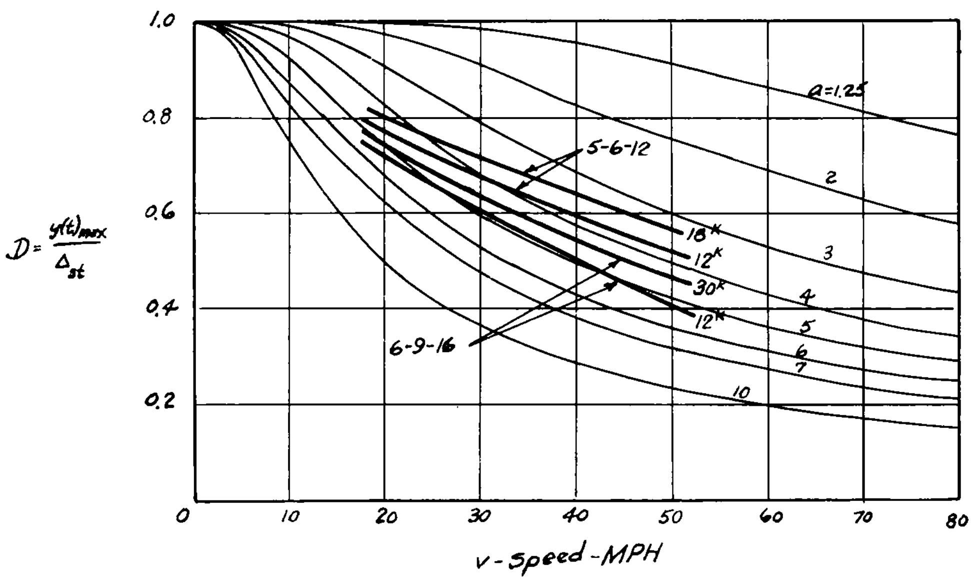

Figure 7. Portions of best-fit curves.

tions of the best-fit curves, computed from the Road Test equation (solid curves in Fig. 6), are reproduced in Figure 7.

It is difficult to conclude on the basis of the above that a reliable procedure has been developed whereby an engineer can predict the effect of vehicle speed on the deflection of highway pavements. This can only be verified by carefully conducted tests. However, the general aspects of these results do demonstrate the ability of viscoelastic theory to supply a mechanistic model of the pavement system.

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Roadside Development Safety Features in Highway Design Standards

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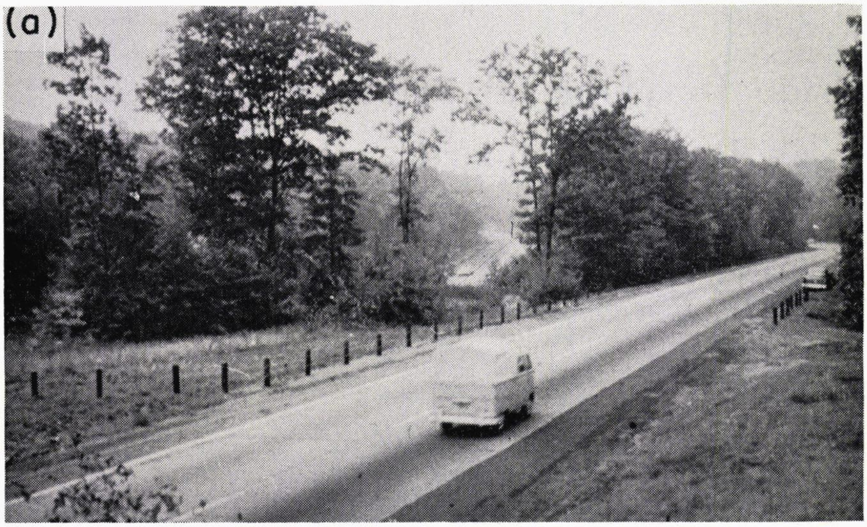
• THERE APPEARED in the October 15, 1961, issue of *Parade*, a Sunday newspaper magazine, the announcement of the 1961 award to the State of Maine for "America's Prize Highway." Quite naturally, this feat was a cooperative effort, involving the help of planners, designers, construction engineers, contractors, and the regional engineer of the U. S. Bureau of Public Roads, as well as the assistance of the landscape engineer in the Maine State Highway Commission. Those responsible for making this a great addition to the Nation's highway system are to be commended.

This section of road has been described in editorial comment in Augusta's *Kennebec Journal* as "the highway *with a soul,* because it was architectured to nature." It is considered a driver's road—a combination of scenery, speed and safety that have been "designed into" the landscape. It thus portrays what is known as the "complete highway," with safety, utility, economy, and beauty blended together for the service of transportation.

This finest section of Interstate Route 95 embodies all the principles set forth in "A Policy on Landscape Development for the National System of Interstate and Defense Highways," adopted by AASHO on January 25, 1961. This highway is an example of collaborative effort on the part of all —the administrator, the planner, the designer, the landscape engineer, and the contractor. This particular section of road embodies the several roadside development safety features that are such a basic part of every highway segment. Most important of these is that "divided highways should be designed as two separate one-way roads to take advantage of terrain and other conditions for safe and relaxed driving, economy, and pleasing appearance. All known features of safety and utility should be incorporated in each design to result in a National System of Interstate and Defense Highways which will be a credit to the Nation."

The alignment of this highway is such that it flows through the countryside—long, easy-flowing curves have been used in lieu of long tangents. Driver interest has been introduced, so that the road lacks the monotony of "sameness"—the one item responsible for a great amount of driver failure and many tragic accidents, because it induces so-called "turnpike trance." There is continually something new and interesting, yet not startling, as the vehicle operator traverses this sort of road.

The grades are easy and fit the topography so that neither the car driver nor the truck operator needs to be concerned with constantly changing speeds. Again, the grades flow through the topography in such a manner that interest is created and tension is reduced. There is ample



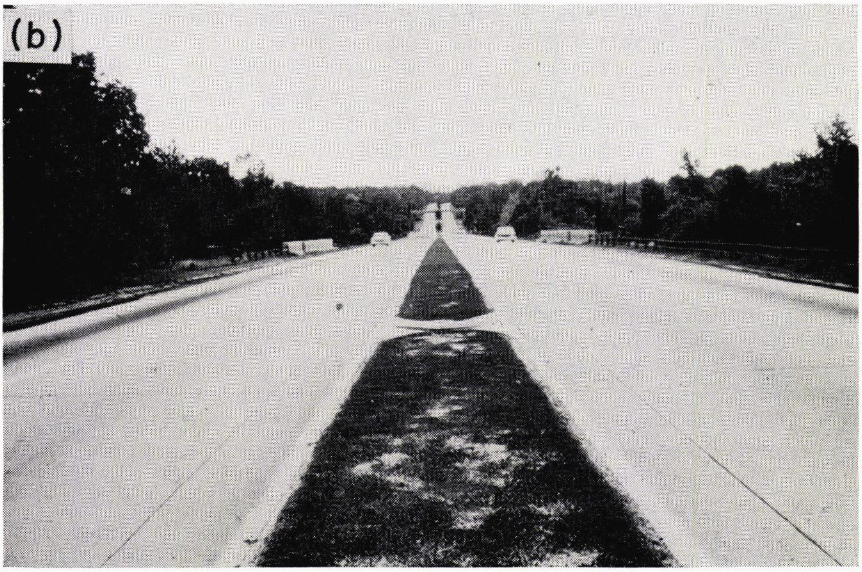


Figure 1. (a) Rural section of highway with two one-way divided roads and conservation features; and (b) road with long tangents, narrow equal-width median.

sight-distance ahead in order that emergency stops for any disabled or slow-moving vehicle may be avoided. And over each gentle roll in the grade, something new and interesting appears on the horizon.

Every highway must have some geometric elements that are the same—the same width of pavement and similarity in the construction elements. Yet the cross-section of the required roadside elements can be

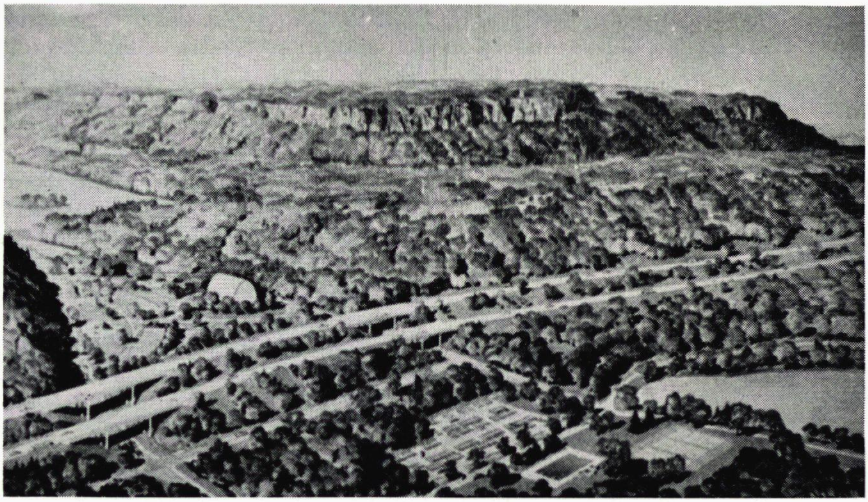


Figure 2. Expressway should fit into existing terrain.

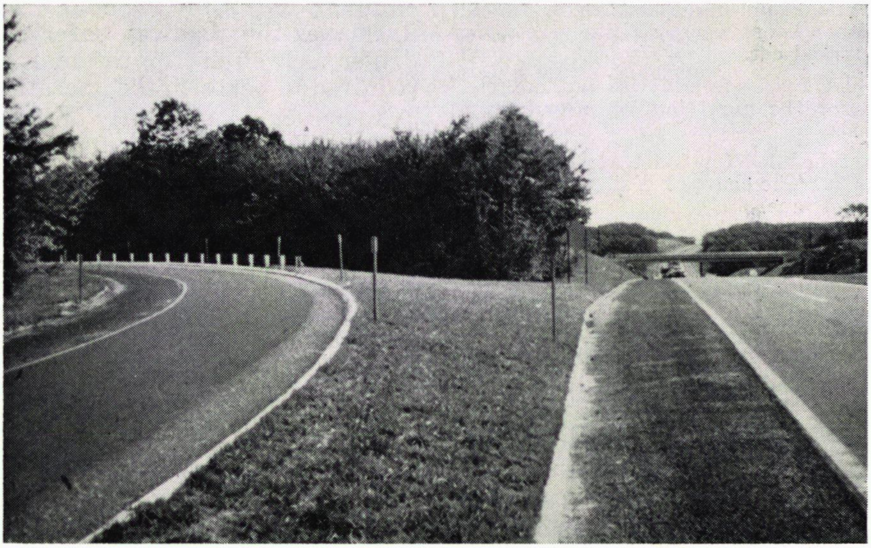


Figure 3. Conservation of natural resources can be an asset to safe travel.

changed to fit the existing conditions. Roadside slopes should be well-rounded and molded into the existing terrain. The flatter the slope, be it cut or fill, the easier it is to stabilize and maintain. Consequently, the vehicle that suddenly goes out of control is damaged less if it traverses an easy slope. Thus the highway is made safer.

Conservation is an important safety factor in highway design. By the preservation of existing natural landscape features, vegetation as well as other important elements, greater safety can be enjoyed by the motorist.

The constant annoyance of headlight glare can be a great detriment to a vehicle operator. By conserving many of the natural resources much of this can be avoided.

As an example of how headlight glare can be a hazard to safe vehicle movement, the following is quoted from an article in a Connecticut newspaper on December 14, 1961.

Auto Rams House, Tree . . . K said he was blinded by headlights of an approaching auto last night with the result that one nearby house lost its front steps and a railing, and a neighboring home its electricity. Police said K's auto swerved off the road here, went 100 feet up an embankment, turned right, crossed the street onto a lawn, sideswiped a tree, went through a fence, and tore down some shrubs and bushes.

The car continued on and tore the steps and railing off a house, went through a hedge and rammed into the electric meter on another house, recrossed the street, and hit a parked car.

The John Lyons family was without electricity for the night but no injuries were reported.

K was scheduled for court appearance December 22. The charge? Failure to drive in the proper lane.

Obviously this is one accident in a million, one that perhaps would not ever happen again—and, thankfully, it did not result in a tragedy. However, it does point emphatically to the need for that roadside development safety feature—conservation of plant materials and the necessary attention given to the blinding headlight glare.

K. A. Stonex, assistant director of the General Motors Proving Ground, recently brought to the attention of a group of engineers in a midwestern city the need to separate lanes widely not only to avoid cross-median accidents but also "to allow vehicle operators to use their high-beam headlights for better, more complete, night-time visibility" without blinding the operator of the on-coming car.

Planting is an important landscape development feature, and, functionally performed with a comprehension of the correct design and use of plant materials, can be a tremendous asset in the esthetic appearance of the "complete highway." The following are a few functional values that may be achieved by well-designed and installed plantings:

1. Protecting the side slopes against erosion. Eroded material that reaches the travelway is a hazard, and every obstruction that can be eliminated from the travelway will make for a safer road.

2. Reducing maintenance operations by erosion control. Every man and every piece of equipment constitute a traffic hazard, and their elimination within the highway limits makes the roadway safer for the driver as well as for the maintainer required to perform the operations.

3. Screening unsightly objects and views. Nothing creates driver tension more than roadside clutter and an unsightly, dirty scene flashing before the driver's eyes.

4. Isolating the highway from the roadside border developments, and thus preventing pedestrian trespass and reducing traffic annoyance for nearby residential areas.

5. Providing advance warning to traffic by indicating structure openings, guiding traffic turns, and framing desirable outlooks and views so that operator interest is created.

6. In the median areas, to supplement valuable conserved vegetation, installing plantings to obscure headlight glare, to function as crash barriers and to prevent the drifting of snow in the snow-belt States.

7. Installing shade trees, where feasible, for comfortable vehicular travel in the hot, glaring sun.

8. Improving the appearance and esthetic interest. A pleasant drive is usually a safe one, and plantings can

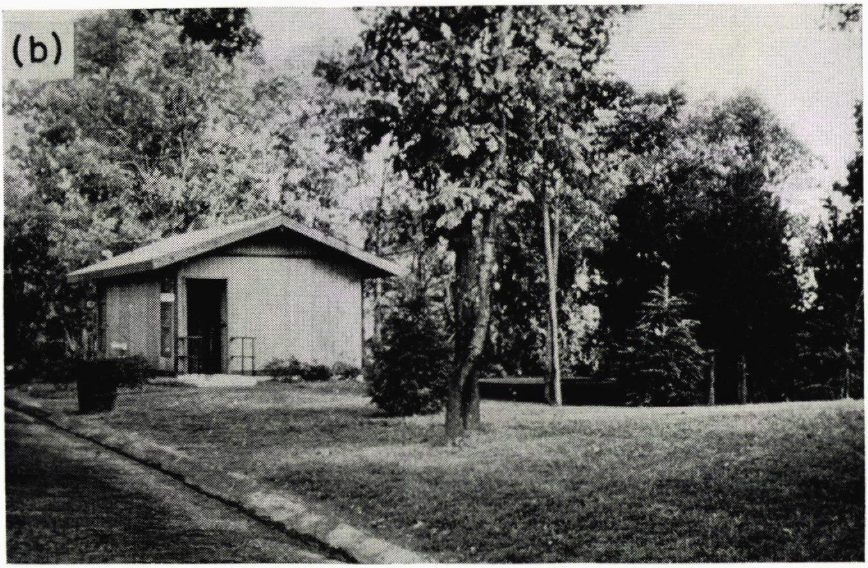


Figure 4. Rest areas (a) are an essential safety factor and should be equipped with modern comfort facilities (b).

create an important asset in motor travel.

These are but a few of the functional values of roadside plantings as an important landscape development safety factor.

Finally, there is the safety rest area, equipped with modern comfort facilities, that is an essential roadside safety factor. The tired driver must have a place to drive off the travelway, stop, rest, and relax. The truck operator needs such a place as much, if not more, than the passenger vehicle driver. Safety experts encourage such stops to promote trouble-free transportation.

Many know of the plea set forth in the recent AASHO sessions in Denver to attain Federal financial participation for modern comfort facilities in safety rest areas. In numerous instances, especially on the midwestern highways, such convenience facilities are lacking. Safe vehicular operation demands that this important roadside safety factor be given immediate and whole-hearted attention and support.

It is the responsibility of every chief highway administrator to develop and maintain his State's highway system with every possible safety factor incorporated into the planning and design. This makes for the "complete highway," such as Maine has recently developed. If the principles of sound landscape development are combined with good engineering practices, safer transportation arteries will be achieved. Maine is an exemplary guide in creating a highway transportation system that is utilitarian, economical, beautiful, and safe.

Esthetic Criteria in Freeway Design

BORIS PUSHKAREV, *Senior Planner, Regional Plan Association, New York, N. Y.*

• ALTHOUGH intensive research is being carried out on the functional and structural aspects of the highway, the form of the highway as such has merited much less attention. The independent study of form is unnecessary in the case of those technological tools whose shape is rigorously determined by structural and functional necessities. The highway, by contrast, like a piece of architecture or industrial design, lies in an order of precision where scientifically determined limitations leave the designer considerable freedom to refine form beyond the bare minima of utilitarian standards. Moreover, the eye of the highway user (and nonuser as well) perceives the form of the highway not as an engineering problem, but as a visual entity; the highway is seen before it can be traveled on. Being seen is an integral part of its purpose; hence, the importance of the formal or visual approach.

By applying methods of formal esthetic analysis, a distinction between the internal and external harmony of the highway can be made. The former concerns the roadway as an abstract ribbon in space. The latter concerns its relationship with the environment. Some relevant criteria to judge internal harmony are continuity of alignment, three-dimensional coordination, and harmony of enclosed areas. Some relevant criteria to judge external harmony are integration with the macroenvironment and the microenvironment, definition of elements, and frequency and progression of focal points.

Continuity of alignment and the sculptural form of the alignment in general is of the utmost importance, because the pavement of the freeway is by far the most insistent feature in the visual field of the driver (Fig. 1). On a conventional country road at moderate speed, the roadbed occupies some 8 percent of the driver's visual field in perspective, while the roadside may take up 80 percent or more, depending on topography and vegetation. On a six-lane freeway at 25 mph, the share of the roadbed rises

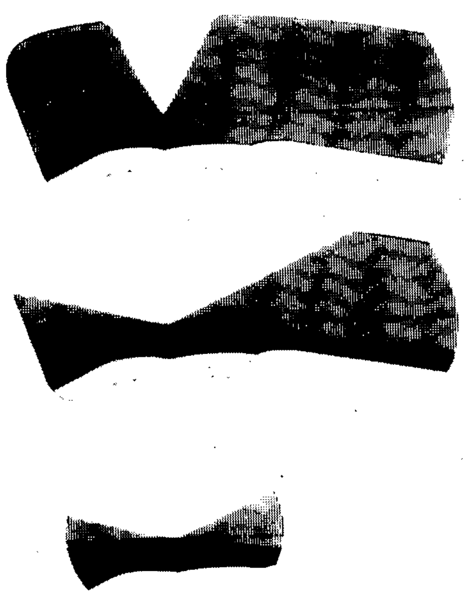


Figure 1. Driver's visual fields.

to 20 percent. Considering the reduction of the cone of vision at 60 mph, the roadbed may occupy close to 30 percent, the sky more than 50 percent, while the share of the roadside shrinks to less than 20 percent of the visual field, and in flat terrain to as little as 5 percent.

In freeway design, the shape of the roadbed itself, as seen in perspective, can become the dominant element of visual, esthetic expression. The alignment of the roadbed consists of curves and tangents, both vertical and horizontal (Fig. 2).

The tangent is esthetically justified in very flat terrain or where the predominant man-made landscape pattern—such as a street grid—is rectilinear. It is easy to design and provides clear orientation, but at the same time, unless aimed at a landmark, it is esthetically uninteresting, because totally predictable; it is monotonous and fatiguing because the view is completely static; it encourages excessive speeds, because the driver tries to "get it over with."

The second most common element of horizontal alignment is the circular arc. It is interesting because it brings more roadside into view, because it shows the driver a changing panorama and arouses a sense of anticipation for what is beyond. A curve encourages attention and a steady hold on the steering wheel. At the same time, the curving roadside provides much better optical guidance, because it is seen ahead, rather than peripherally.

Neither the tangent nor the circu-

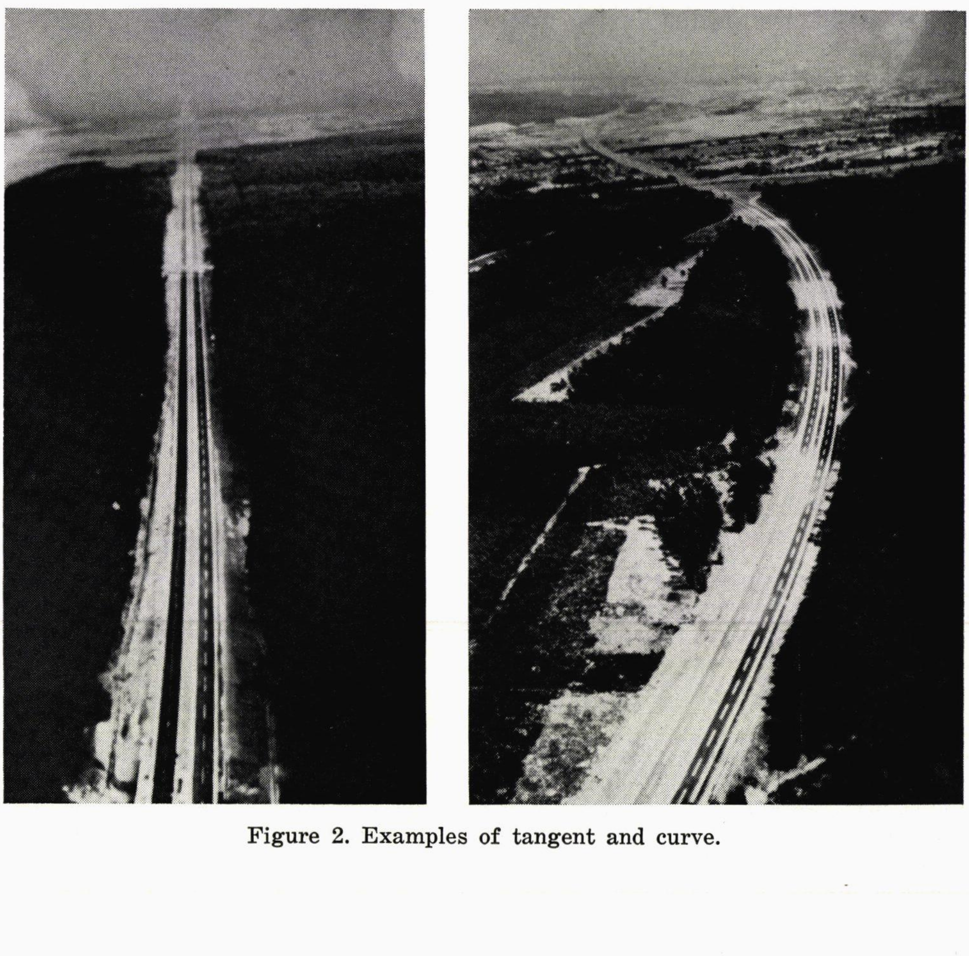


Figure 2. Examples of tangent and curve.

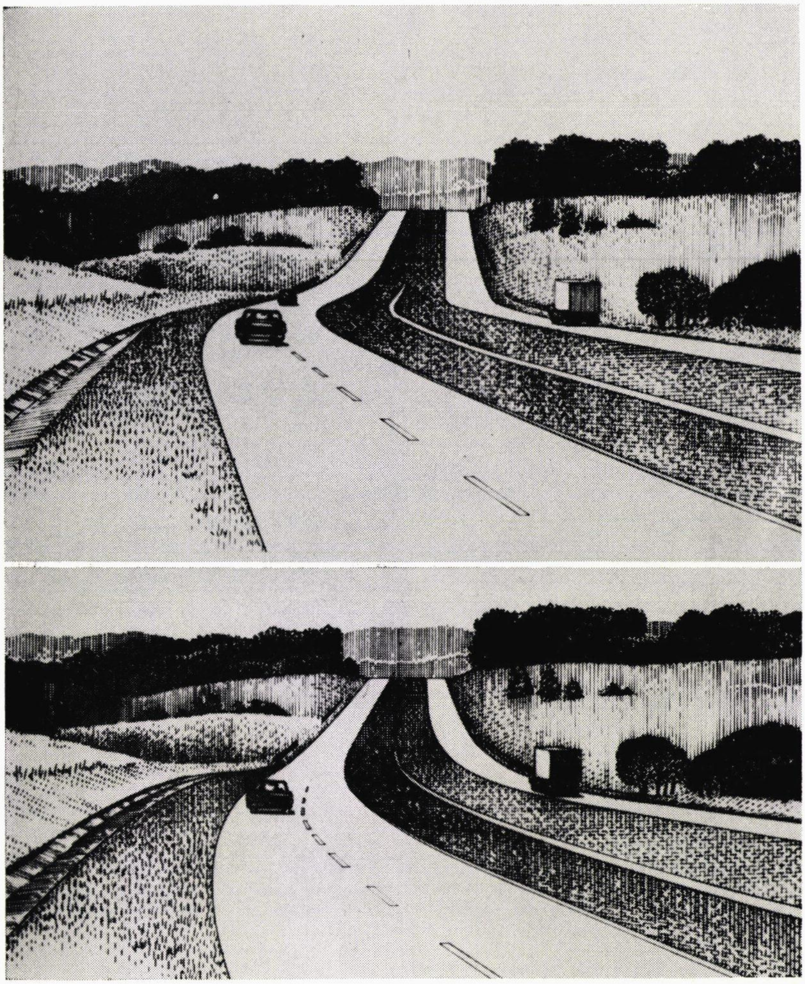


Figure 3. Curve without and with spiral transition.

lar arc, taken in themselves, pose any problems of continuity. The problem arises, when the two are joined, because the straight line, with zero curvature, touches the circle, which appears as an ellipse with a continuously changing curvature, at a point where this curvature appears the sharpest. Perspective foreshortening from a viewpoint only some 4 ft above ground accentuates this discontinuity and makes it, in spite of the huge scale of the curves, visually disturbing.

Clearly, there is visual need for a connecting element whose curvature would gradually change from zero at the tangent to the constant curvature of the circular arc, and join the straight line with the apparent ellipse of the circle in a continuous fashion.

Apart from its visual value, a spiral transition curve of course expresses, in a refined way, the functional performance of a vehicle entering or leaving a curve (Fig. 3). But because spirals in highway design are recognized to be unessential on

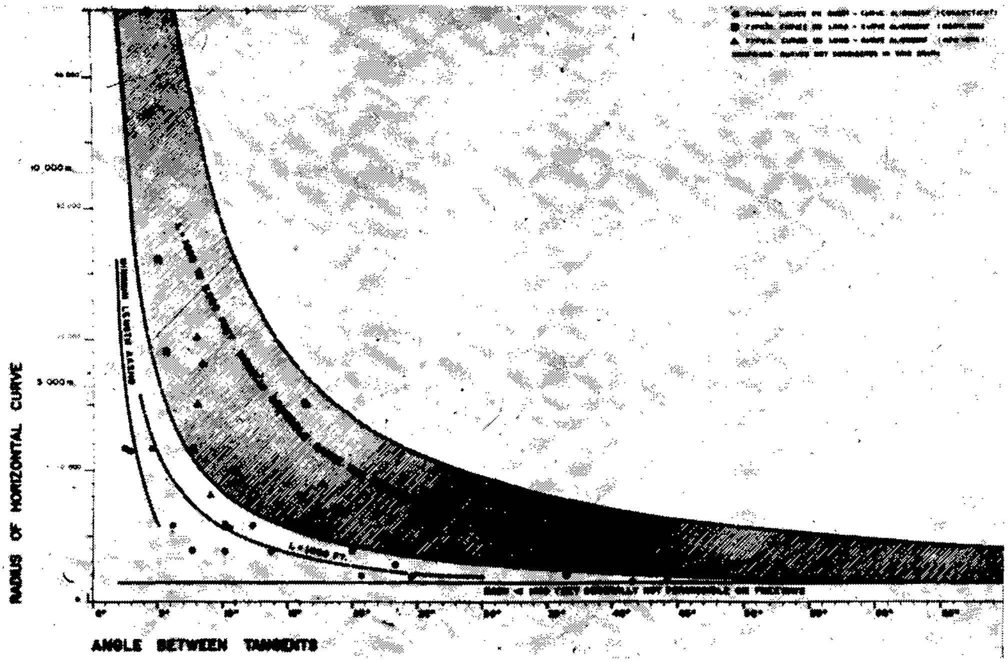


Figure 4. Angle between tangents *vs* radius of horizontal curve.

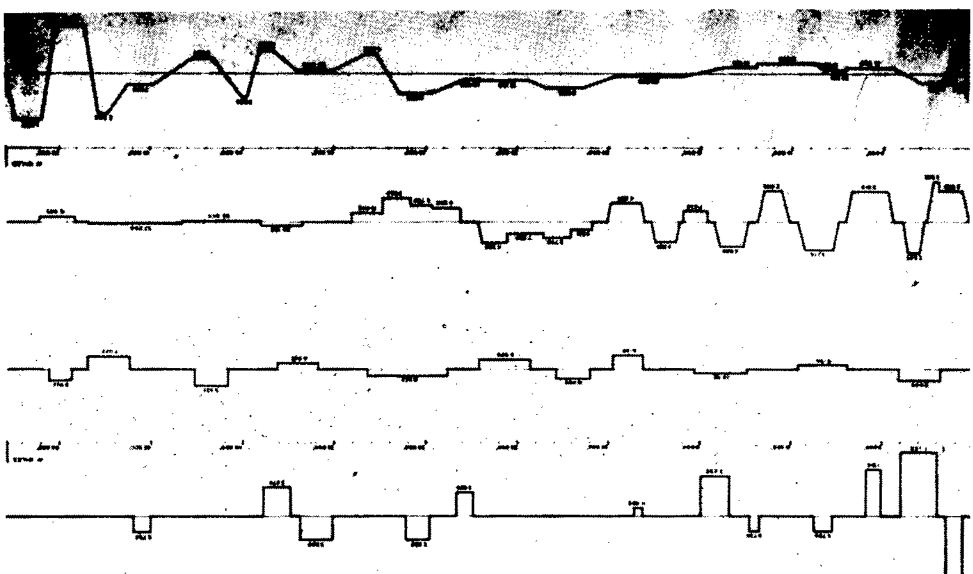


Figure 5. Curvature diagrams for 10-mi stretches of four characteristic kinds of horizontal alignment: Connecticut Turnpike, New York Thruway, Baltimore-Washington Parkway, and Ashaffenburg-Nürnberg Autobahn.

functional grounds, a frankly esthetic standard to determine their use and their minimum length seems justified. It would appear no more arbitrary than the factor C for comfort used in radial acceleration formulas for length of spiral, or the requirement that superelevation runoff be traversed in no less than 2 sec used in calculations tying length of spiral to length of superelevation runoff. For whichever of these two accepted semifunctional calculations governs, the spirals it produces are too short to be visually significant.

The discussion of spirals (that is, the continuity of horizontal form) proceeds to the subject of continuity in horizontal scale. A straight line, in itself, has no scale. But when a straight line has to change direction and is thus interrupted by a curve, the question immediately arises—how long should that curve be to preserve visual continuity? A curve may be nicely transitioned, but if it is too short, it will still appear as an unnatural kink in the road.

Inasmuch as at high speeds the driver focuses some 1,000 to 2,000 ft ahead, it seems that a curve ought to be at least that long to be visually significant while the driver is on it. Systematic observation by several visually trained individuals on a Connecticut freeway indicated that curves shorter than 1,000 ft were generally experienced as "too short." German 1942 geometric standards established 300 m (984 ft) as an absolute minimum for length of curve on the Autobahnen.

In Figure 4, angles between tangents are plotted against radii, and the resulting hyperbolas are lines of equal length of circular curve. The 1,000-ft length is shown as a suggested minimum, and the area between 1,500 and 5,500 ft is shaded as the desirable range for the length of simple circular curves on freeways. Dots indicate radii used on a typical section of a shortcurve freeway alignment in Connecticut; triangles and squares, clustering toward the upper end of the graph, are typical radii on long-curve alignments in New York and Maryland.

Striving for longer and hence flatter curves could lead to making the radius so large that the curve will be hardly distinguishable from a straight line. What is then the longest reasonable radius? Hans Lorenz points out that curves generally cease to be visually significant if the visible part of the curve accomplishes a turn of 2 to 3 degrees or less. This rule seems to be borne out by the visual experience on freeways with very flat curves, such as the Baltimore-Washington Parkway. However, it makes curves with radii up to 80,000 ft quite realistic (Fig. 5).

Another way to approach the scale of horizontal curves is by comparing their length with that of the tangents. Most widely used in America is the long-tangent short-curve alignment. Typically, this consists of straight sections 1 to 3 mi or more long, connected by curves about 1,500 ft long with 2,000- to 12,000-ft radii. From the definition of continuity this is the epitome of discontinuous alignment, for every curve and every tangent is clearly seen as a separate thing. Curves usually make up less than one-fifth of such an alignment.

On divided highways, passing sight distance is of no consequence, and the only reason for the use of long tangents here is probably the tradition of railroad engineering. The fact is somehow often overlooked that if the angle between two tangents is given, a flat curve will result in a shorter and more directional alignment than a sharper one. More familiar functional arguments against the short-curve long-tangent alignment are that curves at the end of long tangents are accident-prone and that long straight sections produce monotony. It appears logical to increase the length of curves to such a degree

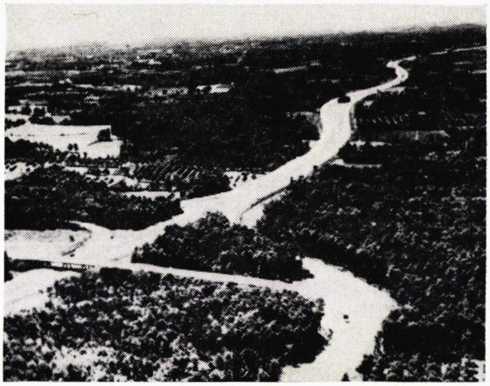



Figure 6. Long-curve short-tangent section of New York Thruway and tangentless section of Ashaffenburg-Nürnberg Autobahn; design speed 160 km per hr.

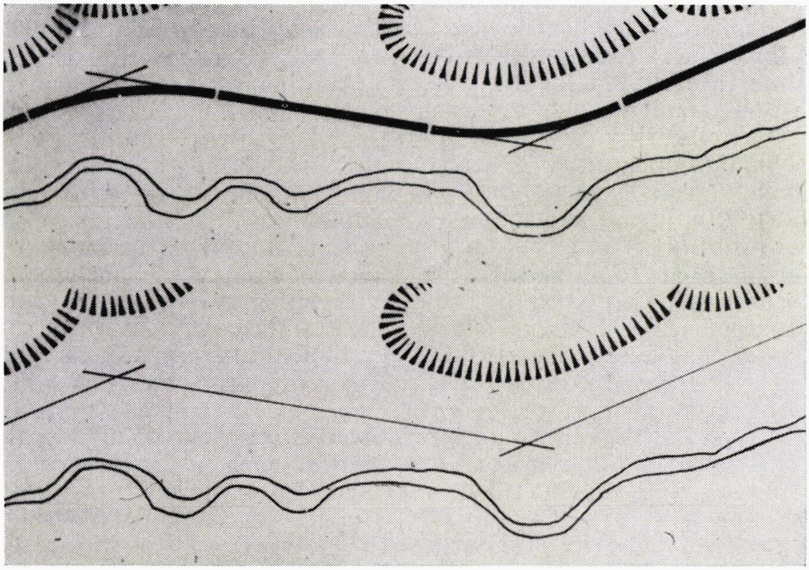


Figure 7. Discontinuous tangent-and-curve alignment.

that they surpass by far the length of tangents, arriving at the long-curve short-tangent alignment.

For this design, not a straight line but rather a flat curve is taken as the basic unit of alignment (Fig. 6). For example, the basis for the geometric design of the Garden State Parkway was a curve with a 15,000-ft radius. Only about one-third to one-fifth of such an alignment consists of straight sections. The latter are not meant to read as tangents visually, in perspective, but rather to appear as a part of a continuous compound curve.

The only esthetic difficulty with this alignment is that unless the radii are extremely long—which is often not permitted by topography and other restrictions—the connection be-

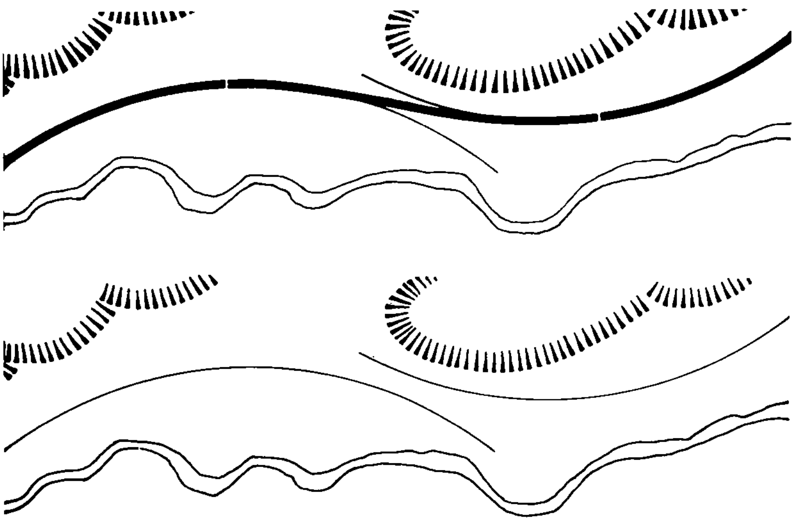


Figure 8. Continuous curvilinear alignment.

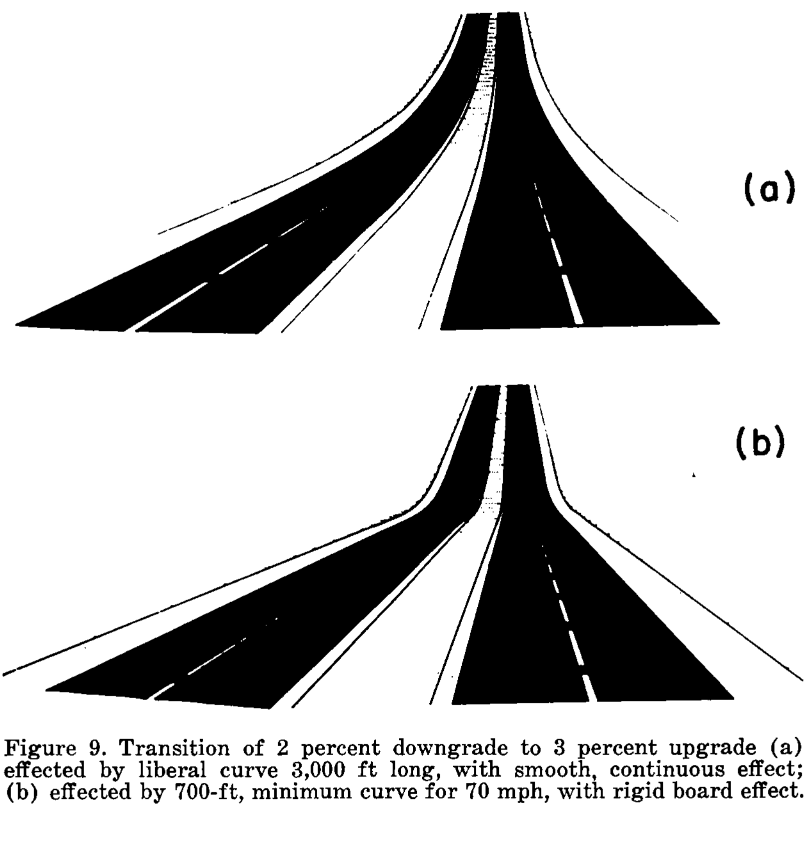


Figure 9. Transition of 2 percent downgrade to 3 percent upgrade (a) effected by liberal curve 3,000 ft long, with smooth, continuous effect; (b) effected by 700-ft, minimum curve for 70 mph, with rigid board effect.

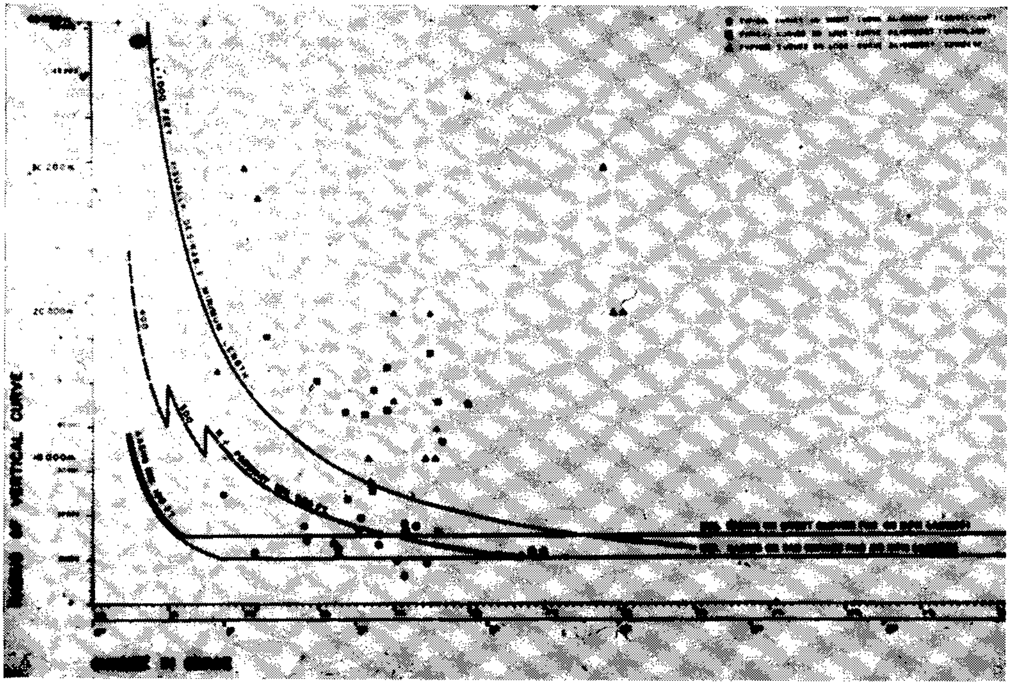


Figure 10. Radius *vs* angle between tangents for vertical curves on three typical kinds of alignment. Vertical radius approximately equal to 100 K for small angles if parabolic curve treated as circular for purposes of comparison.

tween circular curves and tangents appears discontinuous. Hence, spiral transitions have to be introduced. Now, if the tangents are as short as "spline" alignment requires them to be and spiral transitions are made as long as recommended, then very little room is often left for the tangent. The next logical step, then, is the continuous curvilinear alignment.

This consists of long, flat circular curves, simple and compound, connected by fairly long spiral transitions, with about two-thirds of the alignment on circular arcs and one-third on spirals, and approaches the ideal of continuity in form and in scale.

Discarding the traditional, discontinuous tangent-and-curve alignment in favor of the new continuous curvilinear alignment naturally requires a radical break with accepted procedures of design.

The old procedure was to use given topographic controls to establish the basic tangents, and then connect them with circular arcs (Fig. 7).

The new procedure would be to use given topographic controls to establish the basic circular arcs, as flat and as long as practicable, and then join them with suitable transition curves (Fig. 8). The tangent is thus eliminated as a dominant element of design, except under urban or other special conditions.

The continuity of alignment can be expressed and studied graphically and mathematically by means of the curvature diagram, or $1/r$ graph. The method is simple and easy to visualize. One merely plots the centerline of the highway along the horizontal axis

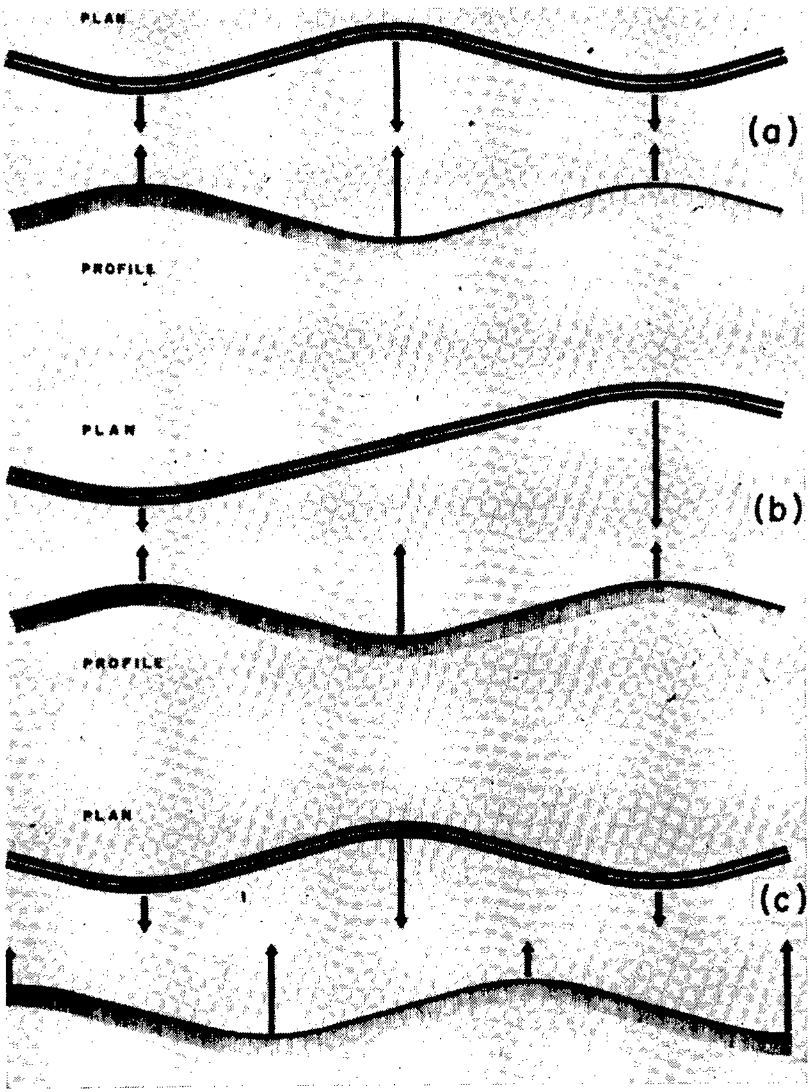


Figure 11. Coordination between vertical and horizontal alignment.

of the graph, and the value 1 divided by radius, along the perpendicular axis. In such a diagram, the tangent appears as a straight line on the horizontal axis. A circular curve appears as a straight line parallel to the axis, but at a distance $1/r$ from it, up or down, depending on whether it is a right or left turn. The spiral appears as a sloping straight line, connecting the two. In essence, the $1/r$ diagram shows the second derivative of the highway curvature, which represents, among other things, the movement of the steering wheel. Areas enclosed between the $1/r$ line and the horizontal axis show L/r; *i.e.*, they are proportional to the angle between tangents.

No less important than horizontal

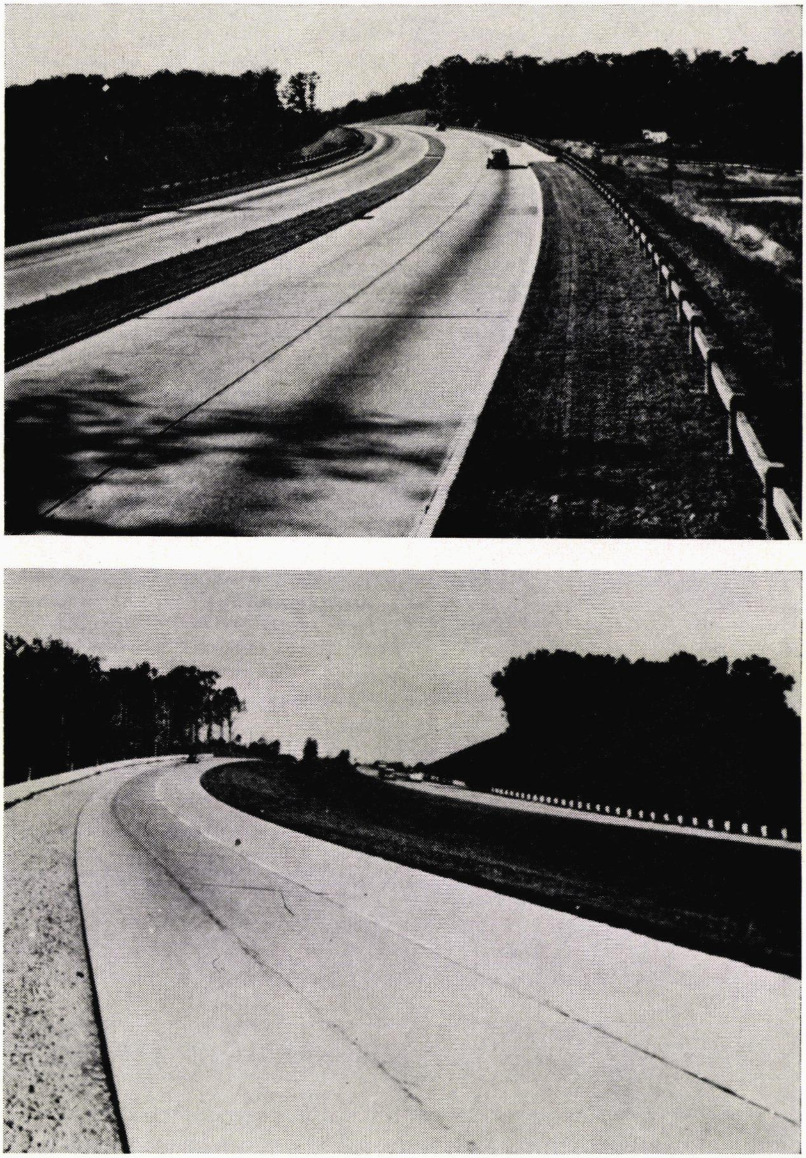


Figure 12. Vertical curvature superimposed on horizontal curvature.

continuity is continuity of alignment in the vertical plane (Fig. 9). Again, the accepted minimum functional requirements for length of vertical curve result in changes of direction that appear visually abrupt and discontinuous. Minimum length of sag curves for differences in grade between 1 and 6 percent ranges from 200 to 900 ft, and such short vertical curves are out of scale both with the adjoining tangents and with the sweeping flow of horizontal curves.

The standard practice on freeways

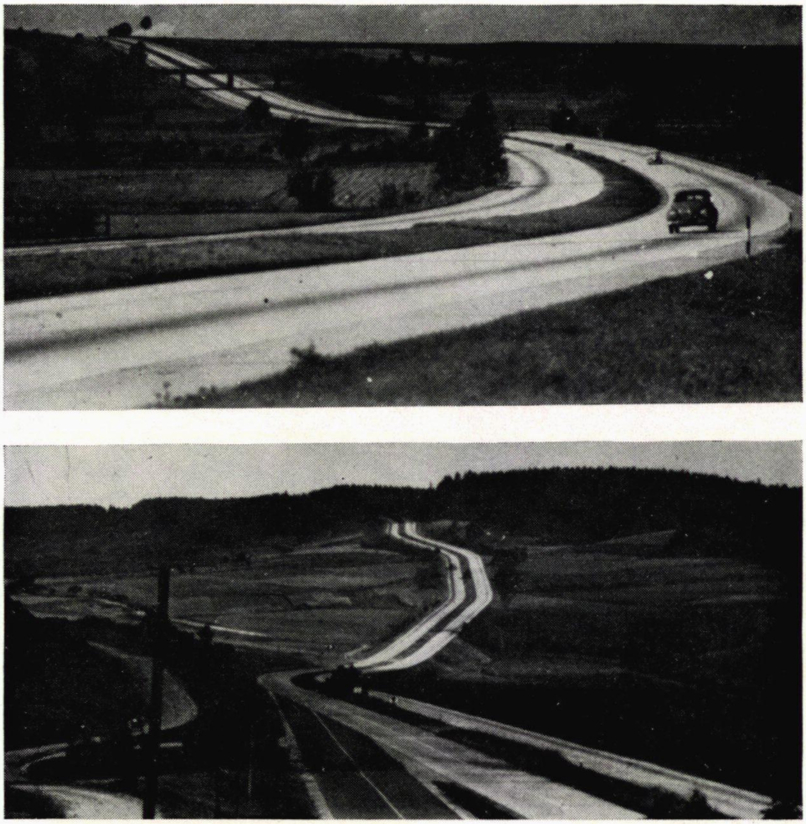


Figure 13. Examples of continuous and discontinuous alignment in Germany.

with a conventional long-tangent short-curve plan is to employ long tangents and short curves in profile as well, making the vertical curves as short as the functional standards will permit, with a deleterious effect on esthetic continuity. Only parkways and freeways with a curvilinear-type horizontal alignment have made use of very long vertical curves so far. The Baltimore-Washington Parkway or the Ashaffenburg-Nürnberg Autobahn, for example, successfully use sag curves 1,500 to 3,000 ft long, which is often 2 to 3 times more than would be required by minimum utilitarian standards (Fig. 10).

The tendency to minimize tangents in profile leads to the continuous curvilinear alignment in the vertical plane. Though curves may make, in moderately hilly terrain, some 25 percent of the conventional vertical alignment, they can make up, under comparable circumstances, as much as 50 percent of the curvilinear vertical alignment. This means, essentially, reducing the number of vertical curves and increasing the length of the remaining ones more than twice. This is particularly important in the case of sag curves and small changes of grade, where perspective foreshortening is acute.

Figure 14. Variable median spoiled by discontinuous alignment and lack of continuity in slope grading.

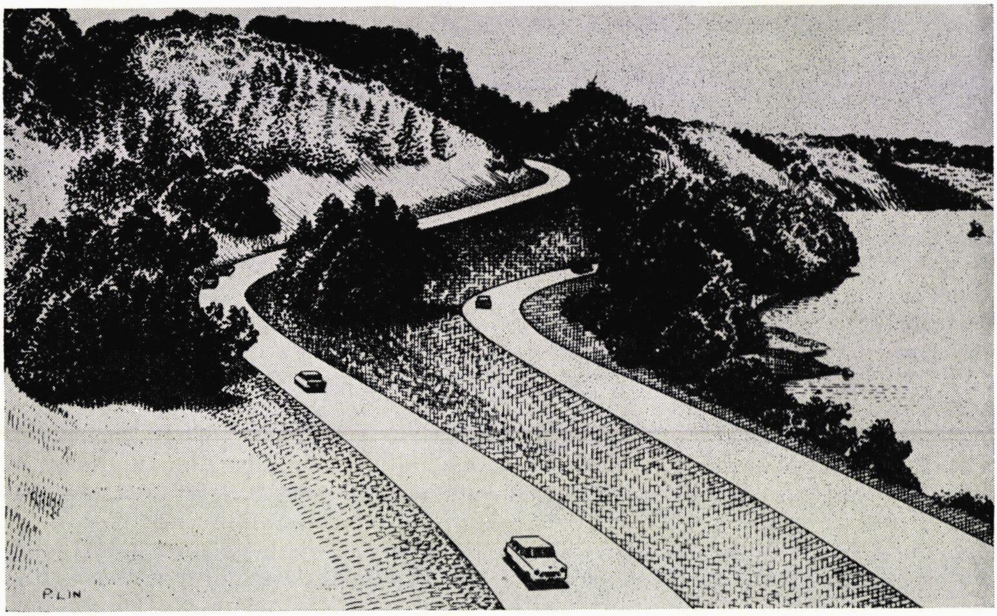


Figure 15. Superior median design, George Washington Parkway.

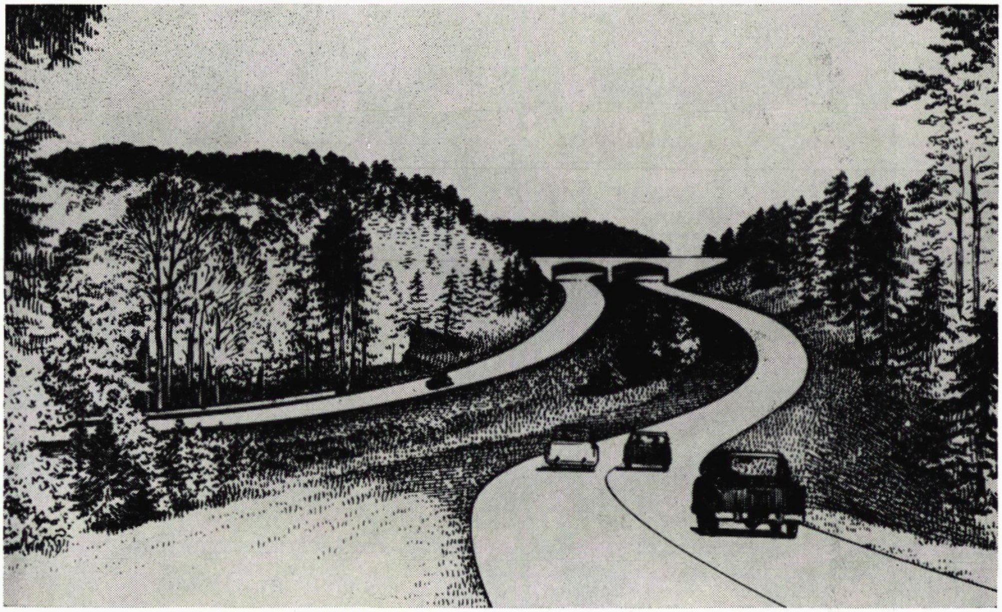


Figure 16. Superior median design, Baltimore-Washington Parkway.

Continuity in plan and continuity in profile will lead to continuity in three dimensions only if the vertical and horizontal elements are carefully coordinated.

1. A generous alignment in one plane does not associate itself with small and frequent adjustments in the other. The length of vertical curves (and grades, for that matter) should be influenced by the length of the horizontal elements on which they are superimposed.

2. Although the scale of vertical and horizontal elements should be related, they should not commence and terminate simultaneously. Fritz Heller points out that the eye may find it disturbing when the vertical and horizontal alignment change at the same time, because any irregularities at this point will be emphasized in perspective and be cumulative in effect. Desirably, horizontal curvature should always "lead" vertical curvature somewhat, and should remain somewhat (but not too much) longer than the latter. This overlapping will also promote safety through optical guidance.

3. Elements of the plan should generally coincide with those of the profile not only with respect to length but also with respect to location. The rule suggested by Hans Lorenz is that the vertex or turning point of a vertical curve should roughly coincide with the vertex of the horizontal curve.

The vertices may be, at times, shifted as much as one-quarter of a phase, but a shift of one-half a phase results in an unsightly situation where the vertical curve lies at the beginning of the horizontal curve, creating the impression of a sharp angle.

Figure 11a shows the classic case of coordination between vertical and horizontal alignment. The vertices of horizontal and vertical curves coincide, creating a rich effect of three-dimensional S-curves, composed of

Figure 17. Example of integration between freeway and river valley.

convex and concave helixes. Figure 11b shows a legitimate case of coordination. One phase is skipped in the horizontal plane, but vertices still coincide. The long tangent in plan is softened by vertical curvature. Figure 11c shows weak coordination. Vertical alignment is shifted one-half of a phase with respect to horizontal alignment, vertices coincide with points of inflection. Superelevation in this case occurs on grade, and crests and sags have normal crowned section; in the first case, superelevation occurs on crests and sags, and grades have normal crowned section. Figure 12 shows two examples of vertical curvature superimposed on horizontal curvature, creating a rich three-dimensional curve, the helix.

Desirably one should think of the alignment (and study it through visual aids) as a continuous sculptured three-dimensional curve and not as a result of random superposition of plan and profile. Every movement of the vehicle along the paved ribbon should be subject to description in one unequivocal three-dimensional term; it should not consist of a series of small movements within a larger one (Fig. 13). Such an approach will eliminate common faults such as the "broken back," "hidden dip," or "roller coaster" alignment. If the hills should not be leveled, the remedy for the "roller coaster" is to introduce some horizontal curvature that will prevent the crest-after-the-next-one from being seen. In effect, this

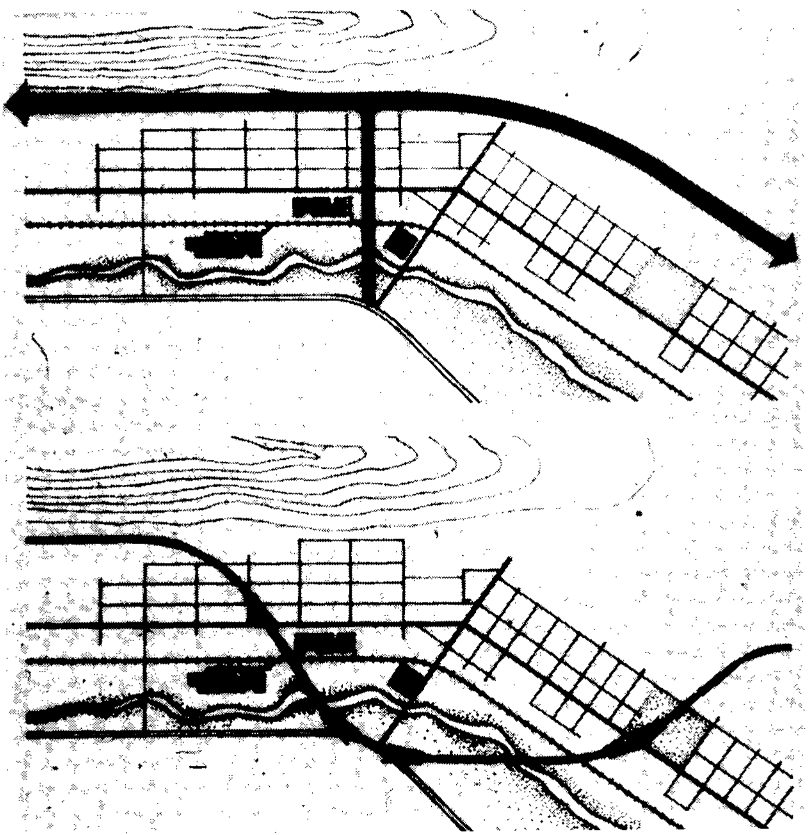


Figure 18. Street grid (top) integrated with freeway; (bottom) violated by freeway.

means setting a limit on sight distance, so that the driver can concentrate visually on the movement he is about to perform now. This increases attention and anticipation, which are indispensible to esthetic enjoyment.

A final note with regard to the internal harmony of a freeway concerns the design of shapes, enclosed by the twin ribbons of pavement, as they appear in perspective. Merely to introduce a varying median, without carefully studying its appearance, is not enough (Fig. 14). For example, differences in width and in elevation should be introduced on curves, never on tangents (Figs. 15 and 16).

Turning now from the subject of internal harmony to that of external harmony, to the integration of the road with its environment, recounting a few of the governing principles is sufficient.

A freeway should have the look of permanence and belonging in the landscape or cityscape (Fig. 17). For this purpose, it should follow, rather than crisscross, the dominant geometric order of its surroundings (Fig. 18). Hills should not be straddled perpendicularly to the contours, but rather at an oblique angle. Location of cuts should be related to natural depressions in a ridge, and if a valley is to be crossed, the in-

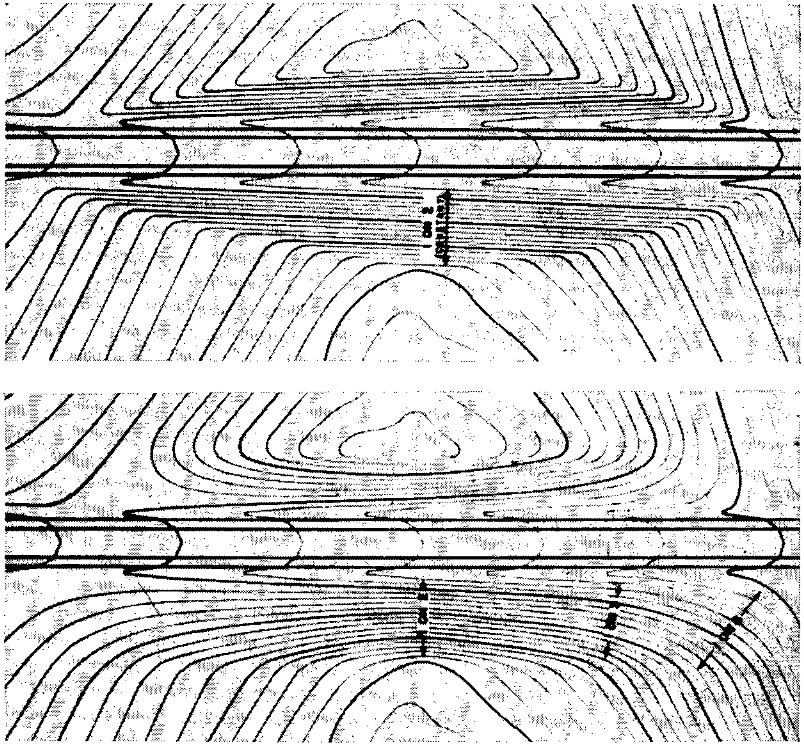


Figure 19. Slope of bank kept constant (top); length of incline kept constant (bottom).

tegrity of its space should be preserved either by an open viaduct design or by a very flat embankment.

The cross-section should be continuous, and the side slopes as gentle as possible. A recent German grading standard suggests that in cuts, instead of keeping the slope of the bank constant, the length of the incline should be kept constant, thus resulting in a varying slope that gently blends into the natural topography (Fig. 19).

Guide rails and similar barriers violate the continuity of the cross-section and should be replaced, as far as possible, by flat slopes.

Separation of the freeway from surrounding development is imperative (Fig. 20). The scale of a freeway is such that it demands a broad strip of land so as to live a life of its own and not conflict visually with residential, industrial, or commercial development. The standard 300-ft right-of-way, leaving a buffer strip of some 80 ft on each side, is barely sufficient to satisfy the requirement for visual articulation. A 400-ft right-of-way could be a desirable minimum for suburban conditions, and still wider green strips, perhaps doubling as recreational or watershed protection lands, are desirable. Rest turnouts on freeways in particular should have ample room.

In dense urban areas, where wide rights-of-way are not feasible, the paramount need is for strong design controls to establish proper massing and setbacks for buildings and to

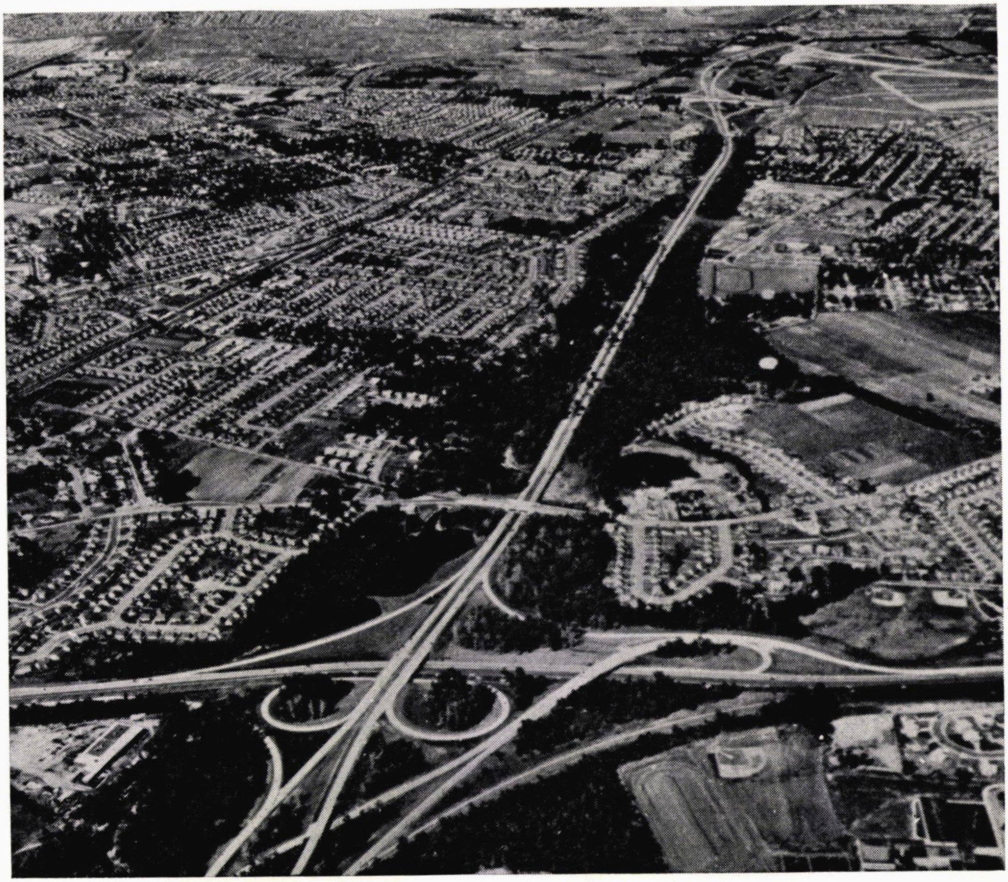


Figure 20. Clean separation between freeway and residential development, Meadowbrook Parkway, Long Island.

eliminate billboards and overhead wiring (Fig. 21). The integrity of the visual space of the freeway should not be violated by extraneous objects.

When all the requirements for continuity of alignment and cross-section for integration with the landscape and proper visual articulation, are taken care of, the freeway will be pleasant, fitting, and clean. However, it may turn out to be somewhat dull, unless vistas and landmarks are provided along the way. In order not to distract the driver by sideways views, the vertical and horizontal alignments should be oriented in such a way as to bring these focal elements into the driver's forward cone of vision. Views of the ocean, of lakes and valleys, of urban panoramas, bridges, industrial and utility structures are all suitable as focal points (Fig. 22), but perhaps giant, abstract pieces of sculpture could provide a fitting link between the fine arts and the art of highway building (Fig. 23).

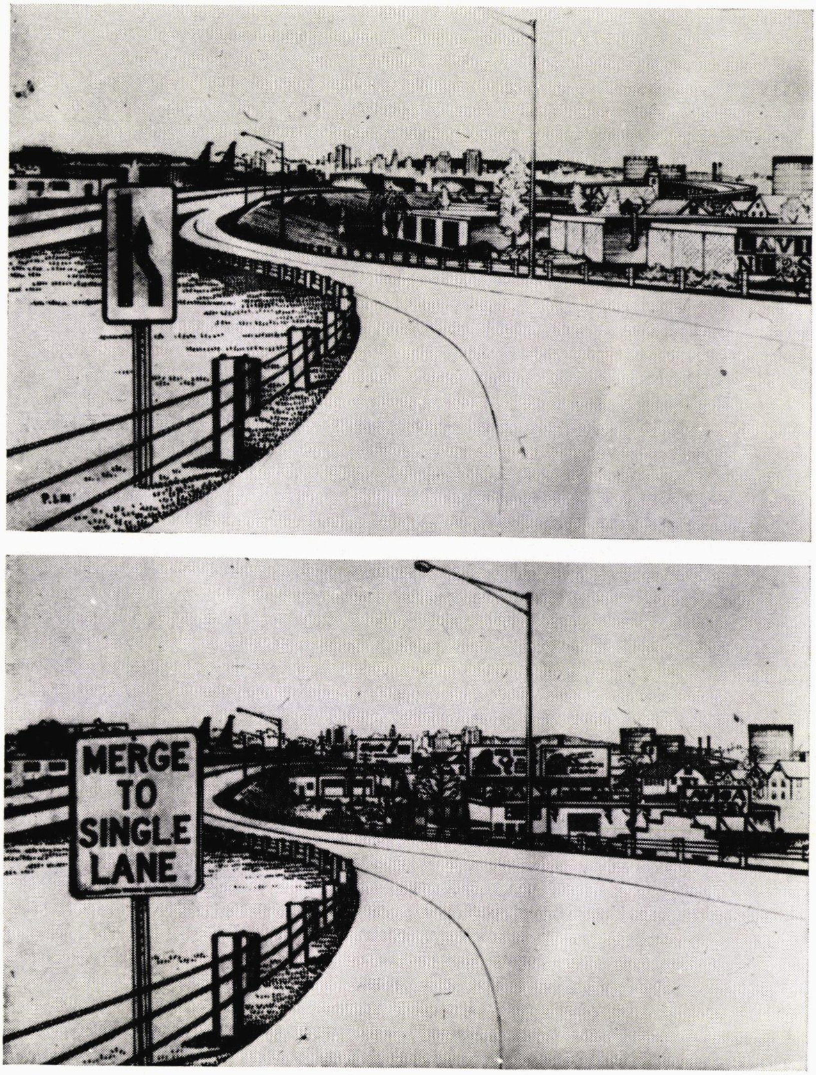


Figure 21. Use of strong design controls (top); integrity of visual space violated by extraneous objects (bottom).

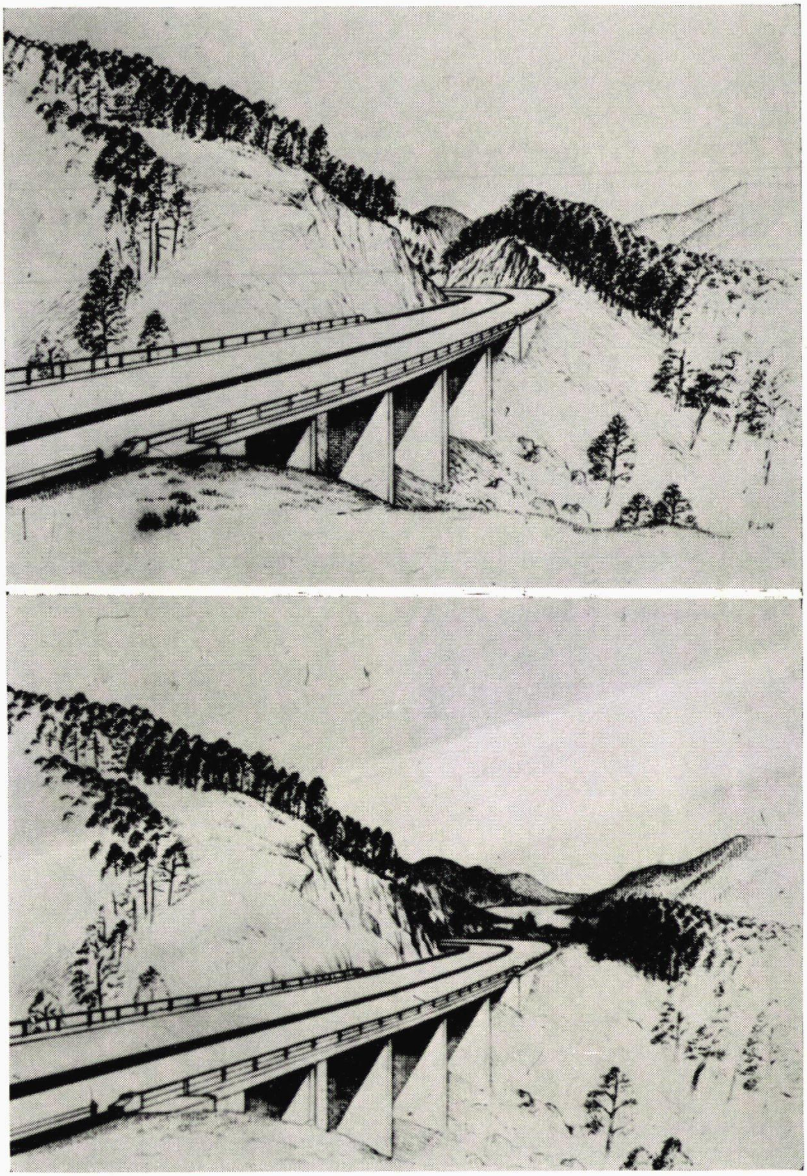


Figure 22. Rock formation removed to open a view.

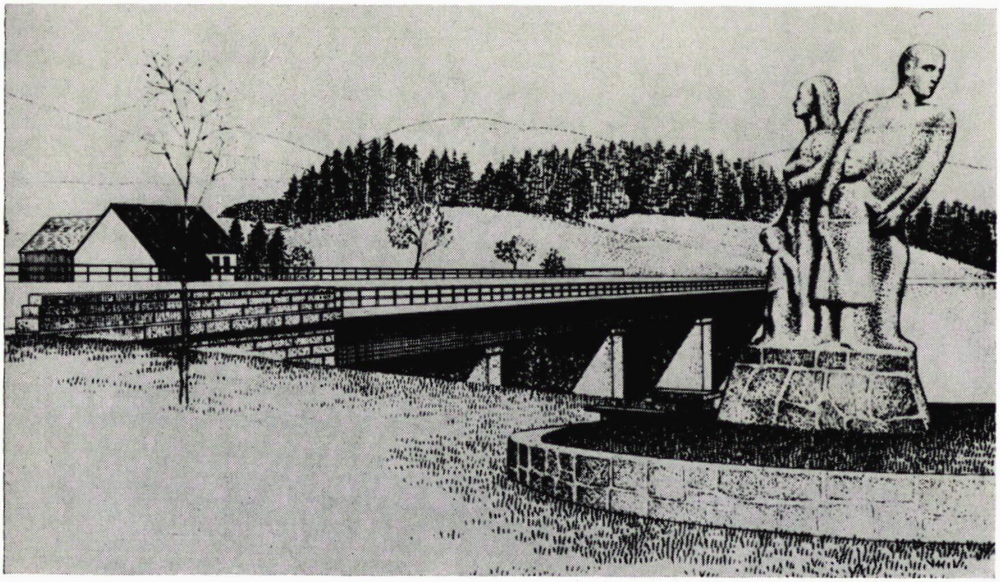


Figure 23. Small sculpture group in rest turnout on Austrian freeway.

A New Field Test for Highway Shoulder Permeability

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Many pavement failures can be traced to a loss of support by the roadbed. This condition is often the result of inadequate drainage through highway shoulders that lose their desired permeability when water becomes trapped beneath the pavement. This paper presents a new method of testing the drainage of highway shoulders in the field. The test is rapid and enables the operator to observe visually the advance of a saturated moisture front throughout a shoulder cross-section by noting the lighting sequence of small neon bulbs arranged in a grid on a panel. These bulbs are electrically connected to steel probes driven into the shoulder at intervals along its width. The test and the equipment were developed with a full-scale laboratory model of a highway shoulder, using various gradations as the single variable. Special graphs were plotted to enable an operator to get a general idea of the permeability of the material in a very short time while the test is in progress. The field tests show good agreement with the laboratory data. Further field work is under way on major highways in several parts of the State of Washington.

• WHEN a granular base course is built to support traffic as well as to drain, it must remain open-graded throughout the pavement life. Considerable subgrade intrusion may occur if free water is present in the base (*1*).

Drainage must be adequate to lower permanently the ground water table to an acceptable depth below the pavement surface (*2*). If the granular subbase does not extend to the ditch line or if the material becomes clogged with fines, water may be trapped and prevented from flowing laterally. This condition causes a partial or total loss of stability that ultimately produces pavement failure.

This paper presents a new approach to the problem of evaluating in the field the ability of a highway shoulder to drain. The new test gives a rapid evaluation of relative permeability with a minimum of disturbance to material and traffic. It analyzes the entire shoulder cross-section rather than localized samples which are not necessarily representative. The test classifies drainage characteristics into categories of permeability that are excellent, acceptable, borderline, and unsuitable. When the data show that a road section cannot drain water at a suitable rate, the area may be subjected to more detailed tests. These should include mechanical analyses and field densities of representative samples.

BASIC TEST

The test equipment consists of a water supply, a perforated water injection pipe, several steel probes, connecting cables, and an observation panel (Fig. 1). The position of a moisture front is indicated on the panel when the water short circuits electrodes in probes that have been driven into the highway shoulder. The electrical shorting of each electrode lights a corresponding neon bulb on the panel (shown as solid black circles). An operator is thus able to "see" the shape and the rate of advance of the water envelope. The dotted line drawn through or directly above the glowing neons approximates the surface and depth of the saturated material.

Figure 2 shows the detection of a case of partial "plugging" within a highway shoulder. Water does not flow as quickly through the clogged areas as through the surrounding material. The timed lighting sequence of the neons shows this.

This new test was developed in a full-scale laboratory model of a highway shoulder. Objectives of this model (described in Appendix A) were the following:

1. To determine a permeability standard for an index of relative permeability.

2. To establish a compaction routine insuring uniformity of test conditions.

3. To compare drainage in highway base courses and shoulders with

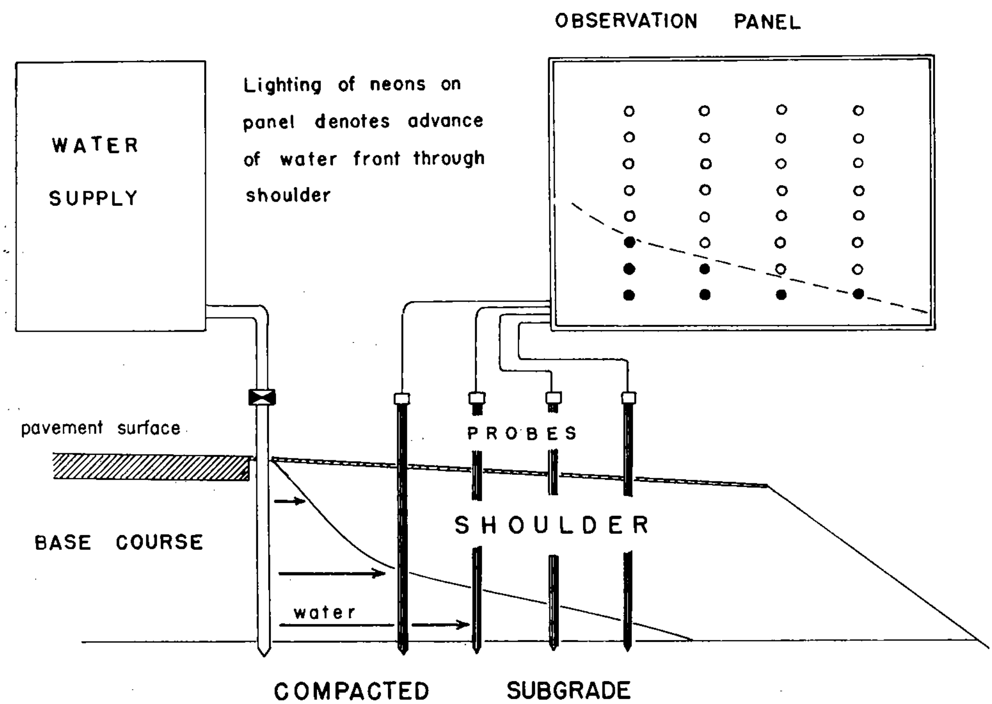

Figure 1. Simplified sketch of test and equipment.

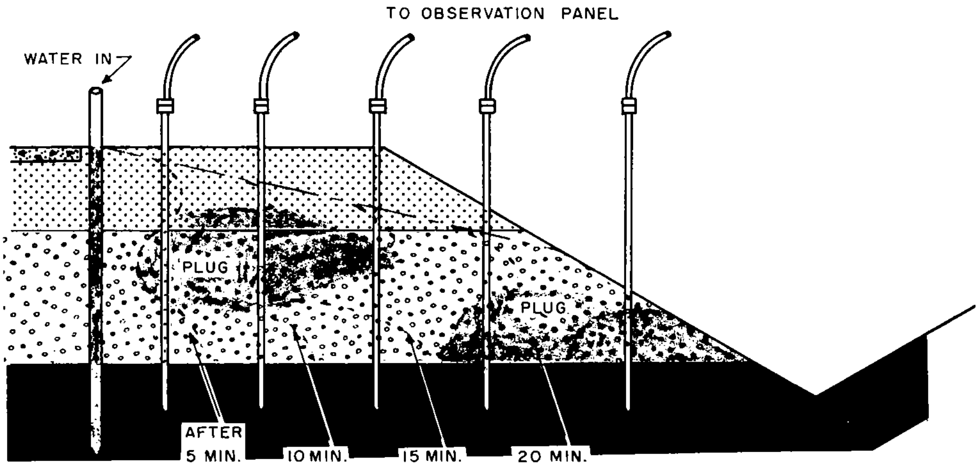

Figure 2. Diagram showing effect of shoulder "plugging" on advancing water front.

drainage in a model consisting of similar material.

4. To study the type of correlation applicable to laboratory model and field conditions.

FIELD TEST PROCEDURES

Setting up the equipment is simple. The perforated water injection pipe and probes are driven into the highway material. The protective caps are removed, and the probes are cable-connected to the observation panel. An alternating current is then introduced into the entire circuit. Direct current should not be used because of polarization. The test begins and is timed from the moment water flows in the injection pipe. The water level in the pipe must be held constant throughout the test to avoid misleading and inaccurate data. All data in this report were recorded from tests made with water in the injection pipe under atmospheric pressure at ground surface level.

If a field test is to be practical, it must quickly bracket the degree of permeability of base course or shoulder material. The amount of time taken to complete a test can be shortened considerably by the proper choice of spacing between the probes and the injection pipe. This calls for an educated guess, based on observation of the material and knowledge of its characteristics. For example, an open-graded base course may be expected to pass water at a velocity of 1 to 5 ft per min, whereas another one with more fines might show velocities of only a few hundredths of a foot per minute. As will be shown later, a definite relation exists between velocity and the amount of fines in a compacted material. The operator need only have a rough idea of the graduation in order to estimate the probable range of velocity and, consequently, to know how far to space his probes.

The probes must be driven through the entire depth of the course whose drainage is to be studied. It is important to have the lowest electrode on each probe imbedded into the subgrade material. Failure to observe this may result in misleading observations, because water can flow over the relatively impervious subgrade but underneath the lowest electrode. It is also possible for water to flow downward at a faster rate than outward. Whether this is due to high vertical permeability or to channeling from internal cracks, the situation produces no neon lighting. The problem can easily be recognized if the operator keeps track of his water input in the material.

DATA INTERPRETATION

Analysis of test results indicates a satisfactory correlation between velocity, relative permeability, and the amount of fines in the material.

Tables 1 and 2 summarize the basic information gathered from tests performed in the model and on the highways. Series I, II, and III were tests done on the same model shoulder using ⅝ in. minus crushed basalt. After completion of a test series, the material was removed from the

TABLE 1

SUMMARY OF DATA, LABORATORY AND FIELD TESTS [1]

Test Series	% Passing Sieve No. 40	No. 80	No. 200	$\overline{V}_s$ (ft/min)	$\overline{V}_d$ (ft/min)	γ Dry (pcf)	K_s (ft/min)	K_d (ft/min)	K_{fh30} (ft/min)
Lab:									
I	3	2	1	0.40	1.5	127	4.8	16.7	0.16
II	8	4	2	0.08	0.30	122	0.72	2.6	0.027
III	18	10	5	0.04	0.13	127	0.33	1.1	0.013
Field:									
A	4	2	1	—	1.5	121	—	—	0.15
B	7	5	3	—	0.4	131	—	—	0.029
C	16	11	9	—	0.15	138	—	—	0.014
D	37	24	17	—	0.03	141	—	—	0.000 [2]

[1] V_s=velocity of water within aggregates, taken below stabilized slope of water envelope; V_d=velocity of advancing front of water; K_s = permeability (from V_s/S in which S = slope of water envelope); K_d = permeability (from V_d/S); K_{fh30} = permeability (from falling head permeameter after 30-min saturation).

[2] Permeabilities between 0.00004 and 0.001 ft/min indicate possible experimental errors.

TABLE 2

COMPARISON OF VELOCITY, DENSITY, AND GRADATION, FIELD TESTS

Test Series	V (ft/min) [1]	γ Dry (pcf)	⅝-Minus Fraction of Sample (% passing) ¼-In.	No. 10	No. 20	No. 40	No. 80	No. 200	K (ft/min) [2]
D	0.025–0.045	141	—	—	—	37	24	17	0.2–0.4
B	0.1–0.2	138	58	28	20	16	11	9	1–2
C	0.3–1.0	131	43	17	10	7	5	3	3–6
A_1	0.6–1.2	122	41	20	11	7	3	3	6–12
A_2	1.2–2.5	121	34	12	6	4	2	1	12–25
A_3	1.1–2.0	118	26	10	7	4	2	1	11–20

[1] Measured from 1 to 2 ft up from shoulder break to 1 ft past shoulder break.

[2] Computed on premise if materials containing similar fractions of minus 40 fines exhibit same velocities and permeameter values, then slope of water flowing through these materials is also similar.

flume, remixed with added fines and recompacted. Permeability was computed from the general Darcy equation in two ways: (a) using dynamic velocity, and (b) using velocity of water within the saturated material after flow conditions had reached stability. Dynamic velocity is defined in this paper as the velocity of the foremost edge, or toe, of a water envelope moving into relatively dry shoulder or base course material. Model test results show that dynamic velocity is approximately 3.1 times as great as the velocity measured with stabilized flow conditions.

The two types of velocities were studied to determine if the dynamic velocity could ultimately be used in the field, because it is observed quickly and easily without additional equipment. Data show that this is possible, because the two velocities are related by a constant.

Observation of velocity within a saturated material, or a fluid within a fluid, was done successfully with a special technique. This consists of the injection of a strong saline solution to the water feeder pipe, and the timing of the passage of its front by noting the sudden illumination of a

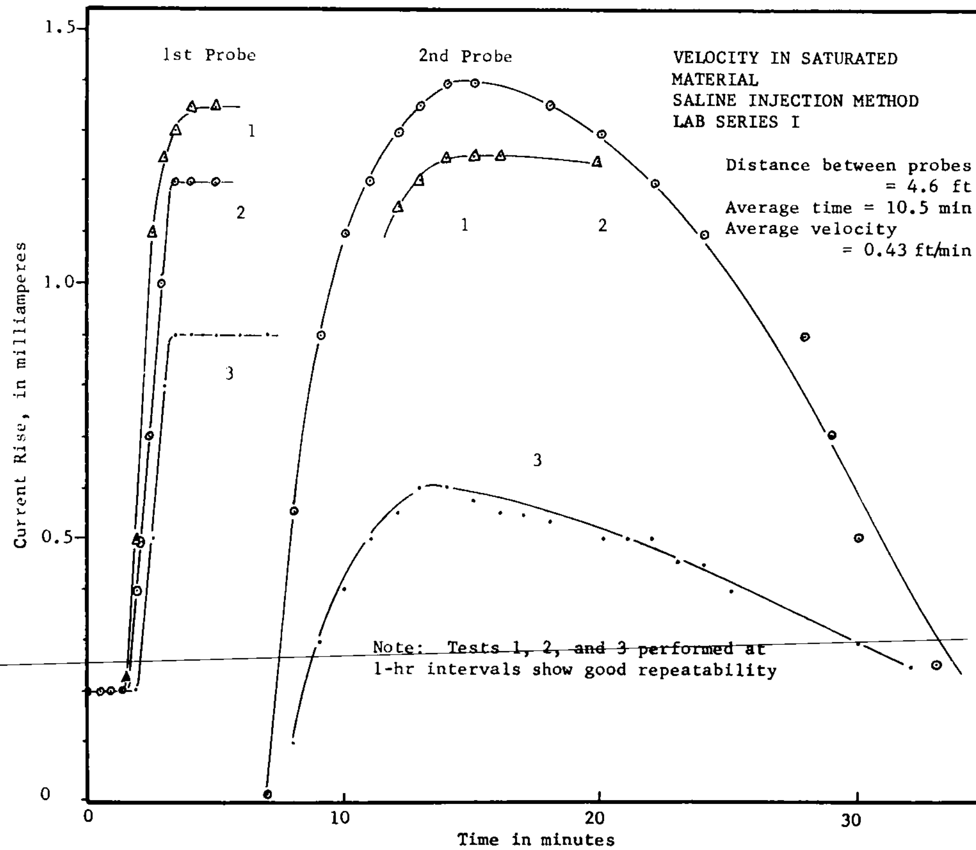

Figure 3. Electrode current vs time.

neon bulb or by plotting a graph of electrode current vs time. Figure 3 shows three typical time-current curves for Series I. Although this technique calls for more skill than field crews may possess, there seems to be little need for this method because use of the dynamic velocity gives results that are reliable enough. By adding salt to the water, the test can be conducted in rainy weather or with a roadbed already saturated. This increases the usefulness of this new test.

Figure 4 presents one of the significant results of the model tests: the agreement between permeabilities obtained in the model and permeabilities recorded from permeameter tests. These values are plotted as a function of the amount of fines in the material.

For each gradation, permeability was calculated in the model from observed velocity and slope, using Darcy's general equation for laminar flow, $K = \frac{V}{S}$. The material for each of the three test series was also recompacted in cylindrical molds at corresponding densities and subjected to a falling head permeameter tests. The resulting permeabilities were calculated from the equation $K = \frac{aL}{A\Delta t}\log_{10}\frac{h_0}{h_1}$. The two graphs are parallel, which indicates the linear relationship, hence the excellent correlation between the two methods.

The dotted vertical lines for the three test series are bandwidths containing the data scatter of all the permeabilities obtained. Each point inscribed by a circle represents a mean value. Thus each bandwidth

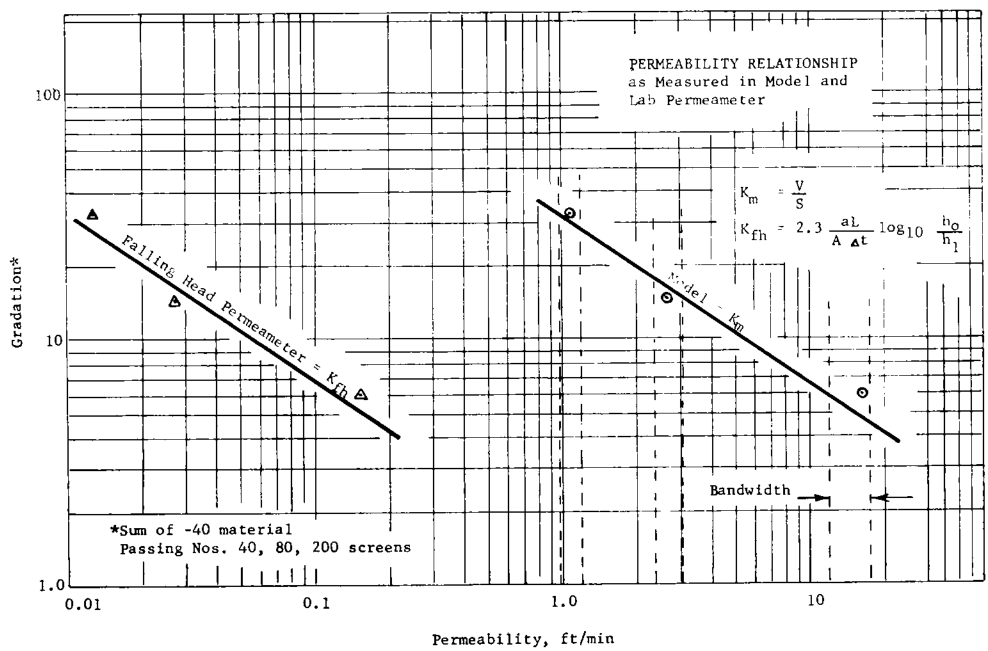

Figure 4. Permeability vs gradation.

serves as an index for estimating the relative accuracy of any one test in the model.

Table 3 gives typical data for the coarsest gradation (Series I) compacted in the model. It illustrates how the permeability bandwidth for Figure 4 depends on the maximum and minimum values of slope observed in the 8 tests listed, or 0.015 and 0.085, respectively. When divided into the virtually constant value of velocity (under stabilized flow) the corresponding permeabilities are 4.1 and 5.1 ft per min. Using dynamic velocities, the maximum and minimum permeabilities are 11.4 and 18.9 ft per min, respectively.

TABLE 3
MODEL DATA FOR TEST SERIES I[1]

Test No.	S	K (ft/min)	V_s (ft/min)	V_d (ft/min)	$\frac{V_d}{V_s}$
1	—	—	—	—	—
2	0.085	5.0	0.43	1.2	2.7
3	0.085	5.1	0.43	1.4	3.1
4	0.094	4.6	0.42	1.6	3.5
5	0.086	5.0	—	—	—
6	0.082	5.2	0.43	1.5	3.2
7	0.105	4.1	0.42	1.5	3.2
8	0.089	4.8	0.44	1.4	3.0
Avg.	0.090	4.8	0.43	1.5	3.1

[1] S = measured stabilized slope of water envelope; K = Permeability measured from $\frac{V_s}{S}$; V_s = velocity of water below stabilized slope of water envelope at ⅓ depth; V_d = velocity of advancing water front, referred to as dynamic velocity.

The velocity of a water front through highway shoulders can show a variety of patterns, as shown in Figure 5. In some cases, velocities are several times higher when measured 1 ft from the injection pipe than when observed 2, 3, and 4 ft away. In other instances, practically no velocity change is observed at any distance between injection pipe and the

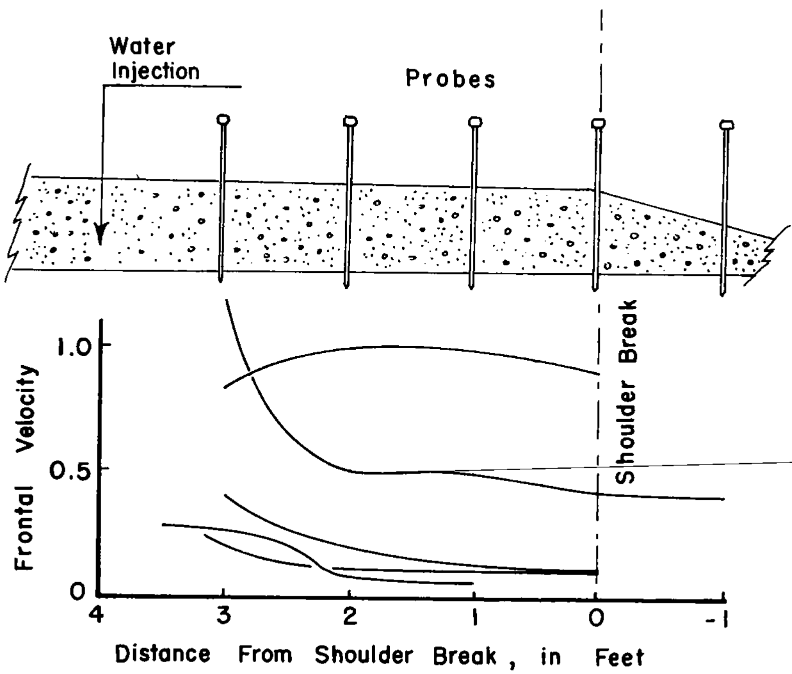

Figure 5. Typical velocity profiles in highway shoulders.

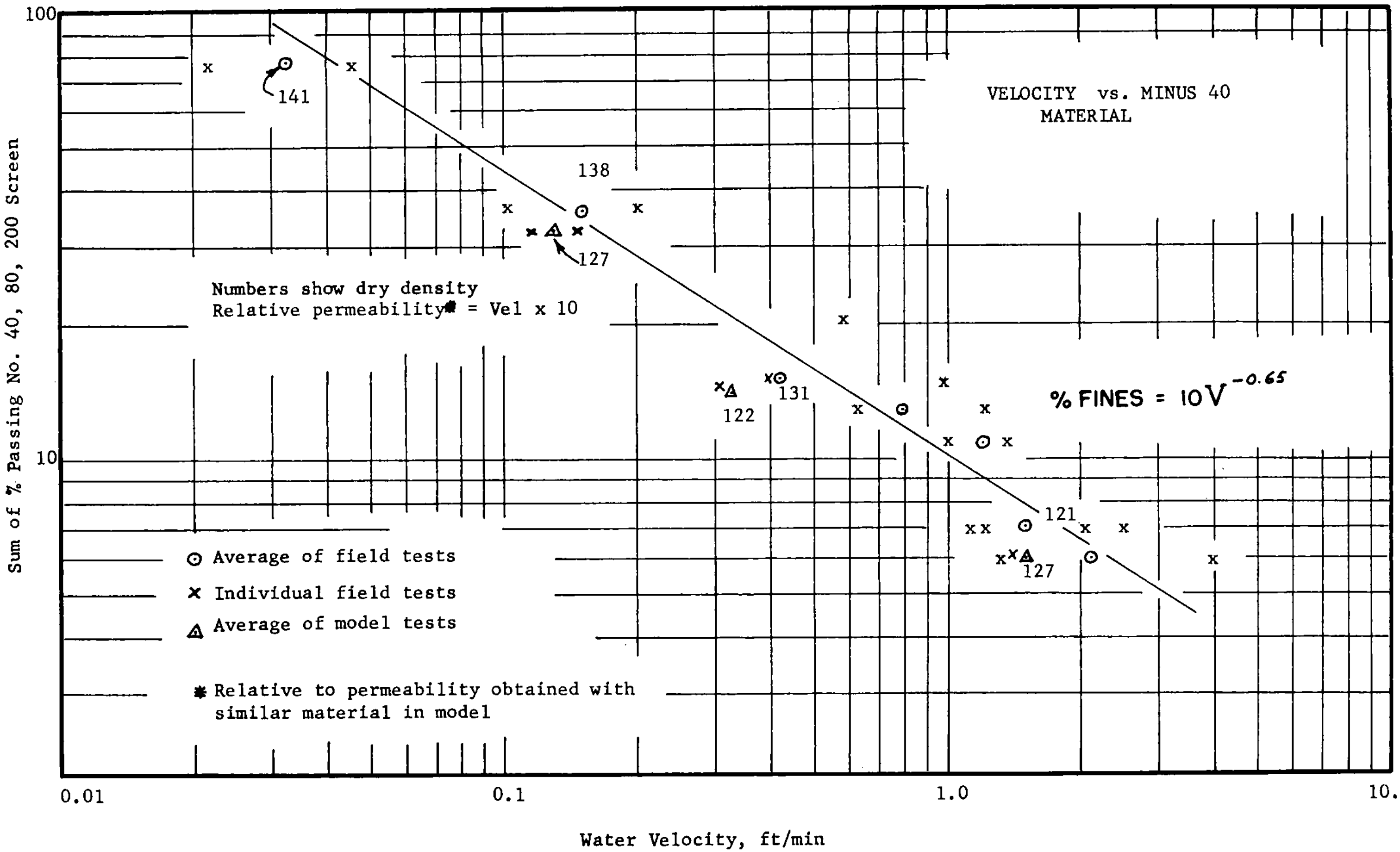

Figure 6. Relationship of fines to velocity.

furthest probe, which may or may not be past the "break" in shoulder slope. These velocity patterns may be partly the result of subgrade geometry which could produce flow conditions ranging from three-dimensional to two-dimensional. Cracks and poorly graded material could also influence the velocity pattern.

In practice, a satisfactory and representative average velocity can be obtained if readings are taken at least 1.5 ft away from the injection pipe.

Velocity is a practical index for determining drainage characteristics in the model and on the highways. This is shown in Figure 6, where it has been plotted as a function of the sum of minus 40 material passing Nos. 40, 80, and 200 screens. Observed velocities in the field are changed to relative permeabilities simply by multiplying by 10. This is legitimate because slopes observed for the three different gradations in the model were in the order of 0.1. Slight deviations have negligible effects on the permeability. For all practical purposes, the slope may be treated as 0.1 without introducing significant errors. It can therefore be stated that $K = 10V$ when model and highway tests are performed identically.

ANALYSIS

Interpretation of data from experiments on permeability—relative density—D_{10} relations conducted by Kane (*3*) and Burmister (*4*) shows that for material with D_{10} values similar to those of the flume model gradations, increasing the relative density from 50 to 90 percent causes the permeability to drop by a factor of 2 to 3. This is far less significant than the 15-fold decrease in permeability observed with model test Series I and III when the amount of fines was increased 5 times. In practice, the relative density of a highway base course or shoulder deviates little from a mean value, although intrusion of fines may raise the percentage of minus 40 material several fold. Figure 7 is a dimensionless graph

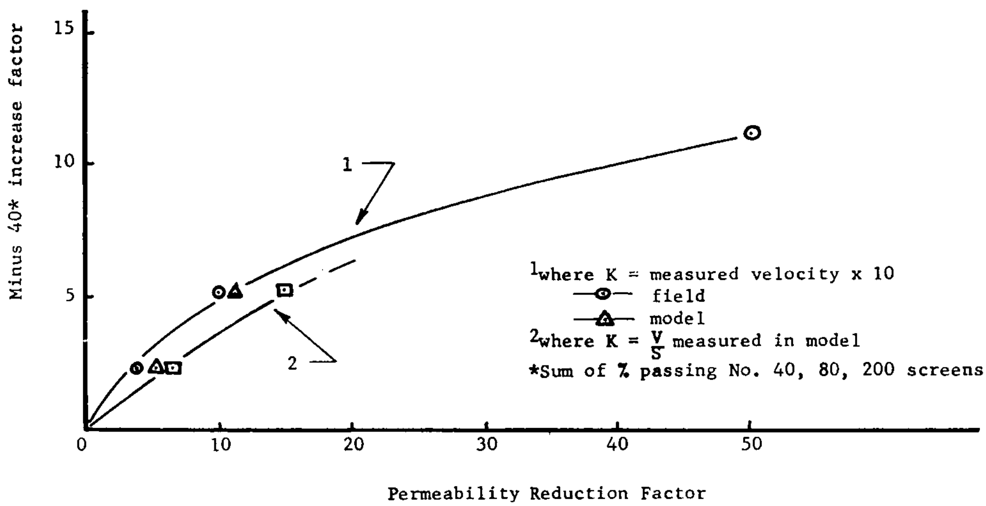

Figure 7. Effect of increasing contents of fines on permeability.

showing that the permeability reduction factor is not a constant in relation to the sum of fines passing Nos. 40, 80, and 200 screens.

The results of single permeameter tests on loose material are often misleading. Difficulties in sampling and in reproducing field conditions in the laboratory may lead to large errors. Many tests are needed. Permeability tests are subject to certain experimental errors (*5*) such as may arise when a filter skin of fine material forms on the surface of the sample, or air bubbles block the voids and reduce the permeability. The data for Series D field tests may reflect such experimental errors. The discrepancies in values observed invalidate the practical value of permeameter tests for materials with high percentages of fines. For the coarser gradations, such a test was useful in establishing the relationship between velocity and permeability, as calculated by the electric probe test method. In the field, however, information on the gradation of fines is simpler to obtain than permeameter test data, and it provides reliable index of permeability. Figures 8 and 9 show the gradation of the materials tested in the model and on the highways.

There has been much investigation made to obtain the value of K in Darcy's $V=KS$. Hazen (*6*) related it to the effective size of the material (D_{10}) which is the grain size shown by mechanical analysis to be smaller than 90 percent of the material. Laboratory permeability tests have been devised for different materials (*7*).

Examination of actual road sections sometimes reveals pronounced and erratic segregation of particle size. It is therefore realistic to visualize that as water seeps outward underneath the pavement and through the shoulder material, the flow may alternately shift from laminar to turbulent conditions. Only the steady state of flow through material fully saturated can be analyzed with precision (*8*). Another type of complex flow occurs below a temporarily elevated ground water level or free water surface having various degrees of saturation and air clogging of the voids. A good example of this is the flow of water through a base course beneath a pavement during rainy periods.

This new test is designed to indicate over-all drainage within a granular complex. The degree of precision need not be as critical as for situations requiring detailed analysis through special laboratory tests. Data show that in the model, flow was essentially laminar for the three gradations tested. It might be interesting from an academic standpoint to observe the velocity-slope relationship for a coarse, open-graded material through which the flow would be in the turbulent range. However, inasmuch as this test is designed primarily for use on highways, such study would not be of immediate importance.

Special pumping tests for the horizontal permeability of various undisturbed sand layers (*9*) in the alluvial valley of the Mississippi River showed a definite relationship between the effective grain size, D_{10}, and horizontal permeability. Table 1 indicates such a correlation between the minus 40 fractions and permeability.

Some decision must be made on the choice of permeability ranges for classifying the drainage capacity of the material under test. In effect, these ranges will represent a set of standards. Whether the test is used

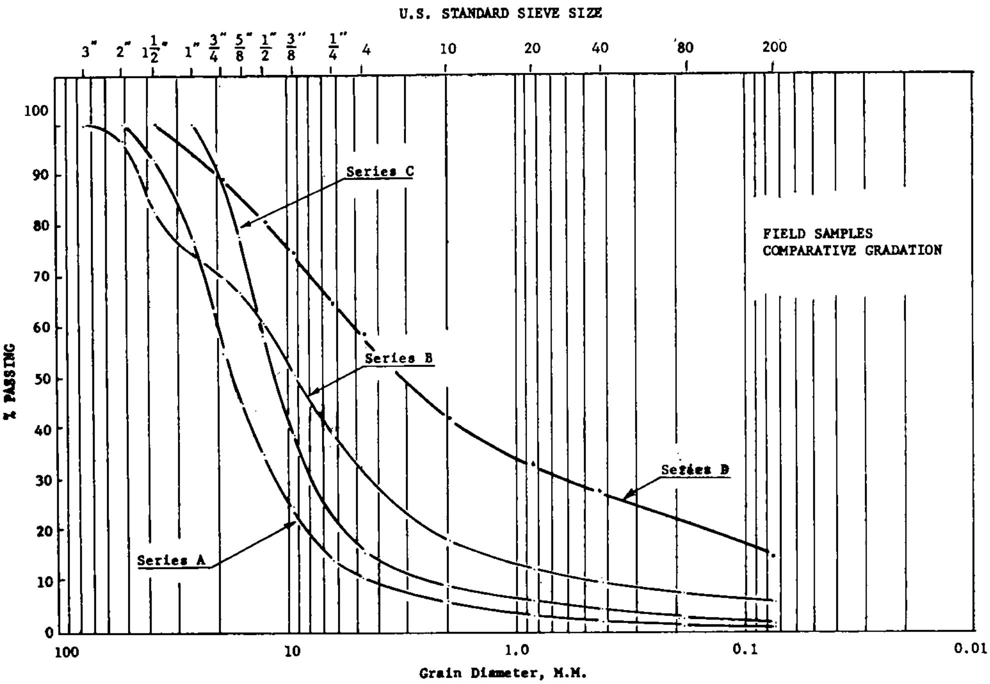

Figure 8. Mechanical analysis of field tests.

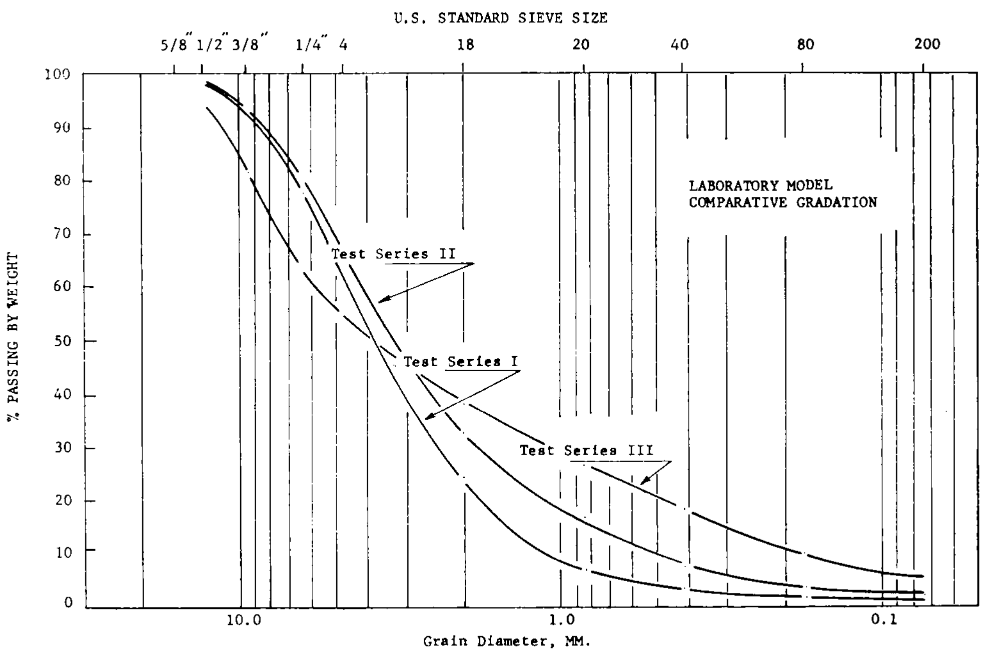

Figure 9. Mechanical analysis of model tests.

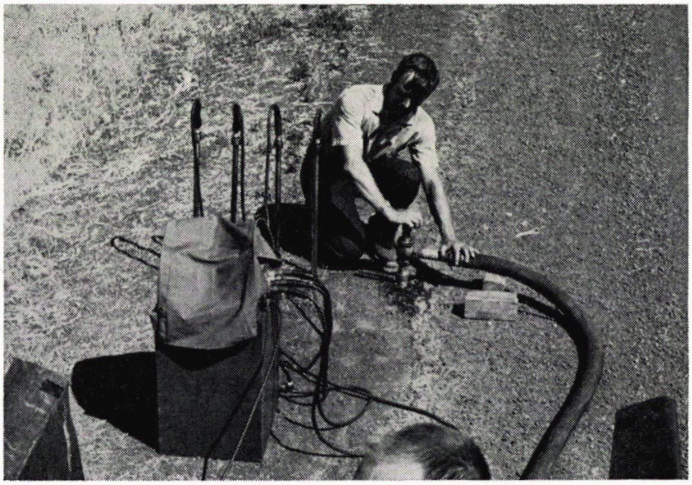

Figure 10. Highway shoulder under test.

to check drainage on a highway construction project, or on existing roads, as shown in Figure 10, the operator must be able to decide right then and there whether to extract samples for further investigation.

CONCLUSIONS

This new test is practical for classifying drainage into such categories as excellent, acceptable, questionable, and bad. These are suggested ratings whose permeability limits remain to be established after additional studies to relate cause and effect of pavement distress, or by the agency adopting this new test.

Studies made in a full-scale laboratory model shoulder and on actual highways indicate permeability can be correlated to the observed water velocity with an accuracy level satisfying the main objective of this field test. Preliminary tests indicate this electric probe method should prove of substantial benefit in highway maintenance. The equipment is easily transported and can be handled by semiskilled personnel with less risk of collecting inaccurate data than with the use of laboratory permeameter tests.

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APPENDIX A

Laboratory Model

The 16-ft wood flume had 4-ft side walls braced with buttresses bolted to the frame, as shown in Figure 11. The maximum lateral deflection recorded anywhere along the side walls was 1/16 in. The concrete floor precluded any deflections in the horizontal alignment of the floor deck on the model.

Because the depth of existing base courses and shoulders seldom exceeds 24 in., the flume was designed for this depth. The 16-ft length allowed testing a full width shoulder section while retaining several feet of clearance between the end bulkhead and the water injection pipe. During the entire testing program, this distance was kept at 4 ft, which proved adequate to minimize back-pooling and possible interference with flow patterns occurring downstream from the point of water injection. It is possible that flow characteristics were defined to some degree by side walls effect. However, because all tests were performed identically in the same flume, this possible variable is treated as an unknown constant.

The roller-vibrator compactor (shown in Fig. 12) has dual 8-in. ribbed steel rollers having 1/4-in. clearance to the walls of the model and mounted on a rectangular steel frame. A pneumatic tamper bolted to the deck of the frame transmits vertical vibrations to the rollers. During compaction, the frequency was approximately 15 cycles per sec which appeared to be the resonance of the

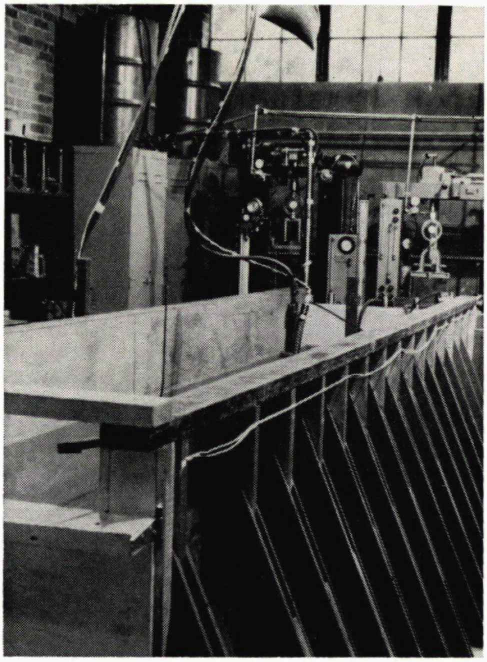

Figure 11. Flume used in model tests.

400-lb compactor. A $\frac{1}{3}$-hp electric motor supplied power to a reduction gear box and chain drive, propelling the unit at 1 ft per sec. The flume automatically reverses when triggered by an antenna atop the compactor. Safety stop switches located behind the reversing switches cut off electric power if the compactor overrides the reversing mechanism.

Compaction Procedure

The aggregate material for all laboratory tests was compacted in $\frac{1}{2}$-in. layers to a depth of 24 in. Each layer was rolled at a moisture content of 5 to 6 percent. The crushed basalt used in all the laboratory tests came from a stockpile of material for road construction and maintenance work. Twenty passes a layer produced a compactive effort within the range of measured field densities.

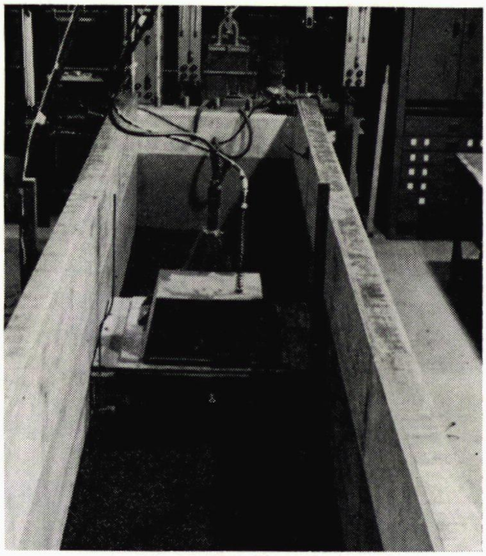

Figure 12. Compactor in flume.

The thin-layered compaction routine formed a uniformly graded and homogeneous deep blanket of material. The maximum density deviation for repeated tests taken along the length and depth of the model was 1.5 lb per cu ft in Series III and 1.0 lb per cu ft in Series I and II. The Washington State Department of Highways densometer was used for all in-place density tests in the model and in the field.

Instrumentation

The electrical monitoring system consists of (a) a source of AC power, (b) an observation and control box, (c) probe assemblies, and (d) interconnecting cables and sockets.

The AC power must supply at least 110 volts to ignite the miniature neon bulb. Wattage rating is determined by the total number of neon bulbs on the observation panel. A vibrator power supply with a 12-volt DC input

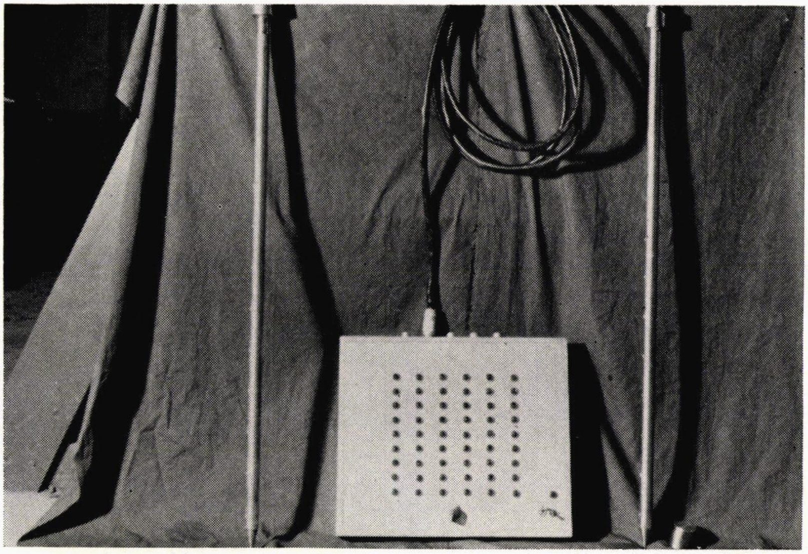

Figure 13. Observation panel and associated equipment.

and 110-volt AC output at 15 watts was used in the field.

The probes used for all tests were 7/8-in. outside diameter steel pipes fitted with a solid steel point at the lower end and a special cap at the upper, or driving, end. The cap slipped over an amphenol socket and rested on the heavy steel collar welded to the probe. It protected the delicate amphenol socket from injury during transit and repeated sledge hammer blows.

Small, evenly-spaced holes were bored through one side of the probe. An insulated wire was centered within each hole and connected electrically to a pin in the socket. Each hole then was filled with an epoxy glue which, after hardening, was filed and contoured flush with the outer surface of the probe. These were the electrodes. The epoxy was not allowed to soften or absorb water. If the epoxy does not possess a high dielectric constant, it may short circuit the electrical potential between the center wire and the grounded steel body of the probe. This would ignite the neon bulb.

The control box, or observation panel, had as many vertical rows of neon bulbs as there were probes. Each vertical row, cabled and connected to a probe, had as many neon bulbs as there were electrodes in each probe. The entire system was grounded through an internal grounding wire as well as through the outer shield of the cable bundle. The wiring circuit was such that shorting any electrode caused the corresponding neon to light on the panel. Figure 13 shows observation panel with two probes and one of the cables.

APPENDIX B

Correlation of Model and Field Data

In Darcy's equation,

$$K_m = \frac{V}{S_m} \qquad (1)$$

The model's equation from collected data is

$$K = \frac{V_m}{S} \qquad (2)$$

In the model and in the field, courses with similar fractions of minus 40 material show similar velocities, or

$$V_m \cong V_f \qquad (3)$$

In the model and in the field, samples from courses with similar fractions of minus 40 material show similar permeabilities with the laboratory permeameter (see Table 1), or

$$K_m' \cong K_f' \qquad (4)$$

It follows from Eqs. 2, 3, and 4 that

$$\frac{K_m'S_m}{V_m} \cong \frac{K_f'S_f}{V_f} \qquad (5a)$$

reassembling,

$$S_f \cong \frac{K_m'S_mV_f}{K_f'V_m} \qquad (5b)$$

Therefore,

$$S_f \cong S_m \qquad (6a)$$

Also

$$K_f \cong \frac{V_f}{S_f} \qquad (6b)$$

Economic Possibilities of Corrosion-Resistant Low-Alloy Steel in Welded I-Section Stringer Highway Bridges

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• THIS IS the second published report on a study, under way at Purdue University in cooperation with the International Nickel Company, Inc., of the economic use of nickel-copper corrosion-resistant types of high-strength low-alloy steels in short- and medium-length highway bridges.

PURPOSE

The objective of the work at Purdue University is to investigate the economic possibilities of nickel-copper high-strength alloy steels in short- and medium-length highway bridges.

A report (*1*) published on the first phase of the study compared designs for the superstructures of typical short-span concrete slab and rolled wide-flange steel stringer highway bridges fabricated from nickel-copper high-strength low-alloy steels with those fabricated from ASTM A7 or A373 structural steels. The results of this first study indicate that this type of highway bridge may, for all practical purposes, be constructed of nickel-copper types of high-strength low-alloy steels at the same first cost in dollars as if A7 or A373 structural steels were used. Thus, any savings in maintenance costs due to the use of nickel-copper types of high-strength low-alloy steels may be obtained at practically no additional cost over the use of A7 or A373 structural steels.

This report covers the second phase of the study, which is an analysis of the comparative economic use of various steels for welded I-section stringers with concrete deck for short- and medium-length highway bridges within the framework of the standard design specifications. The results indicate that structures of this type may be fabricated and erected using a nickel-copper grade of high-strength low-alloy steel at about the same first cost in dollars as if A373 structural steel were used.

The third phase of the investigation, a study leading towards the application of nickel-alloy steels to new forms of highway bridges, is now in progress.

STEELS COMPARED

In this report on the second phase, cost comparisons are made for structural steels with the following ASTM designations: A373, A36, and A242. The structural steel quantities for ASTM A441 steel would be the same as those for ASTM A242 steels. The cost analyses made for ASTM A242 steels are based only on types that contain nickel and copper and provide an atmospheric corrosion resistance recognized to be 4 to 6 times that of carbon steel. Many investigators have confirmed the belief that any improvement in the corrosion resistance of steel produces a beneficial effect on the durability of paint coatings (*1*, p. 66). This may be an important factor in the study of economic maintenance of structures in many locations.

DETAILS OF DESIGN

Because the objective of the study was to make comparative designs in the various steels, the details of the structure used for the comparative analysis should be as generally acceptable as possible. A 7-ft interior stringer spacing was used with a 30-ft clear roadway. Figure 1 shows the cross-section of the typical structure studied for the welded I-section stringer with vertical stiffeners. Channel diaphragms were used for webs up to 38 in. in depth. Cross-frames, made from angles 4 by 3 by $\frac{5}{16}$, were used for webs from 40 to 61 in. in depth.

Bearing details are as found in the Bureau of Public Roads (*2*). The diaphragm details used with the unstiffened web are also from these standard plans.

Designs were made for span lengths of 64 and 68 ft and at 5-ft intervals from 70 to 120 ft in both composite and noncomposite action in accordance with the 1957 AASHO standard specifications for highway bridges. Designs were made at all span lengths for the minimum permissible depth allowed by the span depth ratios. Designs were also made for the following depths based on web buckling ratios: (a) maximum permissible depth for a $\frac{5}{16}$-in. web of A242 steel, (b) maximum permissible depth for a $\frac{5}{16}$-in. web of A373 steel, and (c) maximum permissible depth for a $\frac{7}{16}$-in. web of A242 steel. It is thought that this gives a good bracketing coverage of possible depths for comparative purposes.

Five stringers spaced equally on 8¼-ft centers could also have been used. The total design moment for an exterior stringer is virtually the same as the total design moment for an interior stringer with this spacing as well as with the 7-ft spacing. Thus, an exterior and an interior stringer will generally be the same size regardless of which interior stringer spacing is used. The greater stringer spacing would probably give better

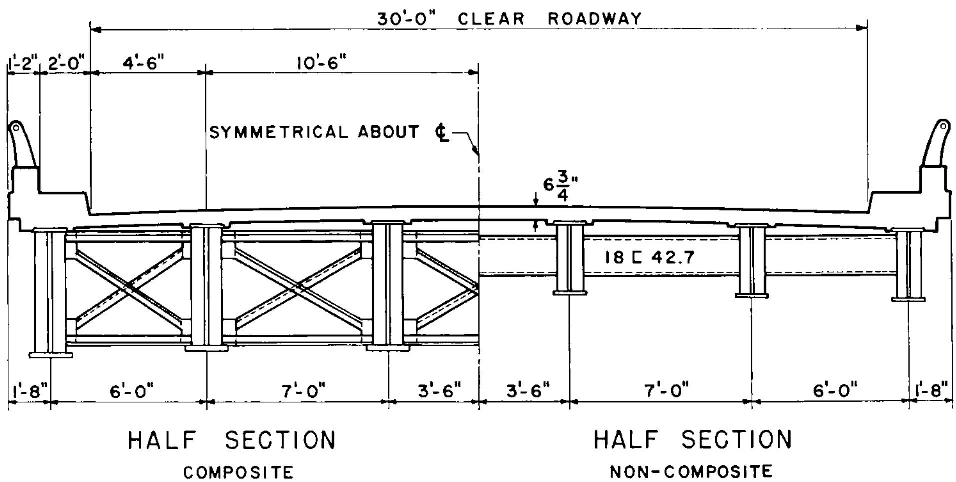

Figure 1. Typical details of comparative designs.

over-all economy as the span length is increased above the maximum length of 120 ft used in this study. The use of the 7-ft interior stringer spacing gives a reasonable comparison between the various steels for the span lengths included in this phase of the study.

DESIGN ASSUMPTIONS

The structures were designed for the H20-S16 live loading in accordance with the AASHO specifications, except that the Indiana practice of using a single-axle load of 32,000 lb for the design of the roadway slab was followed. The slab was further designed for a ½-in. integral wearing surface and a future wearing surface of 15 psf. Many engineers follow slightly different procedures, but the objective was to have designs as comparable as possible. The exterior stringer steel was assumed to be the same as that used for the interior stringer.

The composite designs were made in accordance with the AASHO specifications and the following assumptions:

1. All dead load, including future wearing surface, carried by steel alone;
2. Live load and impact only carried in composite action; and
3. Bottom of concrete slab at top of top flange plate.

No attempt was made to vary the size of the flange plates along the length of the span. This was thought permissible because comparative designs were all that was desired. These assumptions give sufficiently accurate results for comparative estimates using the various steels.

DEFLECTION LIMITATIONS

Section 1.6.10 of the AASHO bridge specifications limits the deflection due to live load and impact to $L/800$ of the span. These deflections may be computed for the standard loading considering all stringers as acting together and having equal deflections, if the diaphragms are sufficient in depth and strength to insure lateral distribution of loads. The standard H20-S16 live loading used consists of three concentrated loads (Fig. 2). The deflection under the

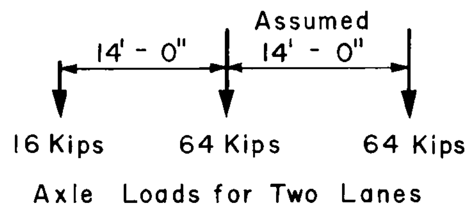

Figure 2. Assumed H20-S16 live loading for deflection computations.

load at the point of maximum moment in the simply supported spans considered here may be taken as the maximum deflection.

Figures 3 and 4 show, respectively, for the stiffened and the unstiffened webs a plot of the required sum of the moments of inertia of all the stringers in a simple span to maintain the specification deflection limitation of $L/800$ of the span for the H20-S16 live loading and impact effect. Also shown are the required sums of the moments of inertia of all the stringers in a simple span to maintain the live load plus impact deflection at both $L/1{,}000$ and $L/1{,}200$ of the span for the H20-S16 live loading. The moments of inertia furnished by all of the stringers for the A242 steel designs with minimum depth in both composite and noncomposite action are shown, as are the moments of inertia furnished by all of the stringers for the A373 steel designs with minimum depth in composite action.

The moments of inertia furnished

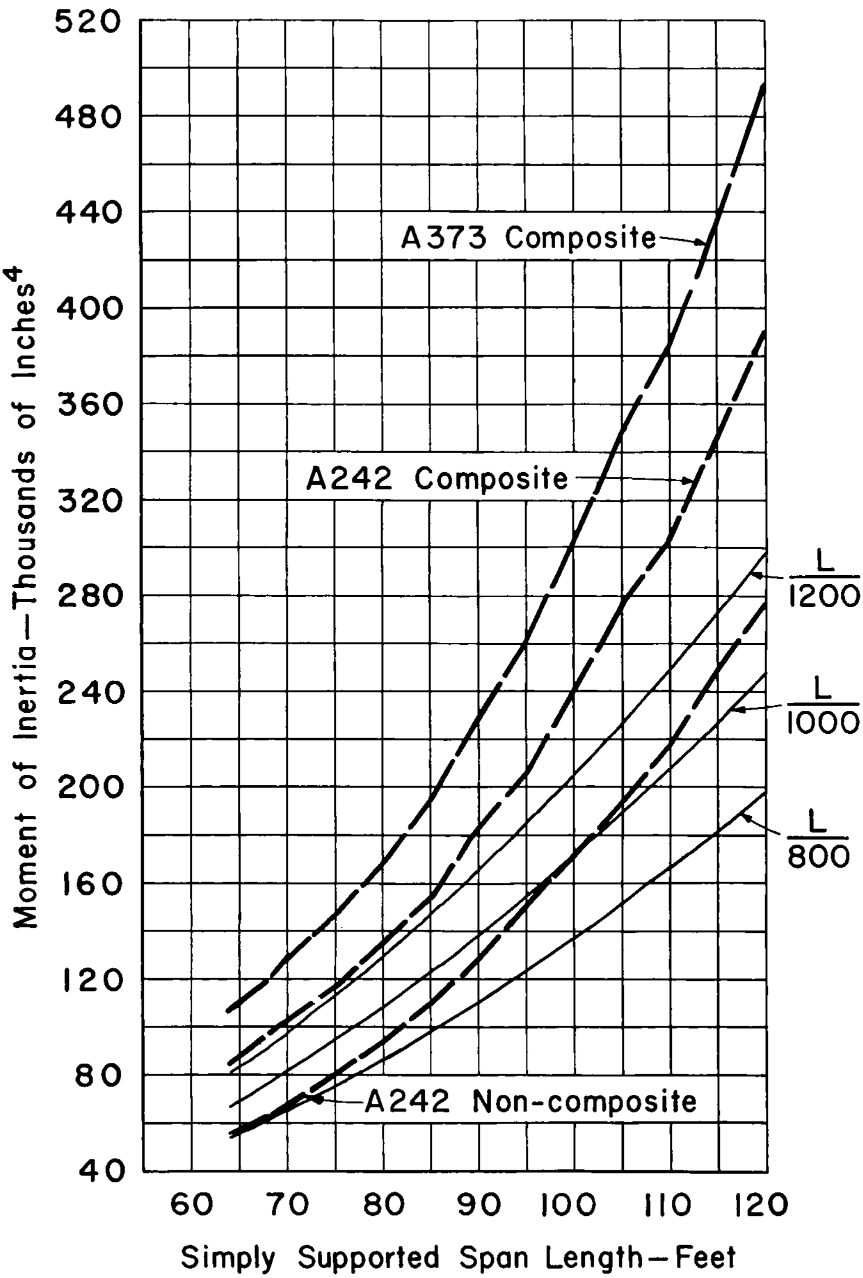

Figure 3. Live load plus impact deflection limitations, vertical stiffeners, minimum depth stringers.

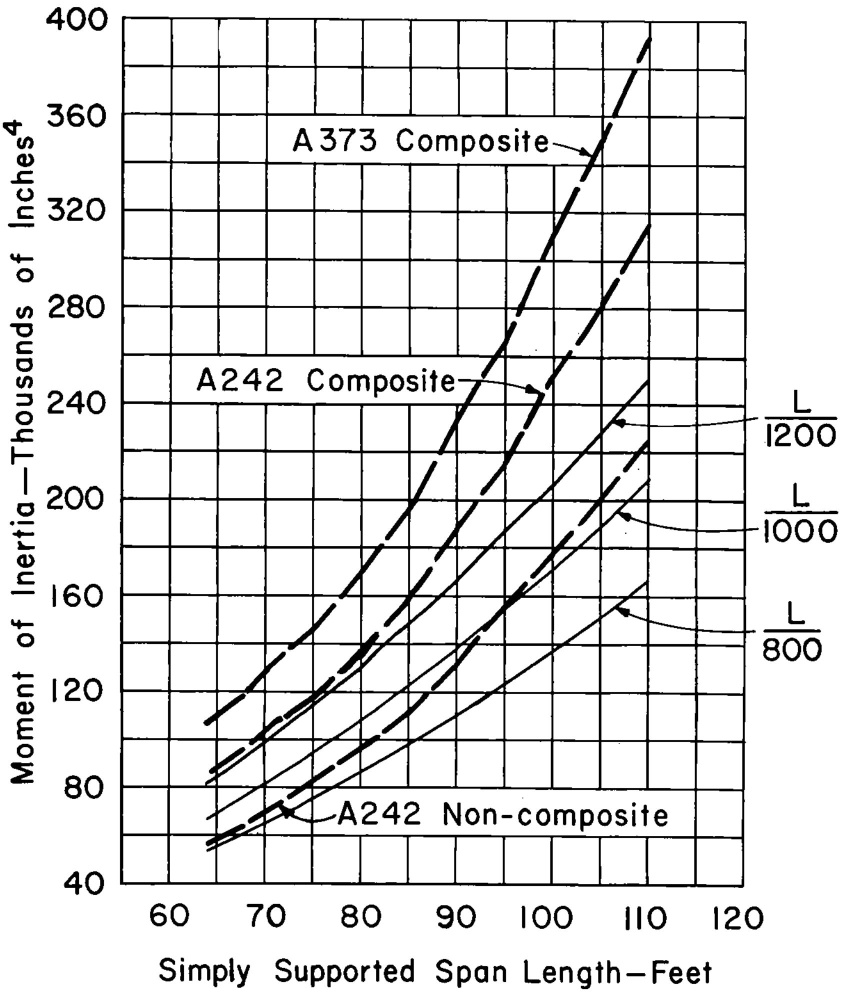

Figure 4. Live load plus impact deflection limitations; no stiffeners, minimum depth stringers.

by all of the stringers for the designs with minimum depth satisfy the $L/800$ deflection requirement. All designs with minimum depth in composite action have deflections less than $L/1{,}200$. The moments of inertia furnished by all of the stringers increases as the depth of the stringers is increased. Consequently, for the deeper stringer designs the deflection limitations are more than adequately satisfied. Thus, the live load and impact deflection limitations of the AASHO bridge specifications are not critical with the welded I-section stringer, if adequate diaphragms are used in order that full live load and impact deflection may be equally distributed to all stringers.

Figures 5, 6, 7, and 8 plot the actual H20-S16 live load and impact deflections in inches for interior

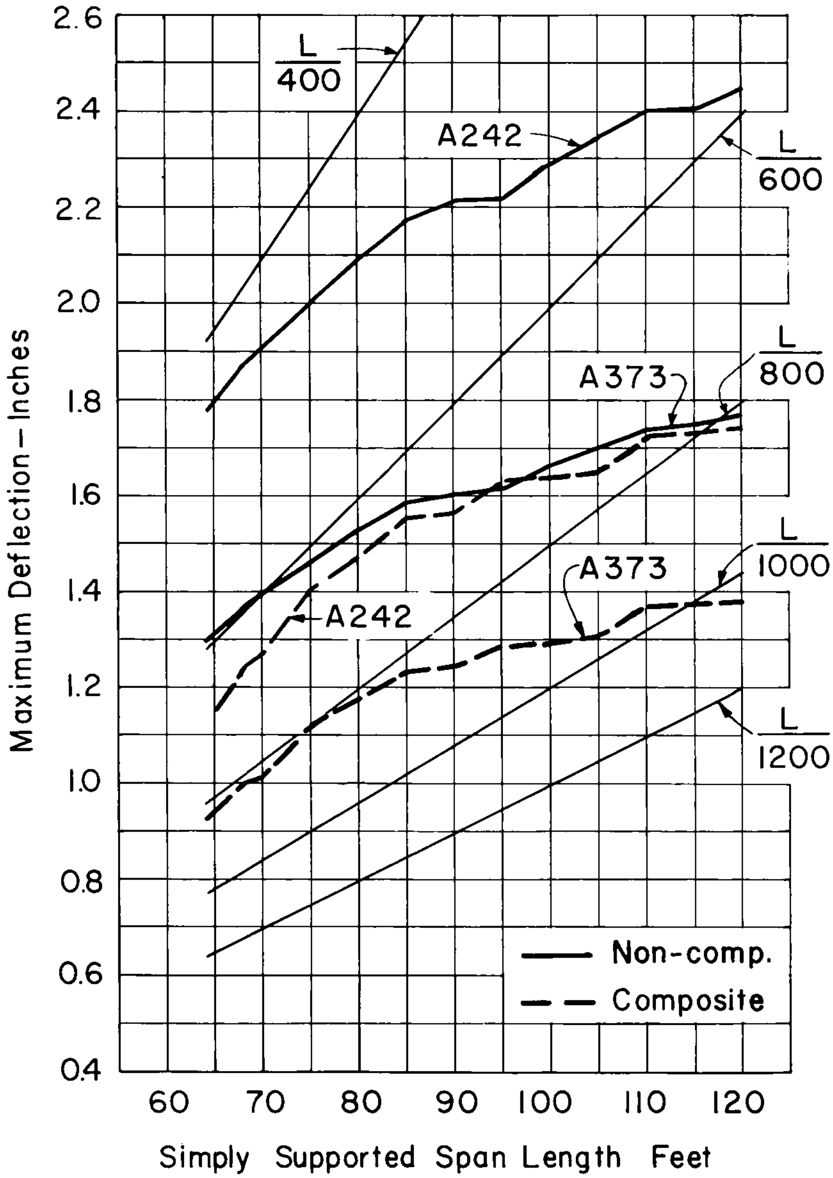

Figure 5. Live load plus impact deflection limitations based on design stress in interior stringer; vertical stiffeners, minimum depth web plate.

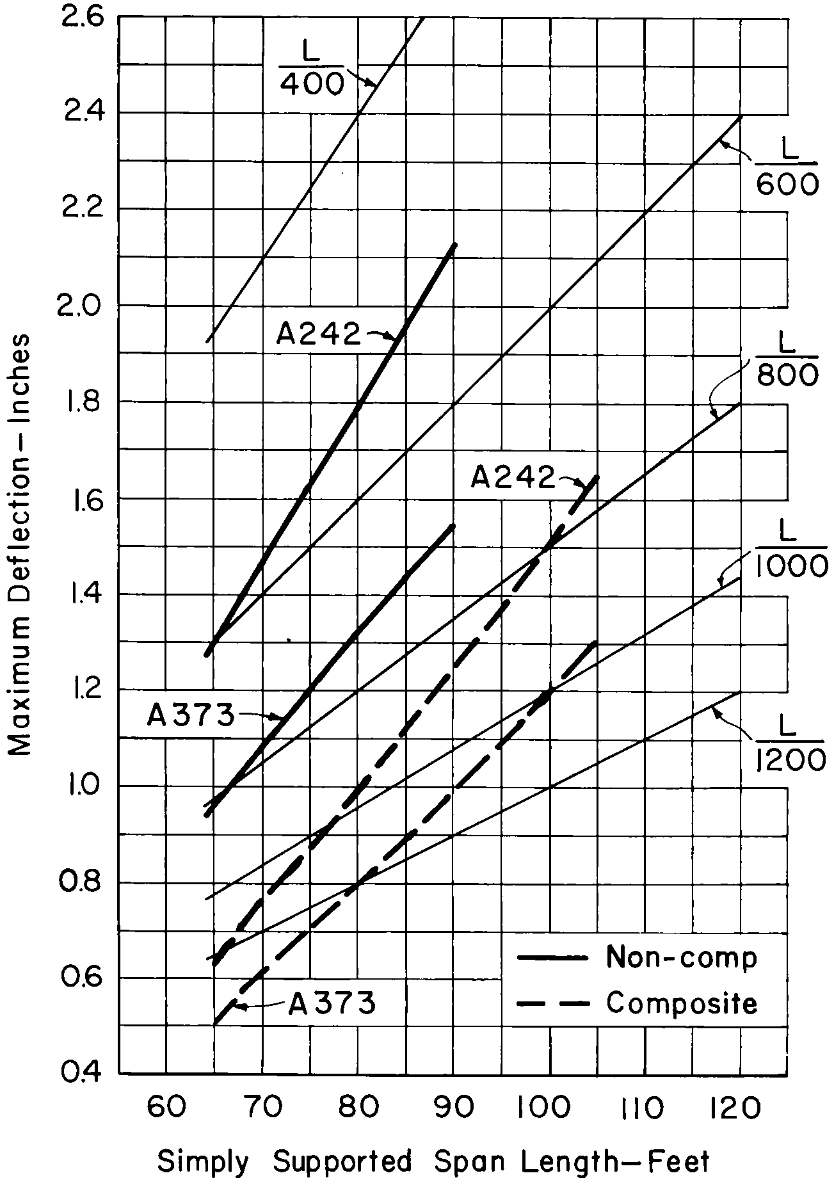

Figure 6. Live load plus impact deflection limitations based on design stress in interior stringer; vertical stiffeners, 43-in. depth web plate.

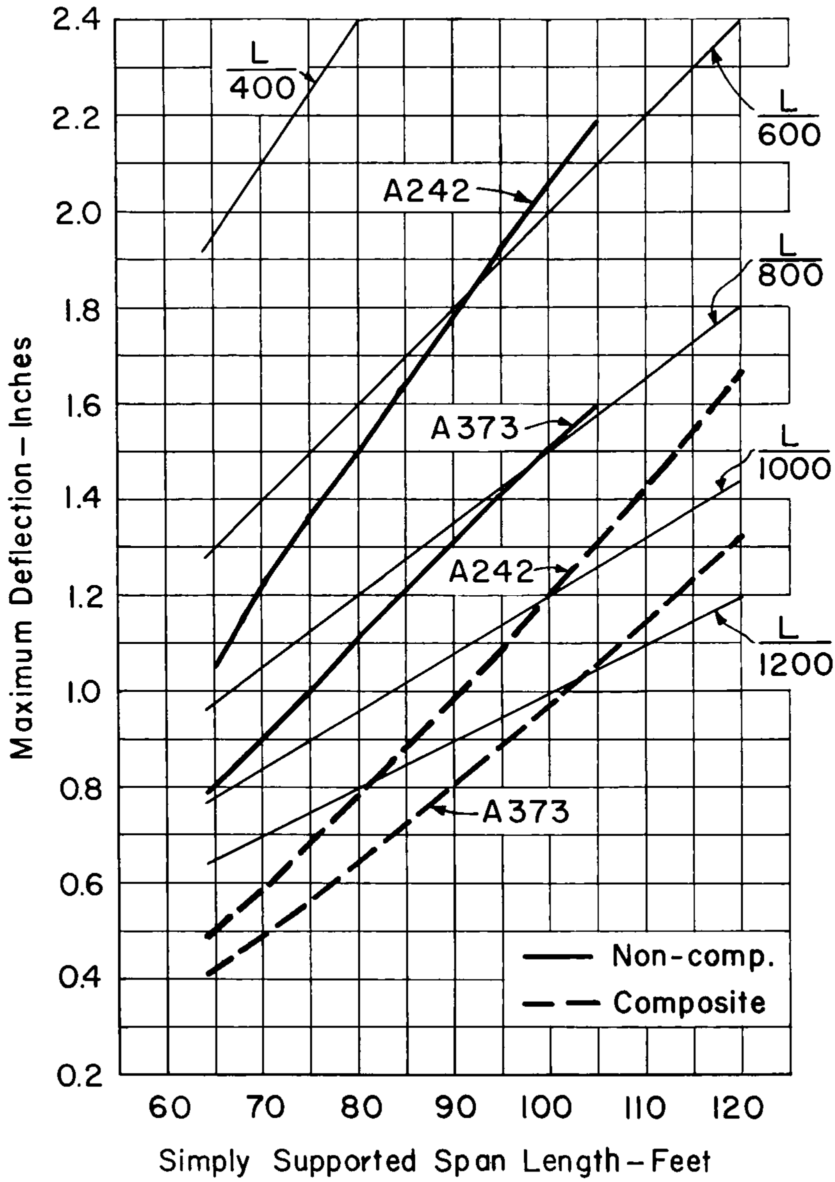

Figure 7. Live load plus impact deflection limitations based on design stress in interior stringer; vertical stiffeners, 52-in. depth web plate.

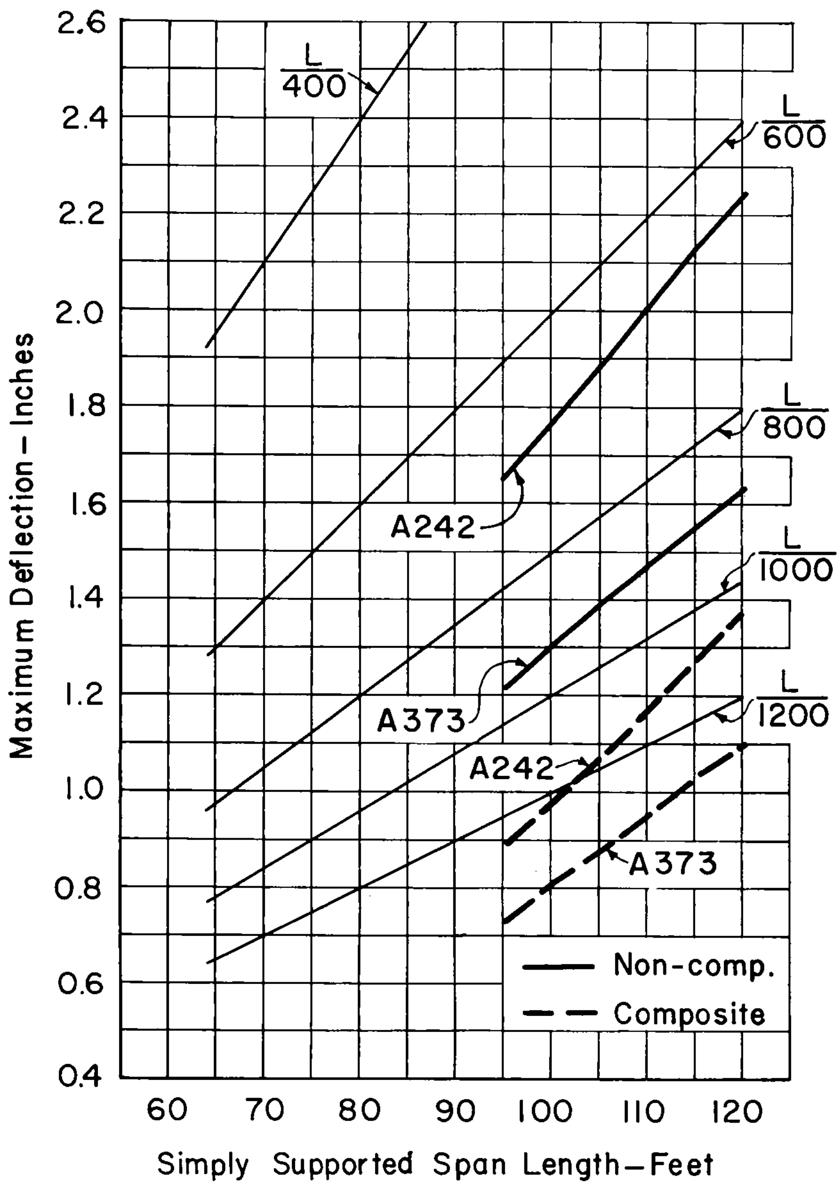

Figure 8. Live load plus impact deflection limitations based on design stress in interior stringer; vertical stiffeners, 61-in. depth web plate.

stringers with 7-ft spacing and stiffened webs where the deflections are computed for the same load distribution as was used in computing the critical extreme fiber stresses. The deflections for the stringers with the unstiffened web are practically the same.

COMPARATIVE QUANTITIES

Tables 1 through 12 summarize the results of the various designs as obtained with an electronic computer. The program used is described in Appendix A. The results were obtained by assuming that the flange plates were each 1 in. thick with the required flange areas concentrated at a distance of ½ in. above or below the top or bottom edge of the web plate. Some of the required areas for the flange plates are too small for a practical-size plate. In general, the smallest practical size of flange plate would be determined by the requirements of erection stability. It is considered that these small flange areas are reliable in obtaining a comparison between the various steels, because a wider stringer spacing could be

TABLE 1
A373 STEEL, NONCOMPOSITE ACTION, VERTICAL STIFFENERS

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)	Weight (lb/ft)		Steel	
	Thick.	Height		I-Section	Total DL	Total Stress (psi)	Moment of Inertia (in.^{4})
64	0.3125	30	24.95	201.53	946.7	18,024	12,691.3
		43	15.95	154.17	905.1	18,038	17,513.7
		52	12.27	138.68	893.6	18,052	20,894.2
68	0.3125	31	26.90	215.83	960.4	17,913	14,546.7
		43	17.77	166.52	916.2	18,080	19,272.0
		52	13.77	148.90	902.4	18,065	23,003.9
70	0.3125	32	27.22	219.07	963.5	17,938	15,672.9
		43	18.74	173.14	922.2	18,071	20,213.3
		52	14.54	154.15	907.1	18,072	24,088.8
75	0.3125	34	28.67	231.09	975.2	17,910	18,584.5
		43	21.23	190.08	937.9	18,077	22,624.9
		52	16.56	167.88	919.4	18,071	26,925.6
80	0.3125	36	30.06	242.67	986.5	17,904	21,792.4
		43	23.85	207.91	954.6	18,088	25,163.3
		52	18.68	182.27	932.6	18,078	29,896.9
85	0.3125	38	31.39	253.85	997.5	17,920	25,304.0
		43	26.67	227.04	972.7	18,076	27,887.2
		52	20.89	197.32	946.6	18,095	33,006.2
90	0.3125	41	31.75	259.46	1,003.5	17,926	29,798.8
		43	29.80	248.30	993.1	17,992	30,913.1
		52	23.27	213.51	961.8	18,079	36,349.9
95	0.3125	44	32.21	265.76	1,010.1	17,907	34,827.7
		52	25.74	230.25	977.7	18,093	39,807.8
	0.375	61	20.39	216.40	967.3	18,063	46,275.2
100	0.3125	46	33.49	276.58	1,021.0	17,938	39,520.3
		52	28.39	248.30	995.0	18,077	43,535.5
	0.375	61	22.52	230.89	981.0	18,097	50,369.6
105	0.3125	48	34.91	288.39	1,032.9	17,908	44,789.2
		52	31.33	268.27	1,014.2	18,001	47,659.2
	0.375	61	24.87	246.88	996.2	18,063	54,890.8
110	0.3125	50	36.21	299.3	1,043.9	17,934	50,340.6
	0.375	61	27.25	263.1	1,011.7	18,084	59,465.7
115	0.3125	53	36.74	306.1	1,051.3	17,906	57,442.8
	0.375	61	29.98	281.6	1,029.7	17,999	64,707.2
120	0.375	55	37.73	326.7	1,072.0	17,921	64,360.2
		61	32.63	299.6	1,047.1	18,022	69,800.1

TABLE 2
A36 STEEL, NONCOMPOSITE ACTION, VERTICAL STIFFENERS

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)	Weight (lb/ft)		Steel	
	Thick.	Height		I-Section	Total DL	Total Stress (psi)	Moment of Inertia (in.^{4})
64	0.3125	30	22.23	183.01	928.22	19,936	11,382.4
		43	14.02	141.02	891.94	20,080	15,641.9
		52	10.73	128.19	883.06	20,048	18,727.2
68	0.3125	31	23.79	194.73	939.25	19,923	12,957.9
		43	15.67	152.27	901.94	20,079	17,243.1
		52	12.05	137.20	890.72	20,079	20,587.2
70	0.3125	32	24.13	198.12	942.55	19,902	13,994.7
		43	16.53	158.07	907.16	20,079	18,067.9
		52	12.73	141.82	894.73	20,095	21,541.7
75	0.3125	34	25.30	208.14	952.22	19,940	16,518.0
		43	18.76	173.25	921.05	20,059	20,229.4
		52	14.55	154.18	905.71	20,063	24,096.0
80	0.3125	36	26.48	218.29	962.12	19,947	19,337.8
		43	21.04	188.79	935.45	20,098	22,440.9
		52	16.40	166.79	917.10	20,093	26,699.4
85	0.3125	38	27.68	228.60	972.28	19,928	22,479.7
		43	23.52	205.61	951.28	20,085	24,836.6
		52	18.39	180.29	929.53	20,084	29,488.7
90	0.3125	41	27.96	233.67	977.66	19,939	26,452.3
		43	26.24	224.12	968.89	20,006	27,470.4
		52	20.46	194.40	942.69	20,088	32,402.5
95	0.3125	44	28.32	239.32	983.69	19,921	30,892.0
		52	22.69	209.55	956.99	20,057	35,532.3
	0.3750	61	17.75	198.51	949.39	20,094	41,217.2
100	0.3125	46	29.48	249.31	993.71	19,922	35,091.3
		52	24.99	225.17	971.84	20,063	38,758.3
	0.3750	61	19.71	211.82	961.89	20,064	44,981.8
105	0.3125	48	30.60	259.06	1,003.53	19,942	39,611.6
		52	27.46	241.98	987.95	20,038	42,228.7
	0.3750	61	21.70	225.33	974.66	20,088	48,800.3
110	0.3125	50	31.77	269.13	1,013.73	19,938	44,567.0
	0.3750	61	23.83	239.81	988.45	20,081	52,891.6
115	0.3125	53	32.19	275.23	1,020.36	19,909	50,816.4
	0.3750	61	26.20	255.91	1,003.95	19,996	57,443.7
120	0.3750	55	32.95	294.17	1,039.48	19,933	56,861.7
		61	28.48	271.41	1,018.89	20,036	61,825.2

used to increase the flange area requirements per stringer without materially changing the relationship between the quantities involved.

Tables 13 and 14 summarize the results of a series of check computations made with the assumption that the top flange plate was $\frac{3}{4}$ in. thick and that the bottom flange plate was $1\frac{1}{2}$ in. thick. The results are not appreciably different from those where both flange plates were assumed to be 1 in. thick. These computations were made only to check the accuracy of considering the flange areas to be concentrated at a distance of $\frac{1}{2}$ in. above or below the top or bottom edge of the web plate and were not used in the cost comparisons. The results are included for information only.

Tables 15 through 18 summarize the results of a series of computations for A242 steels only assuming the flange plates to be $\frac{3}{4}$ in. thick and using a maximum allowable extreme fiber stress of 27.0 ksi. In most instances there are no savings to be reflected here, because the width of flange plate required with the $\frac{3}{4}$-in. thickness would exceed the width permitted by the local buckling ratios. These results are included for information only. They show that no advantage can be taken in this study of the higher allowable stress permitted for A242 steel in thicknesses of $\frac{3}{4}$ in. and less.

The structural steel quantities for complete spans of six stringers with

TABLE 3
A242 STEEL, NONCOMPOSITE ACTION, VERTICAL STIFFENERS

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)	Weight (lb/ft)		Steel	
	Thick.	Height		I-Section	Total DL	Total Stress (psi)	Moment of Inertia (in.^{4})
64	0.3125	30	17.93	153.80	899.01	24,044	9,318.4
		43	11.25	122.17	873.09	24,039	12,957.4
	0.3750	52	7.98	120.56	875.43	23,983	15,601.5
68	0.3125	31	19.29	164.08	908.61	23,911	10,650.3
		43	12.58	131.20	880.87	24,076	14,243.9
	0.375	52	9.06	127.91	881.44	24,044	17,119.5
70	0.3125	32	19.47	166.46	910.89	23,959	11,460.1
		43	13.26	135.84	884.94	24,095	14,904.1
	0.3750	52	9.61	131.67	884.59	24,075	17,896.7
75	0.3125	34	20.48	175.41	919.48	23,906	13,569.4
		43	15.08	148.26	896.06	24,055	16,671.7
	0.3750	52	11.09	141.72	893.24	24,065	19,971.4
80	0.3125	36	21.38	183.60	927.43	23,942	15,846.7
		43	16.95	160.91	907.58	24,090	18,473.4
	0.3750	52	12.64	152.25	902.57	24,052	22,147.7
85	0.3125	38	22.29	191.92	935.60	23,944	18,377.3
		43	18.94	174.49	920.16	24,074	20,406.0
	0.3750	52	14.23	163.09	912.33	24,080	24,386.3
90	0.3125	41	22.37	195.65	939.6	24,046	21,521.9
		43	21.01	188.86	933.3	24,089	22,410.8
	0.3750	52	15.94	174.66	922.9	24,068	26,775.3
95	0.3750	44	22.40	208.41	952.8	23,906	25,340.8
		52	17.70	186.66	934.1	24,076	29,254.3
	0.4375	61	13.44	182.10	933.0	24,083	34,100.0
100	0.3750	46	23.26	216.83	961.2	23,922	28,735.0
		52	19.58	199.46	946.1	24,052	31,897.3
	0.4375	61	15.00	192.76	942.8	24,076	37,113.1
105	0.3750	48	24.16	225.49	970.0	23,909	32,459.8
		52	21.59	213.14	959.1	23,999	34,722.9
	0.4375	61	16.63	203.80	953.1	24,087	40,232.5
110	0.3750	50	25.03	233.95	978.6	23,918	36,457.8
	0.4375	61	18.40	215.88	964.5	24,021	43,646.3
115	0.4375	53	24.85	247.83	993.0	23,925	41,661.5
		61	20.17	227.86	975.9	24,049	47,032.5
120	0.4375	55	25.74	256.82	1,002.1	23,922	46,419.4
		61	22.08	240.88	988.4	24,017	50,712.6

bearings and diaphragms are shown in Figures 9 to 12. These quantities do not include the shear connectors, because it was assumed that the quantities and type involved would be about the same regardless of the steel used. The weights of the welded I-section stringers per foot (Tables 1 through 12) were used in making the estimates. The nearest available size plate would be used in practice. The weights of the web stiffeners and weld material per foot were added to these stringer weights. The weights of the channel diaphragms or the angle cross-frames were also included in the estimates. The quantities for those cases in which at least one of the required flange plate areas would give an impractical size plate (assumed to be those plates with areas less than 8 sq in.) are shown dashed in Figures 9 through 12.

COMPARATIVE COST ESTIMATES

Comparative cost estimates in dollars are shown in Figures 13 through 16. The cost estimates for those cases in which at least one of the flange plate areas would give an impractical size plate are cross-hatched. Detailed cost analyses were made for the 64-, 100-, and 120-ft spans. Each piece of material was individually priced from the mill base with appropriate charges for all extras. Freight from mill to fabricating plant to distribution point was assumed at an average cost of $0.01 per lb. Costs in dollars of engineering, fabricating, erecting, and painting were assumed

TABLE 4

A373 STEEL, COMPOSITE ACTION, VERTICAL STIFFENERS

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)		Weight (lb/ft)		Total Stress (psi)			Moment of Inertia (in.⁴)	
	Thick.	Height	Top	Bottom	I-Section	Total DL	Bottom Steel	Top Steel	Top Concrete	Steel	Composite
64	0.3125	25	12.89	27.42	163.59	906.6	18,090	18,026	626	6,476.4	17,757.5
		43	4.58	14.77	111.48	862.4	17,934	18,050	414	9,903.3	32,046.9
		52	2.52	11.46	102.78	857.6	17,909	17,947	358	11,623.5	40,253.4
68	0.3125	26	14.47	29.52	177.19	919.6	17,986	18,067	629	7,681.9	19,966.8
		43	5.79	16.55	121.65	871.3	17,924	18,064	431	11,318.7	34,354.7
		52	3.48	12.89	110.92	864.5	17,923	17,974	373	13,253.6	42,996.8
70	0.3125	27	14.95	29.89	181.14	923.5	17,963	17,997	621	8,479.4	21,513.8
		43	6.46	17.45	126.96	876.1	17,934	18,051	440	12,073.3	35,502.0
		52	3.96	13.67	115.19	868.1	17,906	18,035	380	14,090.3	44,446.6
75	0.3125	28	17.21	32.47	198.66	940.3	18,016	18,059	627	10,178.8	24,259.4
		43	8.33	19.82	141.40	889.2	17,920	17,930	458	14,156.2	38,486.9
		52	5.45	15.57	126.69	878.2	17,917	17,903	398	16,488.9	47,975.6
80	0.3125	30	18.87	33.70	210.62	952.0	18,080	17,925	614	12,480.9	28,165.8
		43	10.27	22.33	156.52	903.2	17,927	17,969	475	16,319.1	41,540.8
		52	6.87	17.61	138.47	888.8	17,926	18,070	414	18,860.8	51,653.9
85	0.3125	32	20.33	35.05	222.28	963.6	18,081	17,950	602	15,027.4	32,514.3
		43	12.49	24.99	173.13	918.8	17,923	17,937	490	18,728.8	44,722.4
		52	8.69	19.74	151.92	901.2	17,918	17,930	427	21,706.1	55,451.6
90	0.3125	35	20.73	35.14	227.18	968.8	18,090	18,023	578	18,214.8	38,370.9
		43	14.76	27.84	190.52	935.3	17,912	18,055	502	21,208.4	48,031.4
		52	10.57	22.01	166.01	914.3	17,914	17,923	440	24,654.0	59,387.9
95	0.3125	37	22.26	36.49	239.06	980.7	18,090	18,013	567	21,488.0	43,656.4
		43	17.50	30.70	209.56	953.5	17,976	17,977	512	24,030.5	51,344.7
		52	12.46	24.42	180.63	928.1	17,908	18,050	451	27,666.1	63,465.3
	0.375	61	8.51	19.34	172.48	923.4	17,914	17,952	407	31,638.4	77,153.4
100	0.3125	40	22.77	36.73	244.80	986.9	18,077	18,051	547	25,534.8	50,717.2
		43	20.21	33.68	228.92	972.2	18,061	18,078	522	26,849.2	54,716.8
		52	14.74	26.80	196.50	943.2	17,979	17,990	460	31,068.6	67,497.1
	0.375	61	10.18	21.20	184.47	934.5	18,073	18,089	417	35,096.2	81,471.8
105	0.3125	43	23.32	37.02	250.84	993.5	18,072	18,088	528	30,043.8	58,437.2
		52	17.08	29.45	213.46	959.4	17,984	18,031	468	34,628.2	71,865.8
	0.375	61	12.30	23.64	199.98	949.3	17,925	17,954	423	39,529.2	87,037.4
110	0.3125	44	26.02	39.43	269.28	1,011.7	18,088	17,969	524	34,203.1	63,878.5
		52	19.68	32.27	231.87	977.2	17,984	18,029	474	38,510.8	76,456.5
	0.375	61	14.31	26.01	214.87	963.5	17,915	18,028	430	43,759.2	92,328.4
115	0.3125	47	26.68	39.82	276.02	1,019.0	18,056	17,968	507	39,758.1	72,903.2
		52	22.49	35.14	251.14	995.9	18,033	18,034	478	42,579.2	81,062.9
	0.375	61	16.51	28.47	230.71	978.7	17,921	18,070	435	48,294.7	97,768.4
120	0.3125	50	27.30	40.15	282.46	1,026.0	18,060	17,984	492	45,824.3	82,637.6
		52	25.64	38.27	272.53	1,016.8	18,037	17,973	481	47,142.9	86,150.4
	0.375	61	19.23	31.02	248.66	996.1	17,933	17,912	438	53,565.8	103,511.3

TABLE 5

A36 STEEL, COMPOSITE ACTION, VERTICAL STIFFENERS

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)		Weight (lb/ft)		Total Stress (psi)			Moment of Inertia (in.^{4})	
	Thick.	Height	Top	Bottom	I-Section	Total DL	Bottom Steel	Top Steel	Top Concrete	Steel	Composite
64	0.3125	25	10.94	24.31	146.43	889.5	20.069	19,940	651	5,663.2	16,405.6
		43	3.63	12.98	102.18	853.1	19,936	20,018	428	8,705.7	29,671.9
		52	1.78	9.87	94.86	849.7	20,048	19,932	370	10,192.5	37,164.6
68	0.3125	26	12.22	26.00	157.58	900.0	20,060	20,043	656	6,676.9	18,360.2
		43	4.66	14.55	111.01	860.7	19,936	20,086	447	9,916.8	31,774.5
		52	2.59	11.25	102.33	855.9	19,932	19,991	386	11,631.8	39,879.4
70	0.3125	27	12.59	26.23	160.72	903.0	20,085	19,990	649	7,352.1	19,736.5
		43	5.22	15.36	115.67	864.8	19,936	20,081	455	10,570.9	32,836.0
		52	3.07	11.82	105.88	858.8	20,034	19,929	395	12,394.1	40,976.5
75	0.3125	28	14.59	28.61	176.66	918.2	20,056	19,982	655	8,860.7	22,302.3
		43	6.81	17.47	128.25	876.1	19,915	19,971	476	12,366.4	35,560.0
		52	4.22	13.64	116.01	867.5	19,926	20,022	412	14,387.1	44,431.1
80	0.3125	30	15.79	29.79	186.83	928.2	20,061	20,018	643	10,795.7	25,912.6
		43	8.47	19.70	141.46	888.1	19,908	20,021	494	14,237.9	38,356.2
		52	5.56	15.44	126.68	877.0	19,919	20,017	429	16,573.1	47,786.7
85	0.3125	32	17.02	30.94	197.04	938.3	20,068	20,034	631	12,997.8	29,870.6
		43	10.35	22.01	155.70	901.4	19,927	20,010	510	16,293.2	41,196.9
		52	7.10	17.32	138.27	887.5	19,928	19,911	444	19,005.9	51,196.5
90	0.3125	35	17.43	31.12	202.24	943.9	20,013	20,059	607	15,823.9	35,317.1
		43	12.50	24.52	171.55	916.3	19,902	19,908	524	18,603.4	44,212.0
		52	8.70	19.35	150.62	898.9	19,901	19,915	458	21,565.6	54,805.7
95	0.3125	37	18.66	32.18	212.18	953.9	20,063	20,074	596	18,617.2	40,053.9
		43	14.62	27.07	187.44	931.4	19,944	20,029	537	20,888.9	47,213.5
		52	10.30	21.46	163.25	910.7	19,901	20,081	470	24,146.5	58,477.0
	0.3750	61	6.74	16.86	158.04	908.9	19,904	19,977	424	27,664.2	71,397.1
100	0.3125	40	19.37	32.29	218.16	960.3	20,083	19,917	574	22,286.1	46,474.4
		43	17.21	29.67	205.10	948.4	20,034	19,920	547	23,516.4	50,264.4
		52	12.29	23.64	177.40	924.1	19,919	19,982	480	27,159.4	62,232.6
	0.3750	61	8.23	18.72	169.43	919.5	19,912	20,055	434	30,879.3	75,763.3
105	0.3125	43	19.85	32.53	223.78	966.4	20,068	19,952	556	26,238.5	53,533.7
		52	14.29	25.98	192.18	938.2	19,912	20,028	490	30,244.6	66,184.1
	0.3750	61	9.91	20.64	181.62	930.9	19,933	20,065	443	34,373.1	80,228.4
110	0.3125	44	21.83	34.65	238.79	981.2	20,064	20,012	554	29,627.7	58,400.6
		52	16.43	28.32	207.40	952.7	19,980	20,085	498	33,457.8	70,088.6
	0.3750	61	11.79	22.67	194.96	943.6	19,930	19,997	450	38,228.8	84,932.9
115	0.3125	47	22.28	34.81	244.03	987.0	20,097	20,058	537	34,324.7	66,440.7
		52	18.82	30.86	224.17	969.0	20,005	20,072	504	37,008.5	74,275.6
	0.3750	61	13.68	24.87	208.85	956.9	19,906	20,043	457	43,181.9	89,892.5
120	0.3125	50	22.82	35.10	250.04	993.6	20,089	20,069	521	39,581.6	75,267.2
		52	21.43	33.49	242.00	986.2	20,051	20,042	509	40,799.8	78,593.1
	0.3750	61	15.74	27.14	223.57	971.1	19,904	20,058	462	46,405.0	94,974.4

TABLE 6

A242 STEEL, COMPOSITE ACTION, VERTICAL STIFFENERS

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)		Weight (lb/ft)		Total Stress (psi)			Moment of Inertia (in.^{4})	
	Thick.	Height	Top	Bottom	I-Section	Total DL	Bottom Steel	Top Steel	Top Concrete	Steel	Composite
64	0.3125	25	8.10	19.74	121.22	864.2	24,059	24,031	694	4,468.5	14,304.4
		43	2.28	10.35	88.62	839.5	23,922	24,026	454	6,971.6	26,039.7
	0.375	52	0.07	7.20	91.03	845.9	24,076	23,924	396	8,168.8	33,559.2
68	0.3125	26	9.10	21.16	130.51	872.9	23,981	24,068	700	5,281.8	16,027.8
		43	3.13	11.61	95.81	845.5	23,939	23,936	474	7,972.8	27,811.2
	0.375	52	0.74	8.26	96.91	850.4	24,066	23,914	414	9,324.0	35,685.3
70	0.3125	27	9.39	21.35	133.22	875.2	23,997	23,965	693	5,823.8	17,227.2
		43	3.56	12.27	99.53	848.6	23,934	23,981	483	8,478.6	28,731.0
	0.375	52	1.07	8.90	100.20	853.1	23,924	23,980	422	9,933.8	36,955.0
75	0.3125	28	10.88	23.15	145.46	887.1	24,048	23,973	703	6,987.2	19,382.1
		43	4.75	13.98	109.36	857.2	23,938	23,970	506	9,854.1	31,037.9
	0.375	52	2.04	10.21	107.95	859.5	24,059	23,957	443	11,521.3	39,511.5
80	0.3125	30	11.79	24.06	153.75	895.1	24,059	23,984	691	8,515.0	22,484.5
		43	6.01	15.81	119.84	866.5	23,906	24,082	526	11,308.5	33,449.8
	0.375	52	3.07	11.79	116.82	867.1	23,916	24,004	460	13,272.0	42,529.8
85	0.3125	32	12.72	24.94	162.02	903.3	24,075	23,980	680	10,251.5	25,847.6
		43	7.53	17.50	130.78	876.4	24,090	23,934	546	12,931.1	35,658.0
	0.375	52	4.26	13.31	126.01	875.3	23,940.8	23,927.8	477	15,176.6	45,370.0
90	0.3125	35	13.00	25.00	166.38	908.0	24,047	24,013	655	12,474.3	30,527.9
		43	9.04	19.65	143.24	888.0	23,926	24,030	562	14,663.5	38,356.3
	0.375	52	5.53	14.93	135.86	884.1	23,929	23,915	493	17,205.1	48,361.7
95	0.3125	37	13.97	25.86	174.73	916.4	24,065	23,985	644	14,702.7	34,615.3
		43	10.85	21.69	156.31	900.3	23,961	23,909	577	16,580.1	40,888.8
	0.375	52	6.82	16.64	146.09	893.5	23,929	24,026	507	19,298.9	51,440.2
	0.4375	61	3.67	12.65	146.22	897.1	23,911	23,954	457	22,154.7	63,518.9
100	0.3125	40	14.27	25.96	179.29	921.4	24,056	24,024	623	17,485.6	40,139.8
		43	12.67	23.92	170.11	913.4	23,941	23,965	591	18,558.8	43,582.0
	0.375	52	8.30	18.46	157.28	903.9	23,901	24,020	520	21,619.9	54,648.6
	0.4375	61	4.81	14.17	155.27	905.3	23,924	24,087	469	24,676.0	67,169.7
105	0.3125	43	14.60	26.07	183.97	926.6	24,069	24,069	604	20,577.7	46,148.3
	0.375	52	9.94	20.29	169.10	915.1	23,925	23,961	531	24,114.8	57,868.4
	0.4375	61	6.14	15.73	165.10	914.4	23,929	24,080	479	24,470.2	70,914.5
110	0.375	44	15.94	27.49	203.76	946.2	23,996	23,903	605	23,521.9	50,692.2
		52	11.59	22.28	181.45	926.8	23,913	24,047	541	26,672.5	61,272.3
	0.4375	61	7.65	17.38	175.84	924.5	23,911	23,955	488	30,568.3	74,821.9
115	0.375	47	16.30	27.63	209.28	952.2	23,981	23,908	587	27,346.9	57,747.8
		52	13.38	24.34	194.55	939.3	23,924	24,089	550	29,410.9	64,748.2
	0.4375	61	9.16	19.09	186.78	934.8	23,931	23,968	497	33,699.4	78,800.5
120	0.375	50	16.68	27.79	214.94	958.5	23,978	23,919	570	31,549.6	65,369.0
		52	15.39	26.53	208.80	953.0	23,908	24,061	556	32,406.7	68,416.5
	0.4375	61	10.67	20.92	198.14	945.6	23,926	24,092	505	36,899.9	82,985.3

TABLE 7

A373 STEEL, NONCOMPOSITE ACTION, NO STIFFENERS

Span Length (ft)	Web Plate (in.) Thick.	Web Plate (in.) Height	Flange Plate (sq in.)	Weight (lb/ft) I-Section	Weight (lb/ft) Total DL	Steel Total Stress (psi)	Steel Moment of Inertia (in.⁴)
64	0.50	30	23.97	213.99	942.67	18,062	12,642.0
	0.75	45	12.06	196.76	929.83	18,072	18,455.1
	0.875	52	8.02	209.26	944.38	17,917	21,522.2
68	0.5625	31	25.67	233.82	962.04	17,938	14,537.6
	0.75	45	21.38	208.91	941.13	18,088	20,345.1
	0.875	52	9.50	219.31	953.54	18,016	23,597.3
70	0.5625	32	26.05	238.36	966.52	17,906	15,722.0
	0.75	45	14.84	215.64	947.48	18,037	21,393.2
	0.875	52	10.31	224.82	958.64	18,013	24,735.3
75	0.625	34	27.08	256.38	984.31	17,938	18,632.4
	0.75	45	17.20	231.73	962.69	18,082	23,895.6
	0.875	52	12.32	238.46	971.36	18,088	27,553.7
80	0.625	36	28.51	270.24	997.98	17,910	21,866.6
	0.75	45	19.85	249.75	979.94	18,019	26,699.7
	0.875	52	14.58	253.84	985.92	18,039	30,728.5
85	0.6875	38	29.42	288.88	1,016.54	17,932	25,517.1
	0.75	45	22.53	267.93	997.45	18,043	29,528.5
	0.875	52	16.83	269.18	1,000.55	18,085	33,898.3
90	0.6875	41	29.71	297.85	1,025.73	17,910	30,151.2
	0.75	45	25.56	288.57	1,017.49	17,956	32,740.4
	0.875	52	19.46	287.00	1,017.73	17,976	37,578.1
95	0.75	44	29.67	313.96	1,042.07	17,916	35,365.1
	0.875	52	22.11	305.05	1,035.22	17,945	41,307.5
100	0.8125	46	30.59	335.06	1,063.20	17,936	40,372.6
	0.875	52	24.79	323.25	1,052.90	17,984	45,065.9
105	0.8125	48	31.90	349.51	1,077.70	17,924	45,782.0
	0.875	52	27.77	343.54	1,072.73	17,948	49,256.5
110	0.875	50	32.84	372.05	1,100.32	17,936	51,820.2
		52	30.79	364.09	1,092.86	17,972	53,501.9

TABLE 8

A36 STEEL, NONCOMPOSITE ACTION, NO STIFFENERS

Span Length (ft)	Web Plate (in.) Thick.	Web Plate (in.) Height	Flange Plate (sq in.)	Weight (lb/ft) I-Section	Weight (lb/ft) Total DL	Steel Total Stress (psi)	Steel Moment of Inertia (in.⁴)
64	0.5	30	21.30	195.82	924.5	19,947	11,358.0
	0.75	45	10.37	185.25	918.3	19,916	16,663.5
	0.875	52	6.42	198.33	933.5	19,922	19,265.1
68	0.5625	31	22.57	212.73	941.0	19,950	12,950.1
	0.75	45	11.86	195.38	927.6	20,053	18,241.0
	0.875	52	7.78	207.60	941.8	19,969	21,179.7
70	0.5625	32	22.89	216.83	945.0	19,920	13,997.6
	0.75	45	12.73	201.31	933.2	20,007	19,163.5
	0.875	52	8.53	212.72	946.5	19,929	22,236.7
75	0.625	34	23.77	233.92	961.8	19,918	16,608.9
	0.75	45	14.86	215.76	946.7	20,031	21,412.0
	0.875	52	10.35	225.05	957.9	19,988	24,782.3
80	0.625	36	24.98	246.23	974.0	19,902	19,458.1
	0.75	45	17.18	231.54	961.7	19,986	23,867.0
	0.875	52	12.33	238.57	970.7	19,958	27,575.3
85	0.6875	38	25.68	263.45	991.1	19,938	22,672.9
	0.75	45	19.50	247.38	976.9	20,035	26,331.2
	0.875	52	14.31	252.00	983.4	20,034	30,349.5
90	0.6875	41	25.87	271.78	999.7	19,917	26,770.0
	0.75	45	22.15	265.38	994.3	19,953	29,131.6
	0.875	52	16.60	267.55	998.3	19,937	33,562.2
95	0.75	44	25.73	287.18	1,015.3	19,927	31,378.0
	0.875	52	18.90	283.20	1,013.4	19,930	36,793.0
100	0.8125	46	26.50	307.28	1,035.4	19,914	35,860.1
	0.875	52	21.28	299.42	1,029.1	19,948	40,144.6
105	0.8125	48	27.59	320.19	1,048.4	19,911	40,605.8
	0.875	52	23.87	317.02	1,046.2	19,923	43,778.1
110	0.875	50	28.29	341.09	1,069.4	19,931	45,900.2
		52	26.48	334.74	1,063.5	19,966	47,438.0

TABLE 9

A242 STEEL, NONCOMPOSITE ACTION, NO STIFFENERS

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)	Weight (lb/ft)		Steel	
	Thick.	Height		I-Section	Total DL	Total Stress (psi)	Moment of Inertia ($in.^4$)
64	0.625	30	16.49	175.86	904.5	24,077	9,328.3
	0.875	45	6.81	180.21	913.3	23,903	13,853.4
	1.0	52	3.08	197.74	932.9	23,917	16,043.3
68	0.625	31	17.82	187.06	915.3	23,925	10,676.0
	0.875	45	8.11	189.04	921.3	23,953	15,227.9
	1.0	52	4.24	205.65	939.9	23,906	17,676.2
70	0.625	32	18.03	190.62	918.8	23,909	11,525.3
	0.875	45	8.83	193.89	925.7	23,910	15,981.7
	1.0	52	4.82	209.59	943.4	23,934	18,489.9
75	0.6875	34	18.62	206.09	934.0	23,915	13,656.4
	0.875	45	10.58	205.83	936.8	23,931	17,840.5
	1.0	52	6.37	220.09	953.0	23,923	20,658.2
80	0.75	36	19.13	221.88	949.6	23,951	16,010.0
	0.875	45	12.43	218.42	948.6	23,943	19,798.8
	1.0	52	7.97	231.03	963.1	23,929	22,917.5
85	0.75	38	19.98	232.74	960.4	23,919	18,621.3
	0.875	45	14.37	231.60	961.1	23,961	21,849.1
	1.0	52	9.65	242.41	973.8	23,951	25,268.2
90	0.8125	41	19.68	247.10	975.0	23,914	22,026.2
	0.875	45	16.47	245.90	974.8	23,912	24,074.9
	1.0	52	11.43	254.56	985.3	23,936	27,777.4
95	0.875	44	19.37	262.59	990.7	23,919	25,820.1
	1.0	52	13.29	267.20	997.4	23,937	30,389.4
100	0.9375	46	19.80	281.24	1,009.4	23,914	29,469.2
	1.0	52	15.29	280.76	1,010.4	23,900	33,189.8
105	0.9375	48	20.56	292.83	1,021.0	23,920	33,326.4
	1.0	52	17.30	294.42	1,023.6	23,940	36,011.8
110	1.0	50	20.99	312.72	1,041.0	23,004	37,711.6
		52	19.47	309.17	1,037.9	23,930	39,056.7

the same for A373 and A36 steels. These items were assumed less than this amount by $0.0015 per lb for A242 steel.

After pricing each individual piece and determining the total cost of structural steel, an average unit price for structural steel was determined. These average unit prices were used in computing the cost estimates. Unit prices for those spans for which no detailed cost analyses were made were determined by interpolation or extrapolation. A summary of the unit prices for the spans for which detailed cost analyses were made are shown in Tables 19 through 22.

Tables 23 through 26 show the percentage of the estimated costs in dollars for the A36 and A242 steels based on the estimated costs in dollars for the A373 steel as 100 percent.

For the noncomposite action stringers with vertical stiffeners, the percentage variations with respect to A373 steel are (a) A36 steel varies from 2.0 to 7.6 percent less, and (b) A242 steel varies from 3.1 percent less to 6.1 percent more.

For the composite action stringers with vertical stiffeners, the percentage variations with respect to A373 steel are (a) A36 steel varies from 0.1 to 7.4 percent less, and (b) A242 steel varies from 2.9 percent less to 6.5 percent more.

For the noncomposite action stringers without stiffeners, the percentage variations with respect to A373 steel are (a) A36 steel varies from 2.1 to 6.6 percent less, and (b) A242 steel varies from 0.1 to 9.2 percent more.

For the composite action stringers without stiffeners, the percentage variations with respect to A373 steel are (a) A36 steel varies from 5.0 to 7.9 percent less, and (b) A242 steel varies from 3.0 percent less to 0.5 percent more.

These percentages should be considered somewhat relative in nature —merely indicating that, in general, the first cost in dollars of structures

TABLE 10

A373 STEEL, COMPOSITE ACTION, NO STIFFENERS

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)		Weight (lb/ft)		Total Stress (psi)			Moment of Inertia (in.[4])	
	Thick.	Height	Top	Bottom	I-Section	Total DL	Bottom Steel	Top Steel	Top Concrete	Steel	Composite
64	0.4375	25	12.07	26.83	169.45	896.7	18,057	18,019	631	6,405.6	17,810.7
68	0.4375	26	13.78	28.71	183.16	910.0	18,054	17,927	635	7,631.3	19,938.7
70	0.50	27	13.90	28.97	191.67	918.4	17,976	17,924	628	8,434.3	21,610.6
75	0.50	28	16.07	31.51	209.39	935.7	18,034	18,052	634	10,105.2	24,344.1
80	0.50	30	17.45	32.93	222.30	948.4	17,994	18,080	621	12,349.4	28,368.9
85	0.5625	32	18.71	34.02	240.47	966.5	17,996	18,031	609	14,989.0	32,879.1
90	0.625	35	18.99	33.91	254.21	980.5	17,956	17,967	585	18,406.1	39,095.4
95	0.625	37	20.40	35.17	267.55	993.9	17,975	18,000	574	21,696.7	44,459.1
100	0.6875	40	20.42	35.20	282.59	1,009.2	17,912	18,084	554	25,933.3	52,036.4
105	0.75	42	21.63	36.05	303.23	1,029.9	17,985	18,073	544	30,217.9	58,640.6
110	0.75	44	23.21	37.43	318.35	1,045.2	17,968	18,051	532	34,925.6	65,772.0

TABLE 11

A36 STEEL, COMPOSITE ACTION, NO STIFFENERS

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)		Weight (lb/ft)		Total Stress (psi)			Moment of Inertia (in.[4])	
	Thick.	Height	Top	Bottom	I-Section	Total DL	Bottom Steel	Top Steel	Top Concrete	Steel	Composite
64	0.4375	25	10.05	23.92	152.66	879.9	19,914	20,051	655	5,585.2	16,553.1
68	0.4375	26	11.39	25.31	163.44	890.2	20,085	20,039	662	6,593.9	18,389.3
70	0.50	27	11.52	25.59	172.08	898.8	19,929	19,940	655	7,326.4	19,988.7
75	0.50	28	13.32	27.75	187.25	913.5	20,030	20,094	663	8,755.1	22,453.4
80	0.50	30	14.68	28.79	198.80	924.9	20,076	19,942	651	10,751.5	26,055.0
85	0.5625	32	15.54	29.88	215.60	941.7	19,971	20,002	639	13,016.6	30,286.1
90	0.625	35	15.47	29.76	228.15	954.5	19,905	20,076	616	15,902.3	36,018.5
95	0.625	37	16.82	30.58	239.79	966.1	20,035	19,987	605	18,780.6	40,751.3
100	0.6875	40	16.83	30.59	254.72	981.3	19,925	20,020	585	22,530.9	47,770.9
105	0.75	42	17.80	31.22	273.76	1,000.5	20,010	19,994	575	26,254.1	53,771.5
110	0.75	44	19.12	32.40	287.37	1,014.2	19,976	19,966	564	30,349.7	60,260.8

TABLE 12

A242 STEEL, COMPOSITE ACTION, NO STIFFENERS

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)		Weight (lb/ft)		Total Stress (psi)			Moment of Inertia ($in.^4$)	
							Bottom Steel	Top			
	Thick.	Height	Top	Bottom	I-Section	Total DL		Steel	Concrete	Steel	Composite
64	0.50	25	7.01	18.95	130.76	858.0	24,031	24,029	703	4,411.9	14,941.2
68	0.50	26	8.07	20.18	140.27	867.1	24,086	23,919	710	5,234.4	16,147.7
70	0.5625	27	7.93	20.34	147.75	874.5	23,921	24,006	703	5,769.1	17,559.0
75	0.5625	28	9.46	22.00	160.51	886.8	24,058	23,929	714	6,942.9	19,666.0
80	0.625	30	9.99	22.70	174.88	901.0	23,994	23,961	704	8,503.9	22,982.9
85	0.625	32	10.80	23.47	184.52	910.6	24,039	24,018	693	10,230.9	26,427.0
90	0.6875	35	10.70	23.27	197.31	923.6	23,917	23,953	669	12,581.4	31,487.6
95	0.75	37	11.16	23.75	213.05	939.4	23,963	24,036	661	14,856.4	35,866.2
100	0.8125	40	11.03	23.46	227.75	954.4	23,904	24,001	640	17,856.3	41,970.0
105	0.8125	42	11.99	23.98	238.32	965.0	24,078	23,949	631	20,694.6	46,827.9
110	0.875	44	12.47	24.45	256.42	983.2	24,089	24,009	622	23,937.2	52,574.5

TABLE 13

A373 STEEL, COMPOSITE ACTION, VERTICAL STIFFENERS, ¾-IN. TOP PLATE, 1½-IN. BOTTOM PLATE

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)		Weight (lb/ft)		Total Stress (psi)			Moment of Inertia (in.⁴)	
	Thick.	Height	Top	Bottom	I-Section	Total DL	Bottom Steel	Top Steel	Top Concrete	Steel	Composite
64	0.3125	25	12.75	28.33	166.21	909.2	17,927	17,965	621	6,580.4	18,110.7
90	0.3125	35	20.55	36.05	229.63	971.3	17,947	17,992	575	18,438.8	38,958.5
		43	14.69	28.26	191.72	936.5	17,907	17,957	500	21,416.3	48,429.3
		52	10.27	22.34	166.13	914.4	17,919	18,055	439	24,611.0	59,775.4
120	0.3125	50	27.00	40.91	284.01	1,027.5	17,958	18,010	490	46,140.8	83,508.5
		52	25.26	38.86	273.25	1,017.5	17,982	18,039	480	47,305.4	86,850.3
	0.375	61	18.89	31.48	249.04	996.5	17,901	17,986	438	53,653.9	104,188.6

TABLE 14

A242 STEEL, COMPOSITE ACTION, VERTICAL STIFFENERS, ¾-IN. TOP PLATE, 1½-IN. BOTTOM PLATE

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)		Weight (lb/ft)		Total Stress (psi)			Moment of Inertia (in.⁴)	
	Thick.	Height	Top	Bottom	I-Section	Total DL	Bottom Steel	Top Steel	Top Concrete	Steel	Composite
64	0.3125	25	7.94	20.35	122.72	865.7	23,936	24,021	689	[illegible]17.9	14,556.0
90	0.3125	35	12.78	25.56	167.55	909.2	23,955	24,046	652	12,563.6	30,911.1
		43	8.97	19.93	143.92	888.7	23,971	23,913	561	14,781.7	38,609.9
	0.375	52	5.34	15.24	136.27	884.6	23,913	24,000	492	17,234.7	48,741.8
120	0.375	50	16.39	28.26	215.57	959.1	23,933	23,979	569	31,666.6	65,897.6
		52	15.32	26.87	209.74	954.0	23,921	23,939	555	32,670.9	68,850.0
	0.4375	61	10.61	21.23	199.01	946.5	23,928	23,955	503	37,200.9	83,457.1

TABLE 15

A242 STEEL, NONCOMPOSITE ACTION, VERTICAL STIFFENERS, ¾-IN. MATERIAL, ALLOWABLE f_s = 27 KSI

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)	Weight (lb/ft)		Steel	
	Thick.	Height		I-Section	Total DL	Total Stress (psi)	Moment of Inertia (in.⁴)
64	0.3125	30	15.63	138.18	883.4	27,061	8,094.3
		43	9.68	111.54	862.5	27,037	11,339.2
68	0.3125	31	16.79	147.13	891.7	26,936	9,239.8
		43	10.88	119.66	869.3	27,028	12,481.1
70	0.3125	32	17.00	149.61	894.0	26,923	9,971.2
		43	11.47	123.69	872.8	27,062	13,049.1
	0.375	52	8.15	121.71	874.6	27,012	15,730.6
75	0.3125	34	17.79	157.12	901.2	26,953	11,766.7
		43	13.06	134.48	882.3	27,047	14,566.9
	0.375	52	9.44	130.47	882.0	27,029	17,522.4
80	0.3125	36	18.59	164.69	908.5	26,941	13,771.1
		43	14.71	145.73	892.4	27,048	16,149.8
	0.375	52	10.77	139.56	889.9	27,059	19,383.9
85	0.3125	38	19.35	171.99	915.7	26,958	15,960.2
		43	16.44	157.45	903.1	27,062	17,799.4
	0.375	52	12.18	149.11	898.4	27,076	21,337.6
90	0.3125	41	19.53	176.34	920.3	26,924	18,812.6
		43	18.31	170.20	915.0	26,992	19,594.6
	0.375	52	13.64	159.07	907.4	27,096	23,375.6
95	0.375	44	19.33	187.54	931.9	26,924	22,016.8
		52	15.19	169.60	917.0	27,091	25,529.7
	0.4375	61	11.32	167.70	918.6	27,085	29,854.4
100	0.375	46	20.05	194.97	939.4	26,954	24,948.4
		52	16.81	180.63	927.3	27,083	27,786.8
	0.4375	61	12.68	176.93	927.0	27,096	32,440.9
105	0.375	48	20.79	202.58	947.1	26,949	28,162.1
		52	18.54	192.40	938.4	27,039	30,193.8
	0.4375	61	14.10	186.61	935.9	27,095	35,154.2
110	0.375	50	21.57	210.42	955.0	26,911	31,681.7
	0.4375	61	15.66	197.25	945.9	26,995	38,137.4
115	0.4375	53	21.30	223.71	968.8	26,922	36,203.4
		61	17.18	207.56	955.6	27,043	41,029.0
120	0.4375	55	22.03	231.64	977.0	26,927	40,306.8
		61	18.82	218.74	966.2	27,020	44,164.1

of A242 steel are practically the same as those of A373 steel. To determine possible variations in these percentages due to the use of a wider stringer spacing, analyses were made with an 8¼-ft stringer spacing for span lengths from 90 to 120 ft in A373 and A242 steels. These results are summarized in Tables 27 and 28. A detailed cost analysis was made for the 100-ft span in both steels. Table 29 summarizes a comparison of these results for the 100-ft span with those obtained with the use of the 7-ft stringer spacing. It is seen that any changes due to the use of the different stringer spacings are within the limits of accuracy of the study. This comparison does not include the effect of the increase in concrete yardage with the use of the wider stringer spacing. The use of the wider stringer spacing would be best for spans longer than those studied in this investigation. The margin between A373 and A242 steels increases in favor of the A242 steel with longer span lengths and wider stringer spacings. The margin between A373 and A242 steels also increases in favor of the A242 steel with lesser depths of stringer.

These economic evaluations do not include possible long-term maintenance savings due to the increased resistance to atmospheric corrosion and the better paint life of nickel-copper types of high-strength low-alloy steels. This will require individual study for each structure or like grouping of structures in the same locality. The A36 steel was included in the study although it was not classified as a weldable steel for use in highway bridges at the completion of this study in August 1961.

TABLE 16

A242 STEEL, COMPOSITE ACTION, VERTICAL STIFFENERS, ¾-IN. MATERIAL, ALLOWABLE $f_s = 27$ KSI

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)		Weight (lb/ft)		Total Stress (psi)			Moment of Inertia (in.^{4})	
	Thick.	Height	Top	Bottom	I-Section	Total DL	Bottom Steel	Top Steel	Top Concrete	Steel	Composite
70	0.3125	27	7.68	18.73	118.48	860.8	26,918	27,152	731	4,922.4	15,454.5
75	0.3125	28	8.94	20.31	129.21	870.8	26,950	27,110	742	5,912.7	17,397.8
80	0.3125	30	9.69	21.06	136.41	877.8	26,991	27,104	730	7,209.2	20,177.4
85	0.3125	32	10.49	21.85	143.93	885.2	26,965	27,037	718	8,705.8	23,257.7
90	0.3125	35	10.75	21.93	148.28	889.9	26,879	27,001	690	10,640.0	27,512.0
		43	7.36	17.12	128.92	873.7	26,826	26,960	590	12,583.3	34,662.4
95	0.3125	37	11.42	22.40	154.30	896.0	27,163	27,141	682	12,422.3	30,962.5
		43	8.90	18.93	140.29	884.3	26,831	26,825	607	14,218.3	36,942.2
100	0.3125	40	11.70	22.51	158.82	901.0	27,097	27,117	658	14,832.4	35,988.6
		43	10.38	20.76	151.55	894.8	26,929	26,990	623	15,813.7	39,192.5
	0.375	52	6.53	15.93	142.69	889.4	26,806	26,996	546	18,558.0	49,593.2
105	0.3125	43	12.05	22.73	163.93	906.5	26,964	27,043	637	17,579.4	41,577.2
	0.375	52	7.89	17.53	152.74	898.7	26,841	26,992	558	20,639.6	52,430.3
110	0.375	44	12.93	23.94	181.43	923.8	26,847	27,044	641	19,979.2	45,708.5
		52	9.42	19.28	163.70	909.0	26,844	26,881	569	22,933.7	55,400.2
115	0.375	52	10.95	21.06	175.13	919.9	26,810	26,932	579	25,280.6	58,538.8
	0.4375	61	7.03	16.35	170.21	918.2	26,822	26,990	523	28,904.6	71,705.4
120	0.4375	61	8.42	17.91	180.27	927.7	26,824	26,975	532	31,757.2	75,364.5

TABLE 17

A242 STEEL, NONCOMPOSITE ACTION, NO STIFFENERS, ¾-IN. MATERIAL, ALLOWABLE $f_s = 27$ KSI

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)	Weight (lb/ft)		Steel	
	Thick.	Height		I-Section	Total DL	Total Stress (psi)	Moment of Inertia (in.^{4})
64	0.625	30	14.16	160.05	888.7	27,100	8,101.8
68	0.625	31	15.29	169.88	898.1	26,954	9,260.7
70	0.625	32	15.47	173.21	901.4	26,925	10,004.0
75	0.6875	34	15.88	187.47	915.4	26,964	11,840.5
80	0.75	36	16.28	202.52	930.3	26,951	13,911.1
85	0.75	38	16.97	212.29	940.0	26,930	16,170.1

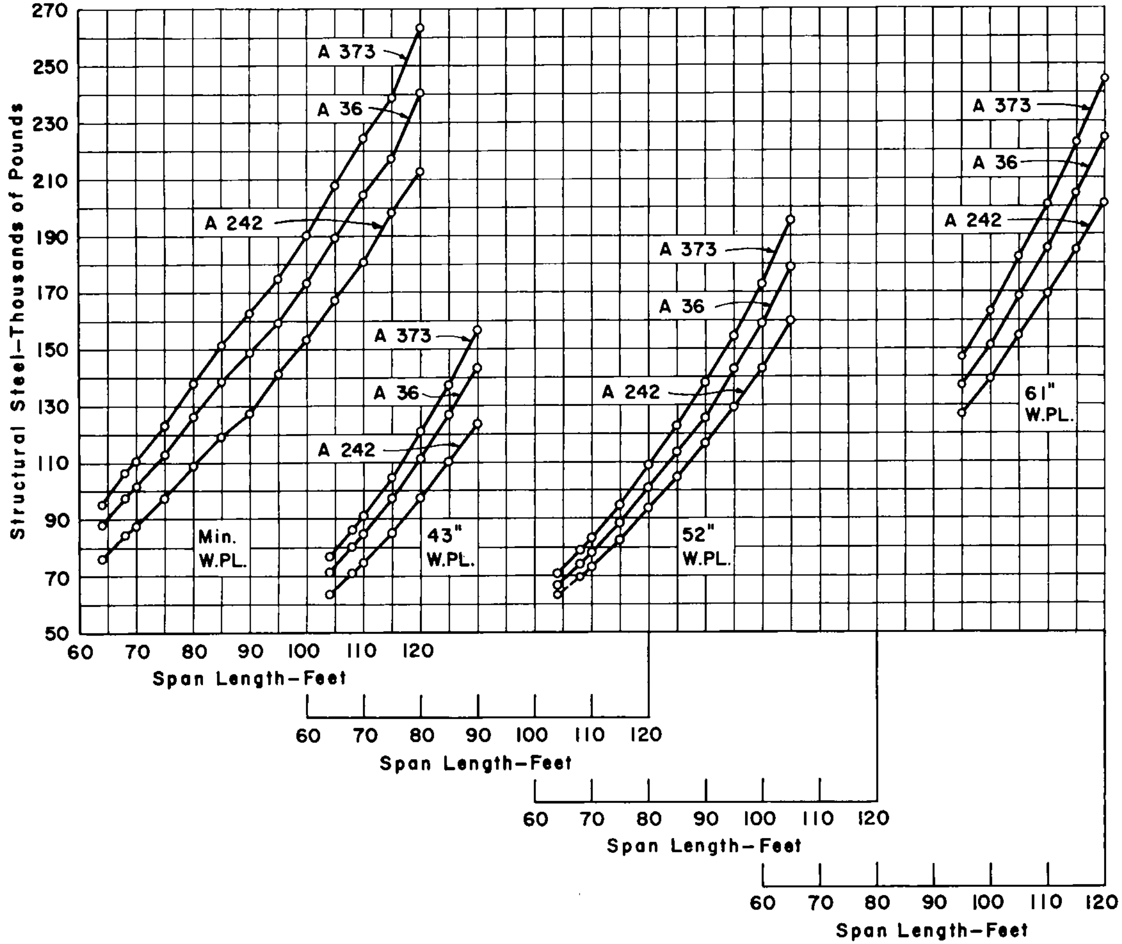

Figure 9. Noncomposite action, vertical stiffeners.

TABLE 18

A242 STEEL, COMPOSITE ACTION, NO STIFFENERS, ¾-IN. MATERIAL, ALLOWABLE f_s = 27 KSI

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)		Weight (lb/ft)		Total Stress (psi)			Moment of Inertia (in.^{4})	
	Thick.	Height	Top	Bottom	I-Section	Total DL	Bottom Steel	Top Steel	Top Concrete	Steel	Composite
64	0.50	25	5.56	16.35	117.00	844.2	27,175	27,107	744	3,722.4	12,947.3
68	0.50	26	6.47	17.49	125.68	852.5	27,135	26,910	750	4,431.8	14,470.0
70	0.5625	27	6.36	17.67	133.33	860.1	26,832	26,831	742	4,920.7	15,858.0
75	0.5625	28	7.40	18.99	143.28	869.5	27,147	27,184	756	5,824.8	17,630.3
80	0.625	30	7.87	19.68	157.45	883.6	26,892	26,987	744	7,208.5	20,732.8
85	0.625	32	8.51	20.26	165.80	891.9	27,012	27,069	734	8,660.9	23,802.5
90	0.6875	35	8.40	20.01	178.42	904.7	26,834	26,850	707	10,715.0	28,441.1
95	0.75	37	8.58	20.43	192.98	919.3	26,828	27,073	700	12,619.1	32,479.5

TABLE 19

UNIT PRICE BASED ON DETAILED COST ANAYSIS, NONCOMPOSITE ACTION, VERTICAL STIFFENERS

Span Length (ft)	Web Plate Depth (in.)	Unit Price (dollars per lb)		
		A373 Steel	A36 Steel	A242 Steel
64	30	0.1788	0.1881	0.2270
	43	0.1930	0.2000	0.2392
	52	0.1993	0.2073	0.2348
100	46	0.1671	0.1701	0.2048
	52	0.1719	0.1743	0.2078
	61	0.1693	0.1759	0.2080
120	55	0.1604	0.1630	0.1969
	61	0.1646	0.1678	0.2004

TABLE 20

UNIT PRICE BASED ON DETAILED COST ANALYSIS, COMPOSITE ACTION, VERTICAL STIFFENERS

Span Length (ft)	Web Plate Depth (in.)	Unit Price (dollars per lb)		
		A373 Steel	A36 Steel	A242 Steel
64	25	0.1916	0.2087	0.2458
	43	0.2182	0.2241	0.2584
100	40	0.1750	0.1818	0.2231
	43	0.1791	0.1851	0.2258
	52	0.1833	0.1920	0.2268
	61	0.1850	0.1920	0.2232
120	50	0.1710	0.1760	0.2124
	52	0.1729	0.1792	0.2141
	61	0.1746	0.1805	0.2147

TABLE 21

UNIT PRICE BASED ON DETAILED COST ANAYSIS, NONCOMPOSITE ACTION, NO STIFFENERS

Span Length (ft)	Web Plate Depth (in.)	Unit Price (dollars per lb)		
		A373 Steel	A36 Steel	A242 Steel
64	30	0.1660	0.1725	0.2053
	45	0.1658	0.1695	0.1948
	52	0.1626	0.1664	0.1868
100	46	0.1460	0.1489	0.1774
	52	0.1416	0.1480	0.1751

TABLE 22

UNIT PRICE BASED ON DETAILED COST ANALYSIS, COMPOSITE ACTION, NO STIFFENERS

Span Length (ft)	Web Plate Depth (in.)	Unit Price (dollars per lb)		
		A373 Steel	A36 Steel	A242 Steel
64	25	0.1784	0.1853	0.2216
100	40	0.1555	0.1585	0.1914

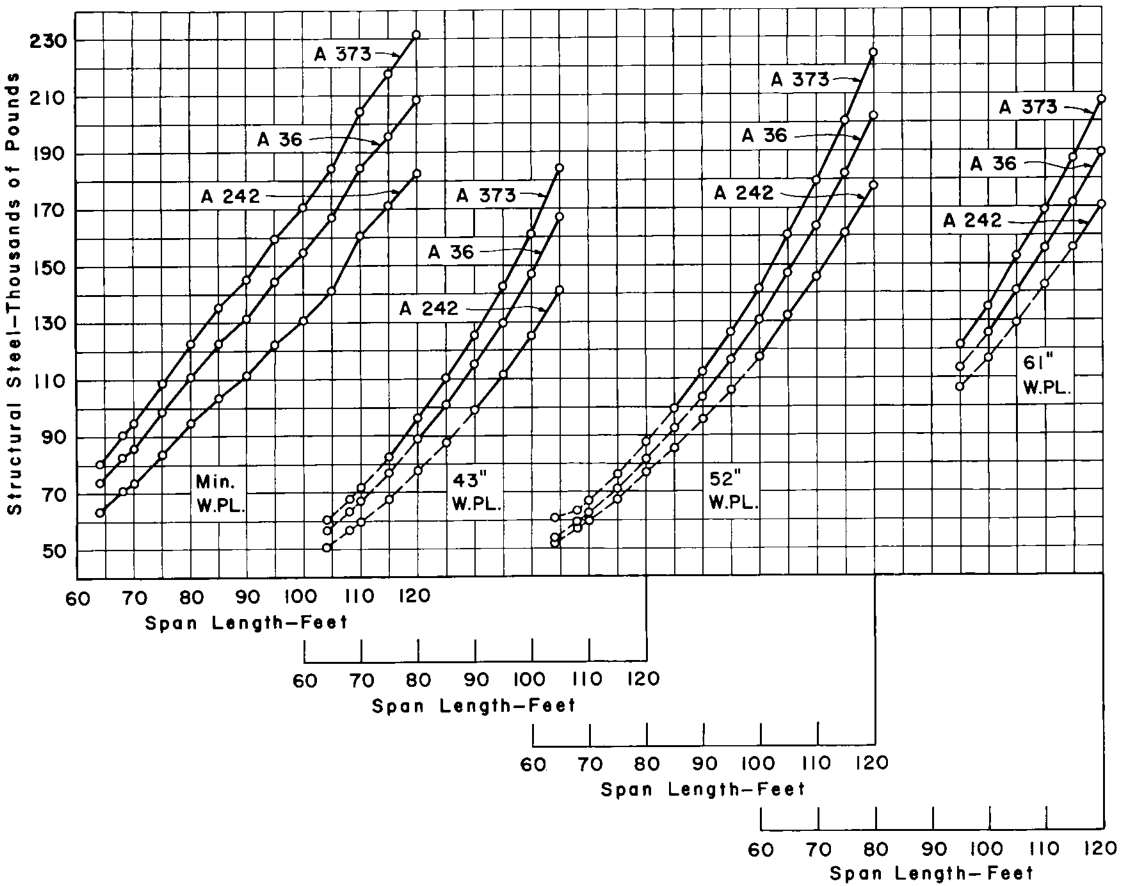

Figure 10. Composite action, vertical stiffeners.

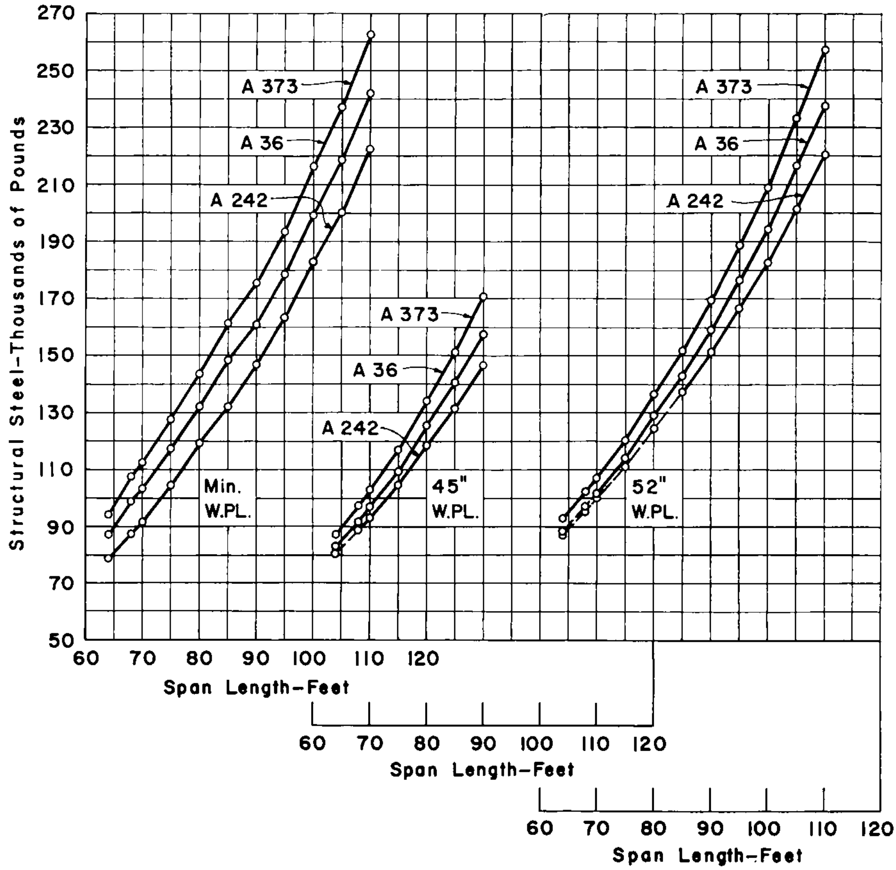

Figure 11. Noncomposite action, no stiffeners.

CONCLUSIONS

It is seen that short- and medium-length welded I-section stringer and concrete slab highway bridges may be fabricated and erected using a nickel-copper grade of high-strength low-alloy steel in accordance with the AASHO bridge specifications at about the same first cost in dollars as if A373 structural steel were used, provided adequate diaphragms are used in order that full live load and impact deflection may be equally distributed to all stringers. Thus any savings in maintenance due to the use of this low-alloy steel may be obtained at little or no additional first cost over the use of A373 structural steel.

The data presented here may be of assistance to highway bridge engineers in their never-ending search for the most economical structure at a specific site—that structure with the best over-all economy in design, fabrication, erection, and maintenance for its location.

ACKNOWLEDGMENTS

This study was made in the Engineering Experiment Station, Purdue University, Lafayette, Ind., in co-

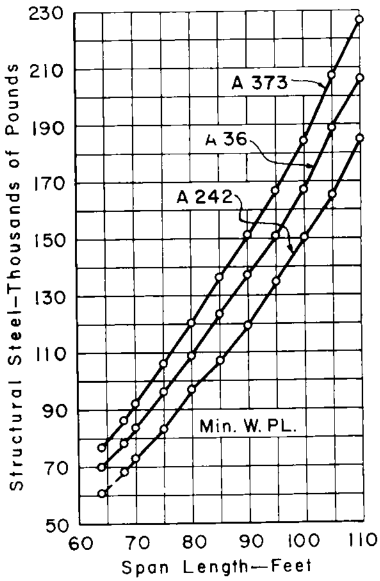

Figure 12. Composite action, no stiffeners.

operation with the International Nickel Company. The authors gratefully acknowledge the assistance of Joseph O. May, Consulting Engineer, Ridgewood, N. J., for his assistance in the unit price cost analyses, and wish to thank L. Anselmini of the International Nickel Company for his assistance in the investigation and the preparation of the report.

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TABLE 23

COST ESTIMATE PERCENTAGES, NONCOMPOSITE ACTION, VERTICAL STIFFENERS

Span Length (ft)	Web Plate Depth (in.)	Percentage of Total Cost Based on A373 Steel as 100%							
		Minimum Web Depth		43-In. Web Depth		52-In. Web Depth		61-In. Web Depth	
		A36	A242	A36	A242	A36	A242	A36	A242
64	30	97.3	101.5	96.7	103.0	98.0	105.4	—	—
68	31	96.3	100.4	96.4	102.1	97.4	104.5	—	—
70	32	96.0	99.9	96.0	101.5	96.9	104.3	—	—
75	34	95.2	99.1	95.6	100.5	96.3	103.4	—	—
80	36	94.7	98.4	95.0	99.7	95.7	102.8	—	—
85	38	94.3	97.7	94.5	98.6	95.2	102.0	—	—
90	41	93.9	96.9	93.8	97.4	94.4	101.3	—	—
95	44	93.4	99.7	—	—	93.9	100.7	97.0	106.1
100	46	93.0	99.1	—	—	93.2	100.0	96.5	105.0
105	48	92.7	98.9	—	—	92.4	99.4	95.7	103.8
110	50	92.6	98.7	—	—	—	—	95.0	102.8
115	53	92.5	101.7	—	—	—	—	94.2	101.2
120	55	92.5	98.9	—	—	—	—	93.4	100.3

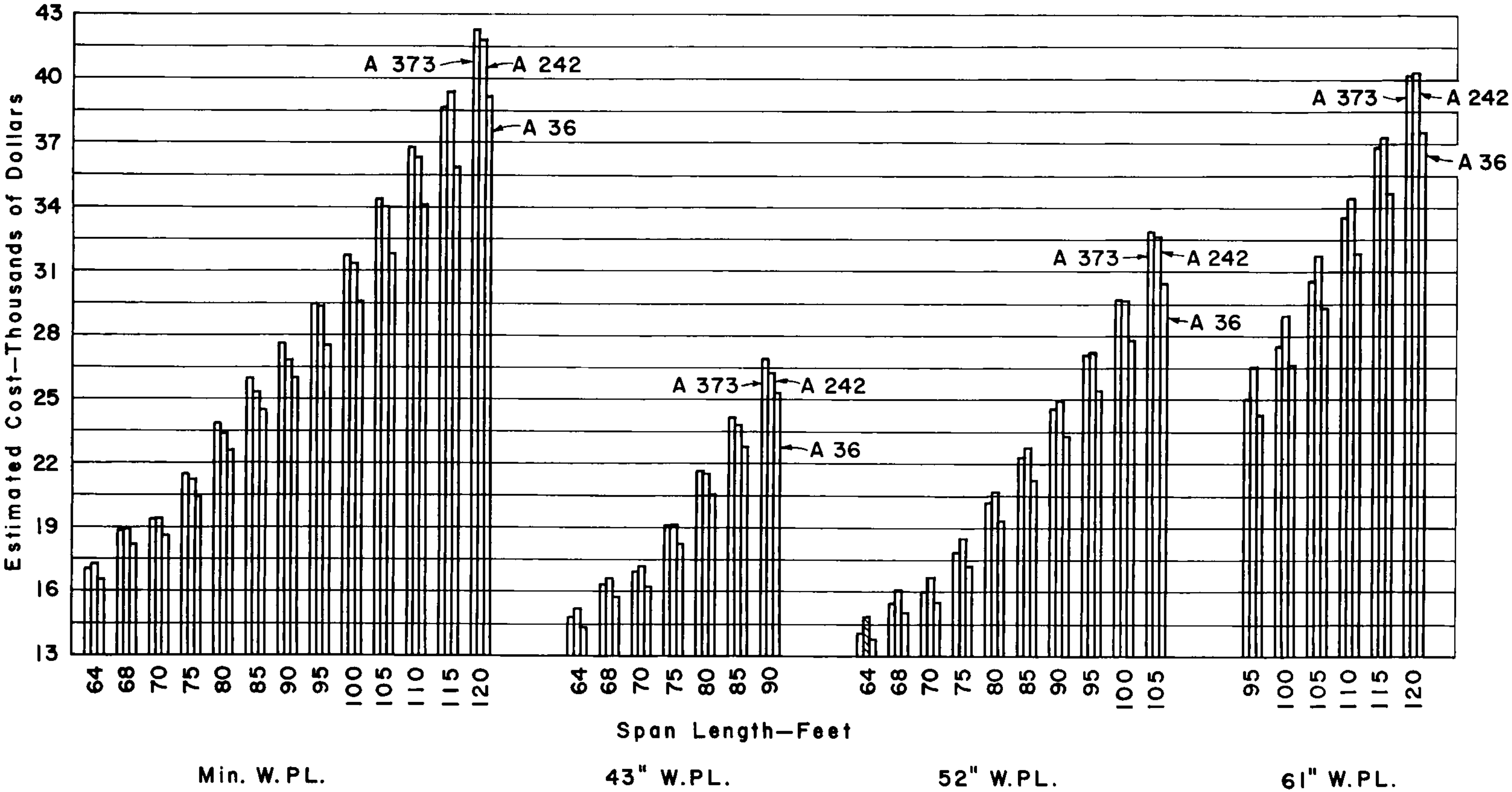

Figure 13. Noncomposite action, vertical stiffeners.

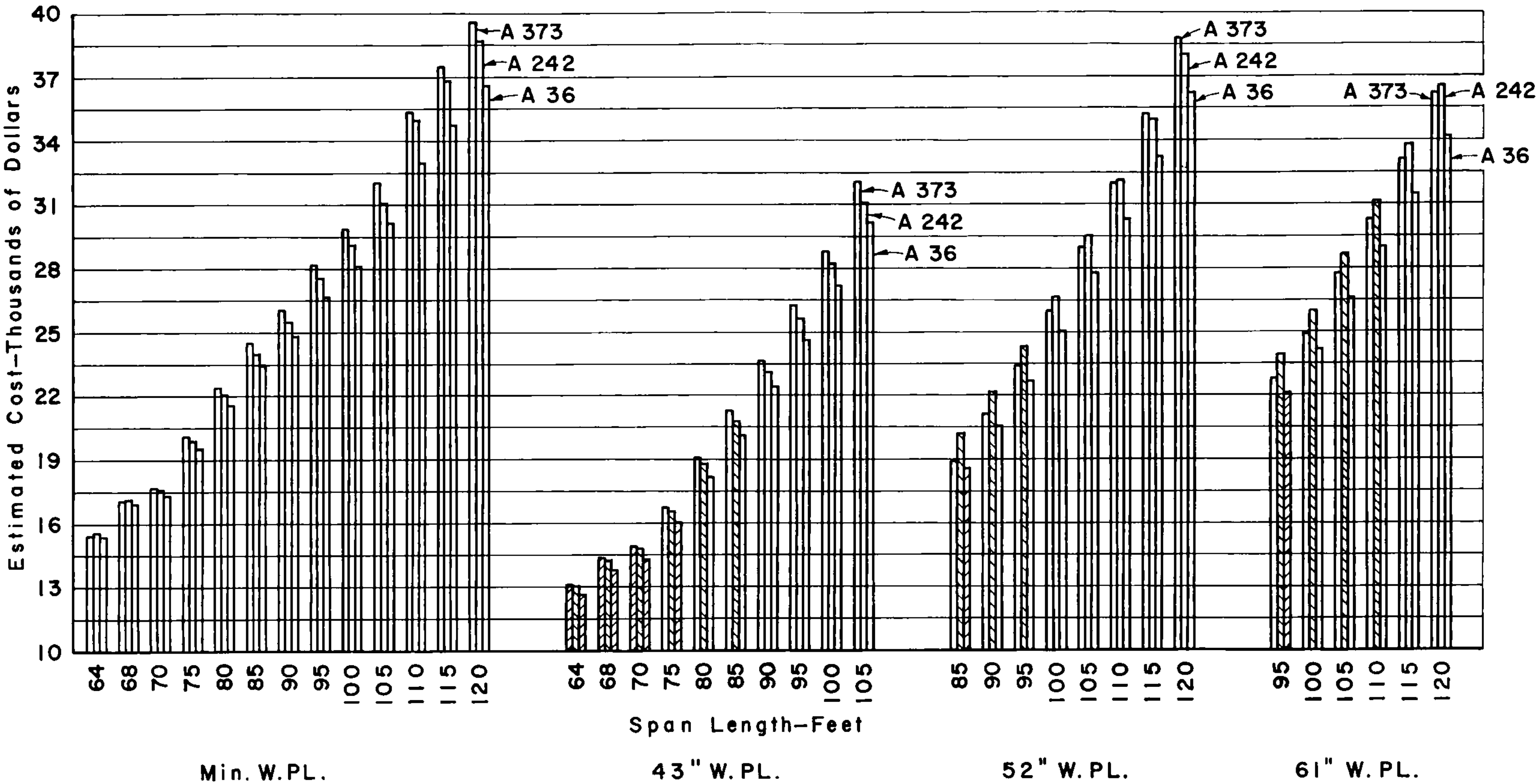

Figure 14. Composite action, vertical stiffeners.

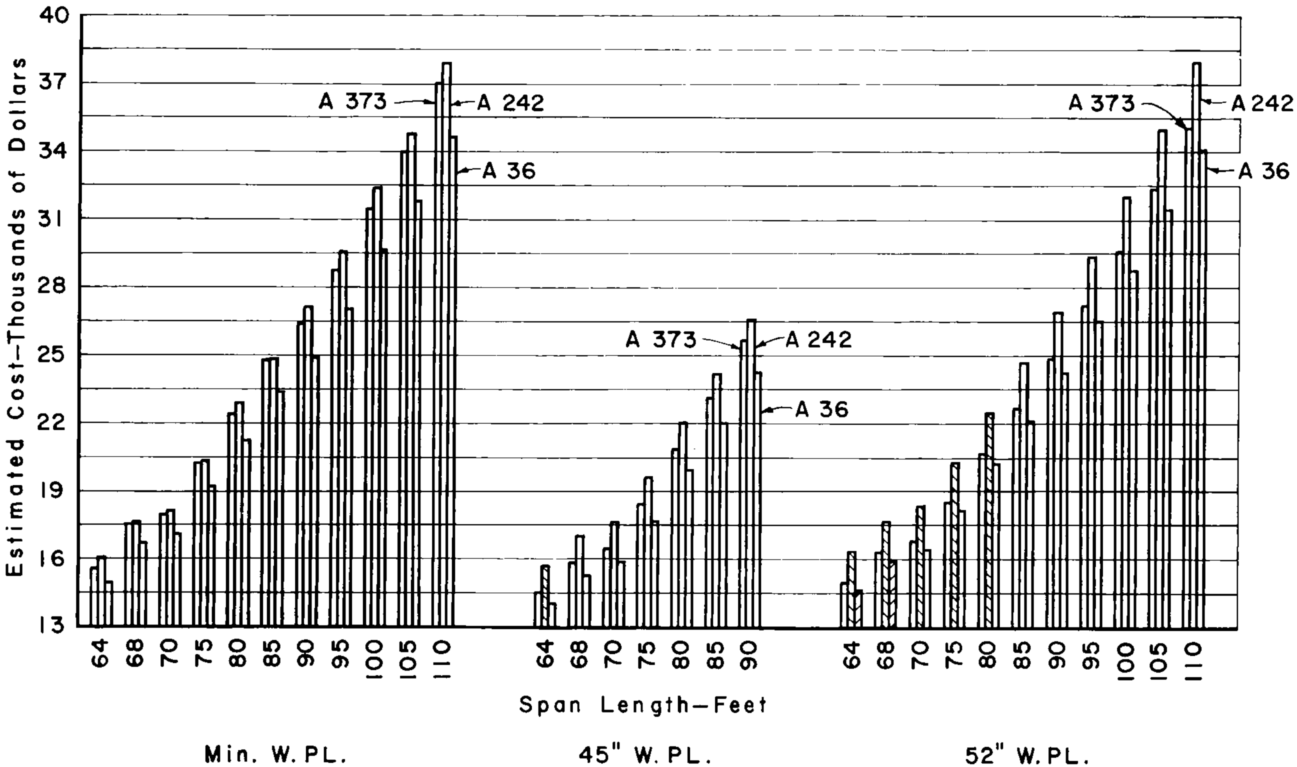

Figure 15. Noncomposite action, no stiffeners.

TABLE 24

COST ESTIMATE PERCENTAGES, COMPOSITE ACTION, VERTICAL STIFFENERS

Span Length (ft)	Web Plate Depth (in.)	Percentage of Total Cost Based on A373 Steel as 100%							
		Minimum Web Depth		43-In. Web Depth		52-In. Web Depth		61-In. Web Depth	
		A36	A242	A36	A242	A36	A242	A36	A242
64	25	99.9	101.2	96.5	99.8	—	—	—	—
68	26	98.7	100.2	96.1	99.4	—	—	—	—
70	27	97.9	99.7	96.0	99.6	—	—	—	—
75	28	97.3	98.8	95.5	98.8	—	—	—	—
80	30	96.5	98.6	95.2	98.7	—	—	—	—
85	32	95.7	98.0	94.7	97.9	98.0	106.5	—	—
90	35	95.4	98.1	94.7	97.9	97.4	105.2	—	—
95	37	94.5	97.6	94.1	97.7	96.7	103.9	96.8	104.7
100	40	94.1	97.5	94.1	97.8	96.2	102.7	96.8	104.3
105	43	94.0	97.1	94.0	97.1	95.7	101.8	95.7	103.2
110	44	93.2	98.9	—	—	94.8	100.5	95.6	102.7
115	47	92.7	98.3	—	—	94.2	99.4	95.2	102.1
120	50	92.6	97.8	—	—	93.4	98.2	94.3	100.9

TABLE 25

COST ESTIMATE PERCENTAGES, NONCOMPOSITE ACTION, NO STIFFENERS

Span Length (ft)	Web Plate Depth (in.)	Percentage of Total Cost Based on A373 Steel as 100%					
		Minimum Web Depth		45-In. Web Depth		52-In. Web Depth	
		A36	A242	A36	A242	A36	A242
64	30	96.1	103.4	97.0	108.1	97.7	108.6
68	31	95.3	100.6	96.4	107.3	97.7	108.8
70	32	95.0	100.3	96.1	107.1	97.9	109.2
75	34	95.0	100.4	95.8	106.3	97.9	109.2
80	36	94.7	102.2	95.4	105.4	97.7	108.9
85	38	94.5	100.1	94.9	104.6	97.6	108.7
90	41	94.2	102.5	94.5	103.6	97.4	108.2
95	44	94.2	102.9	—	—	97.3	108.0
100	46	94.1	102.9	—	—	97.3	108.1
105	48	93.8	102.5	—	—	97.3	107.9
110	50	93.4	102.4	—	—	97.2	108.0

TABLE 26

COST ESTIMATE PERCENTAGES, COMPOSITE ACTION, NO STIFFENERS

Span Length (ft)	Web Plate Depth (in.)	Percentage of Total Cost Based on A373 Steel as 100%	
		Minimum Web Depth	
		A36	A242
64	25	95.0	98.9
68	26	93.9	97.9
70	27	94.1	98.2
75	28	93.5	97.3
80	30	93.2	99.5
85	32	93.1	97.0
90	35	92.9	97.7
95	37	92.4	99.7
100	40	92.6	100.5
105	42	92.5	98.2
110	44	92.1	100.2

TABLE 27

A373 STEEL, COMPOSITE ACTION, VERTICAL STIFFENERS, 8-FT 3-IN. STRINGER SPACING

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)		Weight (lb/ft)		Total Stress (psi)			Moment of Inertia (in.^{4})	
	Thick.	Height	Top	Bottom	I-Section	Total DL	Bottom Steel	Top Steel	Top Concrete	Steel	Composite
90	0.3125	35	26.21	42.97	272.41	1,195.6	17,972	17,938	582	22,396.0	46,327.9
		43	18.90	33.75	224.72	1,151.1	17,972	17,985	507	25,941.1	57,370.8
		52	13.76	26.99	193.80	1,123.7	17,901	17,932	443	30,124.2	70,889.2
95	0.3125	37	27.93	44.33	285.00	1,208.1	18,080	17,999	572	26,246.6	52,456.8
		43	22.03	37.34	247.55	1,178.0	17,987	18,039	518	29,248.0	61,483.1
		52	16.14	29.88	211.72	1,140.6	17,903	18,014	454	33,848.3	75,813.2
	0.375	61	11.49	23.94	198.21	1,130.6	17,920	17,960	409	38,582.2	91,650.1
100	0.3125	40	28.57	44.63	291.37	1,214.9	18,080	18,045	551	31,162.2	60,885.2
		43	25.76	40.88	272.27	1,196.9	18,098	17,932	526	32,941.8	65,570.7
		52	18.73	32.86	230.66	1,158.7	17,939	18,076	463	37,825.8	80,803.9
	0.375	61	13.76	26.46	214.53	1,146.0	17,925	17,918	418	43,289.5	97,539.1
105	0.3125	43	29.24	45.00	298.12	1,222.0	18,085	18,090	533	36,635.5	70,096.9
		52	21.89	35.89	251.69	1,179.0	18,018	17,941	471	42,378.0	85,902.0
	0.375	61	16.05	29.19	231.60	1,162.2	17,908	17,984	426	48,137.9	103,742.8
110	0.3125	44	32.62	47.97	320.74	1,244.4	18,092	17,944	528	41,749.8	76,698.4
		52	25.13	39.27	274.20	1,200.8	18,027	17,916	476	47,144.1	91,448.6
	0.375	61	18.57	32.02	249.79	1,179.7	17,908	18,015	432	53,347.9	110,118.4
115	0.3125	47	33.47	48.50	328.63	1,252.8	18,051	17,931	451	48,571.1	87,586.3
		52	28.26	42.82	296.90	1,222.8	18,048	18,052	481	51,868.6	97,116.5
	0.375	61	21.33	34.96	269.17	1,198.4	17,924	18,019	437	58,933.2	116,690.5
120	0.3125	50	33.87	49.08	335.16	1,259.8	18,014	18,096	495	55,667.7	99,267.7
		52	32.28	46.78	324.05	1,249.4	18,005	17,929	483	57,631.7	103,497.8
	0.375	61	24.38	38.10	290.20	1,218.8	17,928	17,980	441	65,018.0	123,636.9

TABLE 28

A242 STEEL, COMPOSITE ACTION, VERTICAL STIFFENERS, 8-FT 3-IN. STRINGER SPACING

Span Length (ft)	Web Plate (in.)		Flange Plate (sq in.)		Weight (lb/ft)		Total Stress (psi)			Moment of Inertia (in.⁴)	
	Thick.	Height	Top	Bottom	I-Section	Total DL	Bottom Steel	Top Steel	Top Concrete	Steel	Composite
90	0.3125	35	16.53	30.62	197.51	1,120.7	23,918	24,045	662	15,287.6	36,782.9
		43	11.78	24.03	167.44	1,093.8	23,938	24,069	569	17,926.3	45,819.1
	0.375	52	7.72	18.38	155.04	1,084.9	24,085	23,950	498	20,972.3	57,129.5
95	0.3125	37	17.73	31.66	207.21	1,130.4	23,964	24,026	651	17,997.0	41,658.1
		43	13.99	26.39	182.96	1,108.4	24,060	23,940	584	20,227.6	48,762.4
	0.375	52	9.27	20.60	167.86	1,096.8	23,946	24,098	512	23,544.9	61,135.4
	0.4375	61	5.76	15.99	164.66	1,097.0	23,932	23,916	459	27,093.0	74,948.8
100	0.3125	40	18.12	31.79	212.19	1,135.7	23,973	24,079	629	21,382.9	48,251.5
		43	16.27	29.06	199.81	1,124.5	24,050	23,944	597	22,663.8	52,009.1
	0.375	52	11.08	22.62	180.90	1,109.0	24,047	24,075	525	26,307.6	64,763.0
	0.4375	61	7.23	17.63	175.26	1,106.7	24,090	23,970	472	30,147.6	78,996.0
105	0.3125	43	18.58	32.03	217.77	1,141.7	23,948	24,100	609	25,199.3	55,517.7
	0.375	52	13.16	24.82	195.42	1,122.7	24,073	23,935	535	29,400.3	68,659.1
	0.4375	61	8.79	19.53	187.04	1,117.7	24,085	24,026	482	33,477.4	83,580.1
110	0.375	44	20.20	33.67	239.26	1,162.9	23,982	24,015	610	28,629.6	60,728.7
		52	15.23	27.20	210.55	1,137.1	24,067	23,957	545	32,563.3	72,766.2
	0.4375	61	10.55	21.53	199.83	1,129.7	24,062	23,981	492	37,137.0	88,350.5
115	0.375	47	20.66	33.86	245.30	1,169.4	23,979	24,028	592	33,256.2	69,119.0
		52	17.52	29.69	226.83	1,152.8	24,057	23,926	553	35,990.0	77,036.1
	0.4375	61	12.48	23.55	213.25	1,142.5	24,094	23,904	500	41,024.6	93,128.3
120	0.375	50	21.13	34.09	251.49	1,176.1	23,988	24,045	575	38,337.1	78,177.9
		52	19.85	32.55	244.47	1,169.8	23,924	23,992	560	39,620.8	81,774.1
	0.4375	61	14.43	25.77	227.41	1,156.0	24,065	23,981	507	45,059.6	98,267.1

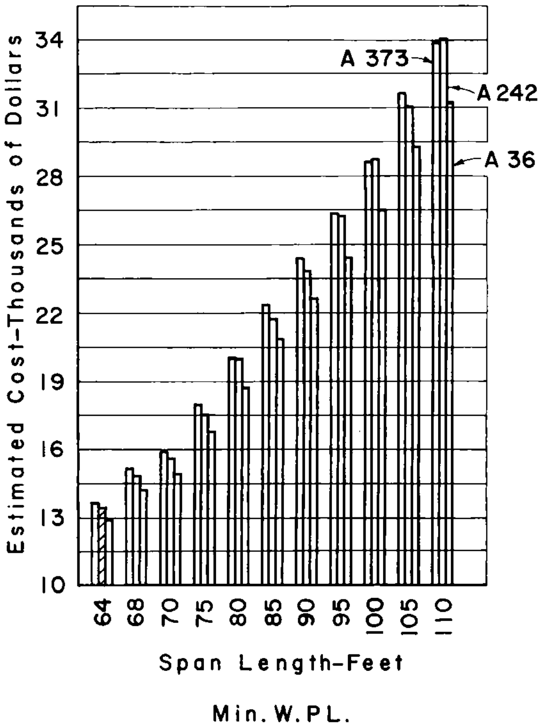

Figure 16. Composite action, no stiffeners.

TABLE 29

DIFFERENCES IN TOTAL VALUES OF STRUCTURAL STEEL QUANTITIES, TOTAL ESTIMATED COST BETWEEN 7-FT 0-IN. AND 8-FT 3-IN. INTERIOR STRINGER SPACINGS, 100-FT SPAN LENGTH

Steel	Web Plate (in.)		7-Ft 0-In. Spacing			8-Ft 3-In. Spacing		
	Depth	Thick.	Total Structural Steel (lb)	Unit Price ($)	Total Cost ($)	Total Structural Steel (lb)	Unit Price ($)	Total Cost ($)
A373	40	0.3125	170,841	0.1750	29,897	168,885	0.1733	29,268
A242	40	0.3125	130,665	0.2231	29,151	128,498	0.2162	27,781
Diff.			—40,176		—746	—40,387		—1,487
A373	43	0.3125	161,276	0.1791	28,885	159,263	0.1758	27,998
A242	43	0.3125	125,150	0.2258	28,259	122,262	0.2176	26,604
Diff.			—36,126		—626	—37,001		—1,394
A373	52	0.3125	141,818	0.1833	25,995	138,302	0.1811	25,046
A242	52	0.375	117,716	0.2268	26,698	112,887	0.2214	24,993
Diff.			—24,102		+703	—25,415		—53
A373	61	0.375	134,887	0.1850	24,954	130,328	0.1820	23,720
A242	61	0.4375	116,661	0.2232	26,039	110,046	0.2215	24,375
Diff.			—18,226		+1,085	—20,282		+655

APPENDIX A

COMPUTER PROGRAM

The program is written to compute the areas of the flange plates for a specified width and thickness of web plate, with the stresses in the top and bottom extreme fibers of the steel I-section reaching their allowable values simultaneously, for the H20-S16 live loading in accordance with the 1957 AASHO standard specifications for highway bridges. The following assumptions are made:

1. All dead load, including future wearing surface, carried by steel alone.
2. Bottom of concrete slab at top of top flange plate.
3. Flange plates same size throughout length of stringer.

The computational procedure, outlined in Figure 17 (see Fig. 18 and Appendix B for a description of terms used), is as follows:

1. Enter as input data: span length, stringer spacing, effective width and thickness of concrete slab, width and thickness of web plate, allowable stress in steel, assumed thicknesses of flange plates, estimated dead load, values of constants K_1, K_5, K_6, and K_7, and trial values of parameters K_2, K_3, and K_4.
2. Compute the live load plus impact moment and a value for the dead load moment based upon the estimated dead load.
3. Compute a first trial value for the portion of the allowable stress in the bottom extreme fiber of the steel available to carry dead load only, $K_3 \times f_{all}$.
4. Compute a first trial value for the distance from the bottom extreme fiber of the steel to the centroid of the steel section, $y_{bs} = K_4 \times h_w$.
5. Compute a first estimate of the value required for the total moment of inertia of the steel section.
6. Compute the required areas of the flange plates with area of the top plate equal to K_2 times the area of the bottom plate, $A_t = K_2 \times A_b$.
7. Compute the value of the dead load moment corrected to the dead weight of the computed steel section.
8. Compute the value of the distance from the bottom extreme fiber of the steel to the centroid of the steel section, y_{bs}. If this value is not within 0.01 in. of the first trial value used for this distance as determined in step 4, use this new value as a second trial value and repeat steps 4 through 8. Continue this process until the last computed value is not greater than, nor less than, the previous trial value by more than 0.01 in.
9. Using the latest computed value of y_{bs}, compute section moduli for the bottom extreme fiber of the steel for both steel action alone and composite action.
10. Compute the value of the total stress in the bottom extreme fiber of the steel Σf_b using the above mentioned section moduli.
11. If the value of this stress is less than the allowable value by more than 100 psi, increase the parameter K_3 by the constant increment K_7 and repeat steps 3 through 11 (here, $K_7 = 0.001$).
12. If the value of this stress is greater than the allowable value by more than 100 psi, decrease the parameter K_3 by the constant increment K_7 and repeat steps 3 through 12.
13. Continue steps 11 and 12 until the value of the stress Σf_b is not greater than, nor less than, the allowable stress by more than 100 psi.
14. Compute the appropriate section moduli and then compute the value of the total stress in the top extreme fiber of the steel, Σf_t.

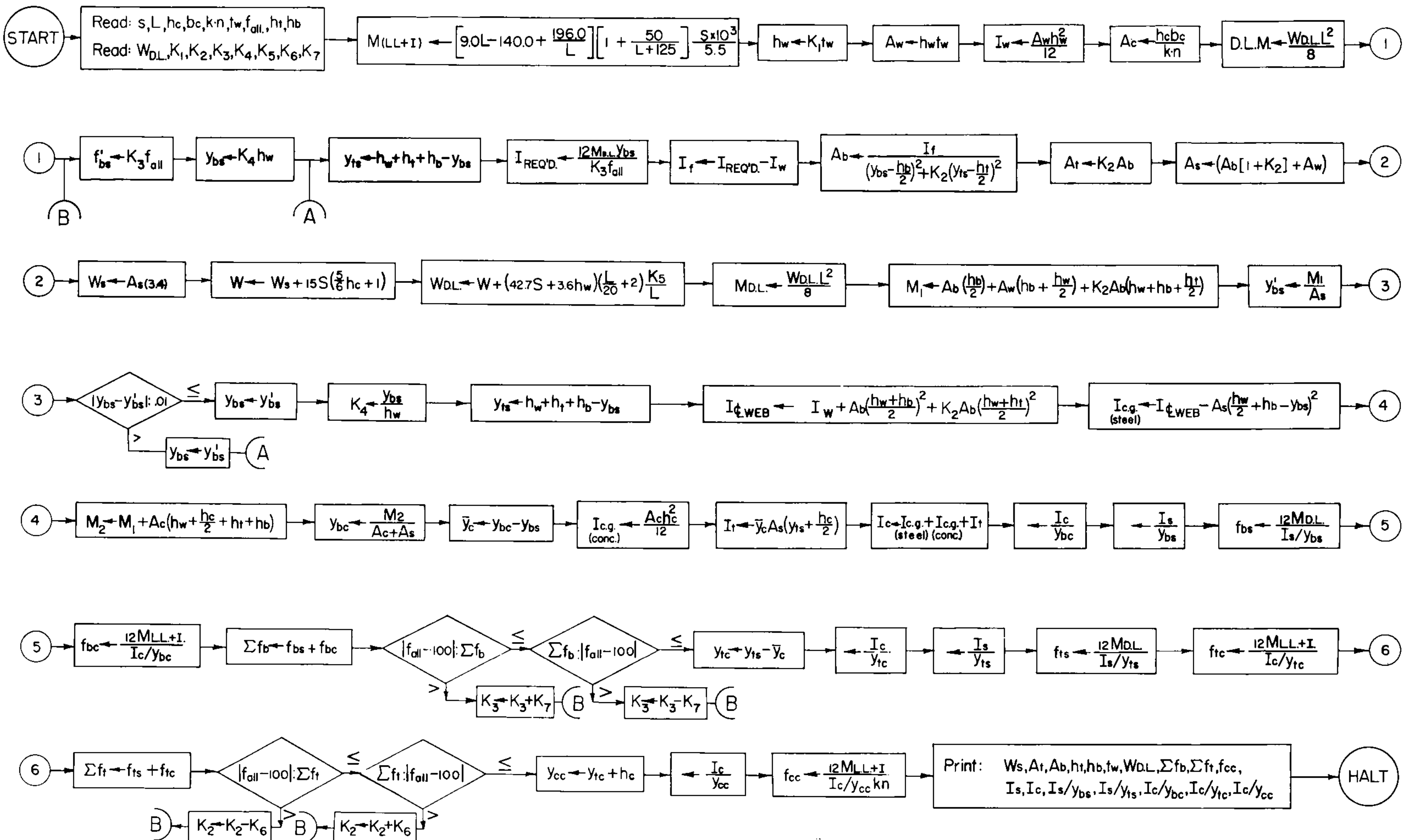

Figure 17. Computer flow diagram.

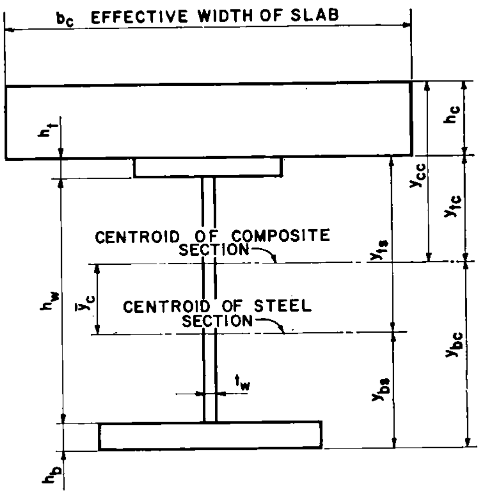

Figure 18. Assumed composite section.

15. If the value of this stress is less than the allowable stress by more than 100 psi, decrease the parameter K_2 by the constant increment K_6 and repeat steps 3 through 15 (here $K_6 = 0.01$).

16. If the value of this stress is greater than the allowable stress by more than 100 psi, increase the parameter K_2 by the constant increment K_6 and repeat steps 3 through 16.

17. Continue steps 15 and 16 until the value of the stress Σf_t is not greater than, nor less than, the allowable stress by more than 100 psi.

18. Compute the value of the stress in the top extreme fiber of the concrete slab, f_{cc}.

19. Print out the information as shown in Figure 17.

This program will work for noncomposite action, if the effective slab width is set equal to zero in the input data. A Burroughs Datatron 205 electronic computer was utilized in the study reported in this paper.

APPENDIX B

LIST OF NOTATIONS

A_b = area of bottom flange plate, sq in.;
A_c = effective area of concrete slab, sq in.;
A_s = area of steel section, sq in.;
A_t = area of top flange plate, sq in.;
A_w = area of web plate, sq in.;
E_c = modulus of elasticity of concrete, psi;
E_s = modulus of elasticity of steel, psi;
$f_{all.}$ = allowable stress in steel, psi;
f_{bc} = stress in bottom extreme fiber of steel due to live load and impact, psi;
f_{bs} = stress in bottom extreme fiber of steel due to dead load, psi;
f'_{bs} = first trial value for the portion of allowable stress in bottom extreme fiber of steel available to carry dead load only;
f_{cc} = total stress in top extreme fiber of concrete, psi;
f_{tc} = stress in top extreme fiber of steel due to live load and impact, psi;
f_{ts} = stress in top extreme fiber of steel due to dead load, psi;

I_c = moment of inertia of composite section, in.^{4};
$I_{cg\ \text{concrete}}$ = moment of inertia of concrete slab with respect to centroid of slab, in.^{4};
$I_{cg\ \text{steel}}$ = moment of inertia of steel section with respect to centroid of steel section, in.^{4};
I_c/y_{bc} = composite section modulus, bottom extreme fiber of steel, cu in.;
I_c/y_{cc} = composite section modulus, top extreme fiber of concrete, cu in.;
I_c/y_{tc} = composite section modulus, top extreme fiber of steel, cu in.;
I ℄ web = moment of inertia of steel section with respect to centerline of web plate, in.^{4};
I_f = moment of inertia of top and bottom flange plates with respect to centroid of steel section, in.^{4};
$I_{\text{req'd}}$ = required moment of inertia for steel section, in.^{4};
I_s = moment of inertia of steel section, in.^{4};
I_s/y_{bs} = section modulus, steel section, bottom extreme fiber of steel, cu in.;
I_s/y_{ts} = section modulus, steel section, top extreme fiber of steel, cu in.;
I_{transfer} = additional moment of inertia to transfer $I_{cg,\ \text{concrete}}$ and $I_{cg,\ \text{steel}}$ to centroid of composite section, in.^{4};
I_w = moment of inertia of web plate with respect to its centroid, in.^{4};
kn = modular ratio kE_s/E_c, where k depends on creep considerations taken equal to unity and E_s/E_c taken equal to 10;
K_1 = dimensionless ratio, h_w/t_w;
K_2 = dimensionless ratio, A_b/A_t;
K_3 = dimensionless ratio, $f_{bs}/f_{\text{all.}}$;
K_4 = dimensionless ratio, y_{bs}/h_w;
K_5 = dimensionless constant, 1.5 with vertical web stiffeners, 1.0 with no web stiffeners;
K_6 = dimensionless constant, increment for K_2;
K_7 = dimensionless constant, increment for K_3;
L = span length, distance between end bearings, ft;
M_{LL+I} = moment due to live load plus impact, ft-lb;
M_{DL} = moment due to dead load, ft-lb;
M_1 = first moment of area of steel section with respect to bottom extreme fiber of steel, cu in.;
M_2 = first moment of area of composite section with respect to bottom extreme fiber of steel, cu in.;
S = spacing center to center of stringers, ft;
W = dead load weight of steel plus slab and future wearing surface, lb per ft;
W_{DL} = total dead load, lb per ft;
W_s = weight of steel I-section, lb per ft;
Σf_t = total stress in top extreme fiber of steel, psi;
Σf_b = total stress in bottom extreme fiber of steel, psi.

DEPARTMENT OF MATERIALS AND CONSTRUCTION

Construction Methods Improvement by Time-Lapse Movie Analysis

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Constant and systematic methods improvement is an area for possible cost reduction that construction contractors cannot afford to overlook. The details of daily operations, including selection of methods, tools, and sizing of crews, are often delegated to the craft foreman. It is proposed that construction management should assume more responsibility for the development and use of improved procedures. The use of time-lapse movies for analysis of operations is suggested as an additional technique besides the uses of stop-watch time studies and cost accounting data. The advantages of this technique are discussed. An actual example involving erection of tubular metal falsework for an elevated freeway structure is used to illustrate a movie analysis study. Both equipment and procedures are described.

• THE AWARD of construction contracts for public works in the United States is based solely on price competition. Detailed plans and specifications are made available when the project is advertised. Each contractor who bids is offering to build the same structure, to meet the same standard of quality for materials and workmanship, and to complete the project within the same time limit. By law the award is made to the lowest responsible bidder. Hence the distinguishing factor between the services offered by one contractor and those offered by another is the total price quoted.

Every contractor treads a narrow path. His prices must be low enough to obtain a share of the work available but high enough to make a sufficient profit to justify remaining in business. Success demands an active and continual search for methods to reduce costs. There are many areas that offer possibilities for cost reduction, and all are important. One of these is the improvement of field methods. Such improvements can be of the more spectacular type that result from a novel approach to a conventional problem, the discovery of a new technique, or a major equipment modification to meet job conditions.

Almost every issue of construction trade magazines contains examples of ingenious solutions that have given one contractor a competitive edge over others for a specific job. The possibility of these solutions makes construction works the exciting and challenging business that it is.

However, a less spectacular type of methods improvement offers even greater possibilities for cost savings. It consists of systematic and sustained efforts to accomplish the routine and detailed steps of each operation in a more economical manner. In view of their importance, the treatment accorded these efforts by construction management generally falls short of what seems justified. Management seems content to pass the responsibility for details on to the craft foreman or the worker himself. Once the general plan of operations has been determined and the major equipment has been selected, it is usually the foreman who chooses the tools, sizes the crew, schedules the daily routine, and selects the procedures to be followed. This delegation of management control is apparently based on the assumptions that the foreman has gained such a degree of "know-how" from past experience that he is the one best qualified to make these decisions and that he will be motivated to draw on the best practices he has learned. These assumptions are questionable. There are about as many ways to perform an operation as there are foremen that can be assigned to do it. The foreman is generally a temporary employee drawn from the union hiring hall. His experience and ability may vary greatly. Often he does contribute or develop better methods than those used previously by his employer, but unfortunately these usually leave with him at the end of the job.

A better situation would be for management to retain a greater share of responsibility for detailed performance as well as for over-all planning. The systematic and continual effort to improve methods should be conducted by permanent and dedicated personnel. The best practices that foremen and workers bring to the job should be retained and used by other crews on future jobs. More sophisticated procedures for analyzing operations and evaluating methods should be developed. When improvements have been developed, better ways of "selling" them to the foreman or worker who will execute them are required.

To justify much special attention and analysis, an operation should be a repetitive one. Highway construction offers greater opportunities for profitable study of methods than perhaps any other type of construction. The highly organized and mechanized paving spreads give proof that this opportunity has been appreciated by the industry. On the other hand, the structures contractor is inclined to feel that most of his operations are unique on each job. This is not as true as often imagined. Although an over-all operation may involve conditions that are not likely to be found in the same combination again, the individual steps required to accomplish the operation are likely to be very similar to the corresponding steps of many past and future jobs. So it is these steps, or sub-operations, that should be subjected to systematic analysis.

Unfortunately the contractor's principal formal tool for evaluating his work is not a very effective one for sub-operation analysis. A good cost accounting system is invaluable for control purposes. However, the unit costs obtained are generally for unique operations composed of many steps. To obtain costs for these separate steps is beyond the capabilities of a cost accounting system. Such a system is based on the time distributions made on foremen's report cards at the end of each day. A too detailed breakdown of accounts would destroy the value of the results obtained rather than increase it. This limits its usefulness in comparing operation methods on one job with those used on another. For the job where the lower unit cost is obtained, there is no conclusive means of pinpointing the reasons for the improvement. The many steps that make up the operations compared are usually present in entirely different proportions and several of them, rather than just one, may have been performed in different ways.

A procedure for effective methods analysis and improvement may be summarized as follows:

1. An operation must be broken down into its separate steps.
2. Quantitative data that may be converted to dollars and cents must be collected for each of these steps.
3. The performance of the steps should be analyzed systematically for possible improvements, using both common sense and any formal work simplification technique available.
4. The best method determined, including any improvements developed, must be "sold" to the men responsible for their execution.
5. A follow-up is desirable to determine if changes made do really result in improvements.

The basic data for such an approach can be obtained by stop-watch time studies. The purpose of this paper is to suggest a different approach that has several advantages over stop-watch time studies.

TIME-LAPSE MOVIES, PROCEDURE AND EQUIPMENT

Many contractors record some of their more interesting operations on movie film. This is often done in an informal manner by job superintendents or engineers and sometimes in a more professional manner by hired photographers. In some cases these films are shown at meetings of company management personnel to pass good ideas from job to job. Such interchange and interest has value but fails to furnish the quantitative working data necessary for detailed study. With little additional effort and equipment, much more useful information may be obtained.

Short-interval, time-lapse movies can often provide the basic data for effective method improvement studies practically and economically. A number of such studies have indicated that 3-sec intervals between exposures generally produce the most useful results. This is a compromise between micromotion studies on the one hand and long-interval time-lapse pictures on the other. Micromotion studies, where movies are taken at normal or accelerated speeds, are useful in studying assembly-line manufacturing operations where hand motions, for example, are highly repetitive. Their use seems hardly justified for most construction operations, and the film produced would be too voluminous for economical analysis. Long-interval time-lapse movies, where individual exposures may be made every 5 min, 30 min, or 6 hr, for example, are useful for progress studies and general analysis. They do not provide sufficient data for detailed analysis.

The field equipment for short-interval, time-lapse studies can simply include a movie camera, a tripod, and a stop watch. These are items that are often already present on the job. A conscientious operator can trip a movie camera manually every 3 sec with sufficient precision to obtain effective data. Precise tripping on each exposure is unimportant as long as the total number of exposures for each minute is correct. However, a battery-operated timing circuit controlling a tripping solenoid eliminates the tediousness of manual operation and permits the operator to devote his efforts to improved documentation. Figure 1 shows the field equipment used for a study. The 16-mm camera was tripped by a solenoid actuated by the small, transistor timing circuit housed in a box mounted on the camera base plate. Power was supplied by a small 24-v storage battery hung from the tripod in a carrying case. The storage battery may be recharged nightly or less frequently as required. The camera may be removed from the tripod and held by the operator to obtain close-up pictures or to get into tight quarters. The timer will continue to operate from the power supply hung from the operator's shoulder.

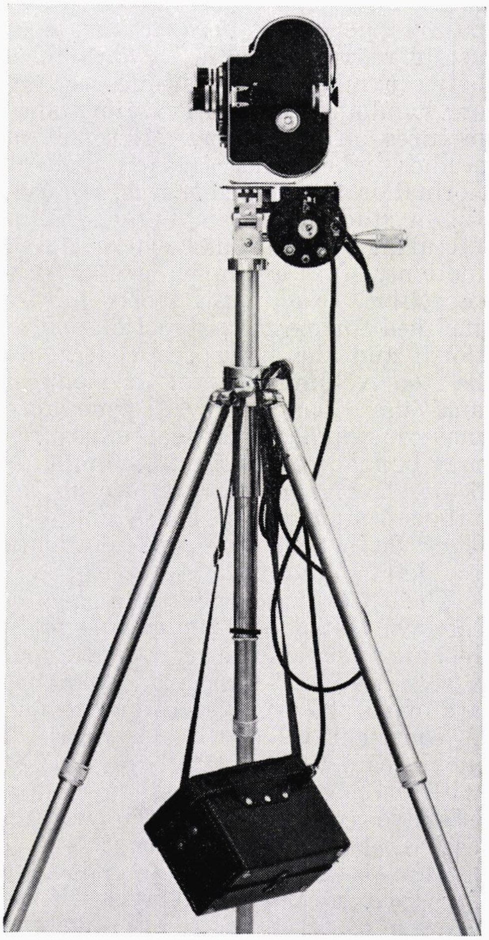

Figure 1. Field equipment for time-lapse movies, including electronic timer and battery power supply.

The office equipment for analysis work includes a conventional film viewer, or editor, equipped with a frame counter (Fig. 2). This enables the analyst to observe single frames as long as he wishes and to advance or reverse the film as slowly as he desires. Having selected cycle end points he may take frame counter readings at the beginning and end of each operation step and obtain time data. Other types of equipment are available for group presentations. Specially equipped stop-motion projectors permit observation of single frames without film damage and allow films to be advanced or reversed a frame at a time as well as at varying continuous speeds. These may range from simple hand-crank projectors to more elaborate, automatically controlled ones such as those sometimes used by football coaches to analyze plays.

ADVANTAGES OF MOVIE ANALYSIS

It was indicated that this approach can have several advantages over the more conventional stop-watch study for analyzing construction operation. Some of these advantages may be described as follows:

1. Most construction operations involve crew activity and one or more pieces of equipment. The simultaneous activities of each man and piece of equipment is important to record. Crew balance studies often indicate the most significant opportunities for improvement. Data for such studies require about as many stop-watch observers as crew members; however, a movie study permits a single observer to record all of the simultaneous activities accurately. By repeated observation of a cycle on film, the analyst may follow the ac-

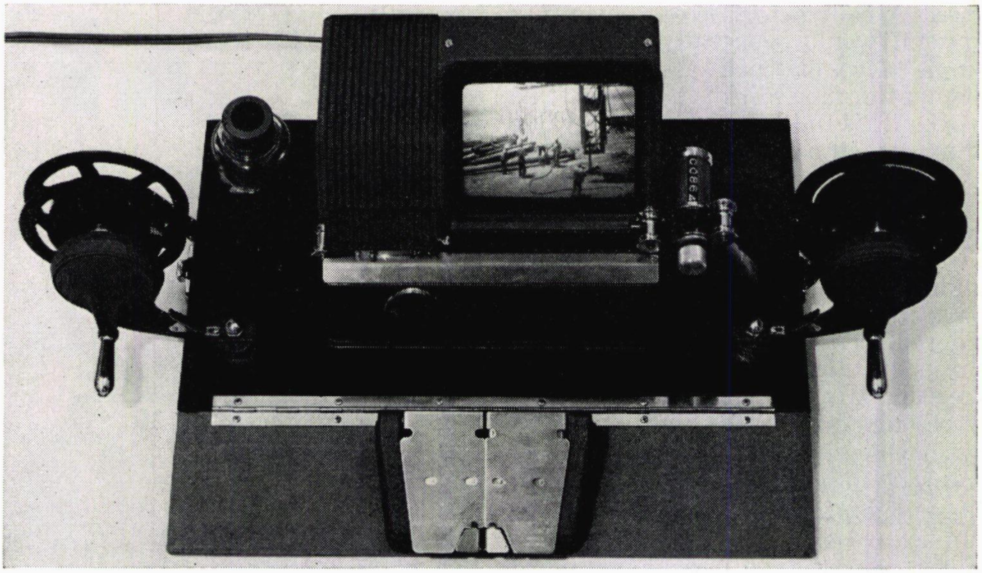

Figure 2. Office analysis of films by means of standard editor with frame counter.

tivities of one man at a time and construct an accurate crew activity chart.

2. Construction cycles are more likely to be irregular than regular making time studies difficult for even experienced industrial engineers. Contractors generally must use personnel that are not highly trained in time-study techniques. Because a film may be observed as many times as desired, away from the job, it is simple for even the amateur to draw useful data from irregular cycles.

3. Construction operations are influenced by changing work conditions, weather, and surroundings to a considerable extent. Complete documentation of the important conditions, or even a recognition of which conditions are important, is difficult. Where data will be used by others who have never seen the job, the conditions cannot be fully appreciated. A movie record gives a complete and easily understood documentation of work conditions.

4. Proposed improvements, supported by arrays of stop-watch data, are difficult to "sell" to the foreman in the field. A graphic presentation, such as that which is possible by looking at a film strip, is easy to appreciate, encourages participation in further method improvement suggestions by the foreman, and allows him to evaluate more intelligently the changes from a practical standpoint in advance of their trial.

5. The movie studies are an economical means of data procurement. At 3-sec intervals a 100-ft roll of film permits a single operator to record completely a total of $3\frac{1}{3}$ hr of continuous operation. Data for a number of cycles of several steps of one operation can usually be obtained on a single film.

TUBULAR FALSEWORK ERECTION

As an illustration, a movie analysis study of a highway structures operation will be described—the erection of tubular metal scaffolding as falsework for an elevated freeway structure. A contractor's cost accounting system would generally include a single account such as "erect and dismantle falsework" to cover this work. This unit cost is of little value from a methods improvement standpoint in determining why this operation is better or poorer than a similar operation on another job. This fact is more understandable when the number of individual steps that are part of this single operation are considered. Table 1 shows 33 separate sub-operations involved in just the erection of falsework towers that were only two frames high.

The movie camera was used to record a number of cycles of each of these operation steps as the opportunity arose. Later the film was edited and the operations arranged in proper sequence. The entire erection operation was covered by four 100-ft rolls of film. The next step was to take the quantitative data collected in the field and to convert it into comparable dollars and cents figures. This was accomplished by viewing the film slowly, picking out cycle end points, and recording frame counter readings. The frame counter readings were converted to elapsed

TABLE 1

ERECTION OF TUBULAR METAL FALSEWORK

Sub-Operation No.	Description
1	Establish stockpiles of materials
2	Stake sill locations
3	Distribute sand for sills
4	Spread and level sand bedding
5	Place sills in approximate position
6	Set sills to string line
7	Mark sills for jack locations
8	Distribute and set bottom jacks
9	Distribute 1st-level frames
10	Distribute 1st-level X-braces
11	Erect 1st-level towers
12	Set grade stakes, ground level
13	Align and plumb 1st-level towers
14	Distribute scaffold planks
15	Distribute 2nd-level frames
16	Distribute 2nd-level X-braces
17	Distribute connecting pins
18	Erect 2nd-level frames
19	Install 2nd-level X-braces
20	Distribute caps with jacks
21	Raise scaffold planks
22	Install caps
23	Set grade line, top of tower
24	Level caps at grade line
25	Level alternate caps, straight edge
26	Hoist stringers
27	Position and toenail stringers
28	Check levels, all jack points
29	Adjust top jacks
30	Final check and adjust
31	Distribute pipe bracing
32	Distribute brace clamps
33	Install pipe bracing

TABLE 2

RELATIVE COST IMPORTANCE OF SUB-OPERATIONS, TUBULAR METAL FALSEWORK OPERATION

Sub-Op. No.	Description	Cost ($)	
		Sub-Op.	Group
26	Hoist stringers (6/load)	19.32	
27	Position and toenail stringers	1.46	20.78
26A	Hoist stringers (4/load)	25.50	
5	Place sills (crane)	11.83	
6	Set sills to string line	2.47	14.30
5A	Place sills (buggy)	12.55	
8	Distribute bottom jacks	3.06	
9	Distribute 1st-level frames	1.36	
10	Distribute 1st-level X-braces	1.54	
14	Distribute scaffold planks	0.62	
15	Distribute 2nd-level frames	(1.36)	
16	Distribute 2nd-level X-braces	(1.54)	
17	Distribute connecting pins	No cov.	
20	Distribute caps with jacks	4.10	
31	Distribute pipe bracing	No cov.	
32	Distribute brace clamps	No cov.	13.58+
13	Align and plumb 1st-level tower	7.60	7.60
3	Distribute sand for sills	4.84	
4	Spread and level sand bedding	2.44	7.28
22	Install caps	5.05	5.05
18	Erect 2nd-level frames	2.84	
19	Install 2nd-level X-braces	1.73	4.57
24	Level caps at grade lines	1.25	
25	Level alternate caps by street edge	3.31	4.56

time. The pictures showed the men and equipment involved. Knowing labor classifications and rates and knowing equipment use rates, costs were developed. To appreciate the relative importance of the costs of the various sub-operations, these costs were developed for a block of work common to all operations. In the example cited, a given number of square feet of supported roadway soffit was the common denominator. Table 2 shows sub-operations grouped in descending order of importance. Knowing the relative importance of the sub-operations, the appropriate amount of attention may be directed towards each.

Having used the films to analyze the operations, improvements are sought. The common-sense questions of Why?, Where?, What?, How?, and When? asked about the details shown may suggest changes in even the simplest tasks. For example, the sub-operation "distribute sand for sills" is about as basic a job as can be encountered. It is accomplished by common laborers and wheelbarrows moving sand from a pile at the edge of the roadway site to sill locations marked by string lines. Foreman directions such as "a couple of you men take a wheelbarrow over and move some of that sand" gave a work cycle in which one man stood idle at the sand pile while the other wheeled and dumped the sand and then returned to help fill the wheelbarrow again. The formal approach of a crew balance study is not required to suggest that costs might be reduced by sending one man to do the same job instead of two. Not only are costs cut in half because the crew is cut in half but costs are further reduced, as shown by other filmed cycles, because one man working alone fills the wheelbarrow faster than two men chatting with each other as they work together. But this is only one possible improvement. Another detail noticed was that the wheelbarrows being used were of the 1½-cu ft garden variety. A laborer could just as easily wheel 5 or 6 cu ft per trip if given a properly sized piece of equipment. When this film was shown at a meeting of job superintendents and foremen, one of the men observed that an entirely different approach had been used on his last job. A special chute had been provided for the tailgate of the truck, and the sand had been dumped directly along the sill lines upon delivery to the site. Another point that came up for discussion at this meeting was "Why do it at all?" Under some conditions a careful grading job might permit the omission of sand bedding. Another alternative is the use of small, individual pads under each tower leg instead of continuous sills. This affects both the distribution of sand bedding and the more costly crane operation of handling the heavy timbers used as sills. There are good reasons for adopting one alternative over another, but these can be profitably documented for future reference to insure that an alternative is not blindly used when the reasons for it are absent.

This simple example would hardly justify much formal attention. But, as is the case with practically all operations filmed, it is effective in making job management conscious of several points. First, there are numerous possibilities for cost reduction through measures that are properly management functions, including giving specific directions as to the manner in which a task is to be performed, sizing the crew properly, and selecting the best tools to furnish the workmen. Second, there is more than one way to accomplish even a simple task. It is only by the continual attempt to seek out the best, retain it, and see that it is used until a better procedure is developed, that real cost reduction can be achieved. Third, there are possibilities on many small, routine tasks of cutting costs not by 5 or 10 percent but in half, or to a fourth or even to a tenth of their present level. Or stated conversely, where management does not assume its proper responsibilities, costs of many sub-operations can increase in the order of magnitude of 100 to 1,000 percent without the reasons being readily apparent even to those in charge of the work.

The same contractor that erected the falsework in the foregoing example was concurrently performing a similar freeway job in another city. Many of the corresponding operations were handled quite differently. Even on the same job, the same operation is performed by different methods from time to time. This is the natural result of the fact that management has turned over the selection of detailed methods to temporary employees with different backgrounds of experience. The sub-operations themselves are often quite similar in scope from job to job. In the case of erecting tubular metal falsework, the procedure for aligning and plumbing a tower, for installing caps on top of it, or for doing practically any of the 33 sub-operations of Table 1 is little affected by whether the job is in San Diego or Philadelphia. Moreover, these are operations that will be repeated by the same contractor and by different contractors many millions of times. More attention to detailed performance seems justified.

Incidentally, comments on the example used for illustration are not a reflection on the abilities of the contractor doing this work; rather, the reverse is true. This company is one of the biggest and best in the United States. Its work receives more detailed planning and is in the hands of more competent supervision than the vast majority of similar jobs. In general, its procedures could serve as models to guide others. So if there are opportunities for method improvements in its work, as it would be the first to agree, then there are even greater opportunities for most other organizations. More important examples than falsework operations can receive similar analysis. For example, building form panels in the yard, erecting and stripping box girder forms, erecting and stripping column forms, handling and driving piles, and placing concrete are all operations that involve greater costs and also have repetitive sub-operations.

SUGGESTED PROGRAM

A systematic procedure for management control of job-level operations might include the following steps:

1. Analyze operations and attempt to develop the best methods for their performance. Draw on job personnel assistance at all levels in evaluating procedures. Just as safety meetings can create safety consciousness, method study sessions can create consciousness of method improvement opportunities. Films can often be used for both of these purposes simultaneously, inasmuch as a revised method cannot be a better one unless it is also a safe one.
2. Maintain a record of the best methods to be used with appropriate documentation of the reasons why it is considered best. (Table 3 gives a sample for the sub-operation discussed.)
3. Go over these methods with supervisory personnel, and workers also when practical, at the beginning of a job to obtain their cooperation in the use of methods.
4. Prepare pre-plans of work in order to assure that the proper tools and materials are available, the personnel is instructed as to how to accomplish the work, the crews are sized correctly, and units of work are performed at the proper times.
5. Encourage job personnel to challenge the "best" ways that have been

TABLE 3
TUBULAR METAL FALSEWORK OPERATION, SUB-OPERATION 3: DISTRIBUTE SAND FOR SILLS

Reference:
Film TTI; Frames 862 to 1190.

Conditions:
Sill locations indicated by string lines. Truck load of sand dumped at side of roadway location. Laborers shovel sand into $1\frac{1}{2}$-cu ft wheelbarrow and dump in piles along string line.

Typical Cycle:

Activity	*Time (min)*
1. Load wheelbarrow	0.90
2. Transport load (avg. 90 ft)	0.62
3. Dump sand	0.15
4. Return empty	0.60
	2.27

Crew and Equipment:

1 Laborer	$3.045
Wheelbarrow and shovel	(No use rate)
Total direct cost	$3.045 per hr

Cost Computation, 3,000 sq ft:

Avg. loads per row of sills	= 10.5
Cycles per 4 rows = (4) (10.5)	= 42
Cost = (42) (2.27) (3.045/60)	= $4.84

Remarks:
This cost based on performance by individual laborer. Cost much higher for two-man crew.
On future operations, try larger wheelbarrow (5 or 6 cu ft).
On future operation, consider tailgate chute to dump sand directly from delivery truck.

determined to date. If a foreman, for example, thinks he has a better approach and can give reasonably convincing arguments, permit him to try his method.

6. Analyze the new methods, including those of competitors, on a quantitative basis and replace existing methods as better ones are discovered.

The use of time-lapse movie methods is a technique that facilitates most of these steps. A basic premise of this paper is that foremen and workers do not necessarily know the best ways to perform their tasks. However,it is not intended to suggest that job management knows better ways and should arbitrarily enforce its ideas on the foremen. Construction management should assume greater responsibility and leadership in working with foremen and labor to develop better methods and to retain and use them. Group viewing of films of familiar operations by job personnel can produce very rewarding results. Foremen and workers are interested in presentations of this sort. Many of them have constructive ideas which they often are not given the opportunity to express. This attention to their work gives it an importance that they appreciate; moreover, it causes them to take a new look at their daily activities.

CONCLUSION

Over the past several years the author has conducted several movie analysis studies on heavy construction work. A by-product of these methods improvement studies has been material of considerable value for educational purposes. Detailed analysis of firms covering construction operations, such as box girder forming and placing, pile driving, or pipe laying, offers an excellent basis for understanding problems and procedures. More recently the National Association of Home Builders has been using 3-sec interval time-lapse movies for very complete studies of home building methods. The goal of its studies is not only methods improvement but also the development of new materials and new tools.

It should be emphasized that the described methods improvement studies are not directed towards making the individual worker work harder. Rather they are directed towards increasing the workers' effectiveness through measures that should be entirely within the realm of management's responsibility and authority. As construction wages continue to rise, it becomes more essential for management to assume this responsibility and for labor to adopt a cooperative attitude.

ACKNOWLEDGMENT

This work was made possible by a research contract awarded to the Civil Engineering Department at Stanford University by the Bureau of Yards and Docks, U. S. Navy. The purpose was to investigate and develop possible cost reduction techniques for construction operations. The use of time-lapse movies for methods analysis is only one of several areas of study undertaken under this contract. For assistance on the tubular scaffolding example, the author is indebted to Henning Jakobsen, a former graduate student in the construction engineering program at Stanford.

Static Carrying Capacity of Steel Plate Girders

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For the past few years an extensive research project has been conducted at Lehigh University to investigate the carrying capacity of thin-web plate girders. The work consisted of analytical studies and a series of tests on full-size, welded steel girders. Design recommendations prepared from the results have been incorporated in the new AISC design specifications. Some of the findings of this research are summarized in this paper and the more important design recommendations are presented briefly. The subject is treated from a physical rather than a mathematical viewpoint.

• PLATE GIRDERS often can be loaded beyond the web buckling load predicted by the classical plate buckling theory. This is due to the fact that the web plate is framed by flanges and transverse stiffeners that allow for a redistribution of stress.

Because a plate girder is usually subjected to bending, shear, or a combination of the two, the stress redistribution will be summarized for these three cases.

BENDING

Actual measurements show that the web of a steel plate girder is seldom a perfect plane and that sudden buckling of the web under bending is usually nonexistent (*1*). Measured cross-sectional configurations of a girder due to increasing moments are shown at the left in Figure 1. (The applied moments are expressed in terms of the yield moment which is the moment causing initial yielding.) The straightening of the tension portion and the gradual lateral deflection of the compression portion of the web is evident.

Examining the corresponding stress diagrams to the right (Fig. 1), two phenomena are observed: (a) the laterally deflected portion of the web does not carry the stresses computed using beam theory (thin, straight lines); and (b) the stresses in the compression flange are greater than the values derived from beam theory. The combination of these two effects indicates a redistribution of stress from the web to the flange. Such a redistribution can be relied on as long as the capacity of the flange is not exhausted; that is, as long as the flange does not fail.

Because the web carries a lower bending stress than the flange and because it is much closer to the neutral axis, the web's contribution to the resisting moment is only a small part of the total. A small area of the compression flange will supply a resisting moment equal to that of the compression portion of the web. This condition leads to the concept of

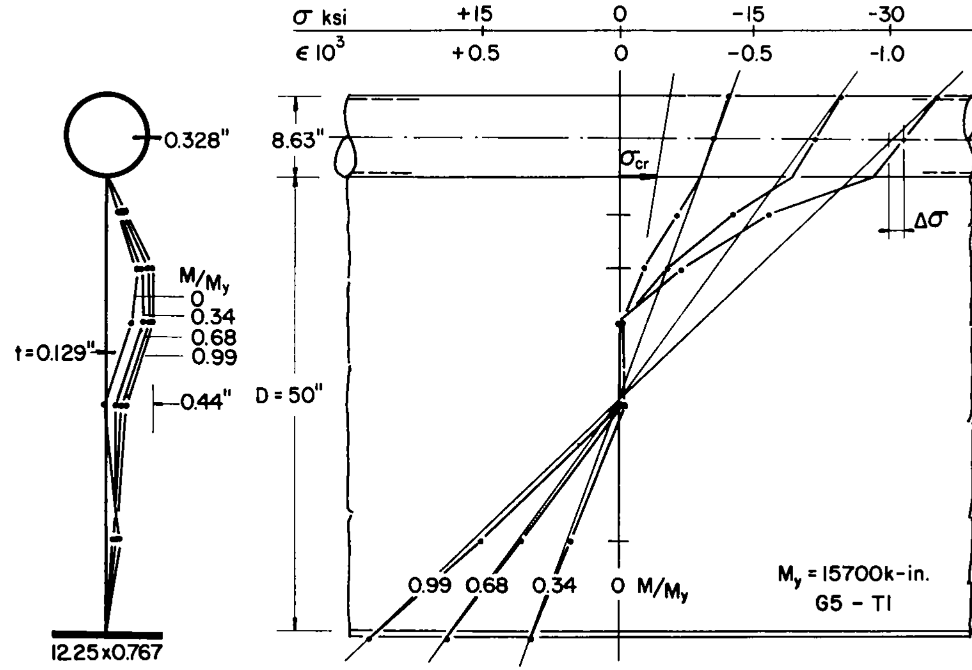

Figure 1. Measured bending stresses and web deflections.

effective width. A portion of the web adjacent to the compression flange is considered as a part of the flange to resist moment.

A compression flange may fail by buckling as well as by yielding. Figure 2 shows the three ways it can buckle; namely, vertically, torsionally, or laterally.

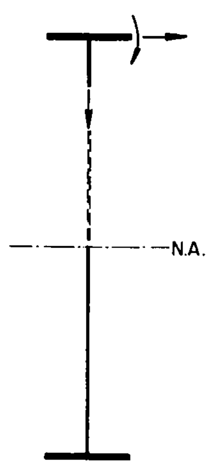

Figure 2. Buckling modes of compression flange.

Vertical Buckling

When a girder is subjected to moment, the compression flange bends toward the web. Having little rigidity in this direction, it depends on the web for support. If the supporting web buckles, the flange will also buckle (Fig. 3). Assuming that the web is strong enough to sustain the pressure from the compression flange so that the latter can be strained to yielding, no vertical buckling will occur before flange yielding. Consideration of equilibrium between the flange and the web under such a condition gives a limitation to the slenderness of the web (*2*).

$$\frac{D}{t} < \frac{0.48E}{\sqrt{\sigma_y\,(\sigma_y + \sigma_r)}} \tag{1}$$

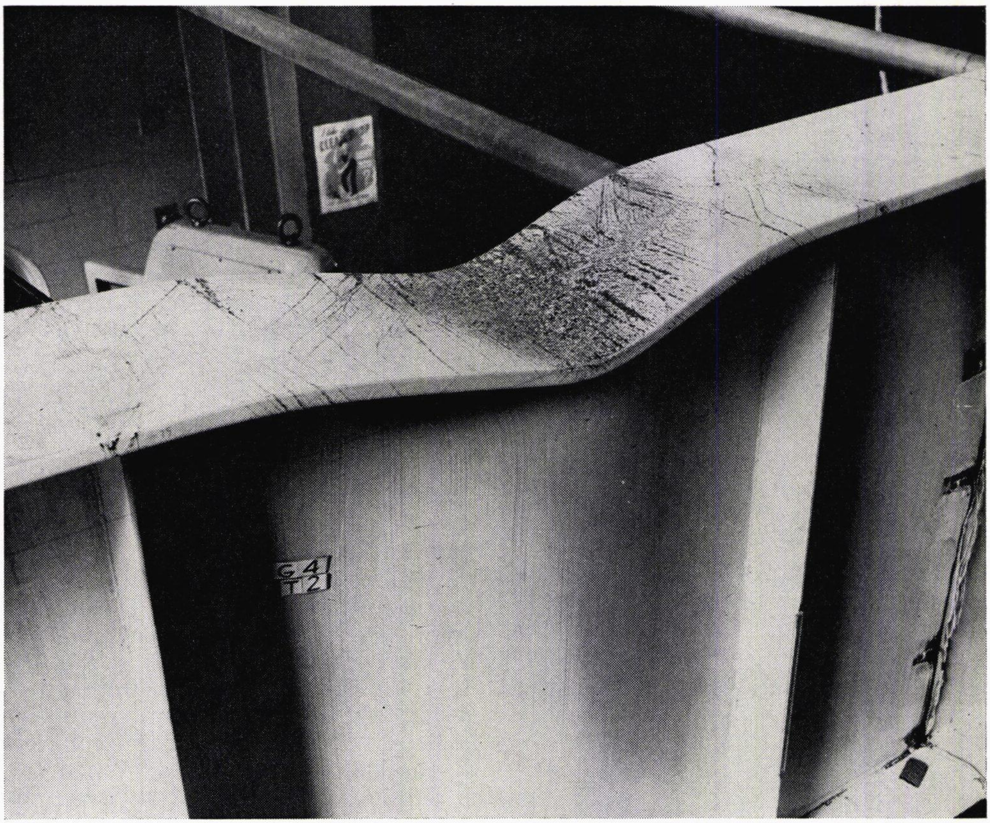

Figure 3. Buckling of flange.

in which D and t are the depth and thickness of the web, E and σ_y are the modulus of elasticity and yield point of the girder material, and σ_r is the residual stress on the flange. For a welded steel girder with $\sigma_y =$ 33,000 psi and σ_r = 16,500 psi, this limiting web slenderness ratio is D/t = 340.

Torsional Buckling

Twisting of the compression flange can only occur between transverse stiffeners and is, in effect, a local buckling of the flange. A frequently used method of preventing this type of failure is to specify a maximum value of the width-to-thickness ratio of the projecting portion of the flange.

Lateral Buckling

For shallow, stocky-rolled shapes, lateral buckling of a flange causes the entire section to tilt because the section is rigid enough to preserve its shape. The LD/bt formula applies for this case. However, preservation of cross-sectional shape cannot be assured for plate girders where the web is slender (Fig. 4). This implies that the buckling resistance is furnished only by the compression flange. The situation is therefore nothing more than a column problem. Using the formula suggested by the Column

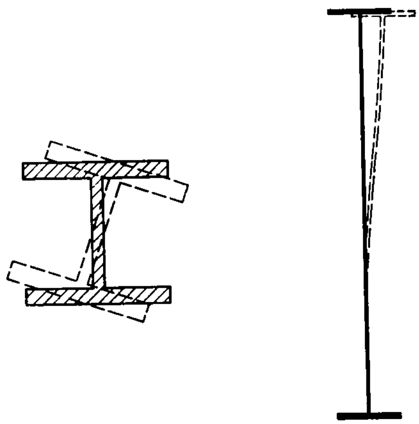

Figure 4. Lateral buckling of plate girder flange.

Research Council (*3*), the recommended formula for allowable stress can be obtained. For example, the allowable bending stress for steel with a yield point of 33,000 psi is

$$\sigma_a = 18{,}000 - \frac{0.52}{C_1}\left(\frac{L}{r}\right)^2 \qquad (2)$$

in which C_1 is a factor dependent on the moment gradient on the member (C_1 can be negative (*3*)), L is the unsupported length of the compression flange and r is the radius of gyration of the effective compression flange,

$$r = \sqrt{\frac{I_f}{A_f + A_w/6}}$$

If rectangular flange plates are used, Eq. 2 can be expressed as

$$\sigma_a = 18{,}000 - \frac{1.04}{C_1}\left(6 + \frac{A_w}{A_f}\right)\left(\frac{L}{b}\right)^2 \qquad (3)$$

which is in the form of the existing AASHO stress formula.

Flange Stress Reduction

It was pointed out that stress redistribution due to out-of-straightness of the web raises the flange stress. As a result, an adjusted allowable bending stress, slightly lower than otherwise permitted, must be used. Because thicker webs deflect less than thinner ones, it is apparent that the reduction is directly affected by the web slenderness ratio, D/t, and the area ratio, A_w/A_f, as is shown in Eq. 4.

$$\sigma'_a = \sigma_a\left[1 - 0.0005\frac{A_w}{A_f}\left(\frac{D}{t} - 170\sqrt{\frac{18{,}000}{\sigma_a}}\right)\right] \qquad (4)$$

In which, σ'_a is the adjusted allowable bending stress and σ_a is obtained from Eq. 2 or is equal to 18,000 psi whichever is smaller (*2*).

When the web is thick enough to resist lateral buckling, no reduction is necessary. The limit is expressed by the last term of Eq. 4. When D/t is smaller than the value given by this term, no reduction is required. For the cases where σ_a is 18,000 psi or 27,000 psi, web slenderness ratios of 170 or 140 are obtained which are the existing upper limits for transversely stiffened plate girders built of structural carbon steel or high-strength, low-allow structural steel, respectively. This is to say that using the new design recommendations, girders with D/t greater than the existing limits are permissible providing the allowable bending stress is reduced according to Eq. 4.

SHEAR

It is physically impossible to have pure shear loading on a plate girder. The effect of combined shear and bending will be discussed later and the effect of shear alone will be summarized now.

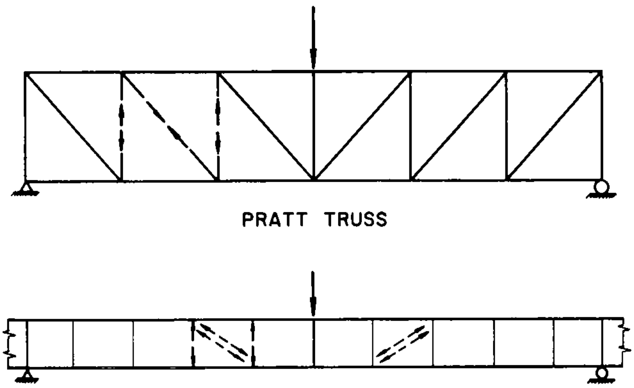

Figure 5. Truss analogy of plate girder.

Tension Field Action

The redistribution of stresses under shear is based on the concept of tension field action that has been used by aeronautical engineers for many years. To explain this action in terms familiar to civil engineers, a comparison between a plate girder and a Pratt truss is helpful.

Figure 5 shows a Pratt truss under load. All the diagonals are in tension with the vertical force components equal to the compressive forces in the neighboring struts. If the flanges and the struts are strong enough, the tension diagonals will fail by yielding. When the plate girder shown below the truss is examined with this in mind, the web of the girder appears to form actual tension fields analogous to the tension diagonals and can be stressed to yielding. The effect of web buckling on these imaginary tension fields is not immediately clear.

Again, test observations will assist in understanding the situation. That a perfectly plane web seldom exists and that a web only deflects gradually has been demonstrated by actual measurements on steel girders (*1*). The wavy pattern of web deflection in the compressive direction leaves fairly straight sections in the tensile direction (Fig. 6) and thus the buckling has little effect on the tension capacity. Hypothetically, a girder resists stress by "beam action" up to the point at which web buckling occurs. The wavy pattern of the web will occur at this point. Therefore, it is assumed that the shear strength (V_u) of a girder panel consists of two parts: the "beam action" shear strength (V_{cr}) and the "tension field action" shear strength (V_{ten}) (*4*).

$$V_u = V_{cr} + V_{\text{ten}}$$

Recalling that the unit stress of the tension field acts with the web buckling stress to cause yielding, the magnitude of the tension field shear force can be evaluated mathematically. Incorporating a factor of safety (1.83) with the total shear strength, the allowable shear stress in a girder web can be expressed as

$$\tau_a = \frac{\sigma_y}{3.18}\left[\frac{\tau_{cr}}{\tau_y} + \frac{1 - \dfrac{\tau_{cr}}{\tau_y}}{1.15\sqrt{1 + (d/D)^2}}\right] \qquad (5)$$

where σ_y and τ_y are the yield points of the girder material due to tension and

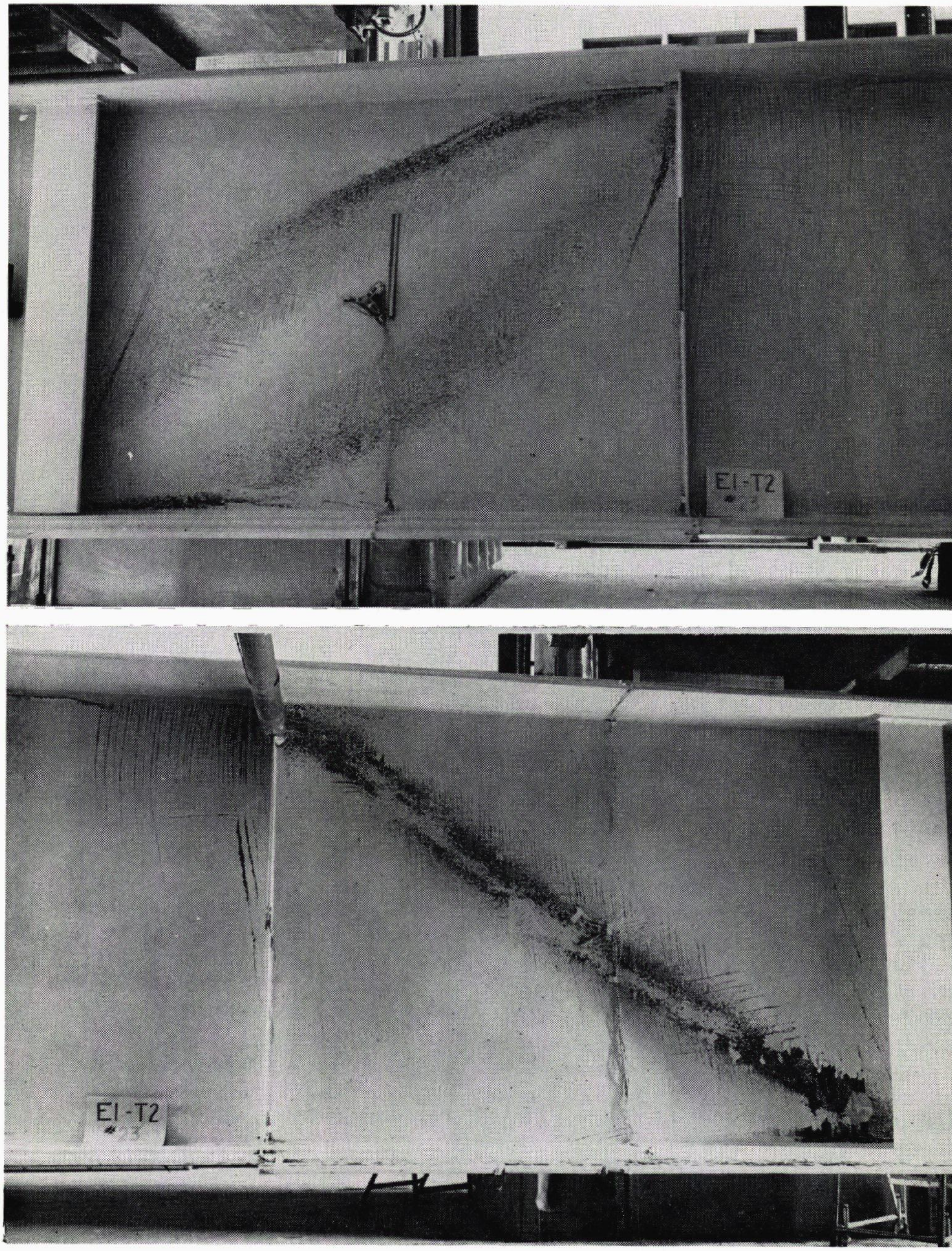

Figure 6. Web deflection.

shear, respectively; τ_{cr} is the web buckling stress; and d/D is the ratio of panel length to web depth. The first term in the bracket corresponds to beam action and the second term is the contribution of tension field action. As the numerator of the second term $(1 - \tau_{cr}/\tau_y)$ shows, tension field action starts only after beam action is developed. For stocky webs with $\tau_{cr}/\tau_y > 1$, beam action can be carried to yielding; hence no tension field action will take place. In such cases, the second term would be omitted.

Eq. 5 may seem too complicated for design purposes. For practical use, this difficulty is overcome by tabulating allowable stresses for different materials (ASTM A7, A36, etc.) and girder dimensions. Table 1 is given as an example. With these tables, it is a simple matter to find the allowable shear stress for a given girder geometry, or conversely, to find the proper stiffener spacing corresponding to a given shear stress.

Size of Intermediate Transverse Stiffeners

The vertical component of the tension field force needs an anchorage. There are only two elements that may possibly serve in such a capacity: the flanges and the transverse stiffeners. The flanges are too flexible to resist any pull in the direction of the web. Consequently, the transverse stiffeners take this responsibility, just as in a Pratt truss the vertical component of the force in a diagonal is transmitted to the neighboring struts.

As the result, the required area of a pair of stiffener plates is (*4*)

$$A_s = \frac{1}{2}\left(1 - \frac{\tau_{cr}}{\tau_y}\right)\left[\frac{d}{D} - \frac{(d/D)^2}{\sqrt{1 + (d/D)^2}}\right] Dt \qquad (6)$$

As in the expression for allowable shear stress, $1 - \tau_{cr}/\tau_y$ expresses the tension field action. The term in the brackets indicates the influence of d/D, the ratio of panel length to web depth; and Dt is the area of the web. This stiffener area requirement is derived to resist the vertical component of the tension field force and is to supplement the existing requirement which provides the necessary rigidity.

Again, the equation can be presented in tabular form for design purposes (Table 1).

Width of End Panel

The horizontal component of the tension field force is transmitted to the neighboring panels, as can be seen by the yield pattern in Figure 6. At the ends of a girder where there is no neighboring panel, there are two methods to cover this situation. The first is to provide an end plate that forms a strong end post over the support to resist the horizontal pull (Fig. 7). The second method is to

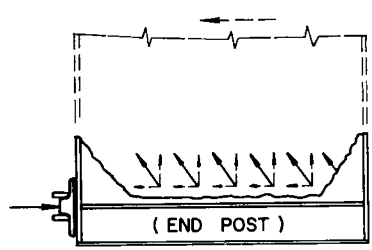

Figure 7. Detail at girder end.

eliminate tension field action in the end panel. To accomplish this, the width of the end panel should be such

TABLE I

ALLOWABLE SHEAR STRESSES IN PLATE GIRDERS (ksi)

REQUIRED GROSS AREA OF INTERMEDIATE STIFFENERS (IN PER CENT OF WEB AREA)

For Yield Point of 33 ksi

Slenderness ratios D/t : web depth to web thickness	Aspect ratios d/D : stiffener spacing to web depth													
	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3	OVER 3
60														11.0
70											11.0	11.0	11.0	11.0
80							11.0	11.0	11.0	11.0	10.8	10.5	10.4	10.0
90						11.0	10.9	10.4	10.2 *0.4*	10.0 *0.7*	10.0 *0.8*	9.7 *0.9*	9.6 *0.9*	8.9
100					11.0	10.5	10.1 *0.8*	10.0 *1.3*	9.7 *1.6*	9.5 *1.7*	9.4 *1.8*	9.0 *1.7*	8.9 *1.6*	7.6
110				11.0	10.3 *0.3*	10.1 *1.2*	9.8 *2.0*	9.5 *2.4*	9.2 *2.7*	8.9 *2.9*	8.6 *3.0*	8.1 *2.9*	7.8 *2.7*	6.2
120			11.0	10.3 *0.5*	10.0 *1.6*	9.8 *2.3*	9.4 *3.2*	8.9 *3.9*	8.5 *4.2*	8.1 *4.3*	7.9 *4.2*	7.4 *3.9*	7.1 *3.5*	5.2
130		11.0	10.4 *0.3*	10.0 *1.6*	9.8 *2.6*	9.4 *3.6*	8.9 *4.8*	8.3 *5.3*	8.0 *5.4*	7.6 *5.3*	7.3 *5.2*	6.8 *4.6*	6.4 *4.1*	4.5
140	11.0	10.7	10.1 *1.3*	9.9 *2.6*	9.5 *3.9*	9.0 *5.1*	8.4 *6.1*	8.0 *6.3*	7.5 *6.3*	7.1 *6.2*	6.9 *5.9*	6.3 *5.2*	5.9 *4.6*	3.9
150	11.0	10.3 *0.7*	10.0 *2.2*	9.6 *3.8*	9.1 *5.3*	8.7 *6.4*	8.0 *7.1*	7.6 *7.2*	7.1 *7.1*	6.8 *6.8*	6.5 *6.5*	6.0 *5.7*	5.5 *5.0*	3.3
160	10.8	10.1 *1.5*	9.9 *3.1*	9.3 *5.1*	8.9 *6.5*	8.4 *7.4*	7.8 *7.9*	7.3 *7.9*	6.9 *7.7*	6.5 *7.4*	6.2 *7.0*	5.6 *6.1*	5.2 *5.3*	
170	10.4 *0.3*	10.0 *2.3*	9.6 *4.4*	9.0 *6.3*	8.6 *7.5*	8.2 *8.2*	7.6 *8.6*	7.1 *8.5*	6.6 *8.2*	6.2 *7.8*	6.0 *7.4*	5.3 *6.4*		
180	10.2 *1.1*	9.9 *3.0*	9.3 *5.6*	8.9 *7.2*	8.4 *8.3*	8.0 *8.9*	7.4 *9.2*	6.9 *9.0*	6.4 *8.6*	6.1 *8.2*	5.8 *7.7*			
200	10.0 *2.3*	9.5 *5.2*	9.0 *7.4*	8.5 *8.7*	8.0 *9.5*	7.7 *10.0*	7.1 *10.1*	6.6 *9.8*	6.1 *9.3*	5.8 *8.8*				
220	9.8 *4.0*	9.2 *6.8*	8.7 *8.7*	8.2 *9.8*	7.4 *10.5*	7.5 *10.8*	6.9 *10.7*	6.3 *10.3*						
240	9.5 *5.5*	9.0 *8.1*	8.5 *9.7*	8.0 *10.6*	7.7 *11.2*	7.3 *11.4*	6.7 *11.2*							
260	9.3 *6.8*	8.8 *9.0*	8.3 *10.4*	8.0 *11.3*	7.5 *11.7*	7.1 *11.9*								
280	9.1 *7.7*	8.7 *9.8*	8.2 *11.1*	7.8 *11.8*	7.4 *12.2*									
300	9.0 *8.5*	8.5 *10.4*	8.1 *11.6*	7.7 *12.2*										
320	8.9 *9.2*	8.4 *10.9*	8.0 *12.0*											
340	8.8 *9.7*	8.4 *11.3*												
360	8.8 *10.1*													

that only beam action takes place. This is specified by the rule,

$$d = \frac{9{,}000\, t}{\sqrt{S}} \qquad (7)$$

INTERACTION BETWEEN BENDING AND SHEAR

At locations in a girder where both bending moment and shear force are high, tensile and shear stresses must be kept within the allowable values.

It was pointed out that a web transfers bending stress to the flange and an adjusted allowable bending stress (σ'_a) is established. As long as this stress (σ'_a) is not exceeded, the transfer does not reduce the factor of safety for the bending stress and the web carries only shear. Because the adjusted allowable bending stress is dependent on the girder geometry, a consideration of the practical range of girder geometry will help to establish a limit that will exclude the possibility of overstressing the girder.

This limit has been established (*5*) and is shown in Fig. 8. When the bending stresses are less than 75

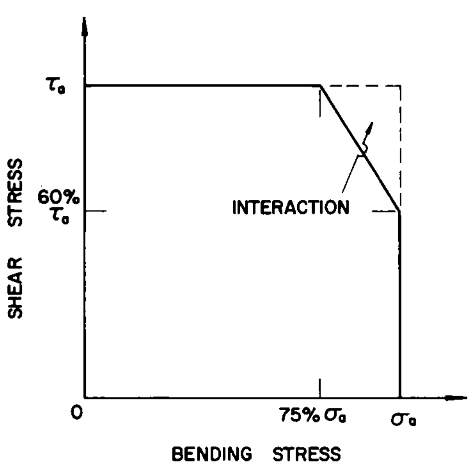

Figure 8. Allowable stresses for interaction

percent or the shear stresses are less than 60 percent of their respective allowable values, no interaction check is necessary. Otherwise, the allowable stress is given by the inclined line in the figure. For ASTM A7 steel girders, for example, this is

$$\sigma = 24.5 - 11 \frac{\tau}{\tau_a} \qquad (8)$$

Where τ is the actual average shear stress in the web.

SUMMARY

The static carrying capacity of a plate girder under bending depends essentially on the compression flange and the capacity under shear depends on the web and stiffeners. The results of the investigation are given in terms of design rules for use with highway bridges. Additional design details not considered here (such as the fastening of stiffeners to the girder) have been investigated and are discussed in the references cited. Some other important features for bridge girders (horizontal stiffeners, combination of different materials, repeated loadings) are presently being investigated.

ACKNOWLEDGMENTS

This report is based on research conducted at Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa., under the direction of Lynn S. Beedle.

The investigation was jointly sponsored by the Pennsylvania Department of Highways; the U. S. Department of Commerce, Bureau of Public Roads; the American Institute of Steel Construction; and the Welding Research Council.

The assistance of Peter B. Cooper and L. S. Beedle in preparing the manuscript is gratefully acknowledged.

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A Study of Hveem Stability *vs* Specimen Height

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This study shows that (a) the measured Hveem stability of dense-graded asphaltic concrete varies linearly with the height of specimen, (b) the Texas gyratory shear compaction method automatically controls the compactive effort so that specimens of various heights of one mix have identical densities, and (c) the strengths of mixes of "unmeasurable" stability can be approximated by a method of extrapolation.

• IN JUDICIOUS testing of asphaltic concrete, many inconsistencies in strengths of specimens from identical mixes have been noted. It was not known definitely whether these variations were brought about by molding, testing, or evaluating techniques. In an attempt to determine the source of strength differences, a study was made of the variation of density and Hveem stability with compacted specimen height.

Six dense-graded asphaltic concrete mixes were tested and evaluated in this study; the reported data for a seventh mix were incorporated for analysis. The descriptions of these mixtures are given in Table 1.

TABLE 1
DESCRIPTION OF ASPHALTIC PAVING MIXTURES

Mix	Location	Material	Max. Aggregate Size: Passing %	Max. Aggregate Size: Passing Sieve	Max. Aggregate Size: Retained (sieve)	Penetration Grade Asphalt
1	THD Dist. 17, Texas 6, Brian	Siliceous gravel, limestone screenings, field sand	1.4	⅝-in.	½-in.	OA-90
2	Leon Co., US75	Iron ore gravel	0.7	⅝-in.	½-in.	OA-90
3	THD Dist. 17, Walker Co.	Siliceous gravel, E. Texas stone screenings, field sand	0.5	⅝-in.	½-in.	OA-90
4 [1]	City of College Station, Walton Dr.	Slag aggregate, limestone screenings	0.9	¼-in.	No. 4	OA-90
5 [1]	City of College Station, Walton Dr.	Slag aggregate, limestone screenings	0.9	¼-in.	No. 4	OA-90
6 [1]	THD Dist. 17, Milam Co., US77	Siliceous gravel, limestone screenings, field sand	—	⅝-in.	½-in.	OA-90
7 [2]	—	—	—	—	—	—

[1] Modified by increasing asphalt content.
[2] Data from THD investigational project 32; design mix $C\text{–}A_{40}\text{–}R_2\text{–}12\text{–}3$.

PREPARATION AND TESTING

The asphaltic concrete mixtures tested were obtained from construction projects in District 17 of the Texas Highway Department and in the city of College Station. All specimens composed of a given mix and covering the range of heights under study were molded during the same afternoon. This was done in order to minimize any changes in stability of the mixtures that might result from the aggregate absorption of the asphalt if allowed to "age" for a varying period of time.

The method of forming the specimens was by the standard manual method of the Texas Highway Department, using the gyratory shear compactor (*1, 2, 3*). Because experience has shown that, in a manual molding method, operator technique is a variable, one man molded all of the specimens for the six mixes tested.

Six specimens 4.00 in. in diameter were molded for each of the five heights which ranged from 1.5 to 2.5 in. The specimens were molded at a temperature of 250 F and measured for height after cooling to room temperature. The ±0.06-in. tolerance in height for a standard test specimen 2.00 in. high was observed for all sets of specimens.

The density of the specimens was determined at room temperature by weighing the specimen in air and in water. No coatings to prevent entrance of water into specimens were used because the surfaces were smooth and essentially watertight.

Testing of the specimens for strength involved the use of the Hveem stabilometer and the method employed by Texas (*4*). Adjustment of air in the stabilometer system for calibration was made so that "initial displacement" was obtained when a pump piston movement of 0.070 to 0.080 in. increased the lateral pres-

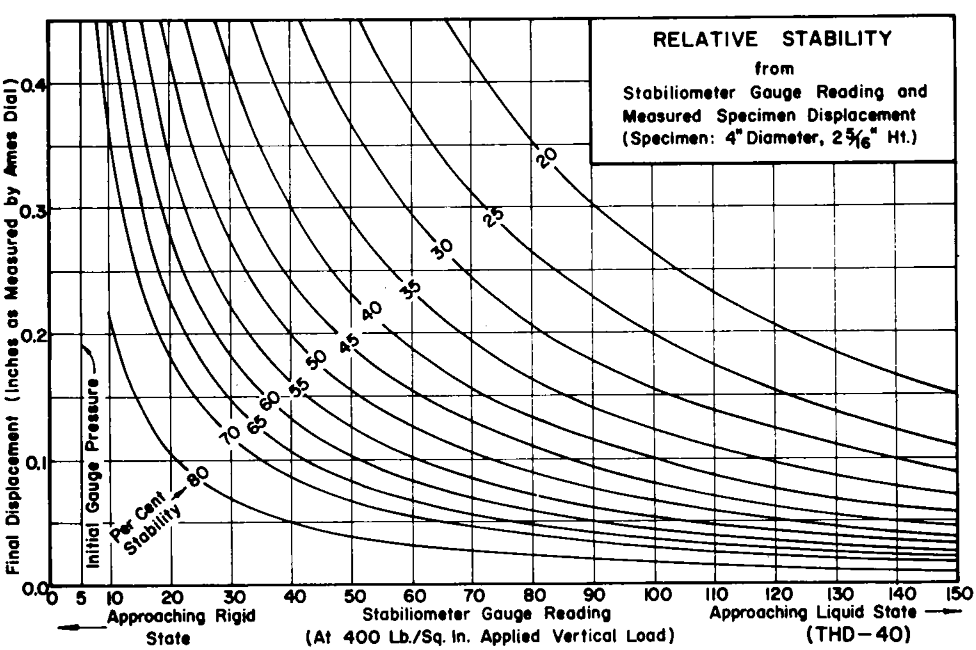

Figure 1.

sure in the stabilometer from 5 to 100 psi and with $1^{13}/_{16}$-in. length of the diaphragm in contact with the dummy.

Before testing, the specimens were placed in a 140 F oven for a period of 4 hr. In testing, load was applied at a rate causing a platen movement of 0.05 in. per min. The maximum applied load was 6,000 lb except in cases where the transmitted pressure reached 200 psi before this maximum was obtained. A record was made of transmitted pressures corresponding to vertical loads. When possible, the stability was obtained by entering the curves in Figure 1 with the values of transmitted pressure at the vertical load of 5,000 lb and the "final displacement." The so-called "corrected stability" for speciment height was obtained by use of the curves in Figure 2. The corrected stability for a specimen was determined by use of both the specimen height and the effective height. The effective height is the height of specimen minus the amount ($^{3}/_{16}$ in.) that bears against a steel diaphragm clamp in the stabilometer.

RESULTS

Density

The compactive effort imparted by the gyratory shear compactor is indeterminate and variable for different mixes, although reasonably constant for specimens of the same height and of the same mix. It is strongly believed that the basic principles of this method for compaction of bituminous mixtures are the best for simulating field compaction economically in the laboratory. Refinements to this compaction method have been made and reported by McRae (*5*) and others. Contrary to common belief, extreme physical effort is not required in this molding procedure. Great physical exertion is evidence of improper molding and a fair warning to the operator to look for defects in his procedure.

Table 2 shows the variation in specimen densities evolving from differences in height and composition. It can be seen from these data and their analyses that density variations are due to differences in mixes and not to specimen height. The reproducibility of density is excellent and meets the normal tolerance range of 0.02 g per cc. This finding appears to be a natural outcome of the compaction method, because compactive effort is applied to the mix until it reaches a certain resistance to a particular load. Undoubtedly, this resistance comes predominantly from internal friction and not from direct bearing on stones, and for this reason the maximum particle size of the aggregate must be limited. It appears that a limited amount of $^{5}/_{8}$- to $^{1}/_{2}$-in. aggregate did not prevent densification of the $1^{1}/_{2}$-in. high specimens of this study.

Stability

Much has been written concerning measurements obtained with the Hveem stabilometer and factors affecting the value of stability computed with the empirical equation applicable to the stabilometer (*6, 7, 8, 9*). A limited study was made of the computed stability at vertical loads other than 5,000 lb. From the research point of view, it is of interest to obtain a value of Hveem stability for specimens that transmit 200 psi (the limit of the standard gage) at vertical loads below 5,000 lb; *i.e.*, the specimens are of "unmeasurable" stability under the standard form. Apparent solutions to the problem are (a) to plot transmitted pressure vs vertical load and extrapolate the curve to obtain the transmitted pressure at 5,000-lb vertical load, or (b) to compute the stability for corresponding vertical and lateral pressures and plot these stabilities vs

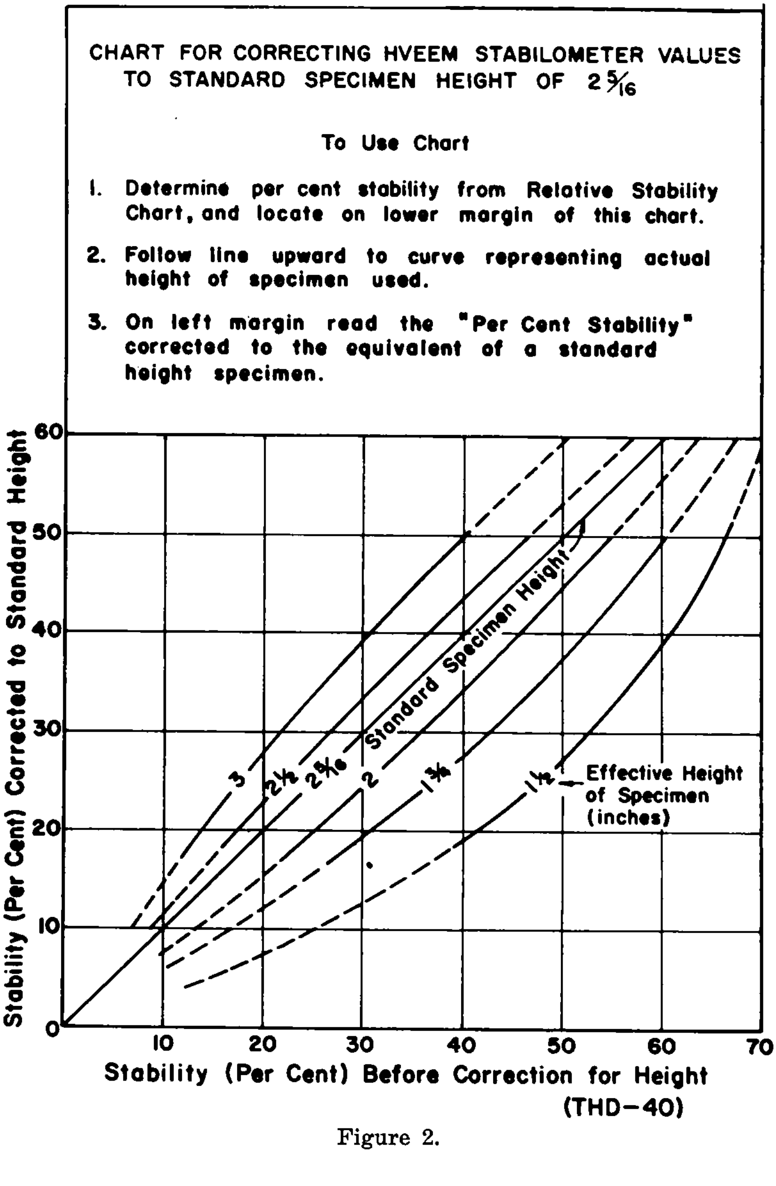

Figure 2.

vertical loads and then extrapolate this curve to obtain the stability at 5,000-lb load. The former method is less time-consuming, but the latter method is much more informative.

Figure 3 plots computed stability vs vertical load at various stages of loading for different mixes (all curves not part of the present study). These curves have three characteristic parts: (a) the high rate of increase in stability during initial loading, (b) the peaking or leveling-off in stability, and (c) a gentle rate of decrease in stability during the final stage of loading. The values of stability computed for the various stages of loading were found using one value

TABLE 2

RECORD OF DATA ON SPECIMEN HEIGHT, DENSITY, AND MEASURED STABILITY ON VARIOUS MIXES[1]

	Height (in.)	Density (gm/cc)	Stability (%)	Height (in.)	Density (gm/cc)	Stability (%)	Height (in.)	Density (gm/cc)	Stability (%)
				(*a*) Mix No. 1					
	1.49	2.387	60.0	1.66	2.407	55.0	2.00	2.385	48.5
	1.51	2.386	62.5	1.71	2.394	60.0	1.96	2.388	57.5
	1.48	2.408	62.5	1.70	2.391	52.0	2.00	2.394	56.0
	1.50	2.389	64.0	1.64	2.405	59.0	2.00	2.371	50.5
	1.45	2.401	61.0	1.71	2.394	53.0	2.02	2.384	50.5
$\bar{x}$	1.49	2.394	62.0	1.68	2.398	55.8	2.00	2.398	52.6
S		0.009	1.5		0.007	3.6		0.008	3.9
$S_{\bar{x}}$		0.004	0.7		0.003	1.6		0.004	1.8
	2.18	2.387	46.0	2.53	2.395	43.0[2]	(avg)	(avg)	
	2.20	2.384	44.5	2.46	2.389	47.0[2]	1.49	2.394	
	2.21	2.377	45.0	2.47	2.381	47.5[2]	1.68	2.398	
	2.18	2.381	46.5	2.47	2.383	45.5[2]	2.00	2.398	
	2.26	2.373	42.5	2.52	2.368	46.5[2]	2.20	2.380	
	2.17	2.378	43.0	2.45	2.371	49.5[2]	2.48	2.381	
$\bar{x}$	2.20	2.380	44.6	2.48	2.381	46.5		2.390	
S		0.005	1.6		0.010	2.1		0.009	
$S_{\bar{x}}$		0.002	0.6		0.004	0.8		0.004	
				(*b*) Mix No. 2					
	1.53	2.580	53.5	1.75	2.589	43.0	2.00	2.547	47.0
	1.53	2.562	57.5	1.74	2.594	50.0	2.04	2.555	49.0
	1.48	2.619	N.D.	1.76	2.587	54.0	2.01	2.584	47.0
	1.55	2.546	56.5	1.77	2.557	53.0	1.99	2.582	56.5
	1.54	2.545	55.0	1.77	2.563	53.0	1.98	2.596	49.0
	1.51	2.561	55.0	1.81	2.559	50.5	2.08	2.558	56.0
$\bar{x}$	1.52	2.569	55.5	1.77	2.575	50.6	2.02	2.570	47.4
S		0.028	1.4		0.017	4.0		0.019	1.3
$S_{\bar{x}}$		0.012	0.6		0.007	1.6		0.008	0.6
	2.21	2.558	42.0	2.40	2.587	33.5	(avg)	(avg)	
	2.20	2.565	42.0	2.40	2.583	31.5	1.52	2.569	
	2.20	2.575	40.0	2.42	2.585	37.5	1.77	2.575	
	2.24	2.553	41.0	2.43	2.568	40.0	2.02	2.570	
	2.21	2.576	46.0	2.44	2.570	46.0	2.21	2.570	
	2.19	2.596	46.0	2.45	2.554	36.5	2.42	2.574	
$\bar{x}$	2.21	2.570	42.8	2.42	2.574	37.5		2.572	
S		0.015	2.6		0.019	5.1		0.003	
$S_{\bar{x}}$		0.006	1.0		0.008	2.1		0.001	
				(*c*) Mix No. 3					
	1.50	2.352	57.0	1.75	2.362	54.0	1.97	2.354	46.0
	1.51	2.341	55.5	1.75	2.352	52.5	1.98	2.334	43.0
	1.51	2.337	56.5	1.76	2.348	49.0	1.98	2.351	47.0
	1.51	2.351	54.0	1.76	2.349	50.0	2.00	2.338	47.0
	1.51	2.351	59.5	1.76	2.350	50.5	2.00	2.345	48.0
	1.53	2.318	51.0	1.76	2.345	50.0	2.01	2.331	45.0
$\bar{x}$	1.51	2.342	55.6	1.76	2.351	51.0	1.99	2.342	46.0
S		0.013	2.9		0.006	1.9		0.009	1.8
$S_{\bar{x}}$		0.005	1.2		0.002	0.8		0.003	0.7
	2.17	2.356	45.0	2.48	2.360	39.0	(avg)	(avg)	
	2.18	2.351	40.5	2.49	2.352	38.5	1.51	2.342	
	2.18	2.351	43.5	2.50	2.339	35.0	1.76	2.351	
	2.19	2.340	41.5	2.50	2.344	26.5	1.99	2.342	
	2.19	2.339	40.0	2.50	2.335	36.0	2.18	2.346	
	2.20	2.338	39.0	2.50	2.342	33.5	2.50	2.345	
$\bar{x}$	2.18	2.346	41.6	2.50	2.345	34.8		2.345	
S		0.008	2.3		0.009	4.5		0.004	
$S_{\bar{x}}$		0.003	0.9		0.003	1.9		0.002	

TABLE 2 (Continued)

RECORD OF DATA ON SPECIMEN HEIGHT, DENSITY, AND MEASURED STABILITY ON VARIOUS MIXES [1]

	Height (in.)	Density (gm/cc)	Stability (%)	Height (in.)	Density (gm/cc)	Stability (%)	Height (in.)	Density (gm/cc)	Stability (%)
				(*d*) Mix No. 4					
	1.46	2.307	46.0	1.66	2.298	41.5	1.94	2.293	39.5
	1.46	2.310	41.5	1.66	2.285	40.5	1.95	2.288	39.0
	1.46	2.294	50.0	1.67	2.284	43.5	1.95	2.278	38.5
	1.46	2.307	49.5	1.67	2.280	43.0	1.95	2.287	40.5
	1.47	2.295	49.5	1.67	2.294	40.5	1.96	2.272	39.5
	1.47	2.301	51.5	1.70	2.208	35.5	1.97	2.267	38.5
$\bar{x}$	1.46	2.302	49.7	1.67	2.275	40.8	1.95	2.281	39.2
S		0.007	2.0		0.033	2.8		0.010	0.8
$S_{\bar{x}}$		0.002	0.8		0.013	1.2		0.004	0.3
	2.14	2.300	29.0	2.42	2.298	27.5	(avg)	(avg)	
	2.14	2.289	31.0	2.42	2.298	30.0	1.46	2.302	
	2.15	2.287	32.0	2.43	2.300	27.5	1.67	2.275	
	2.15	2.279	31.0	2.45	2.283	27.5	1.95	2.281	
	2.15	2.282	32.0	2.45	2.290	27.5	2.15	2.285	
	2.16	2.275	33.0	2.45	2.286	27.5	2.44	2.292	
$\bar{x}$	2.15	2.285	31.3	2.44	2.292	27.9		2.287	
S		0.009	1.4		0.007	1.0		0.010	
$S_{\bar{x}}$		0.004	0.6		0.003	0.4		0.004	
				(*e*) Mix No. 5					
	1.44	2.344	43.5	1.70	2.348	33.0	1.92	2.342	25.5
	1.44	2.349	37.5	1.71	2.345	37.0	1.93	2.344	25.5
	1.45	2.341	37.0	1.70	2.353	36.5	1.93	2.347	26.0
	1.46	2.343	35.0	1.70	2.347	38.0	1.93	2.343	36.0
	1.45	2.346	37.0	1.70	2.346	34.5	1.93	2.345	30.5
	1.45	2.348	35.0	1.69	2.351	32.5	1.92	2.349	31.0
$\bar{x}$	1.45	2.345	37.5	1.70	2.348	35.2	1.93	2.345	29.1
S		0.003	3.1		0.003	2.3		0.002	4.2
$S_{\bar{x}}$		0.001	1.3		0.001	0.9		0.001	1.7
	2.19	2.343	26.0	2.48	2.343	19.5	(avg)	(avg)	
	2.20	2.341	27.0	2.48	2.338	21.5	1.45	2.345	
	2.19	2.343	29.0	2.46	2.349	18.5	1.70	2.348	
	2.19	2.345	20.0	2.47	2.339	16.0	1.93	2.345	
	2.17	2.346	23.0	2.49	2.341	20.0	2.19	2.343	
	2.21	2.341	26.5	2.48	2.341	20.5	2.48	2.342	
$\bar{x}$	2.19	2.343	25.2	2.48	2.342	19.3		2.345	
S		0.002	3.2		0.004	1.9		0.002	
$S_{\bar{x}}$		0.001	1.3		0.001	0.8		0.001	
				(*f*) Mix No. 6					
	1.50	2.396	N.D.	1.72	2.407	14.5	1.99	2.400	12.5
	1.50	2.392	26.0	1.72	2.406	16.0	2.00	2.399	14.5
	1.50	2.394	22.0	1.73	2.405	13.5	2.00	2.397	14.0
	1.51	2.387	28.0	1.73	2.400	17.0	2.01	2.399	17.0
	1.51	2.396	25.0	1.73	2.401	19.0	2.01	2.399	14.0
	1.51	2.396	26.5	1.74	2.397	16.5	2.01	2.442	17.0
$\bar{x}$	1.50	2.394	25.5	1.73	2.403	16.1	2.00	2.406	14.8
S		0.004	2.23		0.004	1.9		0.017	1.8
$S_{\bar{x}}$		0.001	1.0		0.002	0.8		0.007	0.7
	2.17	2.399	9.0	2.47	2.406	6.0	(avg)	(avg)	
	2.18	2.399	11.0	2.48	2.399	4.0	1.50	2.394	
	2.19	2.404	10.0	2.48	2.401	5.0	1.73	2.403	
	2.20	2.396	9.5	2.48	2.403	6.5	2.00	2.406	
	2.21	2.397	6.5	2.49	2.398	6.5	2.20	2.399	
	2.22	2.396	9.0	2.49	2.403	7.0	2.48	2.402	
$\bar{x}$	2.20	2.399	9.2	2.48	2.402	5.8		2.401	
S		0.003	1.5		0.003	1.1		0.004	
$S_{\bar{x}}$		0.001	0.6		0.001	0.4		0.002	

TABLE 2 (Continued)

RECORD OF DATA ON SPECIMEN HEIGHT, DENSITY, AND MEASURED STABILITY ON VARIOUS MIXES[1]

Height (in.)	Density (gm/cc)	Stability (%)	Height (in.)	Density (gm/cc)	Stability (%)	Height (in.)	Density (gm/cc)	Stability (%)
				(g) Mix No. 7[3]				
	61.0		64.0		49.0		54.0	
	59.0		54.0		49.0		53.0	
	58.0		56.0		53.0		50.0	
$\bar{x}$ 1.55	59.3	1.78	58.0	2.00	49.3	2.24	52.3	
	39.0							
	42.0							
	36.0							
$\bar{x}$ 2.47	39.0							

[1] $\bar{x}$ = mean; S = standard deviation; $S_{\bar{x}}$ = standard error.
[2] Stabilometer pedestal set in error of —0.25 in.; therefore, stability data should be for that of average specimen height of 2.23 in., but this set was not used in regression analysis.
[3] No data on individual height or density.

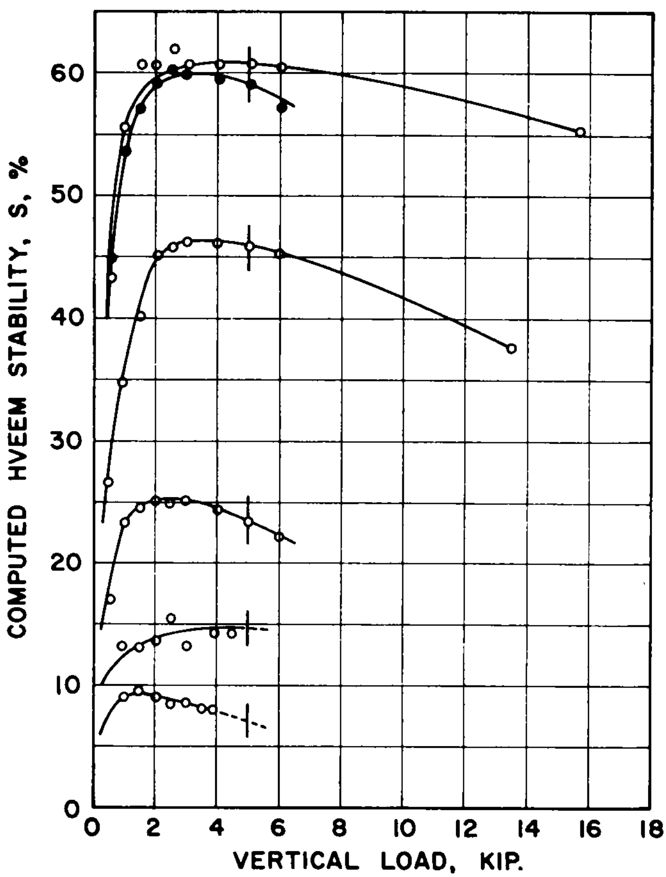

Figure 3. Computed stability *vs* vertical load, showing typical form.

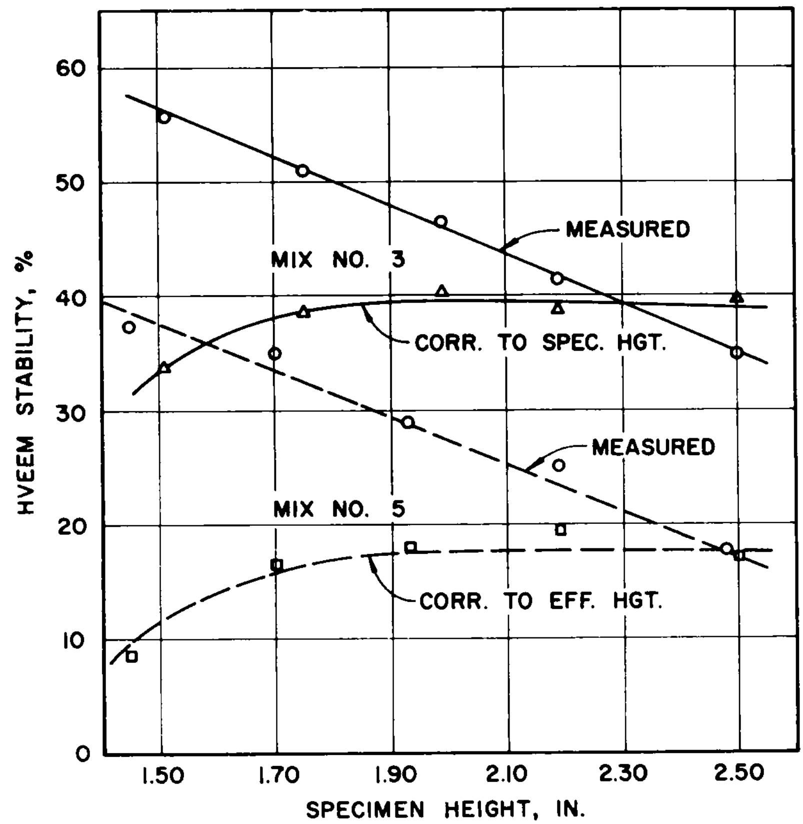

Figure 4. Relationship between Hveem stability and specimen height.

for final displacement, determined after the application of maximum load. The validity in using this one value of final displacement is assumed from the fact that similar specimens yield similar stabilities whether stressed to a maximum vertical load of 6,000 lb or to a maximum lateral pressure of 200 psi (a former method of the Texas Highway Department) which may correspond to a vertical load exceeding 14,000 lb. Also, specimens measured for final displacement after being loaded to 1,500, 2,000 or 2,500 lb have shown this displacement value equivalent to that found on similar specimens tested normally for stability.

The particular point emphasized here for the evaluation of Hveem stability of weak bituminous mixes is that the determination should be made by the extrapolation of the stability vs vertical load curve with particular attention paid to which section of the three parts of the typical curve is being extrapolated. In the height-stability study of this investigation, all stabilities below 22 percent were determined by this method of extrapolation. It is not implied that some stabilities less than 22 percent cannot be obtained by direct use of Figure 1.

Variabilities in stability of seemingly identical specimens, except for height, may arise from the use of height-correction curves similar to those of Figure 2. The use of any correction curves is a necessity in order to reduce the stability of all tested specimens to the strength of a specimen of standard height. This allows the specification of a minimum strength for any mix. Figure 2 shows the Hveem height-correction curves for stability available in 1945 and still used with a slight modification in 1962 by Texas. This modification is that the curve is entered with the height of specimen instead of the called-for effective height which is the height of specimen minus $\frac{3}{16}$ in.; naturally, the correction for stability due to specimen height for these two methods differs because essentially the two methods are using a different standard height for comparison of stability. The standard California specimen for stability comparison is $2\frac{1}{2}$ in. high and that for Texas is $2\frac{5}{16}$ in. high. This difference results in higher values for corrected stabilities by the Texas method.

Examination of the height-correction curves shows two outstanding relationships: (a) for a particular height of specimen, the correction for stability varies with the measured stability, except for the standard specimen height curve, and (b) the correction for stability varies for each specimen height curve at a set value of measured stability. These slopes of tangents to the specimen height curves were of interest in the present study. An investigation by the Texas Highway Department (*10*) showed by an ingenious method that the most used portions of the correction curves of Figure 2 are essentially arcs of circles whose centers are approximately colinear. From this investigation additional specimen height correction curves were developed to include specimen heights of 1.0 in.

Table 2 gives the measured Hveem stabilities for the specimens of the various mixes investigated and also analysis of the data. Incompleteness of sets of data is due to damage to specimens or to improper testing. Reproduction of specimens was limited to the amount of material available. The data for Mix 7 were obtained from an unpublished report (*10*). Comparisons were made on the basis of average height of specimens for a set rather than an individual height. The study was made on this basis because measured stabilities showed as great a range for identical heights (Mix 5, $\bar{H}=1.70$ in.) as for specimen heights having a height range of 0.09 in. (Mix 1, $\bar{H}=2.20$ in.).

Figure 4 plots the relationship between stability and specimen height for two mixes. In the curves, (a) a linear relationship between measured stability and height of specimen is apparent, (b) the two curves of measured stability for Mixes 3 and 5 are approximately parallel, and (c) the curves of corrected stabilities are nearly horizontal (as they should be) between specimen heights of 2.00 and 2.50 in., but fail to maintain this slope for heights less than 2.00 in.

The statistical study (Table 3) was made to substantiate these observations. The analysis of data from this table indicates a linearity for variation of measured stability with respect to specimen height and that the variations of stability with specimen height are approximately identical for the different mixes. The weighted average slope of the seven mixes is —20.1, and the family of curves for stability S vs height H within the limits of the experimental data, may be expressed as $S=-20.1\,H+B$, where B is the intercept for a particular curve or mix.

A variation of the curves of Figure 2 has been obtained from Table 3 and is shown in Figure 5. The term "adjusted" is used in preference to "corrected," because it may be desirable to obtain the stability of a mix at a

TABLE 3
DATA FOR SLOPE DETERMINATION OF HEIGHT vs MEASURED STABILITY AND RELIABILITY OF AN AVERAGE SLOPE[1]

	Mix No. 1		Mix No. 2		Mix No. 3		Mix No. 4	
	X	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*
	1.49	62.0	1.52	55.5	1.51	55.6	1.46	49.7
	1.68	55.8	1.77	50.6	1.76	51.0	1.67	40.8
	2.00	52.6	2.02	47.4	1.99	46.0	1.95	39.2
	2.20	44.6	2.21	42.8	2.18	41.6	2.15	31.3
	—	—	2.42	37.5	2.50	34.8	2.44	27.9
b	− 22.1		− 19.4		− 21.2		− 21.7	
FL_b	± 16.7		± 4.4		± 0.4		± 9.2	
$t_{(b-b_0)}$	0.514		0.99		1.00		0.97	
r	0.97		0.511		8.462[2]		0.556	

Mix No. 5		Mix No. 6		Mix No. 7		Mix	(Σxy)	(Σx^2)
X	*Y*	*X*	*Y*	*X*	*Y*			
						1	− 6.70	0.303
						2	− 9.78	0.503
1.45	37.5	1.50	25.5	1.55	59.3	3	− 12.31	0.579
1.70	35.2	1.73	16.1	1.78	58.0	4	− 12.82	0.591
1.93	29.1	2.00	14.8	2.00	49.3	5	− 11.85	0.651
2.19	25.2	2.20	9.2	2.24	52.3	6	− 11.18	0.591
2.48	19.3	2.48	5.8	2.47	39.0	7	− 10.62	0.529
							− 75.26	3.747
− 18.2		− 18.9	− 20.1	− 20.1				
± 4.0		± 9.4		± 18.1				
1.496		0.404		0			$b_0 = \frac{-75.26}{3.747} = -20.1$	
0.99		0.96		0.90				

[1] b = slope; FL_b = 95 percent confidence limits; X = height; Y = measured stability.
[2] Significant difference.

specimen height other than the standard height. The adjusted stability for specimen height may also be determined from the following:

$$S_A = S_M - 20.1\ (H_D - H_M)$$

in which

S_A = adjusted stability, percent;
S_M = measured stability, percent;
H_D = desired height, in.; and
H_M = measured height, in.

The measured values of S and H are entered from the left ordinate and the abscissa; this point is then extended parallel to the guide lines to intersect the desired H line; the adjusted stability is found on the right ordinate by projecting the second point horizontally.

The differences between correction curves of Figures 2 and 5 may be attributed to differences in molding test specimens and calibration of the Hveem stabilometer.

CONCLUSIONS

From the data presented, the following conclusions are indicated by the study.

1. The Texas method of compacting asphaltic concrete specimens eliminates variabilities between specimen density and specimen height that may arise in other compaction methods that employ a constant compaction effort regardless of mixture or specimen height desired. The reproducibility in density of specimens compacted by one operator is extremely good.

2. The strength of weak mixes

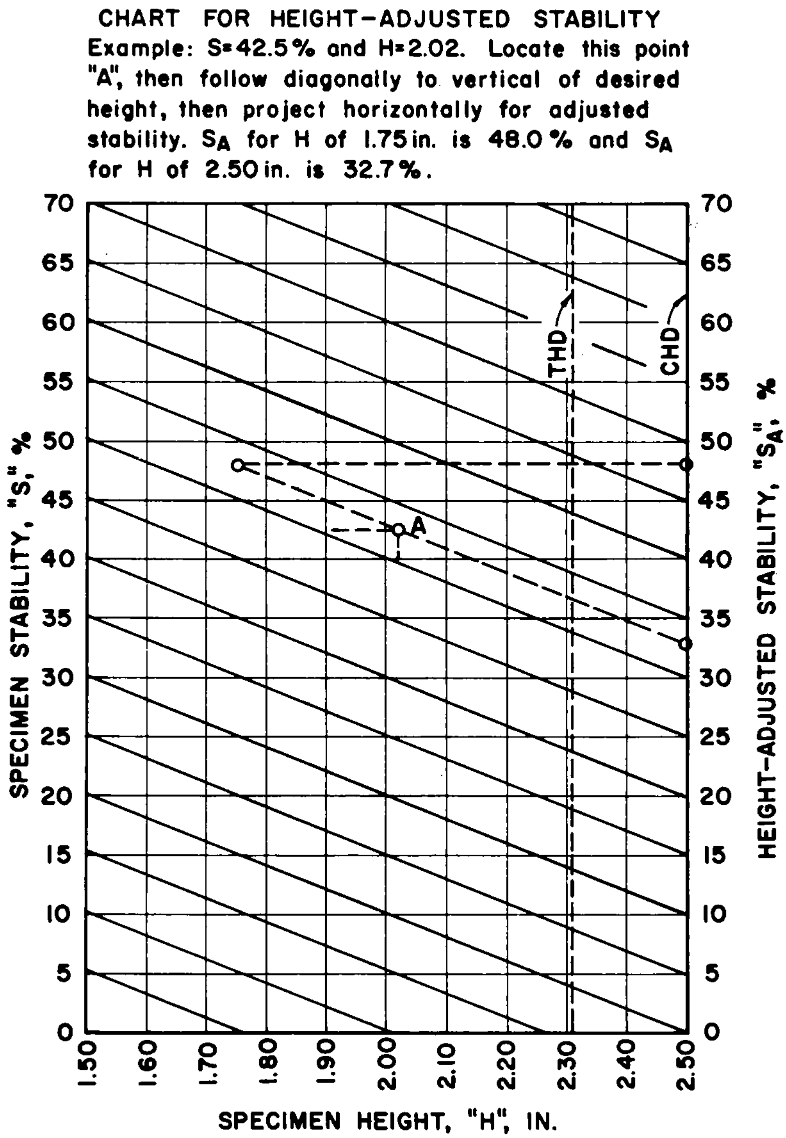

Figure 5.

("unmeasurable" stability or normally below 20 percent) can be approximated by the method suggested.

3. The relationship between measured Hveem stability and specimen height is linear and at a slope of —20.1.

ACKNOWLEDGMENT

The data for this report were obtained as part of a research project for the Texas Highway Department in joint cooperation with the Bureau of Public Roads.

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Cationic Mixing-Grade Asphalt Emulsions

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Cationic asphalt emulsions were introduced in the United States in 1958 for seal coat construction. Their advantages over anionic emulsions gained rapid recognition in the highway construction industry. Recently, mixing-grade cationic asphalt emulsions were made available. The unique properties of these new materials are expected to increase greatly the versatility and usefulness of mixing-grade emulsions in highway base and wearing course construction.

The fundamental physical and chemical properties of aggregate-asphalt emulsion systems are discussed in relation to mixing performance. The advantages in adhesion, dehydration rate, and development of cohesion obtained from cationic emulsifiers in such systems are presented.

Definite field performance advantages of cationic emulsions over anionic emulsions and liquid asphalts include greater permissible variations in aggregate moisture content and mixing temperature and, the ability to compact without excessive reduction of moisture content. These benefits are obtained in all types of emulsion construction and maintenance work. Recommended specifications for two types of cationic mixing-grade emulsions are presented for the first time.

• BEFORE 1958, essentially all asphalt emulsions used for road surfacing and construction in the United States were of the anionic type. The most important characteristic of anionic emulsions is that the emulsified asphalt particles carry negative electrical charges. These charges arise from the ionization of surface active organic acid salts at the asphalt-water interface. The organic acids may occur naturally in the asphalt or may be added as auxiliary emulsifiers. The various types of anionic emulsifiers have been adequately described (*1*).

Cationic asphalt emulsions are the electrical opposites of anionic emulsions. The emulsified asphalt particles in cationic emulsions are positively charged. Cationic emulsifiers never occur naturally and must always be added, either to the asphalt or to the water component, before emulsification. Most cationic emulsifiers are acid salts of organic amines or organic quaternary ammonium compounds. Typical cationic emulsifiers suitable for emulsifying asphalt have been reported by Dybalski (*2*) and in the patent literature (*3*).

The importance of emulsion particle electrical charge for the adhesion of emulsified asphalt to mineral aggregates has been emphasized by several investigators (*4, 5, 6*). Most aggregates are either siliceous or contain a large proportion of silicatetype minerals (*7, 8*) and, hence, become predominantly negatively charged when moist (*4, 5*). Cationic emulsions, therefore, adhere considerably better to the majority of construction aggregates than do anionic emulsions (*9, 10*).

Early in 1958, cationic emulsions, designated as the RS-K grades, were introduced for seal coat construction in the United States. In Europe, the cationics have been used to a minor extent since the early 1950's (*11*). The field performance of the RS-K emulsions has fulfilled the expectations predicted from their chemical nature (*12, 13*).

Last year cationic mixing-grade asphalt emulsions were introduced. The benefits of the cationic-type emulsion were thereby extended to a wide variety of highway construction practices. This paper describes field and laboratory data which indicate promise for these new materials in sand, dense-graded, and coarse aggregate mixing.

FACTORS CONTROLLING MIXING PERFORMANCE

The most important characteristics of an asphalt emulsion-aggregate mixture begin to develop when the emulsion starts to set or coalesce. Thus, mixing stability, rate of set or cure, degree and performance of aggregate coating all depend on the relative magnitude and rate of change of two forces that begin operating at that time. These are the force of cohesive strength in the asphalt or coalesced emulsion phase and the adhesive force at the asphalt-aggregate interface. If the emulsion breaks too rapidly and either the cohesive or adhesive forces are very large, the mix will lack stability or mixability. When the cohesive forces are greater than the adhesive forces, the mechanical stresses of mixing cause the asphalt or coalesced emulsion phase to assume a minimum surface area and ball up. Thus, if, during the process of mixing, the forces become overbalanced in favor of cohesion, asphalt adhering to the aggregate will be stripped away. Stripping failures of this type can be demonstrated easily by vigorous laboratory mixing tests on certain aggregate-emulsion systems. Therefore, in the initial phases of mixing, it is desirable that the adhesive forces be relatively strong and the cohesive forces remain weak. However, when the mix is placed, the cohesive forces must increase at a fairly rapid rate so that the pavement will cure within a satisfactory time.

Emulsion coalescence or rate of set is mainly caused by two phenomena: attractive forces between the asphalt emulsion droplets and aggregate surface, and the transfer of the emulsifier from the emulsion droplets to the aggregate surface. In the first case, emulsion droplets in Brownian motion strike the aggregate surface and adhere because of ionic forces of attraction. In the second case, emulsifier molecules desorb from the asphalt emulsion droplets to the aqueous phase and to the aggregate surface (*14*). If the capacity of the aggregate surface for adsorption is large, a sufficient amount of emulsifier may be transferred to destroy the stability of the emulsion. Both emulsion coalescence mechanisms are highly dependent on the type of emulsifier and its concentration. The surface area and chemical composition of the aggregate are also important. Generally, the larger the aggregate surface area, the greater the difficulty of obtaining sufficient emulsion mixing stability. Thus, the porosity of aggregates and percentage of fines influence the mixing performance with asphalt emulsions.

The correlation between aggregate surface area and the ease of coating several different sands with a cationic asphalt emulsion is given in Table 1,

TABLE 1

SURFACE AREA, ELEMENTAL COMPOSITION AND EASE OF COATING WITH SM-K CATIONIC ASPHALT EMULSION FOR SEVERAL SANDS

Sand	Surface Area (m^2/g)	Major Metallic Constituents (% by wt.)						Ease of Coating
		Na	Mg	Ca	Ti	Fe	Al	
Arizona dune	4.0	3.21	0.48	0.70	0.16	1.99	2.98	Difficult
Ann Arundel	1.2	0.19	0.02	0.08	0.09	0.35	0.47	Difficult
Del Monte	0.21	3.01	0.02	Nil	Nil	0.08	3.11	Easy
Mississippi River	0.03	2.95	0.35	0.35	0.07	1.41	2.84	Easy
Ottawa reference	0.01	0.24	0.02	Nil	Nil	0.14	0.02	Easy

which also gives the amounts of major metallic constituents present in these sands as determined by spectrographic analysis. Surface area rather than variations in chemical composition appears to determine the ease of coating with the emulsion. The surface areas were determined by the nitrogen adsorption method of Brunauer, Emmett, and Teller (*15*) after the sands were dried to remove moisture.

Cationic asphalt emulsions adhere well to a wider variety of aggregates of different chemical composition than do anionic emulsions. Adhesion results with anionic emulsions are often poor with siliceous aggregates and with some "mixed-type" aggregates; best results are obtained when using highly calcareous or dolomitic aggregates. Cationic emulsions, on the other hand, adhere satisfactorily to all but a few aggregates; *e.g.*, dolomitic materials (*10*).

Differences in electrophoretic mobilities may explain the differences in the adhesion of anionic and cationic emulsions with "mixed aggregates." Electrophoretic mobilities determined (in unpublished studies by the American Bitumuls and Asphalt Company) for cationic RS-grade emulsions were up to twice as large as for anionic RS-grade emulsions.

Asphalt emulsion adhesion to clay mineral surfaces is particularly important in base stabilization. Aggregates available at the job site for road base construction often contain clay. The swelling tendency of most clays has, in the past, precluded the use of such aggregates with anionic emulsions. Frequently, suitable aggregates had to be transported long distances to the site. Trouble-causing clays are usually of the montmorillonite type that consists of laminar crystal structures weakly bonded together. Water easily enters the interlaminar spaces and causes swelling (*16*). Cationic mixing grade emulsions are shown in a later section to be effective agents for stabilizing such aggregates. Presumably, this is because water is prevented from entering the interlaminar spacings.

The nature of the asphalt itself is also an important factor in the adhesive-cohesive force balance. The viscosity of the asphalt has considerable influence on the mix cohesive strength. The softer the asphalt, the less difficulty is encountered in coating aggregates. However, ultimate strength requirements sharply limit the permissible variations of asphalt viscosity.

The cohesive properties of the asphalt or coalesced emulsion phase can, fortunately, be regulated easily by a simple method. Inclusion of small quantities of naphtha cutter stock in the emulsion formula increases mixing stability and improves aggregate coating. After the mix is compacted, this relatively minor amount of cutter stock evaporates readily to give a rapid cure. The considerably larger amount of cutter stock present in cutback asphalts is vaporized only slowly and less completely under comparable conditions (*17*).

Water is easily dispersed in the asphalt phase of emulsion-aggregate mixtures and, like cutter stock, also reduces the cohesive strength. Water is, however, sometimes quite difficult to evaporate completely from the final mix. This is particularly noticeable in cool, moist weather. In some instances, extensive blading operations are needed to dehydrate anionic paving mixes satisfactorily before they can be compacted. Cationic emulsions dehydrate and cure at considerably faster rates than do anionics. This is an important advantage when aggregates containing variable and excessive amounts of moisture must be used. Cationic emulsion performance is, therefore, less sensitive to ambient temperature extremes during construction.

MIXING-GRADE CLASSIFICATION

The two types of cationic mixing-grade emulsions described in this paper are designated SM-K and SS-K. The SM-K type was named to indicate its principal use, which is sand mixing; graded aggregate mixing is another important use. The SS-K type is the cationic equivalent of the presently used anionic SS (slow setting) emulsions. The SS-grade emulsions are used principally for slurry seal maintenance, base stabilization, and tack or prime coats.

The specifications recommended for these two new products are given in Tables 2 and 3, respectively. Typical properties for the products now commercially available are also shown. These are the properties of the SM-K and SS-K emulsions used in the present studies. The particle charge test used in the SM-K specification is a relatively new test method described by Mertens, Coyne, and Rogers (*10*) that is used to indicate the cationic character of the emulsion.

The stability and other properties of the SM-K type emulsion are intermediate between the anionic MS (medium setting) and SS emulsion grades. To a certain extent, SM-K emulsions combine the properties of cutback asphalts and SS emulsions and are designed to replace these two materials in road and central plant mixing. SM-K mixes with and coats satisfactorily either wet or dry aggregates. The presence of a minor amount of cutter stock in SM-K formulations gives workability with dry aggregates. The ability to produce mixes that can be immediately compacted without aeration and which develop high, early cohesion are particularly noteworthy features of SM-K emulsions (*18*).

The SS-K emulsions meet all existing anionic SS specifications. Certain additional benefits are obtained by virtue of their cationic nature. Adhesion is improved and is 90 percent or better with all currently used emulsion adhesion tests (according to unpublished studies by the American Bitumuls and Asphalt Company). Pumping stability is better than for anionics. Dehydration of the finished mix is more rapid than with anionics.

Both specifications include product performance tests, which acquaint consumers with the performance properties of the new materials.

COMPARISON OF SM AND SM-K EMULSION WITH CUTBACK ASPHALTS

The SM-K emulsions were designed for use in aggregate mixing work, either at the road construction site or in a central plant. They offer many benefits not available with the conventional binders, anionic emulsions, and cutback asphalts (*19*). These include ease of mixing, ability to coat wet or dry aggregates, faster setting rates, and greater resistance to the deleterious effects of moisture.

TABLE 2

RECOMMENDED SPECIFICATIONS AND TYPICAL INSPECTIONS FOR SM-K EMULSION

Test	Method	Requirement	Typical
Residue (by distillation) (%)	ASTM D 244-60	60 [a]	61.0
Oil distillate (% by wt.)	Calculated [b]	15 [c]	15.0
Viscosity (SSF at 122 F)	ASTM D 244-60	50-500	157.
Sieve (% retained on No. 20 mesh)	ASTM D 244-60 [d]	0.10 [c]	0.01
Particle charge	Described by Mertens et al. (*10*)	Positive	Positive
Settlement (7 days) (%)	ASTM D 244-60	3.0 [c]	0.2
Sand-coating water resistance (% coated):			
Dry job aggregate	— [e]	80 [a]	95
Wet job aggregate	— [e]	60 [a]	85
Distillation residue:			
Penetration at 77 F, 100 g, 5 sec	ASTM D 5-52	50-250	90
Solubility in CCl_4 (or C_2HCl_3) (%)	ASTM D 4-52 [f]	98 [a]	99.1
Ductility (cm)	ASTM D 113-44 [g]	80 [c]	140 +

[a] Minimum.
[b] From volume and specific gravity of distillate obtained in residue determination.
[c] Maximum.
[d] ASTM Method D 244 procedure modified by replacing sodium oleate solution with distilled water.
[e] Aggregate (465 g) and emulsion (35 g) are mixed vigorously for 3 min with spatula in metal pan. After mixture has cured 30 min at room temperature, it is drenched with cold tap water. Mixture then dried by exposure to an electric fan. When surface is dried, percentage of total aggregate surface coated is estimated immediately.
[f] Procedure modified by substituting carbon tetrachloride or trichloroethylene for carbon disulfide.
[g] Run at 77 F for 100 to 200 penetration asphalt and at 60 F for 200 to 250 penetration asphalt.

TABLE 3

RECOMMENDED SPECIFICATIONS AND TYPICAL INSPECTIONS FOR SS-K EMULSION

Test	Method	Requirement	Typical
Residue (by distillation) (%)	ASTM D 244-60	58 [a]	61.2
Viscosity (SSF at 77 F)	ASTM D 244-60	20-100	25
Sieve (% retained on No. 20 Mesh)	ASTM D 244-60 [b]	0.10 [c]	0.01
pH	ASTM E 70-52T	6.5 [c]	3.9
Cement (% broken)	ASTM D 244-60	2 [c]	Trace, < 0.1
Adhesion (% coated)	— [d]	90 [a]	95 +
Settlement (7 days) (%)	ASTM D 244-60	3 [c]	1.0
Coating	— [e]	— [f]	Passes
Distillation residue:			
Penetration at 77 F, 100 g, 5 sec	ASTM D 5-52	40-200	120
Solubility in CCl_4 (or C_2HCl_3) (%)	ASTM D 4-52 [g]	97 [a]	97.5
Ductility (cm)	ASTM D 113-44 [h]	40 [a]	80

[a] Minimum.
[b] ASTM Method D 244 procedure modified by replacing sodium oleate solution with distilled water.
[c] Maximum.
[d] Job aggregate (100-g sample), properly graded to pass ⅜-in. sieve and be retained on No. 4 sieve (for fine aggregates, fraction passing No. 4 sieve and retained on No. 30 sieve is used), and emulsion (10 g), are mixed in 16-oz tin with stirring rod. Mixture held at 200 F for 24 hr and then thoroughly remixed. A 50-g portion of mixture is stirred with 400 cc of boiling distilled water in 600-cc beaker for 1 min. Mixture then air dried on absorbent paper. Percentage of total aggregate surface area coated is estimated immediately.
[e] Sample of the job aggregate passing No. 4 sieve placed in the mixing bowl of mechanical mixer of planetary type (Readco Model P-12 or equivalent). Sufficient water, if necessary, added to darken aggregate. Desired amount of emulsion added and mixture stirred until uniform coating is obtained.
[f] Must mix with dense graded job aggregates without breaking, balling, or segregation.
[g] Procedure modified by substituting carbon tetrachloride or trichloroethylene for carbon disulfide.
[h] Run at 77 F for 100 to 200 penetration asphalt and at 60 F for 200 to 250 penetration asphalt.

Anionic asphalt emulsions present several problems that have hindered their use in central plant mixing. They will not coat most dry, graded aggregates, and the amount of water needed is usually greater than desired at compaction. Long periods of curing at the road site are, therefore, required. Cutbacks, even in the presence of adhesion agents, will not coat wet, graded aggregates very well. The SM-K emulsions overcome both these difficulties and coat either wet or dry aggregates equally well. Aeration is greatly reduced or entirely eliminated. Because no heat is required for drying aggregates, higher pugmill production rates are possible.

The advantages of SM-K emulsions over an RC-3 cutback containing an adhesion agent for graded-aggregate mixing are illustrated by the following tests with an aggregate from Josephine County, Ore. Two series of mixes were made without the addition of water and with equivalent amounts of SM-K emulsion or RC-3 cutback to furnish 4 and 5 percent binder content by weight. After compaction, all the samples were cured for 15 hr at 140 F. *S*-values were then determined in the Hveem stabilometer (*20*). The *S*-value is a measure of the stability or supporting power of asphalt-aggregate mixtures and of their ability to resist deformation under load. The *S*-values for the SM-K emulsions were 36 and 27, respectively, and the RC-3 mixes gave values of only 12 and 14. Two more mixes were then made, each containing 4.8 weight percent water and an amount of SM-K or RC-3 equivalent to 7 weight percent binder. The *S*-value of the SM-K mix was 22, but the RC-3 mix disintegrated before the test could be run. The aggregate used in these tests had a sand equivalent (*21*) of 56 and the following grading:

Sieve Size	Weight Percent Passing
1-in.	100
¾-in.	98
½-in.	77
⅜-in.	64
No. 4	44
No. 10	34
No. 40	16
No. 80	8
No. 200	4

A series of Hubbard-Field stability tests (*22*) was run comparing the rate of cohesion development in RC-1, MC-1, and SM-K mixes. The aggregate used was a sandy silt from San Diego County, Calif. It had a sand equivalent of 28 and the following grading:

Sieve Size	Weight Percent Passing
⅜-in.	100
No. 4	98
No. 8	95
No. 16	87
No. 30	77
No. 50	50
No. 100	25
No. 200	13

The specimens were prepared and tested after various periods of curing. The results are shown in Figure 1. Even though the combined percentage of water and solvent volatiles in these three mixes was equal, the SM-K mixes developed cohesion at a much faster rate than the other systems. The strength obtained after 175 hr was also much better.

Figure 2 shows road construction near Salinas, Calif., using SM-K emulsion applied with a Wood's mixer. Because the SM-K emulsion was able to function with less water present in the aggregate than when using SS emulsions, the mix could be compacted immediately after spreading. SM-K emulsions, therefore, permit mixes to be made at optimum moisture content. In addition, much less mixing was required to give a satisfactory composition with SM-K on this project than with a comparison test section made with a cutback asphalt. A further advantage was obtained in that the emulsion was successfully applied at ambient temperatures while the cutback had to be applied hot. Also, the mixing chamber in the Wood's machine had

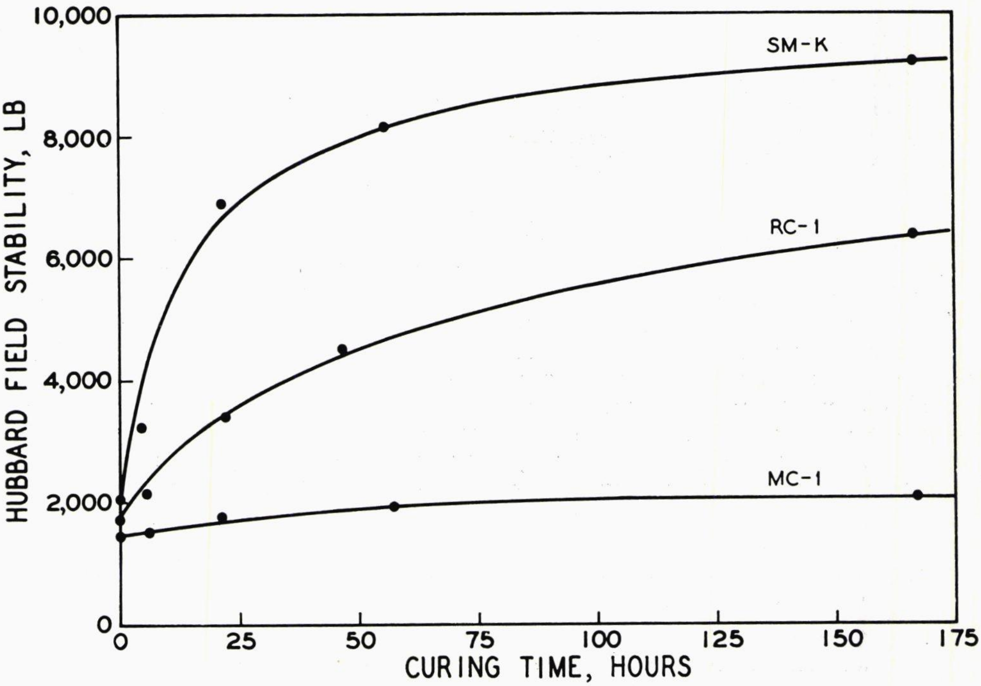

Figure 1. Hubbard-Field stability tests comparing rate of cohesion development in three mixes.

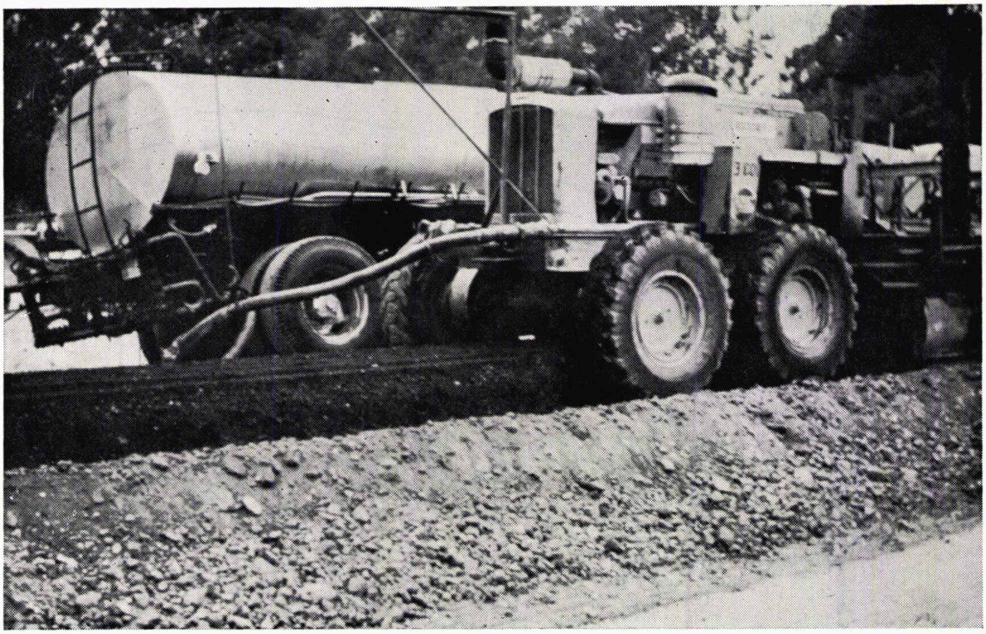

Figure 2. Road construction using SM-K emulsion applied with a Wood's mixer.

to be heated during the use of the cutback. Because an open flame was used, this created a fire hazard.

The incorporation of adhesion agents, some of which are high molecular weight organic amines, in anionic emulsions improves their adhesive properties. However, it does not appear possible to achieve by this means the high level of performance obtainable with cationic emulsions. This is undoubtedly because of the differences in polarity of the amines and the anionic emulsifying agents. In particular, the organic amines are less effective in highly alkaline emulsions, because their greater solubility in asphalt under such conditions tends to remove them from the aggregate-asphalt interface.

The improvement in resistance to deformation under load and in cohesive strength obtained when a cationic emulsion is used in making mixes with clay sand is shown in Figure 3. Comparison is made with anionic emulsion mixes, both with and without an adhesion agent. The results are given in terms of the *R*- and *C*-values obtained in the Hveem stability and cohesion tests, which are measures of the foregoing properties, respectively (*20*). The adhesion agent improved the performance of the anionic emulsion, but failed to give the degree of improvement possible

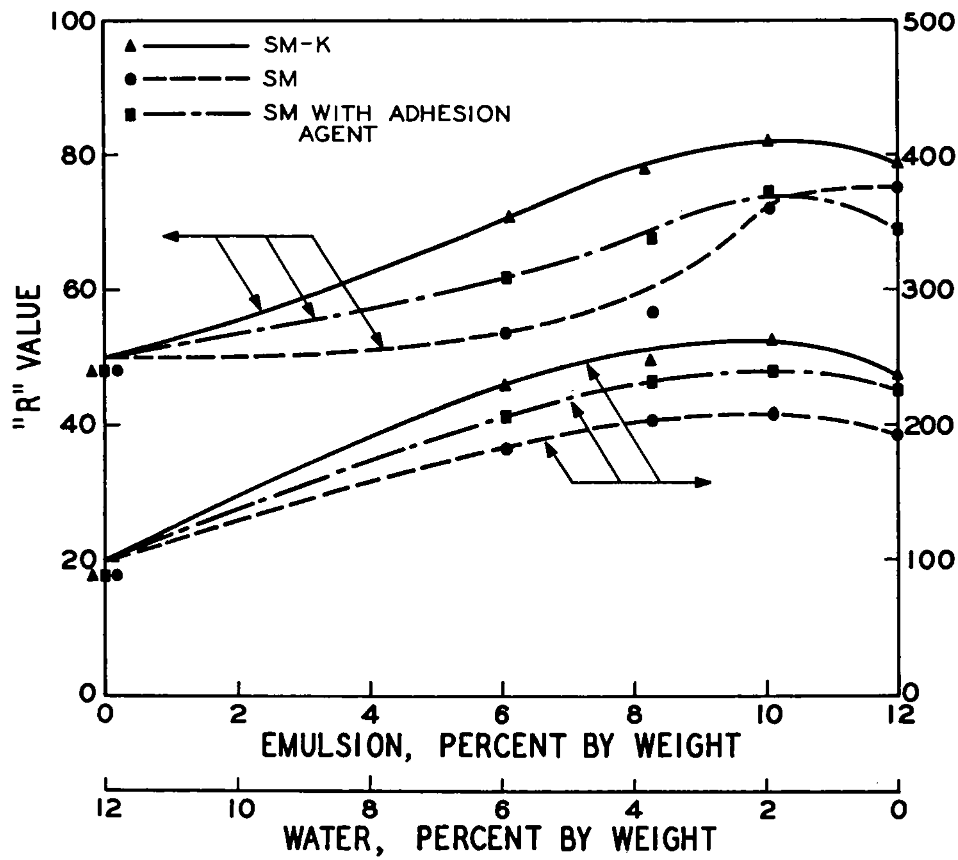

Figure 3. Hveem stability and cohesion values for clayey sand mixes.

with the cationic emulsion. The adhesion agent was used in these tests in an amount equal to the concentration of the cationic emulsifier. The aggregate was obtained from the Coates Pit in Contra Costa County, Calif. It has a sand equivalent of 18 and the following grading:

Sieve Size	Weight Percent Passing
No. 16	100
No. 30	99
No. 50	92
No. 100	32
No. 200	16

The use of amine-type adhesion agents is subject to another disadvantage. Acidic constituents present in most asphalts slowly combine with the amine additives and nullify their effectiveness as adhesion agents. This is true for both cutback and paving-grade asphalts. The low pH of cationic emulsions suppresses the ionization of the asphaltic acids and prevents this reaction.

COMPARISON OF SS AND SS-K EMULSIONS

Slurry sealing is an important maintenance operation for the highway engineer. This type of construction requires aggregates that conform to the following grading (*23*):

Sieve Size	Weight Percent Passing
No. 4	100
No. 8	70-100
No. 16	40-80
No. 30	30-60
No. 50	20-50
No. 100	10-25
No. 200	5-15

Aggregates retained on a No. 4 sieve or larger cause streaking during application. An insufficient quantity of fines results in segregation during application and raveling of the cured mix. An excessive amount of fines makes the mixes brittle. The aggregate should have a minimum sand equivalent of 40.

A seldom recognized cause of slurry seal failure is the use of powered brooms for street sweeping. This equipment, in combination with water, exerts a very severe abrading force. Slurry seals made with cationic emulsions resist this force very well.

The practical value of cationics in resisting the stripping forces of mechanical action and water is shown by the wet track abrasion tests in Figure 4. Slurry seal mixes were made with typical anionic and cationic SS emulsions and tested in accordance with the procedure recommended for this test (*23*). The cationic emulsion gave excellent results, particularly in the range of emulsion concentrations normally employed. With this particular aggregate, chosen for the comparison because of its borderline quality, satisfactory performance could not be obtained with the anionic emulsion under any conditions.

Slurry seal mixes must be fluid enough to fill cracks and small depressions and to be applied in thin lifts; *i.e.*, usually less than ¼ in. deep. This fluidity is a disadvantage in that the water required to obtain this condition must be evaporated before subjection to vehicular traffic. Depending on weather conditions, 4 to 24 hr are required for the water to evaporate from anionic slurry seals. On some construction projects, it is difficult to hold traffic off for such periods of time.

Cationic slurry seals dehydrate at much faster rates than do anionic slurry seals. This is shown by the curing rate curves for anionic and

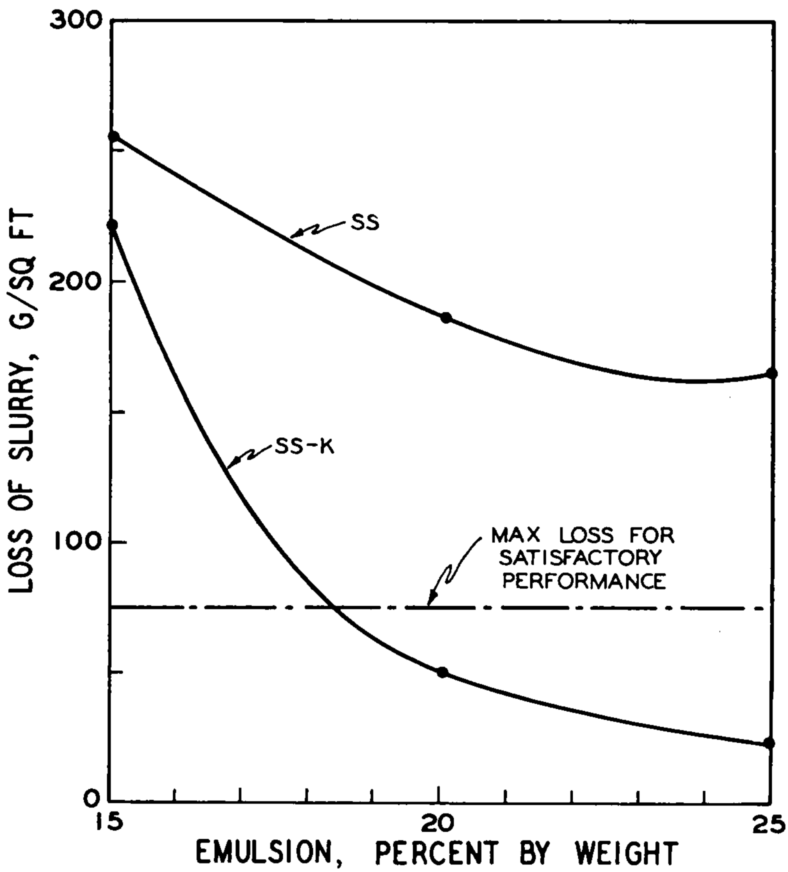

Figure 4. Wet track abrasion test.

cationic slurry seal mixes in Figure 5. Two hours or less was sufficient to set the cationic mix, while all of the anionic specimens required much longer times. Cationic emulsion slurry seals are reasonably resistant to traffic when the residual water content is as high as 2 percent; anionic emulsion slurries must contain less than 0.5 percent water before they are satisfactorily resistant to abrasion.

Another difficulty occasionally experienced with anionic emulsion mixes is "wash-off" due to rainfall before the time the emulsion sets. Unless the water content of such mixes is below a certain low level, addition of more water causes restoration of the partially coalesced emulsion to its original condition. Unexpected rainfall can thereby have disastrous effects on anionic slurry seals.

Cationic slurry seals are essentially immune to this difficulty after initial coalescence has started. This is shown in Figure 6, which was taken after a sudden, heavy rainstorm struck a small-scale slurry seal test section at Richmond, Calif., about one-half hour after completion. The cationic section is in the background, and the anionic section is in the fore-

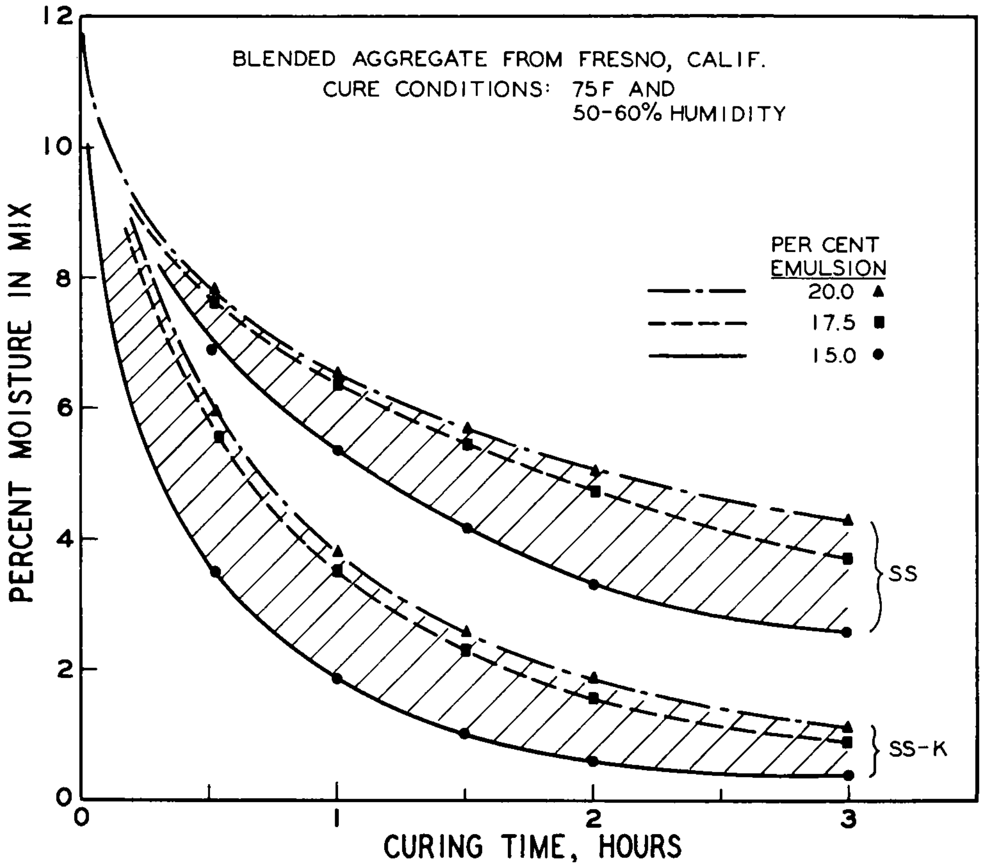

Figure 5. Curing rate curves for anionic and cationic slurry seal mixes.

ground, with a heavy runoff on the roadside from the anionic section. Runoff from the cationic section was slight and would have been entirely absent if the mix could have cured for about another half hour before the rain started. The anionic section would have been susceptible to heavy damage for many additional hours.

The superiority of cationic emulsions for base stabilization is shown in Figure 7. Cationic and anionic SS-grade emulsions were mixed in laboratory tests with a graded aggregate containing a considerable amount of clay. The aggregate had the following grading:

Sieve Size	Weight Percent Passing
¾-in.	100
½-in.	88
⅜-in.	75
No. 4	60
No. 8	51
No. 16	40
No. 30	31
No. 50	23
No. 100	15
No. 200	8

The sand equivalent of the aggregate was 21. The coarse fractions were crusher run gravel from Fairfield,

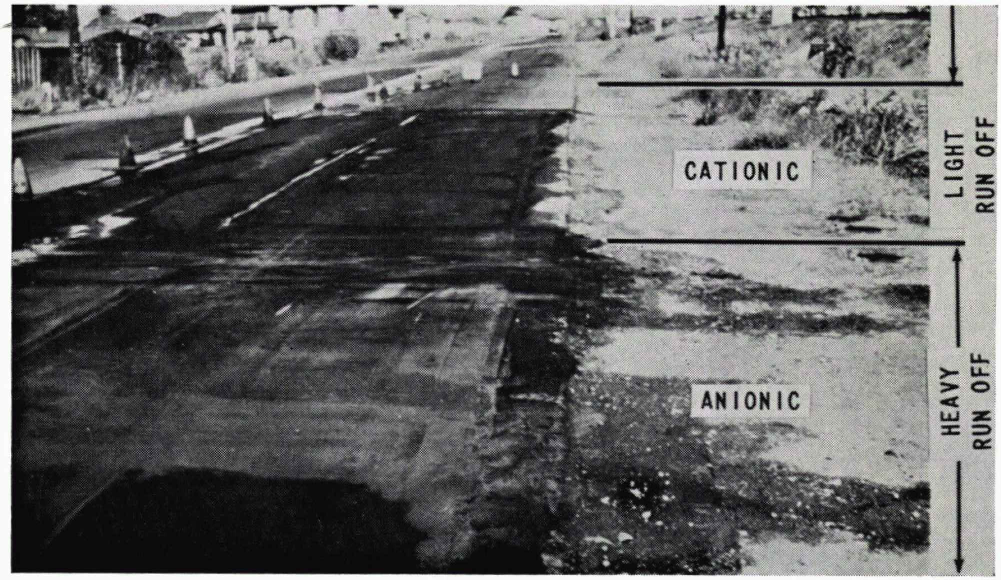

Figure 6. Small-scale slurry seal test section after sudden, heavy rainstorm.

Calif. The 8 percent fraction passing the No. 200 sieve contained the following:

Material	Weight Percent
Bentonite clay	5.0
Limestone dust	0.8
Fairfield nonplastic crusher fines	2.2

Bentonite clay is one of the most plastic types of silicates and was purposely added to make the stability critical.

The mixes were prepared with 7 percent emulsion, compacted at optimum moisture content, and cured at 100 F to one-fourth the compaction moisture. The specimens were then buried in moisture-saturated Ottawa sand for two months before the stability determinations to obtain equilibrium conditions.

PROSPECTUS FOR CATIONIC MIXING-GRADE EMULSIONS

SM-K and SS-K cationic asphalt emulsions are now commercially available in some areas. Their availability is being extended rapidly to all areas where the aggregates available are particularly suited to the use of these new products.

Particularly noteworthy among the properties of the new materials are the ability to coat and adhere to a wide variety of dry or moist aggregates, superior resistance to mechanical stripping action and moisture, and rapid setting tendencies.

These properties mean that contractors will be able to use a wider range of aggregate types and will have greater flexibility in their choice of construction practices. It appears likely that mixing-grade cationics will speed highway construction procedures. The amount of mixing or blading is in some cases much less

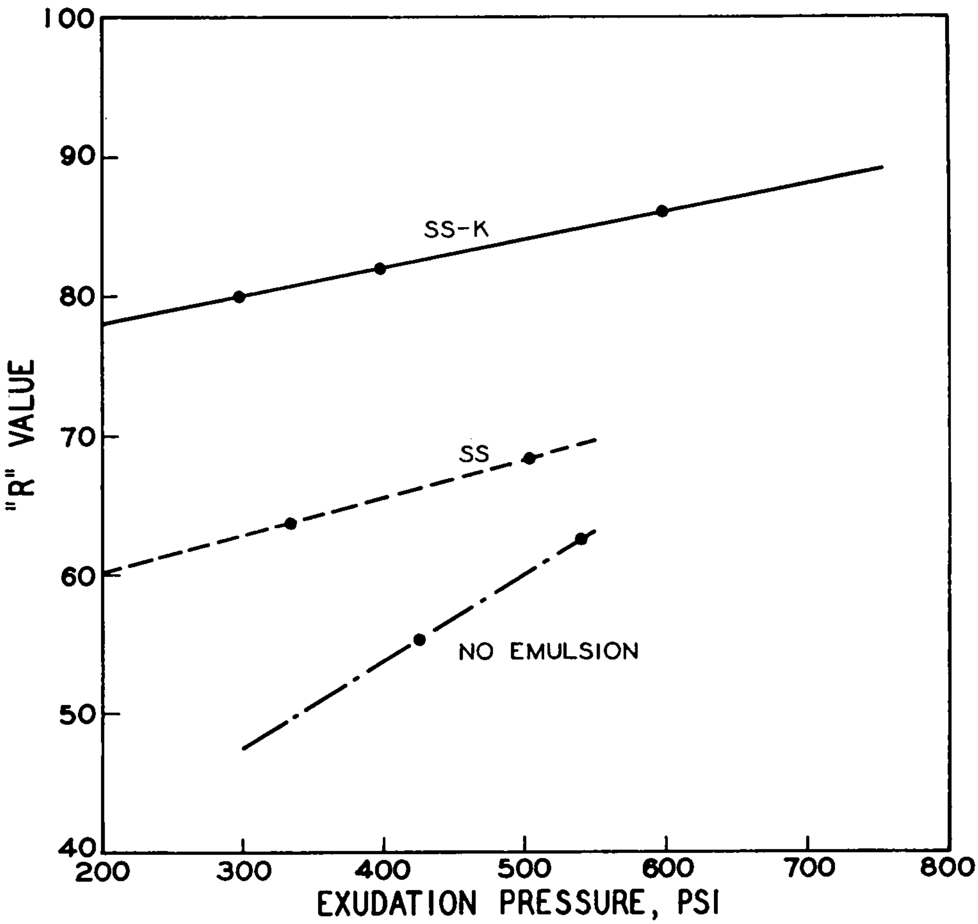

Figure 7. Hveem stability of graded aggregate containing clay.

than that required with anionic emulsions or cutback asphalts.

The mixing-grade cationic emulsions are expected to be successful in applications where anionics have given marginal results in the past. The future demand for these new emulsions will not develop, however, entirely at the expense of the anionics. Anionics, which are inherently somewhat less costly than cationics, will continue to be used where application conditions are favorable. The properties of the cationics are such that their use will expand into applications now employing nonasphaltic materials.

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DISCUSSION

K. E. McCONNAUGHAY, *K. E. McConnaughay, Inc.*—As a manufacturer of asphalt emulsions whose experience with these materials dates back some 35 years, K. E. McConnaughay, Inc., is much interested in research designed to develop the full potential of these materials. A continuing research program is maintained for this purpose and in this light the research efforts that have produced this paper have been supported. However, some aspects of the presentation require comment.

Throughout the paper the authors refer to anionic emulsions and compare the properties of these with cationic materials. If in so doing, test results, experience, and opinions of the American Bitumuls and Asphalt Company is being cited with respect to the materials of their own manufacture, the paper should be written so as to make this limitation clear. On the other hand, the writer objects to the inference that all anionic emulsions are inferior to cationic ones. The performance advantages claimed by the authors for the cationic materials over the anionic ones they tested are just as valid for certain anionic emulsions over other anionic ones. Specifically, the weatherproof type of anionic MS materials which the writer's organization long has favored have many of these performance advantages as compared to SS and MS anionic types which are subject to washing by rain.

An SS-K emulsion is presented as a cationic material and the slurry seal made from it is referred to as cationic. The SM-K cationic material is characterized as cationic in the proposed specification for this material by requiring a positive charge result in the particle charge test, but the proposed specification for the SS-K material does not include this requirement. Therefore, it may be that the SS-K emulsion is truly cationic. Certainly it could meet the requirements of the specification and not be cationic.

In the introduction to the paper the statement is made, "The most important characteristic of anionic emulsions is that the emulsified asphalt particles carry negative electrical charges." Contrary to this statement, the writer considers that the most important characteristic of any asphalt emulsion is its performance in use. He agrees that asphalt emulsions should coat aggregate readily in a wet or dry condition and should produce mixtures with good workability that are not water susceptible, do not require the addition of water for mixing, and compact readily without excessive reduction of moisture content. For these reasons his organization has not generally recommended the use of SS types of emulsions or of MS materials that are susceptible to washing with water. However, he believes that these properties are achieved in emulsions of his manufacture which fall into the anionic class.

The writer does not wish to leave the impression that he does not recognize that cationic asphalt emulsions can be satisfactory road-building materials. His experience to date with the cationic materials of his manufacture has been entirely satisfactory, and some projects of this type are more than four years old. However, satisfactory performance of pavements he has constructed with anionic emulsions that are over 30 years old can be cited. Also, where both cationic and anionic emulsions of his manufacture were used in a comparative way in the same project, superiority of cationic emulsions has not been demonstrated. Such projects have included siliceous as well as carbonate aggregates. Therefore, particle charge of the asphalt emulsion is not the controlling factor in performance.

M. J. BORGFELDT AND R. L. FERM, *Closure.*—The authors have evaluated in their laboratory cationic and anionic emulsions made by a number of manufacturers. It was not intended to infer that anionic emulsions are inferior or that they are unsatisfactory for highway construction and maintenance. The superiority of cationic asphalt emulsions is best demonstrated by their rapid acceptance by consumers, and tests run by unbiased groups. For example, the Highway Department of North Carolina used some 100,000 gal of cationic emulsion in 1958, in 1959 they used 4,000,000 gal, and in 1960, 8,500,000 gal. The Montana Highway Department has run adhesion tests on some one hundred different aggregates. Anionic emulsions reacted more favorably than cationic emulsions on only 2.8 percent of these aggregates, which include both siliceous and calcereous types. Cationics were superior on some 64 percent of the aggregates. The Department of Highways of Ontario, Canada, conducted coating tests with various limestone aggregates, all showed a preference for cationic emulsion. In a paper presented at the Sixth Annual Conference of the Canadian Technical Asphalt Association at Victoria, B.C., November 1961, A. E. Holberg stated:

> In the meantime, I obtained comments from Dr. Ing Letters and according to his experience in Western Germany, cationic emulsions do just as well with dolomite carbonates as with any other aggregates as far as field performance is concerned.
>
> Mr. G. Bertlet of Esso-Standard-France, commented on this as follows. "To date we never came across a material giving better results when using anionic emulsions than when using cationic emulsions."

Cationic emulsions have been used in France since 1953, and in 1960 accounted for 38 percent of the total emulsion market.

Mr. McConnaughay asked if the SS-K emulsion is cationic, as the specification does not include a particle charge test requirement. The SS-K emulsion will not migrate under the conditions of the particle charge test. Using microelectrophoresis techniques, the cationic nature of this emulsion can be demonstrated. This method however is not suitable for specification purposes, as it requires equipment not usually available to control laboratories. More definitive tests are being investigated.

The authors agree with Mr. McConnaughay that the most important characteristic of an asphalt emulsion is its performance in use and feel that the essential chemical difference between anionic and cationic emulsions is the sign of the charge on the asphalt particle. Inasmuch as a majority of road building aggregates have a negative charge, the value of a binder bearing a positive charge is apparent.

Changes in Physical Properties of Asphalt Pavement with Time

J. R. BISSETT, *Professor of Civil Engineering, University of Arkansas*

This paper reports the study of approximately 60 mi of asphaltic concrete pavement under actual traffic conditions. The pavements vary from 3 to 8 years of age and represent seven construction projects. Recovery of the asphalt cement was accomplished by a simplified Abson method. Check tests were made on this method to determine any effect it might have on Arkansas asphalts. Density, asphalt content, and gradation of aggregate of the pavements were determined. The asphalt cement recovered from the pavements were subjected to penetration, ductility, softening point, and ash content tests. In some cases the thin-film oven test was also made. Generally, the results indicate a rapid reduction in the ductility of the asphalt with time. There is also a decided but less rapid reduction in the penetration of the asphalt. Samples of the pavement were heated, remolded, and tested by the Marshall method. Usually, the stabilities were quite high.

• THIS REPORT is part of a study at the University of Arkansas on the performance of flexible pavements. It is a joint effort of the University of Arkansas, the Arkansas Highway Department, and the U.S. Department of Commerce, Bureau of Public Roads. The purpose of the study is to evaluate the performance of flexible pavements and to determine some of the factors that cause pavement failures. The physical properties and conditions of the various parts of the pavement structure are being examined and evaluated so that the design methods may have a more rational basis. This paper is concerned with the asphalt pavement only. A separate report of the study of other parts of the pavement structure is being prepared.

PAVEMENT SAMPLING

All pavements studied were hot-mix asphaltic concrete. Some of the pavements were made with uncrushed gravel as the aggregate, some were made with crushed gravel, and others with a mixture of crushed gravel and crushed syenite.

Six construction projects totaling 47 mi are discussed in this report. All of the projects were regular State highway jobs constructed by the highway department under contract in the normal way. At the time the jobs were constructed it was not known that they would be used as future test projects. At the time the samples were taken, the pavements varied in age from 2 to 7 years. Samples about 15-in. square were cut

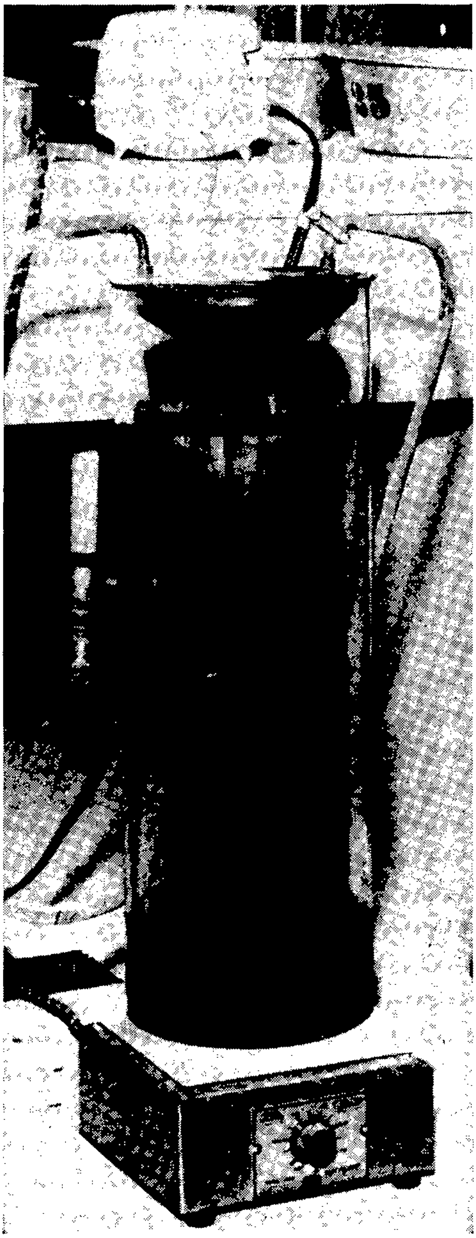


Figure 1. Faulwetter apparatus.

from the pavement at intervals of 0.4 to 0.6 mi. All pavement samples were taken in the center of the traffic lane between the two wheelpaths. The samples were taken in the late fall or early spring while the pavements were cold. They were cut from the pavement with an ordinary paving breaker equipped with a spade bit. The samples came out in one solid piece and showed no cracks or evidence of fracturing. Reasonable care was taken in handling and transporting the samples to the laboratory. The samples were placed in paper bags and kept in a horizontal position to reduce disturbance as much as possible.

Numbers were painted on the pavement at each 0.2 mi for the purpose of identifying test locations. Samples were then taken at intervals of either two or three 0.2-mi stations.

METHOD OF ASPHALT CEMENT RECOVERY

A modification of the Abson process was used for recovering the asphalt cement from the pavement samples. For this recovery the Faulwetter apparatus was used to extract the asphalt cement from the pavement. This apparatus is shown in Figure 1. Trichloroethylene was the solvent used. The Faulwetter apparatus consists of two wire cone-shaped baskets suspended in a glass jar. The solvent is placed in the bottom of the jar and a metal cone-shaped water-cooled condenser is used for a cover at the top of the jar. The jar is 18 in. high and 6 in. in diameter. The sample is placed in the wire baskets with three-ply filter paper between the sample and the wire of the basket. An electric hot plate is used to heat the apparatus. The solvent is condensed on the cone-shaped condenser at the top of the apparatus and drips into the baskets containing the sample. If medium density filter paper is used, the solvent will flow out of the baskets at a sufficiently slow rate so that the baskets will remain full of the solvent. This leaching process is continued until the solvent flowing from the baskets is clear. This usually requires about 3 hr. The solution is settled by centrifuging as specified in the standard method AASHO T 170-55 before the asphalt cement is recovered from the solution by distillation.

Method AASHO T 170 for recovering the asphalt cement from solution was modified only to the extent that the oil bath is eliminated and a glass tube with a bulb on the end is substituted for the brass ring in admitting carbon dioxide to the boiling solution. The solution boils at 190 F until practically all the solvent is driven off The rate of distillation is not controlled while the temperature is maintained at 190 F. The temperature rises rapidly from 190 F to 300 F. Carbon dioxide is introduced into the distillation flask when the temperature reaches 220 F. Figure 2 shows the apparatus used for recovering the asphalt cement from the solution.

Abson and Burton (*1*) report that trichloroethylene did not change the

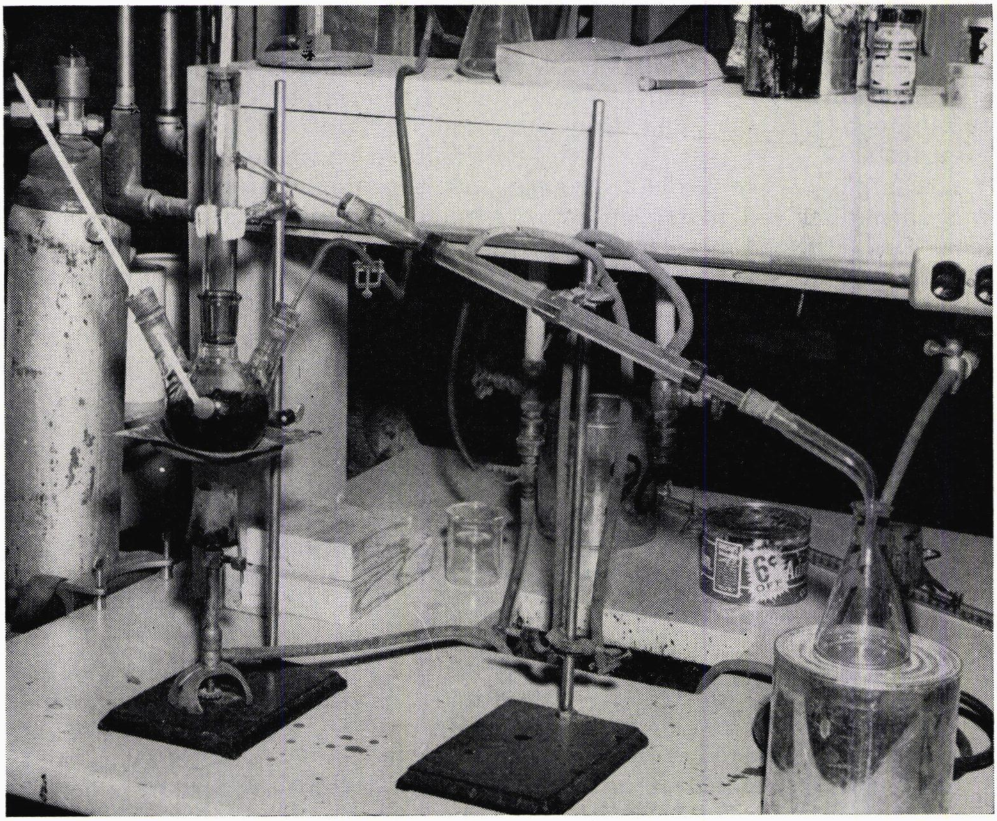


Figure 2. Distillation apparatus.

TABLE 1

CHECK ON METHOD OF RECOVERY

Brand	Penetration			Ductility			Softening Point			Ash	Thin-Film Oven		
	Orig.	8-Hr	7-Day	Orig.	8-Hr	7-Day	Orig.	8-Hr	7-Day	Orig.	Orig.	8-Hr	7-Day
A	73	73	74	150+	150+	150+	48	49	48	0	0	0.06	0.10
B	71	71	71	150+	150+	150+	51	51	52	0	0	0.03	0.07
B	63	62	59	150+	150+	150+	52	52	55	0	0	0.03	0.02
C	65	65	65	150+	150+	150+	50	48	49	0	0	0.03	0.10

physical properties of asphalt cement when used as the solvent for recovering the asphalt cement. A series of check tests were made on Arkansas asphalts to determine any changes that might be caused by the method of recovery that was proposed. Table 1 gives the results of these tests. The three brands of asphalts are the only ones produced in Arkansas. For this test, samples of the asphalts were subjected to penetration, ductility, softening point, ash, and thin-film oven tests.

A sample of the asphalt cement was then dissolved in trichloroethylene. One series of distillations were then made completing the entire process and testing in an 8-hr period. Another series of tests were made in which the solution of asphalt cement and trichloroethylene was prepared and stored in the laboratory 7 days. The results are the averages of the three sets of tests in each case. In recovering the asphalt cement from the pavement samples as reported hereafter, the entire process was completed in one 8-hr day.

PAVEMENT TESTS

Part of the samples of pavement taken from the roadway were used for the purpose of determining the condition of the mixture at the present time. A sample of the pavement weighing from 500 to 700 g was used to determine the specific gravity of the pavement. The sample was weighed in air, then coated with paraffin and weighed in water for this determination. Another sample of the pavement was heated in an oven to approximately 240 F and then molded for Marshall stability specimens. The percent voids, Marshall stability, and flow were then determined. Only two Marshall stability test specimens were made from each sample because there was not enough material available to make the third specimen. The asphalt cement content of the pavement was determined by Method AASHO T 164-55. Carbon tetrachloride was used as the solvent. A screen analysis was made of the aggregate.

ASH CONTENT

Investigations under way in the laboratory indicate that ash contents up to about 1.3 percent have no effect on the penetration or ductility of asphalt cement. Both limestone dust and portland cement were used to mix with the asphalt cement to produce the ash. This percent may not be the upper limit as there have been no tests completed in which the ash content was above 1.3 percent.

TABLE 2
PROPERTIES OF RECOVERED ASPHALT CEMENT

Job	Age (yr)	Tests on Orig. AC			Location	Tests on Recovered AC			
		Pen.	Duct.	Brand		Pen.	Duct.	Soft Point [1]	Ash Content
M	2.8	64	100+	C	1	50	67	59	0.87
					4	34	21	64	0.81
					8	42	8	67	1.20
					12	30	7	65	1.09
					14A	36	24	67	2.27
					17	41	39	63	0.76
					20	42	24	63	1.08
					23	39	36	62	0.79
					26	35	10	64	1.11
					29	35	13	65	1.63
					34	42	32	63	1.27
					37	43	33	62	1.36
					Avg.	39	26	64	1.19
J	5.7	68	100+	A	1	28	8	58	0.53
					4	25	5	—	0.44
					8	20	4	—	0.41
					10	27	9	65	0.51
					13	20	7	69	0.59
					17	26	40	62	0.54
					20	27	26	62	0.47
					23	26	20	62	0.68
					26	25	15	64	0.78
					28	27	14	63	0.52
					Avg.	25	15	63	0.55
F	5.3	65	100+	C	2	24	5	77	0.47
					8	29	6	—	—
					11	30	8	74	0.97
					14	26	6	—	—
					17	16	2	—	—
					20	26	6	72	0.43
					23	23	5	—	—
					26	25	8	77	0.49
					29	29	9	—	—
					32	20	3	—	—
					35	25	6	73	0.12
					Avg.	24	6	75	0.50
I	7.2	66	100+	C	2	40	30	61	0.35
					5	33	14	64	0.45
					8	23	5	74	0.29
					11	39	24	61	0.34
					14	40	33	58	0.53
					17	34	12	63	0.30
					20	29	8	62	0.31
					24	22	5	72	0.26
					27	21	4	75	0.28
					29	36	19	59	0.42
					33	28	8	68	0.28
					36	27	3	70	0.39
					42	25	5	69	0.32
					45	30	7	66	0.33
					Avg.	31	13	66	0.35
A	6.8	75	100+	C	3	25	0	74	0.40
					5	26	0	—	—
					7	23	0	—	—
					9	23	0	75	0.51
					11	24	5	—	—
					13	29	7	—	—
					15	24	6	73	0.65
					17	24	4	—	—
					19	20	5	—	—
					21	13	2	78	0.99
					23	18	3	—	—
					25	23	5	—	—
					27	27	7	—	—
					29	18	3	—	—
					31	—	0	—	—
					33	22	5	—	—
					35	25	5	—	—
					39	26	5	—	—
					Avg.	23	3	75	0.64
B	7.7	74	100+	C	2	20	6	75	0.41
					5	19	0	—	—
					8	20	5	72	0.56
					12	24	6	—	—
					15	24	6	—	—
					18	19	5	74	0.35
					21	24	5	—	—
					24	23	5	—	—
					Avg.	22	5	74	0.44

[1] R&B.

RESULTS

The properties of the recovered asphalts are given in Table 2. The penetration, ductility, and softening point were determined by standard AASHO methods. Tests on the original asphalt cement, as taken from the job records, are also given. The original penetration as indicated is an average of all the job tests. In no case was the maximum variation more than ±3, and in only one case was the variation this large.

Table 3 summarizes the results given in Table 2. Figure 3 shows the ductility and percent decrease in penetration with age. It was not possible to determine the amount of change in ductility and softening point. The original ductility was determined as 100+ cm. The softening point test was not made as a routine

TABLE 3
SUMMARY

Job	Age (yr)	Penetration			Ductility	Ash (%)	Softening Point
		Orig.	Final	Decrease (%)			
M	2.8	64	39	39	26	1.19	64
J	5.7	68	25	63	15	0.55	63
A	6.8	75	23	69	3	0.64	75
F	5.3	65	24	63	6	0.50	75
I	7.2	66	31	53	13	0.35	66
B	7.7	74	22	70	5	0.44	74

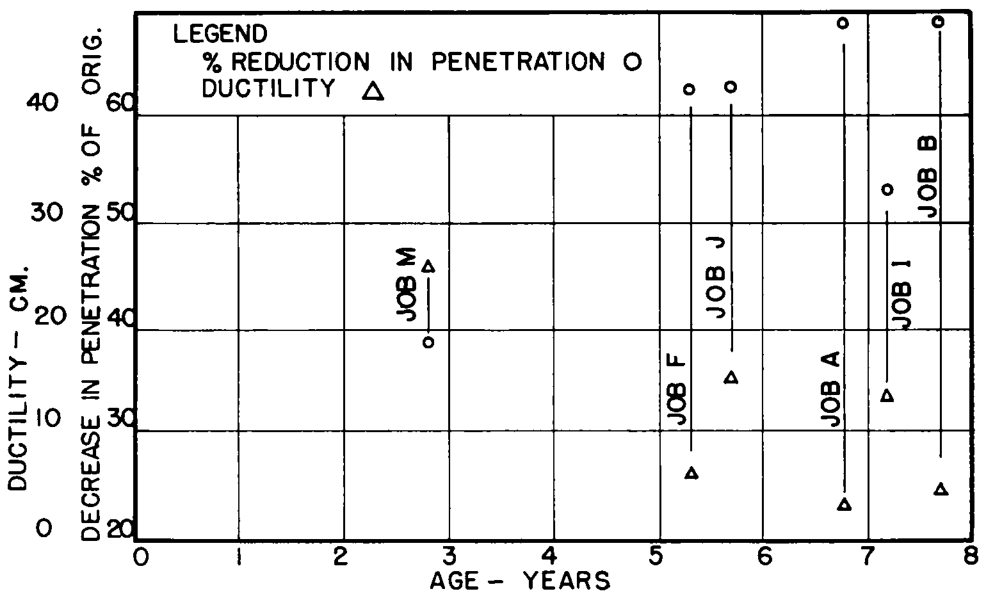


Figure 3. Change in ductility and penetration with age.

procedure by the highway department laboratory. The decrease in penetration is shown as a percent of the original penetration. It was necessary to show the change in penetration this way because two different penetration grades were used on the projects. The asphalts showing the highest percent decrease in penetration were both 74 or 75 penetration when placed.

The results are interesting when compared with the condition of the pavement at the present time. Jobs A, B, and F show the lowest ductility and the highest decrease in penetration. The loss in penetration for Job J is high, but the ductility is still comparatively high. Jobs A, B, and F have extensive surface cracking. These are single, isolated cracks and are roughly parallel to the centerline or normal to it. The minimum spacing of the transverse cracks is about 17 ft. The longitudinal cracks appear near the centerline of each traffic lane and near the centerline of the pavement. Not all of the pavement is cracked this way but the cracking is so extensive that it is difficult to find a quarter-mile section on which there are no cracks. Often the transverse cracks will extend from the edge of the pavement to the centerline and in some cases the crack extends the entire width of the pavement. Jobs M, J, and I do not show any of this type of cracking.

Job I is somewhat older than some of the other jobs, yet it is in excellent condition and shows very few patches. The few patches that do show are in the outer wheelpath.

A comparison of Jobs A and F shows the effects of different types of maintenance. Job A was sealed about three years ago with a light shot of asphalt cement and covered with pea gravel. Within one year the cracks were showing through the seal coat, but at the end of the second year traffic had sealed most of these cracks to the extent that there were no signs of distress in the pavement. The cracks on Job F were not sealed. Rainwater has seeped through the cracks on this job and caused numerous base failures. The failures are so extensive that the surfacing is in very poor condition; in fact, it is almost a total loss.

A search of the construction records does not indicate any reason why some of the jobs have very low ductilities and others such as Job I in the same group still have appreciable ductility remaining. It is doubtful that the ash contents as given in Table 3 are having any material effect on the measurement of the ductility.

Current investigations in the laboratory show that the loss in penetration during mixing and placing was about 28 percent. The original penetration averaged 63, and the penetration of asphalt cement recovered from the pavement one week after placing was 45. The ductility of the recovered asphalt cement was 104. These figures are the average of several tests on each of two construction projects. The aggregate used was limestone and the temperature was closely controlled at 300 F during mixing.

The conclusion drawn from these data is that the early loss of penetration and ductility causes cracking of the pavement surface. It may be the loss of ductility that causes this cracking. The results of Job J indicate this as the ductility is still high, but the decrease in penetration has been considerable. This job does not

TABLE 4
PAVEMENT SAMPLE TEST RESULTS

Job	Design	Job Tests				Sample Tests					
		AC Brand	Pen.	AC (%)	Voids (%)	Loc.	Voids (%) Lab.	Voids (%) Pvt.	AC (%)	Marshall Stab. (lb)	Flow (0.01 in.)
M	Marshall stab., 1,400 lb; flow, 9; voids, 3.5-4.0%; AC, 5.5%; crushed limestone and local sand	C	64	5.3-5.5	4.0-6.0	4	4.6	7.4	5.9	2,060	11
						12	3.3	3.9	6.0	3,050	8
						17	4.8	7.3	5.5	2,950	9
						23	5.2	7.8	5.5	2,220	9
						29	4.0	6.4	5.0	2,220	12
						37	1.3	6.7	6.2	4,430	9
						Avg.	3.9	6.6	5.7	2,822	10
J	Marshall stab., 1,300-1,400 lb; flow, 9; voids, 4.0%; AC, 5.8%; crushed syenite and local fine sand	A	68	5.5-5.7	9.5-10.5	4	7.0	10.1	6.1	2,100	12
						13	8.4	10.6	5.9	2,890	11
						20	9.2	12.8	5.9	2,110	9
						28	5.3	8.2	5.6	2,870	10
						Avg.	7.5	10.4	5.9	2,493	11
A	Marshall stab., 950 lb; flow, 5; voids, 6-7%; AC, 6.0%; partially crushed gravel, local pit	C	75	5.7-6.0	9.1-11.6	5	7.0	10.5	6.7	3,360	7
						9	8.9	10.9	5.4	3,900	15
						21	8.8	9.8	6.0	4,750	13
						25	6.2	11.6	6.1	2,650	7
						33	6.9	8.5	5.9	3,650	7
						Avg.	7.6	10.3	6.0	3,662	10
F	Marshall stab., 1,050 lb; voids, 6.5%; AC, 6.0%; partially crushed gravel, local sand	C	65	5.9	8.2-10.2	2	9.0	12.0	5.7	1,820	8
						11	10.2	9.0	6.4	2,650	13
						20	6.4	10.9	6.2	2,700	8
						26	6.4	10.7	5.9	3,150	10
						35	6.9	11.3	6.1	2,800	9
						Avg.	7.8	10.8	6.1	2,624	10
I	Marshall stab., 1,200 lb; flow, 10; voids, 6.4%; AC, 6.0%; crushed syenite and local fine sand	C	66	5.7-6.0	8.5-9.5	8	8.3	11.2	6.4	3,920	7
						14	4.7	7.5	6.9	2,220	7
						20	5.2	10.4	6.5	2,350	9
						29	3.2	8.6	6.6	2,200	12
						36	1.2	5.4	6.1	3,360	8
						45	7.9	13.0	6.0	3,150	10
						Avg.	5.0	9.4	6.4	2,867	9
B	Not available	C	74	5.7-5.8	7.3-11.4	2	7.8	10.1	5.9	3,260	7
						8	7.6	9.9	6.0	2,860	7
						21	8.7	13.7	6.0	2,630	8
						Avg.	8.0	11.2	6.0	2,917	7

TABLE 5
COMPARATIVE SCREEN ANALYSIS

Sieve	Percent Retained									
	Job I		Job J		Job M		Job A		Job F	
	Constr.	Present	Constr.	Present	Constr.	Present	Constr.	Present	Constr.	Present
½	0	0	0	0	8	7	24	20	21	0
⅜	3	3	3	1	22	18	32	28	30	25
4	24	24	29	22	40	37	46	41	44	38
10	50	46	51	48	53	51	52	48	52	48
40	77	76	76	76	77	74	77	74	78	90
80	90	90	90	93	87	90	94	97	93	96
200	93	94	94	96	92	91	95	98	95	98
AC (%)	5.75	6.4	5.5	5.9	5.3	5.9	5.9	6.0	5.6	6.1
Voids (%)	9.0	9.4	9.5	10.4	4.2	6.6	11.6	10.4	10.2	10.8

show any cracking of the surface at this time even though it is as old as the job that shows serious cracking. The base material, subgrade, and climatic conditions are approximately the same for all of these jobs.

Table 4 gives the results of the tests made on the pavement samples. The percent voids are those determined in the laboratory from a remolded sample and from an undisturbed sample of the pavement. The job tests made by the inspector at the time the pavement was constructed, and the original job mix design are also given. These results show that there is good agreement between the tests made on the pavement at the time of construction and tests made after several years' service. These tests indicate that there has been practically no increase in the density of the pavement between the time it was placed and the present.

It should be remembered that all these samples were taken in the center of the traffic lane between the wheelpaths. It is probable that densification has taken place in the wheelpaths. All of these samples were taken in the same relative location for the sake of uniformity. The increase in Marshall stability is attributable to the decrease in ductility and penetration of the asphalt cement. All of these samples had good workability when heated for remolding. The samples were remolded by being placed in an oven at 240 F for a period of about 1½ hr.

Table 5 shows a comparison of screen analyses made during constuction and at the present time. The asphalt cement content and the percent voids determined on these samples are also indicated. In many cases agreement between construction analyses and the present analyses is as good as that between the analyses made at different times during the construction. Little or no degradation of aggregate is apparent from these tests.

The present void content of the pavement indicates that there is little or no densification in the pavement between the wheelpaths. The tentative conclusion here is that the flow may contribute to this, as the flow on all of these jobs was 10 or below at the time they were placed. This lack of densification does not seem to impair the quality of the pavement in any way. The riding surface remains excellent.

CONCLUSIONS

Two tentative conclusions are drawn from the results of these tests.

1. An early loss of ductility and penetration causes surface cracking of the pavement.

2. Very little densification occurs in the pavement outside the wheelpaths where the asphaltic concrete mix has low flow.

REFERENCE

1. ABSON, G., AND BURTON, C., "The Use of Chlorinated Solvents in the Abson Recovery Method." *Proc., Assn. Asphalt Paving Tech.,* 29:246 (1960).

Effect of an Inhibitor on the Corrosion of Autobody Steel by De-Icing Salt

G. O. GRANT, *Commissioner of Roads, Municipality of Metropolitan Toronto, Canada*

The Council of Metropolitan Toronto became increasingly concerned with the corrosion of automobiles and the claims of individuals and organizations, such as labor groups and taxpayers, that it was caused by the salt used for de-icing roads during the winter months. Various claims were made by suppliers of inhibitors that this corrosion could be reduced to a reasonable amount. A review of a number of studies made on salt corrosion indicated a lack of complete data, and a research program was authorized by the Council to study this subject.

The study was undertaken as a joint venture with the Ontario Research Foundation, the Metropolitan Toronto Department of Roads, and the Ontario Department of Highway. Corrosion Service, a company acting as a consultant on corrosion problems, was also used in this study. Testing was done by means of mechanical rigs operating daily with test panels attached to the rigs, and the total project was designed to simulate, as closely as possible, the actual conditions existing on roads. It was possible, however, to accelerate the program by running the equipment daily and applying snow and chemicals to attain the final results.

The test coupons were removed periodically and weighed, to determine the loss of metal due to corrosion. The results indicated that (a) a reduction in loss of metal was obtained by the use of an inhibitor; (b) the test panels shaped to more closely resemble automobile body conditions showed the greater amount of corrosion; (c) the higher the corrosion rate, the lower the efficiency of the inhibitor; (d) the greatest corrosion caused by salt solution occurs when the vehicle is stored in a heated enclosure; (e) corrosion due to de-icing salt is dependent on time and not related to mileage traveled; and (f) the inhibitor and the de-icing salt tend to segregate, so a method of mixing to prevent this would be necessary.

On the basis of the results, a report was presented to the Council of Metropolitan Toronto, to the effect that there is insufficient justification for an expenditure of this nature, as the use of an inhibitor will not eliminate corrosion, but only retard that portion caused by salting, and the amount of retardation does not justify the cost. Also that the problems of corrosion in motor vehicles can be handled more economically and more thoroughly by the automobile manufacturers.

• THE ROADS and Traffic Committee of Metropolitan Toronto Council, in October 1960, approved a research project on the effect of an inhibitor on the corrosion of automobile body steel by de-icing salts used for snow removal on city thoroughfares. The program was planned and conducted by the Ontario Research Foundation, in association with the Metropolitan Department of Roads and the Ontario Department of Highways.

The problem of autobody corrosion has been recognized for many years and has been of considerable concern to urban and suburban car owners who must bear the burden of this form of attrition as it affects their vehicles. The use of sodium or calcium chloride to clear the roads of snow and to prevent road icing is a necessary practice to keep traffic moving and enable a large urban community to function efficiently during the winter months. It is most unlikely that despite the corrosion action of de-icing salts many city dwellers would favor the abandonment of its use for this purpose. Consequently, it is believed de-icing salts are here to stay, and means will be sought to minimize the deteriorating effect of de-icing salts on motor vehicles both by the auto manufacturers through design and application of better body coatings, and by the road authorities through constant improvement of methods to remove snow and ice with substances less corrosive than these salts, or through the use of additives to minimize their corrosive effects.

The corrosion occurring on autobodies has been studied by Holzwarth (*1*) of General Motors Laboratories with particular emphasis on corrosion in sheltered areas where a nonprotective iron oxide forms and corrosion is accelerated. Most of the salt corrosion testing involving automobiles has employed flat coupons located in well-exposed areas on the vehicle. In view of Holzwarth's work, the test procedure adopted in this program included a crevice-type coupon to simulate sheltered conditions. The crevice coupon was also paired with a flat coupon so that the corrosion loss in a given area could be compared.

In some large cities in the United States, the addition of inhibitors to de-icing salt has been and is being practiced. Also a special committee of the National Association of Corrosion Engineers has been established to study and correlate the problem of corrosion by de-icing salts. However, very few data are available that would provide a quantitative advantage for the use of inhibitors to de-icing salts, and the report of the NACE Committee T-4D, March 1959, indicated that the effect of adding inhibitors to de-icing salts was quite inconclusive. Examination of data from cities using an inhibitor (such as Rochester, Detroit, and Akron) did not provide convincing evidence in support of inhibitors, although Rochester's experience favors the continued use of an inhibitor though the data are not quantitatively substantiative.

In Metropolitan Toronto where some 80,000 tons of salt are used in a winter season, the use of an inhibitor presents a substantial item of expense. Consequently, because supporting data showing an inhibitor would be beneficial were lacking, this program was initiated to obtain quantitative evidence of corrosion of autobody steel in various environments.

The original plan was to employ vehicles fitted with corrosion panels, operating on portions of highways under construction that would not be used by normal traffic during the winter season. This plan was abandoned in favor of special rigs designed to operate on a circular asphalt pad under more controlled conditions. This decision proved wise as the low precipitation pattern, which began in the fall, continued into the winter months and would have made road operation difficult. By using the rigs, a snowfall could be created daily at will by accumulating at the test site truck loads of clean snow whenever sufficient snow occurred in the area. Later in the season, ice scrapings from the various operating skating rinks were secured, but finally in April city water was used on the rig pathways.

CONCLUSIONS

1. Within the limits of the field tests carried out, it has been shown that the sodium hexametaphosphate inhibitor will definitely reduce corrosion of autobody steel due to de-icing salt solutions, the reduction of corrosion ranging from 10.5 to 77 percent if all samples are considered. The calculated average reduction of corrosion losses for all samples regardless of type was 55.7 percent.
2. Coupon shape had a major effect on corrosion in unsalted solutions and inhibited salt solutions. Corrosion of V- or crevice-type coupons was more severe than for the flat coupons, and this was due primarily to the formation and retention of dirt poultices in the V-coupons. It is felt that the V-coupon more closely represents automobile body conditions where sheltered corrosion occurs.
3. Apparently the higher the corrosion rate, the lower the efficiency of the inhibitor in general and vice versa.
4. Both rig and automobile test coupons showed that the greatest corrosion rate caused by salt solutions occurs when the car is stored in a heated enclosure or garage each night. The efficiency of the inhibitor is also erratic under these conditions and may be negligible.
5. Coupons placed on automobiles in normal use showed that corrosion due to de-icing salt is dependent on time and not related to mileage traveled.
6. Dry mixing of the fine inhibitor with coarse salt in a dry blender showed that there is a tendency for the inhibitor to segregate. If inhibitor is to be used on roads, a mixing method will be required to assure even mixing of salt and inhibitor at the point of application.
7. The only laboratory test that proved useful in evaluating the inhibiting effect was the intermittent or dip test. This test would be useful for comparing different inhibitor efficiencies, but the results obtained would not obviate the need for conducting field trials.

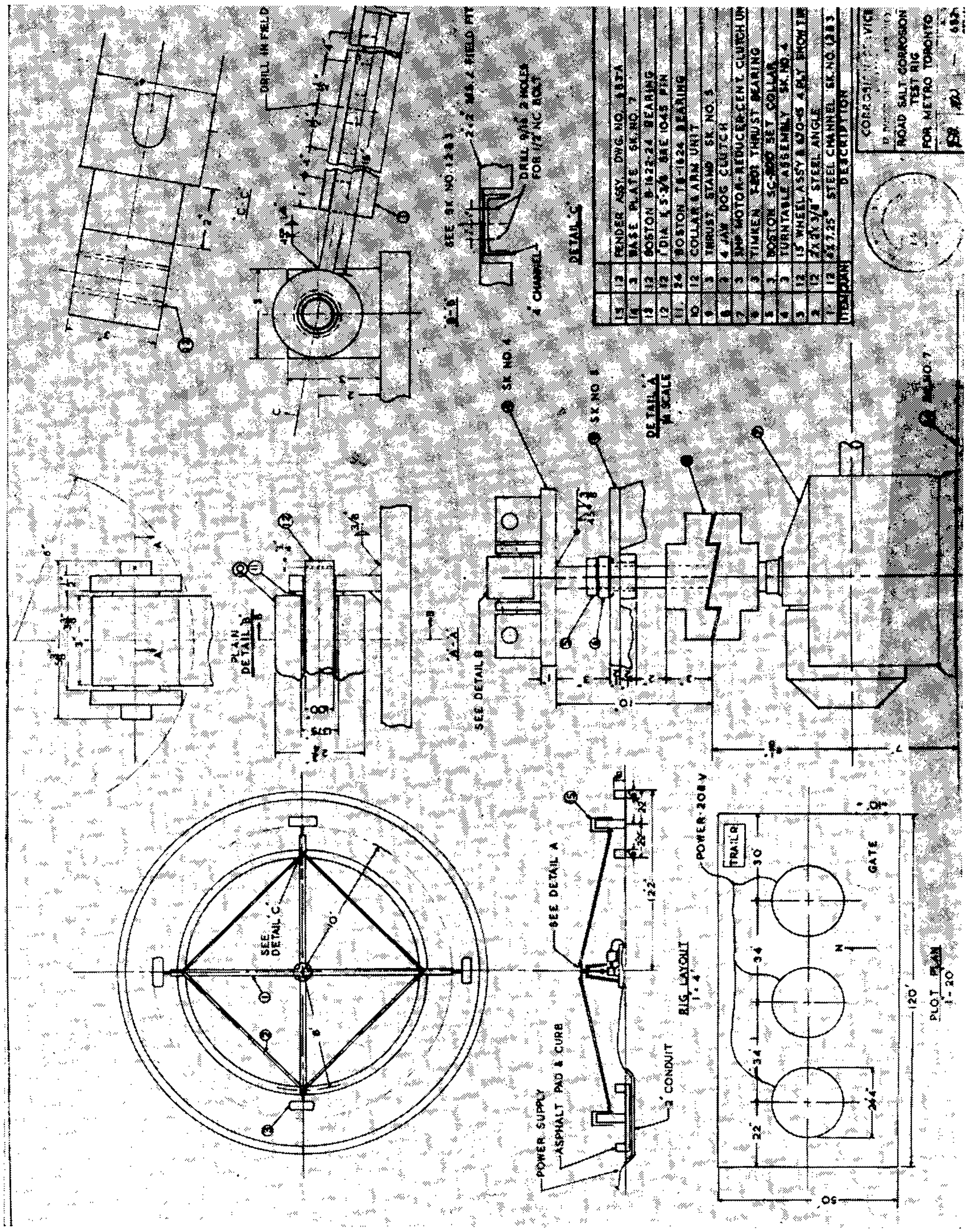
In assessing the conclusions the following should be noted:

1. The rig tests represented accelerated corrosion conditions and would compare to an extreme winter season when de-icing salt was used regularly throughout the winter.
2. The test period was not long enough to produce any major pitting corrosion, and therefore the efficiency of the inhibitor at this stage of corrosion was not illustrated.
3. All test coupons used were clean autobody steel, whereas in the great majority of automobiles, body surfaces would be dirty and in some cases corroded.

DESIGN AND OPERATION OF TEST RIGS

The corrosion program was initiated late in the season, but the design and fabrication of the test rigs were executed to enable the test to start on February 1, 1961.

Three rigs were built to operate under three conditions: unsalted, salted, and inhibited salt. Figure 1 shows the design drawings of the rigs, and Figures 2 and 3 show the rigs operating on circular asphalt pads. The rigs were arranged with unsalted rig A to the west, salted rig B in the middle, and the inhibited salt rig on the east, so that the prevailing westerly winds would not blow spray


ROAD SALT CORROSION
TEST RIG
FOR METRO TORONTO
DETAIL "C"
DETAIL "A"
SEE DETAIL B
SEE DETAIL A
SEE DETAIL C
POWER SUPPLY
ASPHALT PAD & CURB
3" CONDUIT
RIG LAYOUT
POWER 208 V
TRAILR
GATE
PLOT PLAN
DESCRIPTION

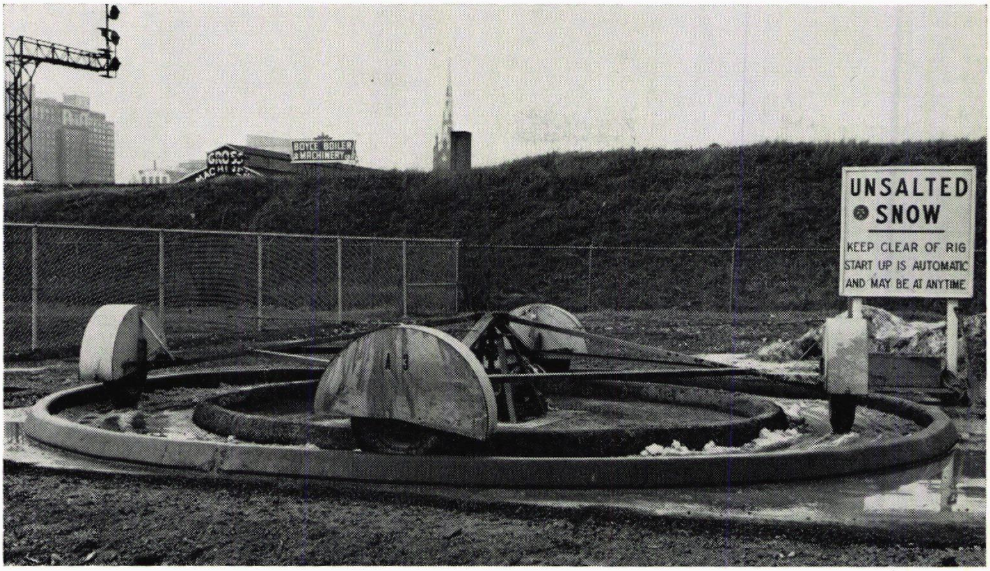


Figure 2. Rig operating on circular curbed asphalt pathway.

Figure 3. General view of the three rigs.

into the unsalted rig. Early in the test it was necessary to place polyethylene shields around the salted and inhibited rigs to prevent splashing and to retain the fluids in their respective tracks. Due to the centrifugal spray pattern caused by the circular motion of the test rig, the fluid splashed out of the wheelpath. In order to retain the fluids, the proposed distance of 50 mi per day was reduced to 25 by operating for 15-min periods. This, in addition to the plastic shield, made the operation more satisfactory, and the reduced time was less damaging to the rigs. It was not possible to reduce the speed without a drastic design change, although a lower speed (around 15 mph) would have been preferable.

This design of corrosion rig functioned quite well throughout the test, and equipment problems arising were not too serious. Repairs were quickly performed and the rigs were promptly restored to operation.

To obtain conditions in the region of 40 F, comparable to those expected in garages heated directly, or by heat indirectly received through the dwelling walls as in an attached garage,

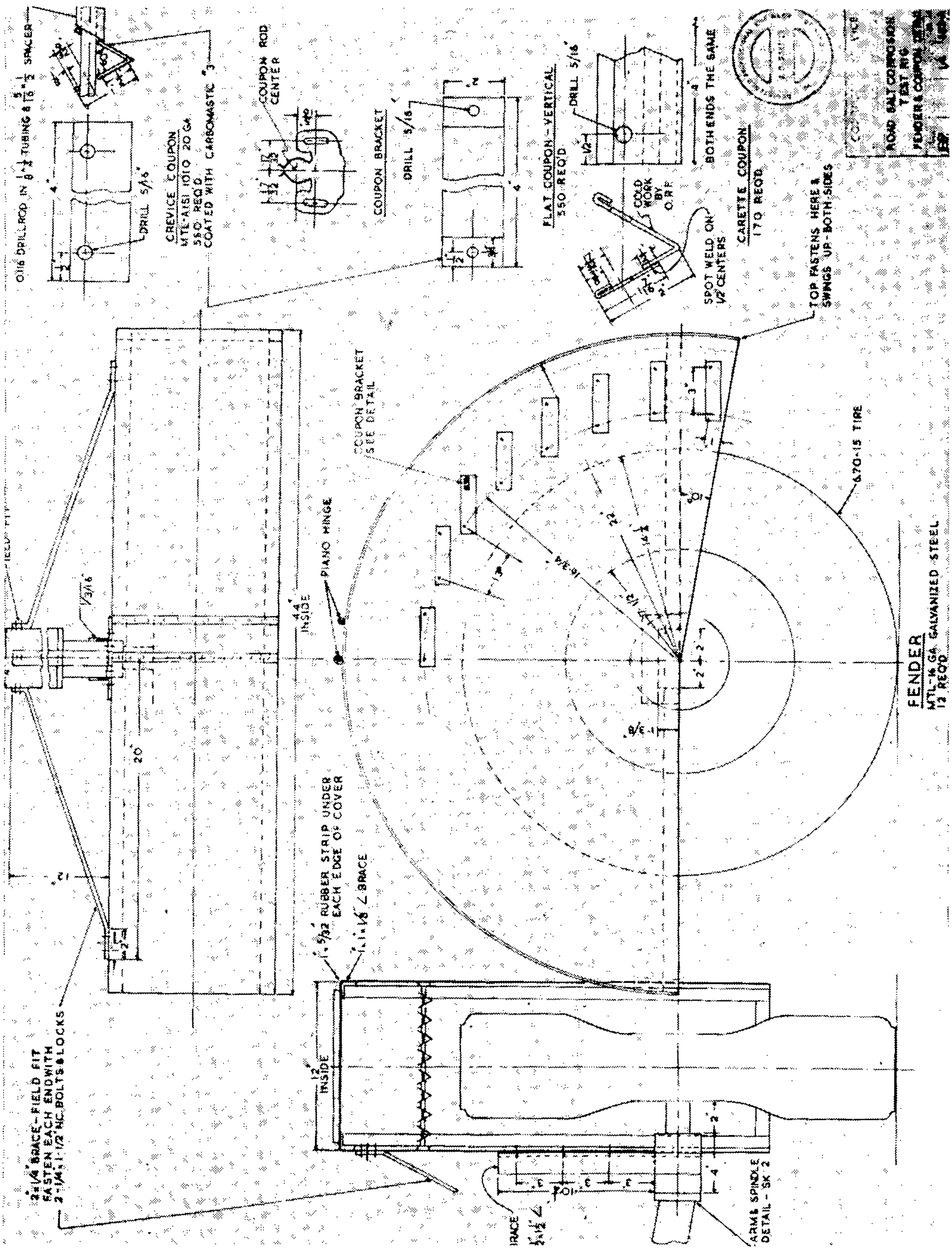


Figure 4. Design drawings of fender and coupons.

No. 4 fender of each rig was enclosed in a plastic envelope. During the down period, the fender was heated by a thermostatically controlled calrod unit. The heating of the No. 4 fenders was not begun at the start of the test, but was put in operation after a period of three weeks on February 21, 1961.

FABRICATION AND PREPARATION OF CORROSION PANELS

The corrosion panels were made from autobody steel. The steel was aluminum killed and conformed to the following chemical analysis: C, 0.08 percent; Mn, 0.40 percent; P, 0.010 percent; S, 0.10 percent; Si, 0.018 percent; Cr, nil; Ni, nil; and Cu, 0.04 percent. The steel was in the cold-rolled, annealed condition with a hardness of Rockwell B-39-43. The metallographic structure consisted of fine grain ferrite in which the carbides were in the spheroidized condition. The sheet tested in the direction of rolling possessed the following physical properties, characteristic of deep drawing quality: tensile strength, 45,200 psi; yield strength, 23,900 psi; and elongation in 2 in., 43.0 percent.

Three types of corrosion coupons were selected—the V-type to approximate crevice conditions, the flat type to represent plain surfaces, and the caret type which was to be used only in a qualitative manner and not for weight loss measurements. The caret type was made similar to the V-type with the exception that 180° cold-worked beads were formed on the two edges; one side was overlapped and spot welded, and the other side was cold worked along the side by bending and straightening twice through a 90° angle increasing the hardness of the cold-worked zone to Rockwell B 59-61. Figure 4 shows the design drawings of the various coupons and of the fender inside which the coupons were fitted. Figure 5 shows a typical coupon rack composed of both V, flat, and caret coupons, mounted on a polyethylene-coated rod and insulated from one another with suitable polyethylene spacers. Figure 6 shows the arrangement of the rods in the fender, the inside region to the right.

The coupons were sheared from the sheet stock and fabricated to the required shape. Holes were punched in the coupons through which the polyethylene-coated rods passed. All test coupons were stamped according to the numbering system shown in Appendices A and B.

The V and flat coupons were sandblasted to ensure that the plastic coating applied to the panels would adhere, and both coupons were masked with suitable tape before plastic spraying. The V-coupons were masked in the crevice only covering an area of 26.3 sq cm. The flat coupons were masked in the midsection

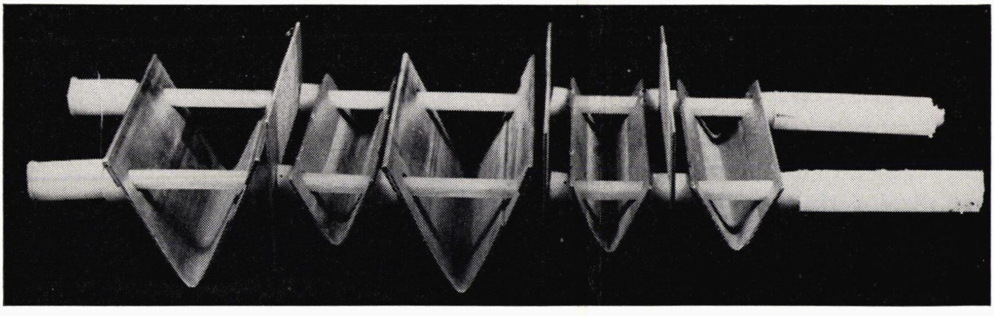


Figure 5. Set of coupons on polyethylene-coated rods.

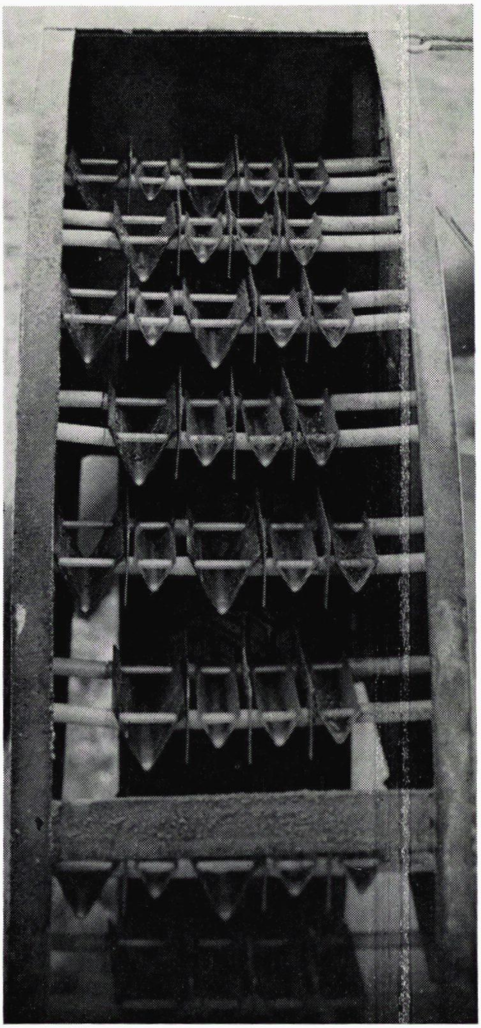


Figure 6. Fender and coupon rods. Inside location to right was sampled from top to bottom position.

covering an area of 64.0 sq cm. The caret coupons were not sandblasted or plastic coated and were placed on the rods following vapor degreasing in trichlorethylene. The V and flat coupons were sprayed with a thermal-setting, clear epoxy resin and baked at 200 F for 15 min between coats. Figures 7, 8 and 9 show the shape and appearance of the three types of corrosion coupons used. It is emphasized that the exposed surface of all the weight-loss coupons was sandblasted, hence this surface would be more chemically active than the original cold-rolled surface. Plastic coating of the V-coupons was done to concentrate the crevice effect, whereas, in the case of the flat coupons, plastic coating eliminated crevice effects in the region of the punched holes.

The epoxy-coated coupons were stripped of masking tape and carefully cleaned in alcohol, dried, and weighed. The number of coupons required to load the rods and to take care of replacement samples is shown in Appendices A and B, which tabulate how the coupons were placed in rigs A, B, and C, and how they were removed at the end of the various periods. As coupons were removed, others were put on. This permitted obtaining data for the initial set of coupons, and also for those coupons placed on, starting two weeks later. For the program 1,300 coupons were prepared for the rigs and the sets of coupons placed on 10 automobiles traveling over the salted city streets for the period February to May 1961.

DESCRIPTION OF INHIBITOR

Banox was chosen as the inhibitor for the program chiefly because it had been used by several cities in the United States in their studies of corrosion inhibitors added to de-icing salts. No attempt was made to evaluate other types of inhibitors, and the scope of this program did not include research to develop other types.

As previously mentioned, data pertaining to the use of inhibitors in road salt are not readily available. In the case of Rochester, where an inhibitor is used, the data were compared with Buffalo and Syracuse where an inhibitor is not used. These data were published in a report by NACE (*2*) and the corrosion rates in

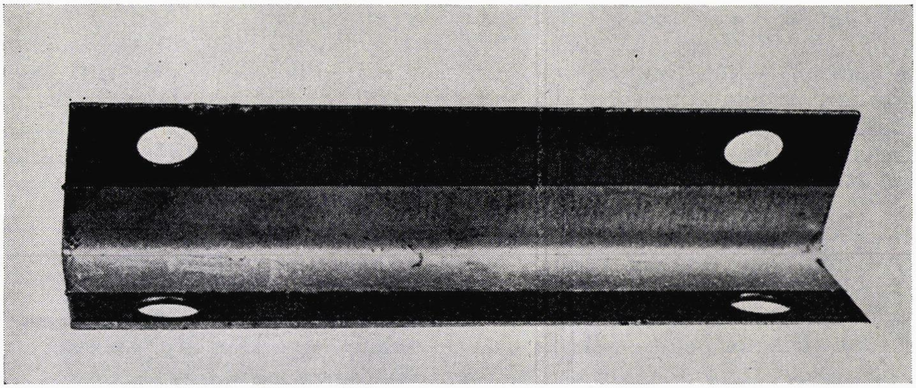


Figure 7. V-type coupon for crevice conditions. Bare portion sand blasted; full size.

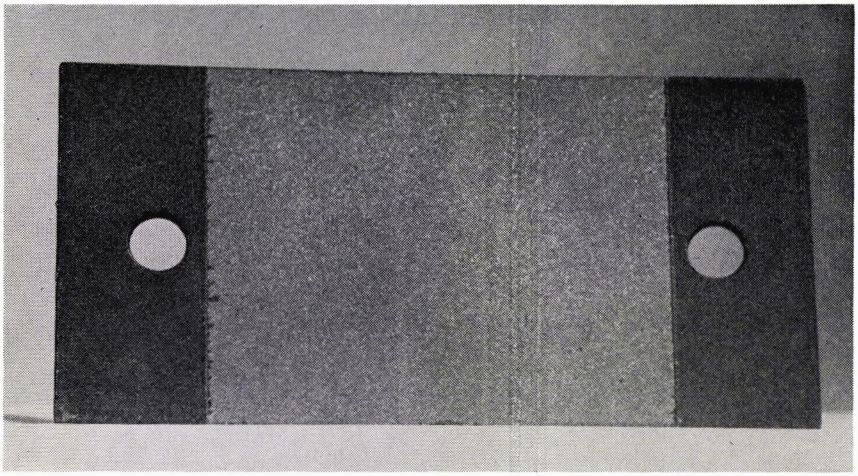


Figure 8. Flat-type coupon. Bare portion sand blasted; full size.

TABLE 1

City	Condition	Corrosion Rate Uncorrected (mpy)	Average Weight Loss ($mg/dm^2/day$)
Rochester	Inhibited	4.7 (spread 1.8—7.8)	25.7
Syracuse	Uninhibited	4.1 (spread 1.4—9.1)	22.4
Buffalo	Uninhibited	4.7 (spread 2.8—7.9)	25.7

mills per year (mpy) only are shown (Table 1), with the corresponding average weight loss in brackets.

Table 2 shows the results of field tests in Cuyahoga County, Ohio (*3*), where polyphosphate inhibitors were used.

TABLE 2

Salt	Rate of Corrosion ($mg/dm^2/day$)	
	Range	Average
Without inhibitor	8.0—14.4	10.2
With inhibitor	5.0— 7.1	6.2

Most of the field experience employing inhibitors made use of a polyphosphate of which Banox is typical. Banox is chiefly composed of sodium hexametaphosphate with additives such as nitrites, compounds of zinc, and a small amount of calcium chloride. The inhibitor is believed to be of the cathodic type, forming a protective film on the metal surface reducing the strength of current that occurs during the action of corrosion. However, it is not proposed to discuss the subject of inhibitors here, as this subject has been well covered by Evan (*4*) and Putilova et al. (*5*). The inhibiting characteristics of sodium hexametaphosphate have been studied by Hatch (*6, 7*) in some detail.

The solubility of Banox was not examined thoroughly, but it was observed in the laboratory that, though most of the inhibitor dissolved readily in saline solutions, some of the crystals went into solution slowly at room temperature. The solution rate determination at various temperatures, particularly in the region of 32 F would be of value. The solubility factor, being related to the concentration of the inhibitor in the saline solutions, directly influences the ability of the inhibitor to retard corrosion rates.

The salt content desired in rigs B and C was approximately 5 percent. This value varied considerably, but this variation was more or less a parallel condition in both rigs. Solutions taken from various city thoroughfares following snow storms also showed a considerable degree of variation of salinity. In the case of the inhibited salt, the Banox addition was 2 percent of the salt, and the two materials were dry mixed in a twin dry blender. In this way, a uniform mixture was secured. However, it was observed that vibration caused a separation of the salt and inhibitor, which would be expected because of the wide difference in the particle size of the coarse salt and

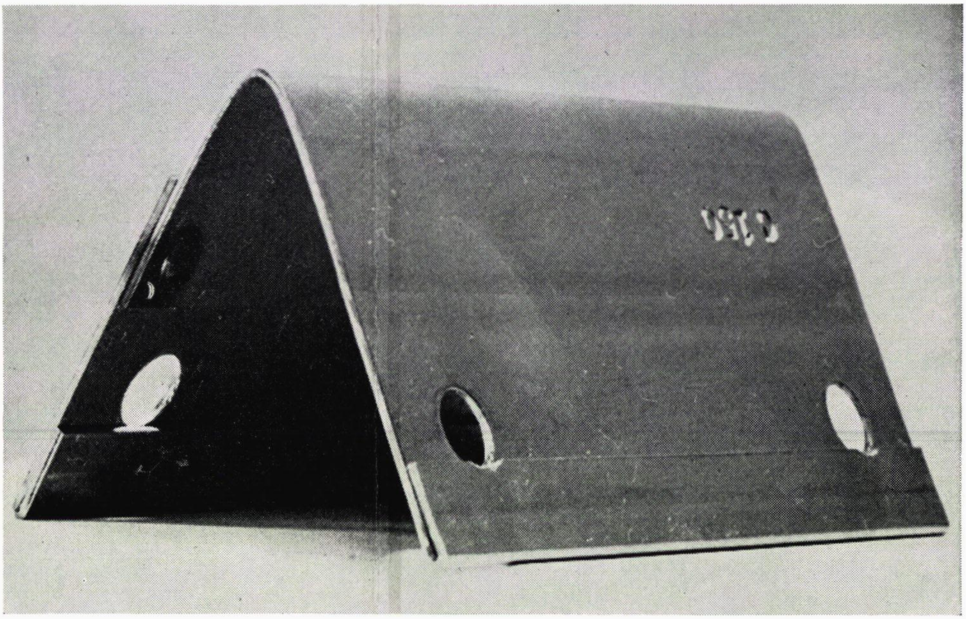


Figure 9. Caret-type coupon, not plastic coated. Surface cold rolled; full size.

fine Banox. The Banox consisted of coarse and fine particles, and it was noted that the fine particles tended to adhere to the salt crystals following the blending operation.

EXPERIMENTAL PROCEDURE

The following outline of experimental procedures describes how the coupons were dealt with at the rigs, as well as how they were cleaned after removal. Certain coupons were assigned to special racks for placement on cars, and others were used for atmospheric corrosion tests. Road and rig solutions were checked for salt content and pH, and in the case of the inhibited salt, the amount of Banox was also determined. Laboratory corrosion tests on coupons exposed to saline and inhibited solutions were also carried out under different conditions to determine the effect of the inhibitor.

Test Rig Procedures

All the fenders were fitted with rods carrying weighed, numbered, and cleaned coupons. Appendix A shows the coupon layout for each rig. On February 1, 1961, during the coldest period of the winter, operations were begun.

It was decided to remove specimens for weight loss every seven days. However, it was found that after the first week very little corrosion occurred, although both rigs B and C contained solution, while rig A contained dry snow. To all three rigs some clay soil was added to simulate road dirt. After the first week, which showed very little corrosion activity, it was decided to defer coupon removal until the fourteenth day. The weather moderated during the second week and corrosion activity increased quite noticeably. From the fourteenth day, coupons were then removed every consecutive seven days until the termination of the test.

Although the coupons were placed on in a consecutive manner, the method of removal was arranged to randomize the coupon selection. Starting on the side of the fender facing the motor drive and corresponding to rods 1—2, the method of removal was based on the latin square with A in the top position and D in the bottom position. One V and one flat coupon were taken from each wheel of each rig according to the removal plan of Appendix B for weight-loss determinations. To these positions additional V and flat coupons were placed on racks for removal at the end of the test, thus providing corrosion data under conditions differing from those experienced by the original coupons.

The conditions of the test involved running the vehicles for 15-min periods intermittently throughout the day for four periods and traveling a distance of 25 mi. In this way, the conditions of the test were held reasonably uniform during the run. Occasionally during a snow storm or heavy rain, this cycle could not be maintained. Time was also lost when the rigs failed mechanically and had to be shut down for repairs. Of the total scheduled 91 hr, the rigs operated some 78 hr, or 86 percent of the scheduled time.

Cleaning of Coupons

Each removal period required cleaning of four V and four flat samples from each rig. All coupons were in pairs so that the corrosion of the V and flat coupons could be compared for the same location. The coupons were cleaned mechanically of all loose deposit and as much oxide as possible removed using a metal scraper. All coupons were washed, scrubbed, and dried, and the plastic was wiped with carbon tetrachloride to remove road tar. The coupons were finally cleaned by chemically dissolving the remaining oxide in a cold 6 N

HCl solution containing 2 percent by volume of rhodine 60 inhibitor (*8*), followed by thorough washing and drying with alcohol. Before weighing, the coupons were placed in a dessicator overnight to remove all traces of moisture. Blanks were cleaned for various periods of time in the inhibited acid, and the metal loss was considered negligible being within the limits of error for this method.

During cleaning of the coupons over the period of the test, the ease of cleaning was as follows: unsalted rig, most difficult; salted rig, most easily cleaned; inhibited rig, readily cleaned under the dirt poultice, more difficult where no poultice formed. It was also observed that the coupons removed from the No. 4 fender equipped with a heater to maintain 40 F during the nonoperating period were all more difficult to clean than the coupons from the unheated fenders.

The accumulation of dirt on the coupons built up during the test, and at the end of 21 days (Figs. 10 and 11) the poultice effect was well established, particularly in the case of rigs B and C. In rig A, due to lack of fluids during the cold periods the poultice build-up was slower. Toward the end of the test period, the dirt had bridged across the top of the coupons filling the V-type coupons almost completely. The flat coupons, which were in a vertical position, tended to be cleaner on the lower side due to washing by the wheel spray.

The weight losses found for the various coupons were determined in terms of milligrams per square decimeter, and the accumulated losses were later plotted to show the various trends encountered in each rig.

Corrosion Coupons on Cars

To obtain corrosion losses due to car operation on city streets, ten cars were selected on which were mounted coupon racks similar to those placed in the test rigs. These racks were mounted near the rear right wheels to be on the curb side of the car. In the case of eight cars, the location was well exposed to the spray from the wheels. In the case of two cars, it was found that the rack was not as exposed to road splash as the other eight.

A record of the mileage and conditions of the road was kept for each car, together with garage conditions. From time to time, coupons were removed from various cars during the test period and terminal samples were removed at the end of the run. The coupons were cleaned as outlined previously, and the corrosion rates in milligrams per square decimeter per day were determined.

Atmosphere Tests

To determine the corrosion losses due to atmospheric effects, a group of coupons were placed on the roof of a small building at the site. Flat coupons were used for this test, and only one side was exposed, the under side being coated entirely with epoxy resin. Most of the coupons were exposed with one flat side facing south inclined at an angle of 40°. Several flat coupons were exposed on edge with both sides to the weather in a manner similar to the way in which the flat coupons were mounted in the fenders of the rigs and under the cars.

Coupons were removed from the test site from time to time, and the weight losses due to industrial atmosphere were then determined.

Solutions from Rigs and City Streets

For purposes of checking, solutions were taken from rigs B and C from time to time and analyzed for salinity and pH, and, in the case of rig C, the amount of Banox present was also determined. Following snow storms in the city, solutions were collected from various locations and analyzed

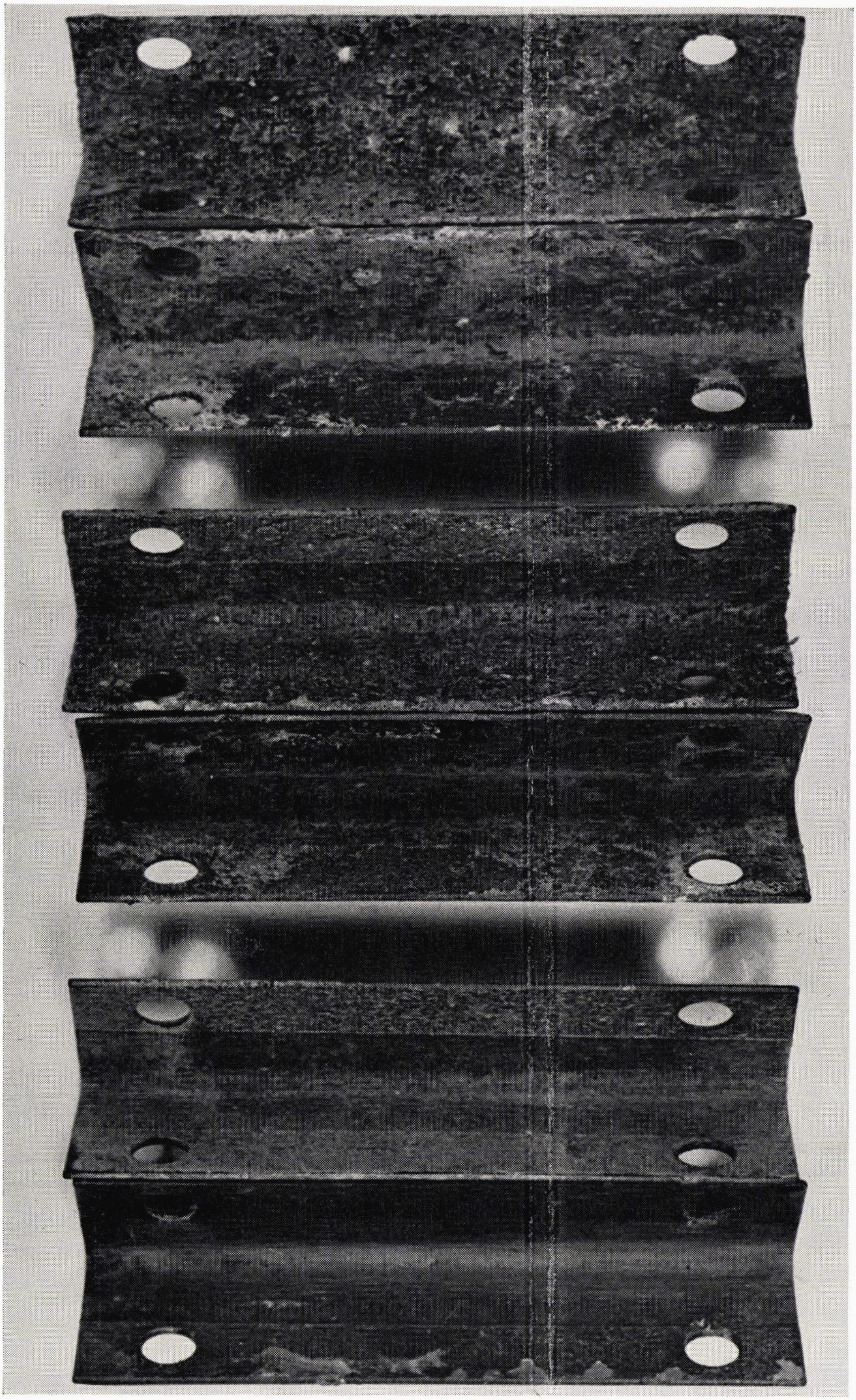


Figure 10. Poultice build-up at end of 21 days, V-type coupons; (left) no salt, (center) salted, (right) salt with inhibitor.

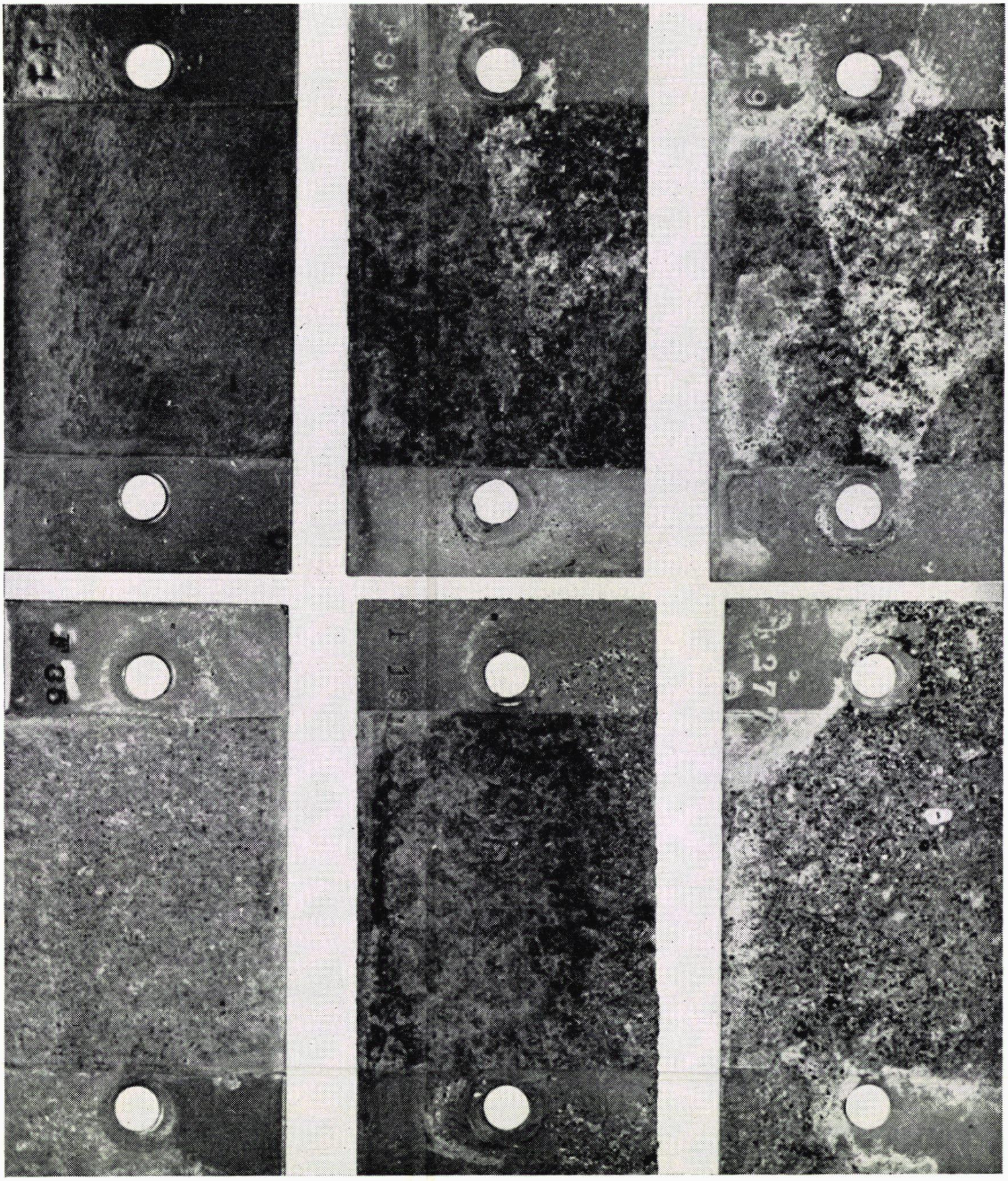


Figure 11. Poultice build-up at end of 21 days, flat-type coupons; (left) no salt, (center) salted, (right) salt with inhibitor.

for salt content and pH to determine the variation and concentration in several areas of the city and on the throughway.

From the various locations, about a quart of solution was collected in such a manner as to be representative of the road or rig mixture. In the case of road samples, to obtain a truly representative sample was not possible. The rig samples were more truly representative of conditions, and when the salinity was found to be high, corrections were made immediately with the aid of the conductivity meter.

Laboratory Corrosion Tests

To examine the inhibiting effect of Banox in saline solution, laboratory tests were conducted by three procedures: (a) salt spray test, (b) modified Corrodcote test, and (c) intermittent immersion test.

Slush samples taken from Toronto roads were submitted for analysis for sodium chloride content and for pH determinations. Samples of slush from test rigs were submitted for sodium chloride and pH determinations, and, where applicable, for determination of inhibitor content. Several samples of deposits removed from test panel were submitted for determination of sodium chloride and pH determination.

DISCUSSION OF RESULTS

The object of this corrosion test program was to evaluate quantitatively the effect of adding an inhibitor to de-icing salt on the corrosion of autobody steel under conditions similar to those encountered on salted streets. Although the test rig operation cannot be considered entirely comparable to street conditions, this method of testing was considered to be sufficiently valid to yield data of a significant nature.

The following discussion of the test results will deal first with the rig operation, followed by examination of data obtained from coupons mounted on cars, atmospheric corrosion tests, laboratory corrosion tests data, and solution analyses.

Corrosion Coupons on Rigs

The weight losses for each removal period of the original coupons placed on February 1, 1961, for each rig are shown in Figure 12. The values up to February 21 represent the average value of four V-type coupons, and four flat coupons taken off in pairs according to the removal program in Appendix B. After February 21 the values for wheel 4 of each rig were excluded as this wheel was heated to 40 F after this date. This was done because the heating reduced the corrosion slightly in rig A (unsalted), increased corrosion very markedly in rig B (salted), and produced erratic corrosion in rig C (inhibited).

The corrosion of the salted rig is definitely more severe than the other two rigs, and at the end of the 91-day period, the rates were still ascending steeply. The curves for each coupon type are somewhat erratic, and one crosses the other at several points. By and large, the corrosion rate for both the V-coupon and the flat coupons does not differ to any great extent, although the poultice effect in the case of the V-coupon was much more severe than the flat coupon. It was previously pointed out that coupons from this rig were easily cleaned. The rust formation was loosely adhering and porous, consequently the saline solutions were able to penetrate the layer of dirt and rust to attack the underlying metal. Even after 91 days, the rate of attack appears to be undiminished.

In the unsalted rig A, the corrosion losses show a divergence as the test proceeds with the V-coupon showing a more severe loss than the flat coupons. Towards the end of the 91-day

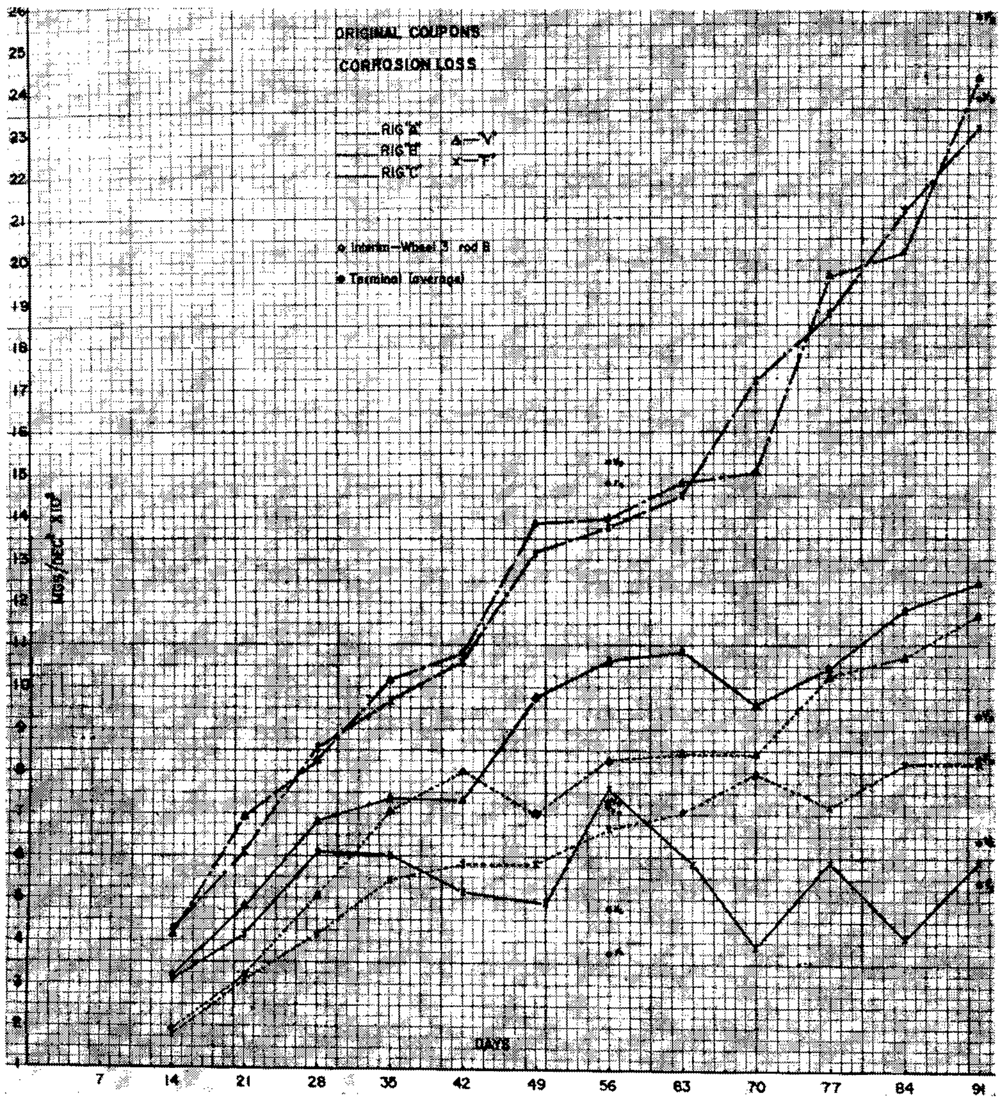


Figure 12. Corrosion losses of original coupons in the three rigs including interim losses from wheel 3 rod 8 and losses of terminal coupons.

period, the tendency was to approach a constant rate of attack. These curves show that the crevice type of surface represented by the V-coupon corrodes at a slightly higher rate than a plain surface represented by the flat coupon. The accumulation of dirt in the V-coupon acting as a poultice would contribute to the increase in corrosion in this instance.

In rig C, inhibited salt, the two curves are somewhat more erratic showing greater divergence. At the end of 91 days, there is also a tendency for the two rates to reach a constant value. The losses for the

V-coupons are above those for rig A, and the losses for the flat coupons reach a point below those for rig A. The inhibitor is tending to reduce corrosion losses from the salted level to the unsalted level. In the case of the flat coupons, the inhibited rig shows that the losses are lowered below those for the unsalted rig.

The results found in rig C showed considerable spread, no doubt due to some variation in the concentration of Banox in the saline solution in the rig, and also due to the effect of poulticing by the added soil in the rig pathway. The flat coupons particularly showed that where the top portions were poulticed, corrosion rates were high as deduced from the etched texture of this area of the coupon. The bottom faces being less poulticed were coated with a tight uniform oxide difficult to remove and under which the metal remained relatively smooth. Consequently, on the flat coupons the corrosion losses represent an average of areas where corrosion rates were high and low, a condition impossible to control, but which is very representative of conditions occurring on the under parts of an automobile body.

At the termination of the test, many of the original coupons were still in place. The losses for the terminal coupons were determined and the average values obtained. The terminal coupons from rig A were partially assessed. Terminal coupons from one wheel from rig B were examined, and all terminal coupons from rig C were examined. The average values are shown on the respective curves in Figure 12 as terminals. In rig A, they fall in the general region between the two types of coupons. In rig B, the terminals are reversed and above the position of their respective curves. In rig C, the greatest shift occurred in the V-coupon while the location of the terminal results for the flat coupon lies close to its respective curve.

The removal plan adopted selected coupons from the side of the fender facing inwards and from positions top to bottom. Examination of the data indicated that within the fender corrosion rates appeared high at the top inside, decreasing in the central region, and rising at the bottom inside. Also the rates appeared higher on the inside regions, decreasing midway across the fender, and flattening off toward the outside of the fender. Although the general results would remain the same had sampling included random removal across the fender as well as in the vertical direction, the curves for the three rigs would have altered their shape slightly in the case of rigs A and B, but more so in the case of rig C. This effect was attributed to the spray pattern. The wheels moving in a circular path would be unlike an automobile wheel because the spray would tend to angle vertically toward the outside of the fender. The spray distribution as visualized inside the rig fender would be less on the inside, greater in the central region, and maximum at the bottom outside. The spray pattern would affect rig C the most if proper coating of the coupons by the inhibited salt solution was not effective. However, examination at the site indicated that, although the above spray pattern existed, the fenders appeared to have been well covered over most of their internal surfaces.

This effect attributed to position was examined at the end of 56 days when the bottom rod was removed from No. 3 wheel of each rig. The weight loss data at this time showed samples from both rigs A and B to be on their respective curves (Fig. 12), whereas in the case of samples from rig C the weight losses were well below rig A and considerably below the curve for rig C. This position effect points out that the inhibited salt solution must have free access to the areas to be protected in order to be effective in reducing corrosion.

The corrosion rates in milligrams per square decimeter per day (mdd) over the period of the test are shown in Figure 13. Included on the chart are the corrosion rates for No. 4 wheel which had been heated to 40 F from February 21 to May 2. From this chart the corrosion rates for the three rigs can be compared; these rates pertain only to the original coupons placed on the rigs February 1. The interesting aspect of these data is the increase in corrosion rate of the heated wheel on rig B where the chemical activity of the salt solutions has been increased by raising the temperature of this fender when not operating. Heating tends to reduce the rate of the unsalted rig, whereas the condition in the inhibited rig C is somewhat erratic. The V-coupons in rig C show wide variation possibly related to the poultice effect reducing the efficiency of the inhibiting process.

It was pointed out earlier that the poultice effect appeared to be most noticeable on the flat coupons of rig C. Under the poultice area at the top, the corrosion rate was higher as judged by the etching effect. On the lower surface a tighter oxide coating was evident and here the corrosion rate appeared lower. Figures 14, 15, and 16 show photomacrographs of the surface of flat panels from each rig after the full period of 91 days. The surface of the sample from rig A (Fig. 14) is still relatively smooth. In this case pit-type corrosion is developing at the dark spots but the surface is not severely etched. Figure 15 showing a sample from rig B indicates severe etching more or less uniformly over the entire surface. Figure 16 (rig C) shows some area of etching at the top whereas at the bottom there are smooth patches. The rough patches were located under the dirt poultice and here the scale was very similar to that formed on the coupons from rig B. The flat coupons from rig C showed the greatest variation of attack ranging in appearance similar to Figures 14 through 16.

The appearance of Figure 14 (rig A) indicates the onset of pit-type corrosion. Although not too well established at the end of 91 days it would be expected that, had the test continued, pit-type corrosion would be more evident a few weeks later.

Analyses were also made of the weight losses and corrosion rates of the replacement coupons over most of the test period. It will be recalled that these coupons were placed on the rods two weeks following the start of the test on February 14. The first coupons placed on remained on until May 2. Thus the replacement coupons have different exposure conditions than the original coupons, and corrosion conditions were much more active on February 14 than they were on February 1, when the temperature was around 0 F. Although the replacement coupons occupied the same locations as the originals, they were exposed in reverse time to the originals.

The replacement coupon weight losses for each rig appear in Figure 17, and it can be readily seen that, although the curve for each rig occupies a similar location to those in Figure 12, the losses are somewhat higher, and there is little or no tendency to level off. Rig C, the inhibited rig, shows some retardation of corrosion, but after 49 days the corrosion losses remained above those for rig A (unsalted).

Figure 18 shows the corrosion rates for the replacement group. In this case the values for rig A and B are the average of the V and the flat coupons. This average was taken as the rates for each type of coupon were not too widespread. In the case of rig C the rates for each type of coupon are shown, as there was a considerable spread in their rates.

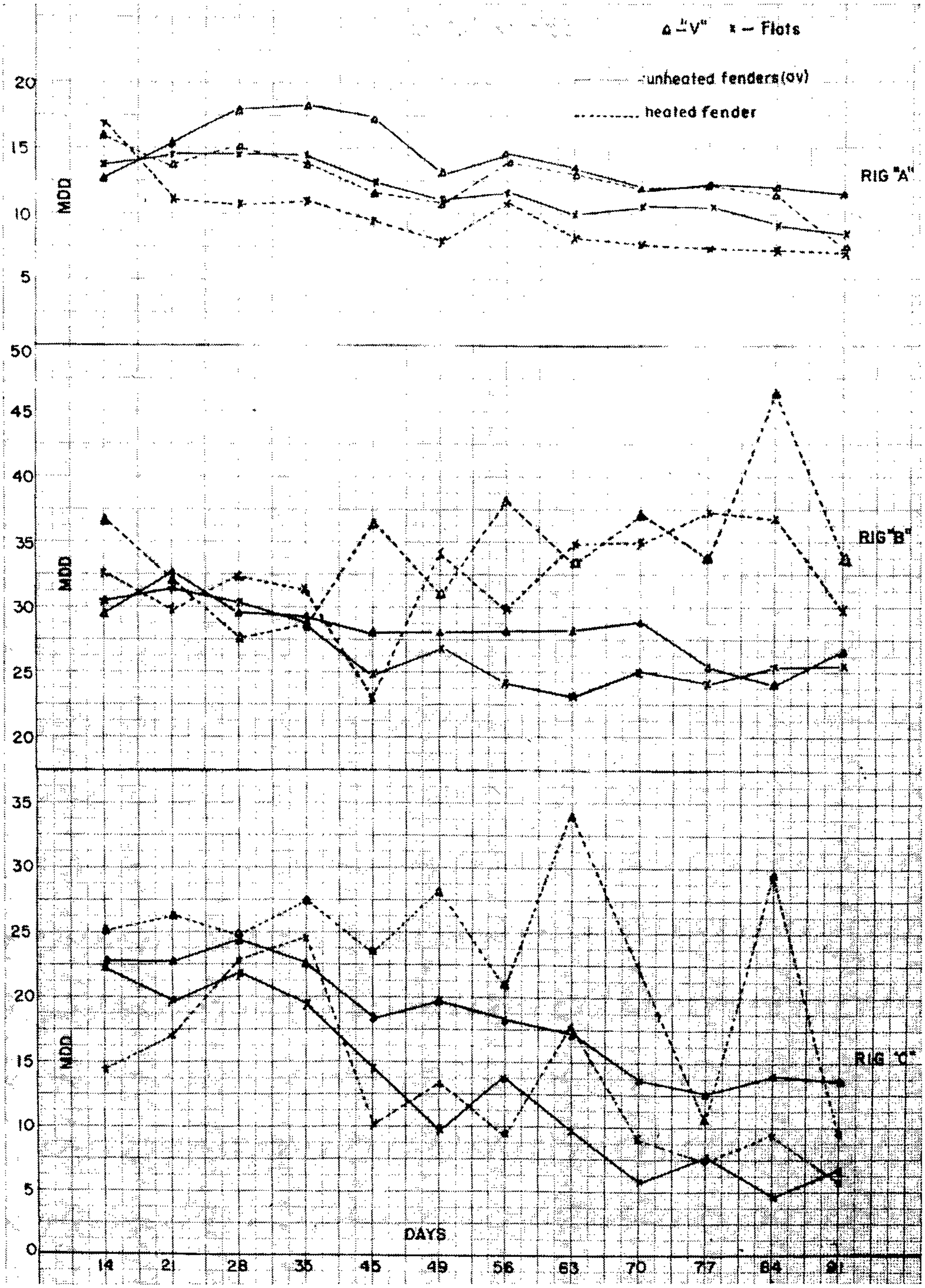


Figure 13. Corrosion rates for coupons in the three rigs and in the heated fender.

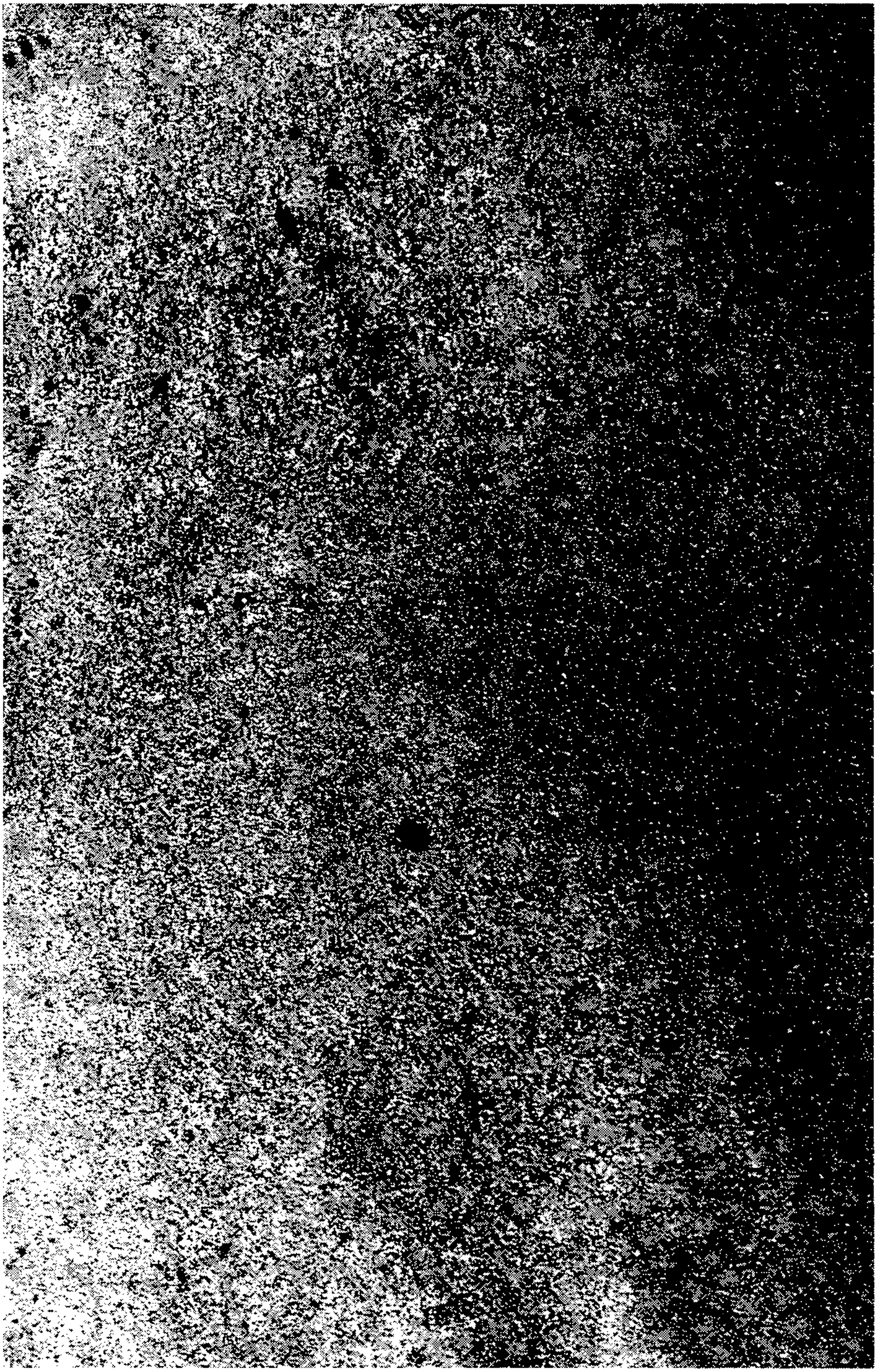


Figure 14. Photomacrograph of flat-type coupon after 91 days, rig A (unsalted). Pitting just beginning. (Mag. × 3¼)

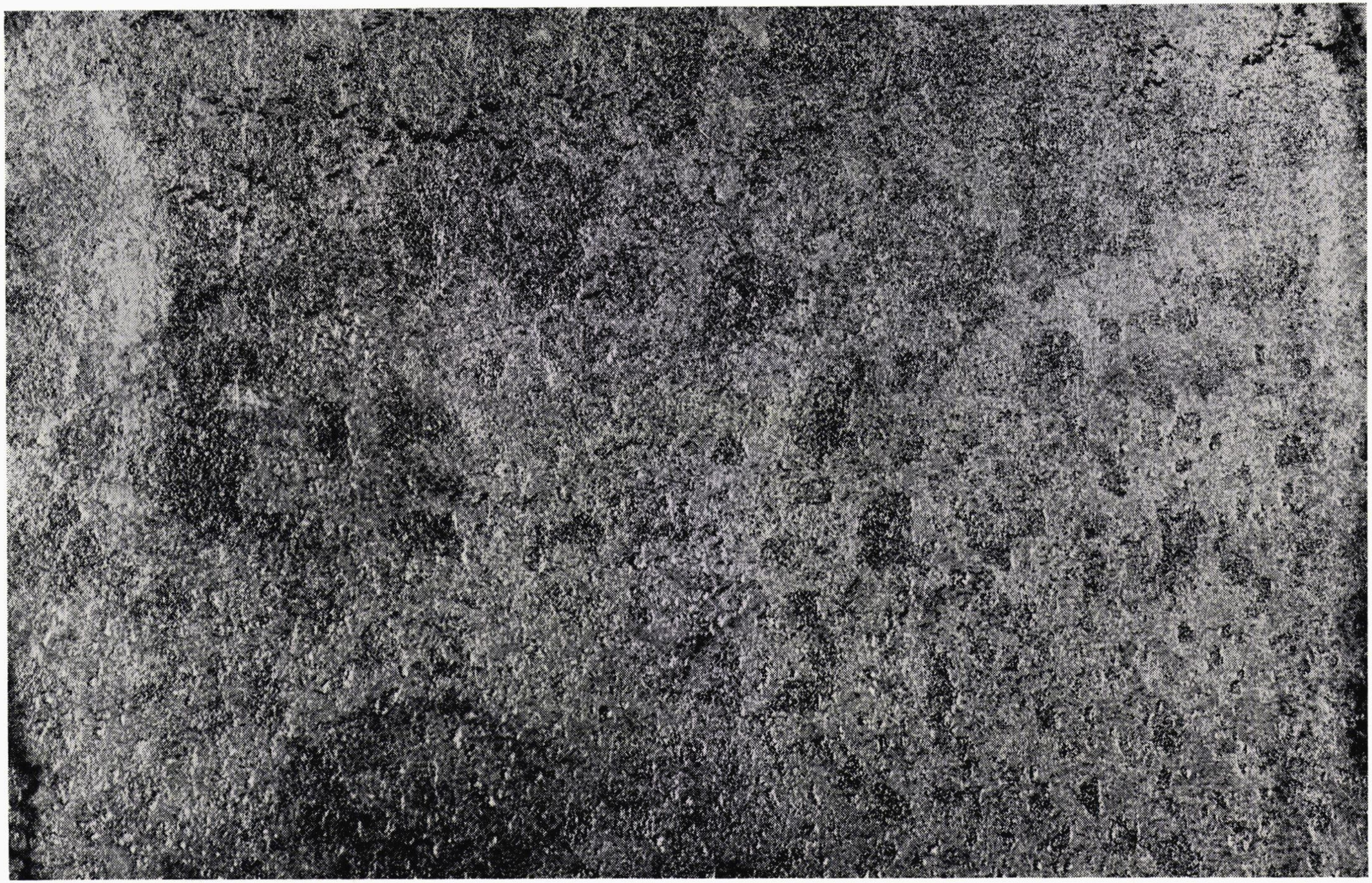


Figure 15. Photomacrograph of flat-type coupon after 91 days, rig B (salted). Surface generally etched. (Mag. × 3½)

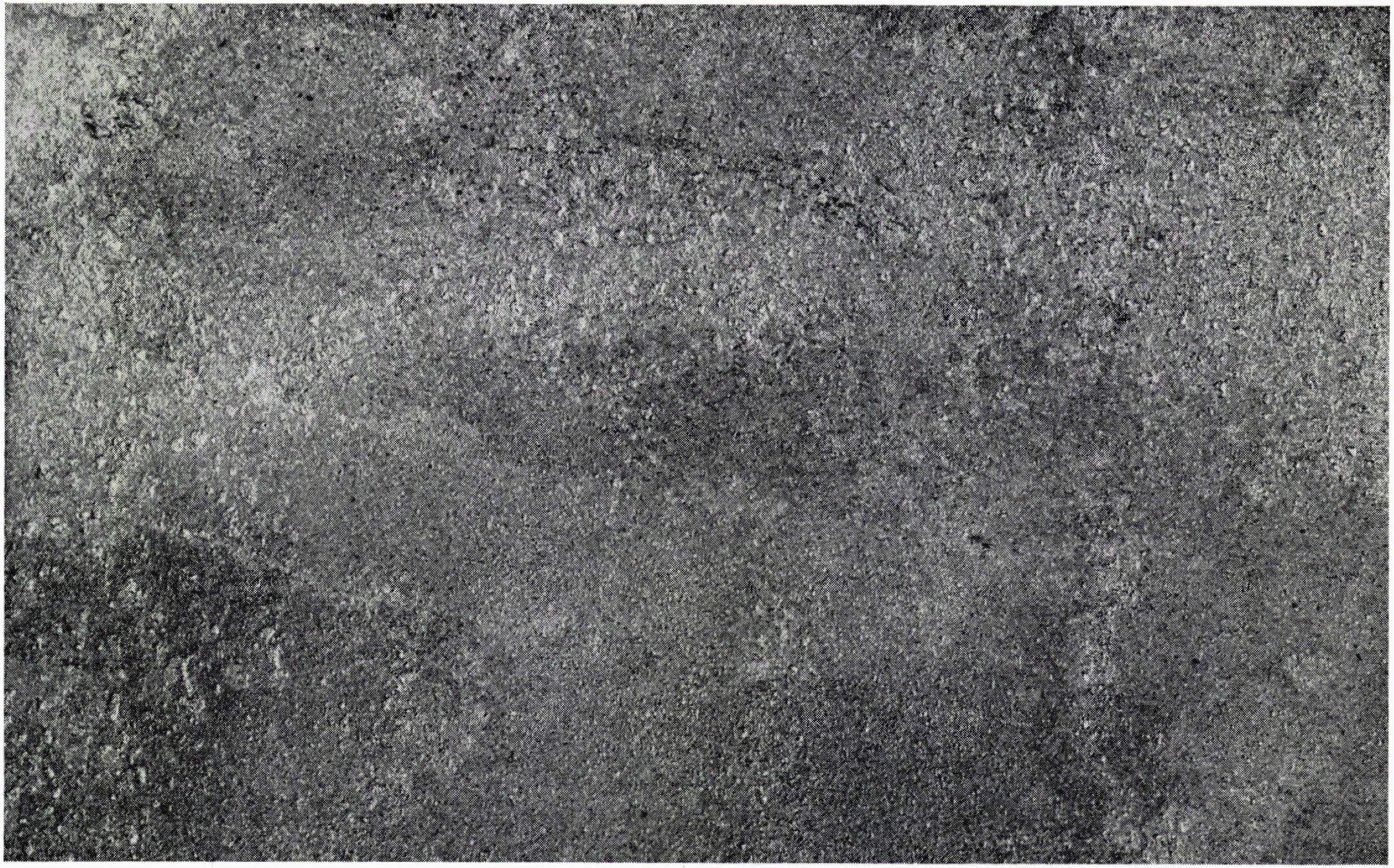


Figure 16. Photomacrograph of flat-type coupon after 91 days, rig C (inhibited salt). General etching under poultice at top; lower unpoulticed areas less attacked. (Mag. × 3¼)

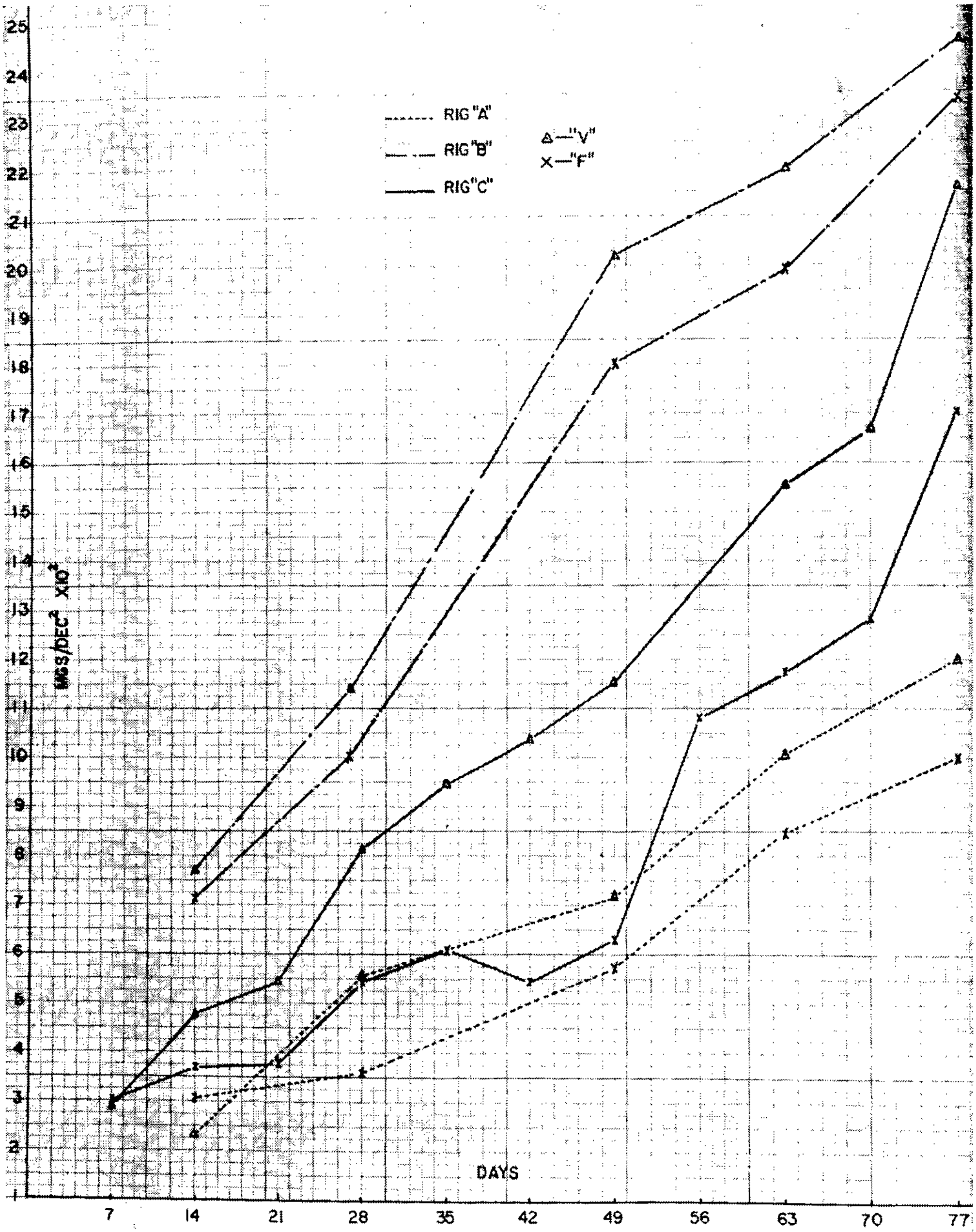


Figure 17. Corrosion losses of replacement coupons.

The rates for the flat-type coupons of rig C appear very close to those shown for rig A.

From these data it is apparent that the different period of placing on additional coupons has resulted in a higher corrosion activity of the replacement coupons. This could be due

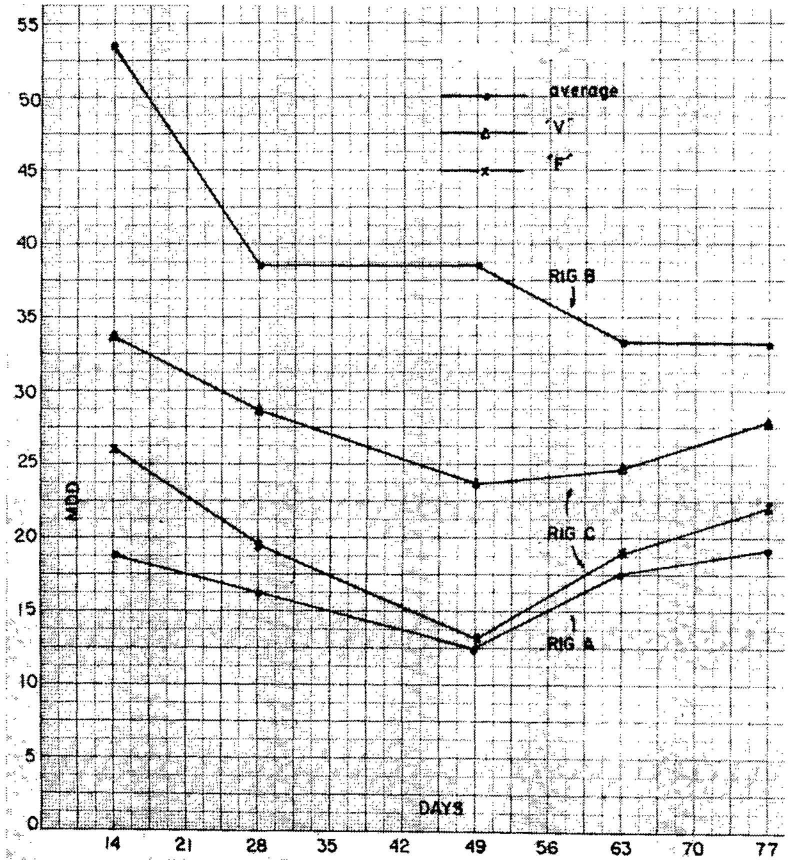


Figure 18. Corrosion rates of replacement coupons.

to a higher temperature environment at the time of placement, and also due to a faster poultice formation during this period. These two conditions were absent in the case of the original coupons when the temperature was lower and poultice rate appeared slower.

The caret-type coupons, it will be recalled, were placed on the rods for a qualitative examination only. It was found that these coupons did not exhibit any significant corrosion under the beads, under the spot welded lap joint, or along the cold worked area. The period of 91 days was too short to develop undue corrosion in these areas, and consequently no time was devoted to any specific examination of the caret coupons.

The foregoing discussion has dealt with the corrosion in the three rigs as measured by two types of corrosion coupons. Comparison of coupons placed on at different periods was also made. From the data presented, it is evident that the inhibitor employed in this test has tended to reduce the corrosion rate of the coupons in rig C to values equivalent to, and lower than, those in rig A. The replacement coupons show the same tendency, though not as markedly. It is evident in rig B that saline solutions corrode autobody steel very actively. The effect of heating influences the corrosion rates, leading to increased corrosion for the salted rig, and a diminished rate in the unsalted rig. In the case of the inhibited salt, it is believed that the rate would be diminished if poulticing by the road dirt were absent.

If the rigs had continued to operate through the summer and fall under atmospheric conditions, as would a car, the corrosion effects can only be a matter of conjecture. Under these conditions, the rainfall would eventually flush away the saline solution in rigs B and C. Corrosion in all the rigs would proceed at different rates for a time, possibly all three reaching a common level later in the summer. In the case of rig C, the corrosion would depend on the stability of the inhibited areas on the coupons. If maintained, the corrosion in rig C would be retarded. If the protection broke down, corrosion would proceed in much the same manner as the other two rigs. With the return of winter, the coupons would have an entirely different surface condition built up to begin the next winter cycle, when compared with the first winter cycle. How the coupons in rig C would respond to these new conditions cannot be answered by the data of this program as it is not possible to extrapolate such short-term data through the following seasons, but which is an important consideration in the use of an inhibitor to protect an automobile. In view of the results obtained for the replacement coupons (Fig. 17), it is probable that in rig C the degree of protection during the second winter season would not be as high as that achieved in the first.

Corrosion Coupons on Cars

The corrosion losses were obtained from coupons placed on cars that operated in the city during the winter months. The test racks mounted near the right rear wheel were well exposed to road solutions except in two cases. Figure 19 is a photograph of the well-poulticed rack representative of eight cars, and Figure 20 is representative of the two cars that did not receive the same build-up of road dirt. A record of distance traveled was maintained and also the garage conditions, if not parked outside. These data are shown in Table 3, with the results in terms of corrosion rates for the several periods of sample removal.

Two significant points were noted from these data:

1. The corrosion is independent of mileage, being chiefly time dependent.
2. The car parked in a heated garage showed a higher corrosion rate than all the others, which were left outside or in an unheated attached garage.

It was also observed that the flat coupons on the car racks showed more attack than the V-type. This may be due to the flat panel being more exposed to the wheel spray than the V-type and receiving a mild sand blast on the lower side of the coupons.

The corrosion rates of the car coupons were found to be lower than those rates for the unheated fenders in rig B. The coupons from car No. 9 which had been parked in a heated garage showed a higher rate of corrosion comparable to conditions found in the heated fender of rig B, although the values for car No. 9 were slightly lower than the results from the heated fender.

The conditions of these tests, of course, differed somewhat from the rig operation. However, the results and comparison of the data derived from each car proved interesting and informative. The reason for the lower corrosion losses experienced by the coupons on the cars when compared with those in rig B is at-

TABLE 3

CORROSION OF CAR TEST COUPONS

Car	Corrosion Rate						Mileage (mi)		Condition of Rack	Parking
	First Period			Second Period			First Period	Second Period		
	Days	Mdd		Days	Mdd					
1	25	V-20.8	F-23.8	90	V-20.2	F-20.9	402	1,451	Heavy poultice	Outside
2	25	V-23.6	F-23.2	90	V-20.5	F-20.1	804	2,573	Heavy poultice	Attached garage, unheated
3	22	V-24.6	F-24.2	87	V-18.7	F-22.6	640	4,084	Heavy poultice	Outside
4	21	V-23.7	F-27.1	85	V-24.0	F-25.0	967	3,571	Heavy poultice	Outside
5	46	V-22.6	F-22.6	98	V-21.6	F-21.6	1,675	3,522	Heavy poultice	Outside
6	44	V-20.4	F-17.8	85	V-21.4	F-20.2	812	2,021	No poultice	Outside
7	43	V-18.9	F-21.2	86	V-17.3	F-21.0	1,259	2,268	Heavy poultice	Inside, unheated
8	64	V-10.1	F-15.3	86	V-15.8	F-13.8	1,683	2,058	No poultice	Outside
9	64	V-28.7	F-30.0	85	V-31.7	F-32.0	1,523	2,055	Heavy poultice	Heated garage
10	64	V-16.5	F-15.9	85	V-18.7	F-18.0	1,852	2,463	Heavy poultice	Outside

tributed to the fact that, during mild periods, slight snowfall or rain would occur and, inasmuch as salting would not be needed, the car coupons would be then rinsed of salt solution periodically slowing down the corrosion rate. In rig B the tendency would be to have the salt solution up to strength most of the time, thus maintaining the corrosion conditions for all periods of exposure. Thus, in the rig test the exposure conditions

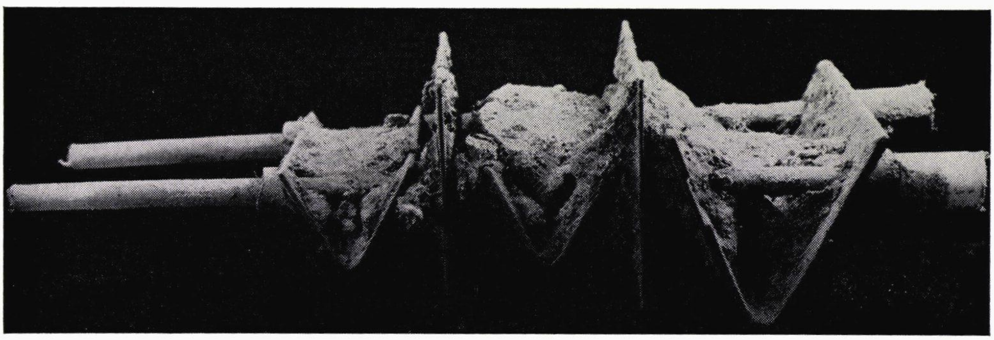


Figure 19. Poultice build-up typical of 8 cars (Table 3).

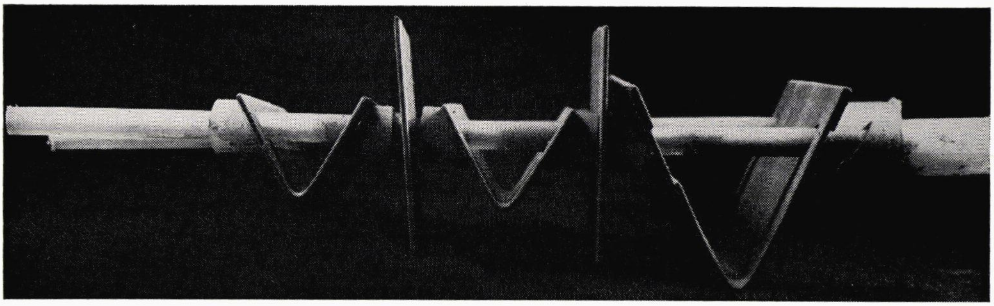


Figure 20. Light poultice build-up on 2 cars (Table 3).

would represent the worst condition of a severe winter, whereas in the car test the conditions were equivalent to a moderate winter season.

Atmospheric Corrosion Tests

To measure the effect of corrosion by the industrial-type atmosphere at the site, suitable coupons set up as described previously were evaluated from time to time. The corrosion losses were higher for these exposed coupons than for the more sheltered coupons in the unsalted rig A. The data for the atmospheric coupons are given in Table 4, together with a few points comparing the panels mounted in a flat and vertical position. Over a long period the losses on the vertical samples would be slightly higher than those mounted in the flat position.

The atmospheric corrosion of the exposed coupons is considerably higher than that found under the fender of rig A. The values lie more nearly in the area slightly above the V-coupons in rig C. These losses are somewhat comparable to those incurred by the test coupons mounted on cars in Table 3. It would be expected that the losses in the well-exposed area would be greater than those in rig A protected as they are by the fender and thus being able to dry off between exposures and to build up a more protective coating. The corrosion loss in the type of atmosphere at the site corresponds to about 9 g per sq dec per year, which is normal for industrial atmospheric corrosion. It is pointed out, however, that the short-term results of this test may not extrapolate to this total loss for the long-term exposure.

QUANTITATIVE CONSIDERATION OF THE CORROSION DATA

The foregoing discussion of results of the corrosion tests in the rigs, on the cars and atmospheric tests related the data somewhat qualitatively by means of graphs and tables. In order to provide a numerical relation of the effect of the inhibitor and the relation of corrosion weight losses of the car coupons and atmosphere tests to the losses found in the salted rig, the following data will serve to show the magnitude of these differences.

Reduction of Corrosion Loss in Salt Solutions Due to Inhibitor

Dealing with the effect of adding an inhibitor to the salt solutions in rig C, the percentage reduction of the corrosion losses for the three groups of data obtained from rig B (salt) and C (salt and inhibitor) is given in Table 5.

For the 91-day period, the effect of the inhibitor is very marked in the case of the original coupons and terminals with the flats showing a lower loss than the V-coupons. This was not the case for the replacement coupons, although they occupied the same locations as the originals. In this instance, as was pointed out previously, the replacements were put on at a different period during which temperature and poulticing conditions

TABLE 4
ATMOSPHERIC CORROSION TESTS

Period		Corrosion Loss (mg/dec²)	Corrosion Rate (mdd)		
Days	Dates			Flat	Vertical
25	2/3/61 - 2/28/61	630	25.2	—	—
46	2/3/61 - 3/21/61	1,082	23.6	—	—
74	2/3/61 - 4/18/61	1,462	19.8	—	—
88	2/3/61 - 5/2/61	1,652	18.8	—	—
21	3/21-22/61 - 4/11/61	—	—	25.5	21.6
42	3/21-22/61 - 5/2/61	—	—	24.8	27.4

TABLE 5

Data Group	% Red. of Wt. Loss $\left(\frac{\text{rig B-rig C}}{\text{rig B}} \times 100\right)$ V-Coupons	Flat Coupons	Remarks
Original coupons	48.5	74.4	For 91-day period
Original coupons plus terminals	61.0	77.0	For 91-day period
Replacement coupons	10.5	25.4	Extrapolated to 91 days

were believed to influence the action of corrosion in the initial stages promoting higher corrosion rates and a faster poultice rate which interfered with the action of the inhibitor.

The reduction of losses shown when the original take-off coupons were combined with the originals left on as terminals is greater than that for the original take-offs only. This result is due to the original take-offs being located at the inner side of the fender where corrosion rates were the highest. The results of the terminals indicated that other regions not sampled during the test period had lower losses due to more efficient coverage from the wheel spray. Thus the combination of all the losses for the original coupons showed lower average loss. This position effect was not as pronounced in rig A and rig B as shown by the location of the terminals in Figure 12.

An attempt was made to summarize the reduction of corrosion losses due to the inhibitor in rig C by calculating the average reduction of losses for both the V-type coupon and the flat-type coupon. The resulting percent reduction of the corrosion losses, thus calculated, was found to be as follows: originals, 62.2 percent; terminals, 76.5 percent; replacements, 17.7 percent; originals plus terminals, 76.3 percent; and originals plus terminals plus replacements, 55.7 percent.

In this case the spread ranges from 17.7 to 76.5 percent for the reduction of the combined losses of both types of coupons. For the test conditions prevalent in rig C probably the best representative figure for the over-all reduction of corrosion losses would be the general average for all the coupons, 55.7 percent.

Comparison of Corrosion Losses With Those of Rig B (Salted)

Based on the corrosion weight losses of the coupons from rig B, the percentage differences for the original and terminal coupons and the replacements of rig A (unsalted) were calculated. In addition, a similar comparison was made of the corrosion losses of the car tests and the atmosphere test. The percentage difference for each of the previous test conditions is given in Table 6 with the foregoing data for rig C included for comparison.

For the 91-day term of the various test conditions, the figures in Table 6 show the weight losses found under these conditions when related to the salted rig B. There is a substantially lower loss in rig A in which the coupons were protected under the fender than for the atmosphere coupons which were boldly exposed to the weather. The losses on the car coupons were lower than those in the salted rig indicating that the conditions in the rig were more severe than on the salted streets.

ROAD AND RIG SOLUTION DATA

To obtain data for salt content of road solutions, samples were collected

TABLE 6

		Wt. Loss, % Diff., V-Coupons	(Rig B — WL)/Rig B, Flat Coupons	Remarks
Rig C (salt plus inhibitor)	Original coupons	48.5	74.4	For 91 days
	Original coupons plus terminals	61.0	77.0	For 91 days
	Replacements	10.5	25.4	Extrapolated to 91 days
Rig A (unsalted)	Originals	51.6	64.4	For 91 days
	Originals plus terminals	58.5	65.6	For 91 days
	Replacements	50.5	56.7	Extrapolated to 91 days
Car test coupons (salted streets)		24.0	27.5	Extrapolated to 91 days; 10 cars tested
Atmospheric test coupons (industrial)			26.4	Extrapolated to 91 days; flat coupons with underside coated

from four sites in the city following snow storms and after the streets had been salted. The four locations were as follows:

Code	*Location*
1	Mount Pleasant at Cemetery
2	Yonge at Dundonald
3	Bloor at Church
4	Queen's Park at Ontario Research Foundation

Several locations in the Metro area were also included to compare the salting within the city limits and the Metro area.

Code	*Location*
A	Gardiner Expressway
B	Eglinton at Weston Road
C	Sheppard at Kennedy Road
D	Danford at Birchmount
E	Highway 27
F	Kingston Road West at Victoria Park
G	Kingston Road East at Victoria Park
H	Dufferin at Lawrence

The salt content and pH-value of the solutions determined in the Service Laboratory are given in Table 7. There was a considerable variation in the salt content taken at various periods during the program.

Solutions were collected from rigs B and C from time to time, and analyzed for Banox content. Some of the laboratory results show a high salt content, and it is pointed out that when this condition appeared it was possible to make immediate correction at the site to reduce the salt to

TABLE 7
SALT CONTENT OF ROAD SOLUTIONS

Date Collected	Location	Salt (%)	pH
12/21/60	1	1.24	7.40
	2	2.19	7.40
	3	3.69	7.32
	4	2.16	7.39
1/4/61	1	1.75	7.39
	2	2.01	7.19
	3	3.01	7.32
	4	1.93	7.58
1/26/61	1	4.42	7.10
	2	7.33	6.70
	3	2.40	7.60
	4	4.83	7.50
1/27/61	1	3.63	7.20
	2	3.62	6.95
	3	5.27	7.10
	4	3.80	7.10
2/5/61	1	3.82	6.30
	2	6.83	7.05
	3	1.28	7.32
	4	0.56	7.45
2/16/61	1	0.18	8.36
	2	0.08	8.34
	3	1.90	9.32
	4	1.67	8.02
3/9/60	1	0.50	8.11
	2	0.30	8.01
	3	0.55	8.71
	4	0.43	7.85
2/4/61	A	7.61	7.03
	B	5.03	7.00
3/9/61	C	2.31	8.31
	D	1.07	8.20
	E	1.68	7.85
	F	0.99	8.22
	G	0.43	8.15
	H	1.31	8.11

the operating range, by the aid of the conductivity meter.

The analysis of the solutions from rigs B and C are shown in Table 8. The interesting point is the variation of the Banox content. Although it was added in the amount of 2 percent of the salt, at no time did the Banox reach the 2 percent mark in the solution, with the exception of that period when the solution was prepared in drums using city water. The value of 2.5 percent Banox in the April 25 test was believed due to using the inhibited salt toward the bottom of the container where excess Banox had concentrated by handling and jarring causing the Banox to settle out toward the lower parts of the container. This possibility was pointed out under the section dealing with inhibitors.

TABLE 8
SALT AND INHIBITOR CONTENT OF RIG SOLUTIONS

Date	Rig	Salt (%)	Meter[1]	pH	Banox (%)
2/1/61	B	6.14		7.50	
	C	6.98		6.62	0.5
2/2/61					
AM	B	15.4		7.35	
	C	17.0		5.88	1.7
PM	B	7.71		7.13	
	C	9.67		6.02	1.9
2/4/61					
AM	B	6.23		7.37	
	C	6.32		5.92	1.8
PM	B	4.79		7.20	
	C	5.20		6.20	1.7
2/5/61					
AM	B	4.29		7.30	
	C	4.87		6.30	1.7
2/12/61	B	3.00	3.4		
	C	2.07	2.7		
2/16/61	C	3.68	(Solution 2 PM)		1.9
		3.65	(Solution 4 PM)		1.65
		2.07	(Slush 2 PM)		1.07
		1.32	(Slush 4 PM)		1.7
3/16/61	B	11.8			
	C	11.7			0.67
3/25/61	B	7.37	7.0		
	C	5.20	5.2		0.52
3/26/61	B	6.29	5.5		
	C	3.99	4.0		0.26
3/27/61	B	2.72	2.8		
	C	2.49	2.5		1.1
4/2/61[2]	B	3.8		8.00	
	C	3.37		7.51	2.5

[1] Michi-Mho conductivity meter.

[2] City water used for rig solutions during this period and to May 2, 1961.

LABORATORY CORROSION TESTS

The corrosion tests employed to evaluate the inhibitor were (a) salt fog test, (b) modified Corrodcote test, and (c) intermittent immersion test. The first two tests indicated that under these conditions the inhibiting effect of Banox was found to be negligible. Both these test procedures are carried out under conditions of saturated humidity, and at no time did the coupons undergo a drying period. Under the conditions of these tests the inhibitor was unable to establish a protective film capable of reducing the corrosion activity of the saline solutions.

The conditions of the intermittent immersion test established protection to the extent that the corrosion loss in the Banox solution was one-tenth that of the salt solution. This test procedure which involves drying between exposures to the corroding environment is more representative of conditions established in the test rigs, although the results are not comparable quantitatively. This method of testing would be useful to obtain data for evaluating the inhibiting characteristics of inhibitors to qualify such compounds for further investigation. This test would by no means indicate inhibiting efficiency as applied to rig testing, but would assist in selecting other compounds for field tests.

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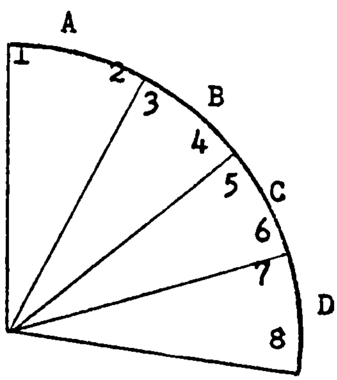
Appendix A
Sample Location Plan

Rig	Wheel	Position on Rod	A Rod 1	A Rod 2	B Rod 3	B Rod 4	C Rod 5	C Rod 6	D Rod 7	D Rod 8
A	1 (Inside dia.)	1	V 1	V 2	V 7	V 8	V 13	V 14	V 19	V 20
		2	F 1	F 2	F 7	F 8	F 13	F 14	F 19	F 20
		3	V 3	V 4	V 9	V 10	V 15	V 16	V 21	V 22
		4	F 3	F 4	F 9	F 10	F 15	F 16	F 21	F 22
		5	C 1	V 6	C 4	V 12	C 7	V 18	C 10	V 24
		6	V 5	F 6	V 11	F 12	V 17	F 18	V 23	F 24
		7	F 5	C 2	F 11	C 5	F 17	C 8	F 23	C 11
		8	C 3		C 6		C 9		C 12	
Condition—No salt, no inhibitor	2	1	V 25	V 26	V 31	V 32	V 37	V 38	V 43	V 44
		2	F 25	F 26	F 31	F 32	F 37	F 38	F 43	F 44
		3	V 27	V 28	V 33	V 34	V 39	V 40	V 45	V 46
		4	F 27	F 28	F 33	F 34	F 39	F 40	F 45	F 46
		5	C 13	V 30	C 16	V 36	C 19	V 42	C 22	V 48
		6	V 29	F 30	V 35	F 36	V 41	F 42	V 47	F 48
		7	F 29	C 14	F 35	C 17	F 41	C 20	F 47	C 23
		8	C 15		C 18		C 21		C 24	
	3	1	V 49	V 50	V 55	V 56	V 61	V 62	V 67	V 68
		2	F 49	F 50	F 55	F 56	F 61	F 62	F 67	F 68
		3	V 51	V 52	V 57	V 58	V 63	V 64	V 69	V 70
		4	F 51	F 52	F 57	F 58	F 63	F 64	F 69	F 70
		5	C 25	V 54	C 28	V 60	C 31	V 66	C 34	V 72
		6	V 53	F 54	V 59	F 60	V 65	F 66	V 71	F 72
		7	F 53	C 26	F 59	C 29	F 65	C 32	F 71	C 35
		8	C 27		C 30		C 33		C 36	
	4 (Outside dia.)	1	V 73	V 74	V 79	V 80	V 85	V 86	V 91	V 92
		2	F 73	F 74	F 79	F 80	F 85	F 86	F 91	F 92
		3	V 75	V 76	V 81	V 82	V 87	V 88	V 93	V 94
		4	F 75	F 76	F 81	F 82	F 87	F 88	F 93	F 94
		5	C 37	V 78	C 40	V 84	C 43	V 90	C 46	V 96
		6	V 77	F 78	V 83	F 84	V 89	F 90	F 95	F 96
		7	F 77	C 38	F 83	C 41	F 89	C 44	V 95	C 47
		8	C 39		C 42		C 45		C 48	

Sample Positions on Rods

Position	1	2	3	4	5	6	7	8
Rod 1	V	F	V	F	C	V	F	C
Rod 2	V	F	V	F	V	F	C	

Section and Rod Positions



Rig	Wheel	Position on Rod	A Rod 1	A Rod 2	B Rod 3	B Rod 4	C Rod 5	C Rod 6	D Rod 7	D Rod 8
B	1	1	V 97	V 98	V103	V104	V109	V110	V115	V116
		2	F 97	F 98	F103	F104	F109	F110	F115	F116
	Inside dia.	3	V 99	V100	V105	V106	V111	V112	V117	V118
		4	F 99	F100	F105	F106	F111	F112	F117	F118
		5	C 49	V102	C 52	V108	C 55	V114	C 58	V120
		6	V101	F102	V107	F108	V113	F114	V119	F120
		7	F101	C 50	F107	C 53	F113	C 56	F119	C 59
		8	C 51		C 54		C 57		C 60	
	2	1	V121	V122	V127	V128	V133	V134	V139	V140
		2	F121	F122	F127	F128	F133	F134	F139	F140
		3	V123	V124	V129	V130	V135	V136	V141	V142
		4	F123	F124	F129	F130	F135	F136	F141	F142
		5	C 61	V126	C 64	V132	C 67	V138	C 70	V144
		6	V125	F126	V131	F132	V137	F138	V143	F144
Condition—Salt		7	F125	C 62	F131	C 65	F137	C 68	F143	C 71
		8	C 63		C 66		C 69		C 72	
	3	1	V145	V146	V151	V152	V157	V158	V163	V164
		2	F145	F146	F151	F152	F157	F158	F163	F164
		3	V147	V148	V153	V154	V159	V160	V165	V166
		4	F147	F148	F153	F154	F159	F160	F165	F166
		5	C 73	V150	C 76	V156	C 79	V162	C 82	V168
		6	V149	F150	V155	F156	V161	F162	V167	F168
		7	F149	C 74	F155	C 77	F161	C 80	F167	C 83
		8	C 75		C 78		C 81		C 84	
	4	1	V169	V170	V175	V176	V181	V182	V187	V188
		2	F169	F170	F175	F176	F181	F182	F187	F188
		3	V171	V172	V177	V178	V183	V184	V189	V190
	Outside dia.	4	F171	F172	F177	F178	F183	F184	F189	F190
		5	C 85	V174	C 88	V180	C 91	V186	C 94	V192
		6	V173	F174	V179	F180	V185	F186	V191	F192
		7	F173	C 86	F179	C 89	F185	C 92	F191	C 95
		8	C 87		C 90		C 93		C 96	
C	1	1	V193	V194	V199	V200	V205	V206	V211	V212
		2	F193	F194	F199	F200	F205	F206	F211	F212
	Inside dia.	3	V195	V196	V201	V202	V207	V208	V213	V214
		4	F195	F196	F201	F202	F207	F208	F213	F214
		5	C 97	V198	C100	V204	C103	V210	C106	V216
		6	V197	F198	V203	F204	V209	F210	V215	F216
		7	F197	C 98	F204	C101	F209	C104	F215	C107
		8	C 99		C102		C105		C108	
	2	1	V217	V218	V223	V224	V229	V230	V235	V236
		2	F217	F218	F223	F224	F229	F230	F235	F236
		3	V219	V220	V225	V226	V231	V232	V237	V238
		4	F219	F220	F2225	F226	F231	F232	F237	F238
		5	C109	V222	C112	V228	C115	V234	C118	V240
		6	V221	F222	V227	F228	V233	F234	V239	F240
		7	F221	C110	F227	C113	F233	C116	F239	C119
Condition—Salt + Inhibitor		8	C111		C114		C117		C120	
	3	1	V241	V242	V247	V248	V253	V254	C259	V260
		2	F241	F242	F247	F248	F253	F254	F259	F260
		3	V243	V244	V249	V250	V255	V251	V261	V262
		4	F243	F244	F249	F250	F255	F256	F261	F262
		5	C121	V124	C124	V252	C127	V258	C130	V264
		6	V245	F246	V251	F252	V257	F258	V263	F264
		7	F245	C122	F251	C125	F257	C128	F263	C131
		8	C123		C126		C129		C132	
	4	1	V265	V266	V271	V272	V277	V278	V283	V284
		2	F265	F266	F271	F272	F277	F278	F283	F284
		3	V267	V268	V273	V274	V279	V280	V285	V286
	Outside dia.	4	F267	F268	F273	F274	F279	F280	F285	F286
		5	C133	V270	C136	V276	C139	V282	C142	V288
		6	V269	F270	V275	F276	V281	F282	V287	F288
		7	F269	C134	F275	C137	F281	C140	F287	C143
		8	C135		C138		C141		C144	

	Samples V	Samples F	Samples C
Per wheel	24	24	12
Per rig	96	96	48
3 rigs total	288	288	144

Appendix B
Sample Removal and Replacement Plan

Wk.	Rig	Wheel	Sect.	Rod No.	Pos. on Rod	Take off Pairs V-F	Put on Pairs RV-RF
1	A	1	B	3	1-2	7	351
		2	A	1	1-2	25	352
		3	C	5	1-2	61	353
		4	D	7	1-2	91	354
	B	1	B	3	1-2	103	355
		2	A	1	1-2	121	356
		3	C	5	1-2	157	357
		4	D	7	1-2	187	358
	C	1	B	3	1-2	199	359
		2	A	1	1-2	217	360
		3	C	5	1-2	253	361
		4	D	7	1-2	283	362
2	A	1	A	1	1-2	1	363
		2	D	7	1-2	43	364
		3	B	3	1-2	55	365
		4	C	5	1-2	85	366
	B	1	A	1	1-2	97	367
		2	D	7	1-2	139	368
		3	B	3	1-2	151	369
		4	C	5	1-2	181	370
	C	1	A	1	1-2	193	371
		2	D	7	1-2	235	372
		3	B	3	1-2	247	373
		4	C	5	1-2	277	374
3	A	1	D	7	1-2	19	375
		2	C	5	1-2	37	376
		3	A	1	1-2	49	377
		4	B	3	1-2	79	378
	B	1	D	7	1-2	115	379
		2	C	5	1-2	133	380
		3	A	1	1-2	145	381
		4	B	3	1-2	175	382
	C	1	D	7	1-2	211	383
		2	C	5	1-2	229	384
		3	A	1	1-2	241	385
		4	B	3	1-2	271	386
7	A	1	A	2	1-2	2	423
		2	B	4	1-2	32	424
		3	C	6	1-2	62	425
		4	D	8	1-2	92	426
	B	1	A	2	1-2	98	427
		2	B	4	1-2	128	428
		3	C	6	1-2	152	429
		4	D	8	1-2	188	430
	C	1	A	2	1-2	194	431
		2	B	4	1-2	224	432
		3	C	6	1-2	254	433
		4	D	8	1-2	284	434
8	A	1	B	4	1-2	8	435
		2	C	6	1-2	38	436
		3	D	8	1-2	68	437
		4	A	2	1-2	74	438
	B	1	B	4	1-2	104	439
		2	C	6	1-2	134	440
		3	D	8	1-2	164	441
		4	A	2	1-2	170	442
	C	1	B	4	1-2	200	443
		2	C	6	1-2	230	444
		3	D	8	1-2	260	445
		4	A	2	1-2	266	446
9	A	1	A	1	3-4	3	447
		2	C	5	3-4	39	448
		3	D	7	3-4	69	449
		4	B	3	3-4	81	450
	B	1	A	1	3-4	99	451
		2	C	5	3-4	135	452
		3	D	7	3-4	165	453
		4	B	3	3-4	177	454
	C	1	A	1	3-4	195	455
		2	C	5	3-4	231	456
		3	D	7	3-4	261	457
		4	B	3	3-4	273	458

Wk.	Rig	Wheel	Sect.	Rod No.	Pos. on Rod	Take off Pairs V-F	Put on Pairs RV-RF
4	1	1	C	5	1-2	13	387
		2	B	3	1-2	31	388
		3	D	7	1-2	67	389
		4	A	1	1-2	73	390
	2	1	C	5	1-2	109	391
		2	B	3	1-2	127	392
		3	D	7	1-2	163	393
		4	A	1	1-2	169	394
	3	1	C	5	1-2	205	395
		2	B	3	1-2	223	396
		3	D	7	1-2	259	397
		4	A	1	1-2	265	398
5	1	1	D	8	1-2	20	399
		2	A	2	1-2	26	400
		3	B	4	1-2	56	401
		4	C	6	1-2	86	402
	2	1	D	8	1-2	116	403
		2	A	2	1-2	122	404
		3	B	4	1-2	152	405
		4	C	6	1-2	182	406
	3	1	D	8	1-2	212	407
		2	A	2	1-2	218	408
		3	B	4	1-2	248	409
		4	C	6	1-2	278	410
6	1	1	C	6	1-2	14	411
		2	D	8	1-2	44	412
		3	A	2	1-2	50	413
		4	B	4	1-2	80	414
	2	1	C	6	1-2	110	415
		2	D	8	1-2	140	416
		3	A	2	1-2	146	417
		4	B	4	1-2	176	418
	3	1	C	6	1-2	206	419
		2	D	8	1-2	236	420
		3	A	2	1-2	242	421
		4	B	4	1-2	272	422
10	1	1	D	7	3-4	21	459
		2	A	1	3-4	27	460
		3	B	3	3-4	57	461
		4	C	5	3-4	87	462
	2	1	D	7	3-4	117	463
		2	A	1	3-4	123	464
		3	B	3	3-4	153	465
		4	C	5	3-4	183	466
	3	1	D	7	3-4	321	467
		2	A	1	3-4	219	468
		3	B	3	3-4	249	469
		4	C	5	3-4	279	470
11	1	1	B	3	3-4	9	471
		2	D	7	3-4	45	472
		3	C	5	3-4	63	473
		4	A	1	3-4	75	474
	2	1	B	3	3-4	105	475
		2	D	7	3-4	141	476
		3	C	5	3-4	159	477
		4	A	1	3-4	171	478
	3	1	B	3	3-4	201	479
		2	D	7	3-4	237	480
		3	C	5	3-4	255	481
		4	A	1	3-4	267	482

Take Off Carets

C 1
C 19
C 34
C 40
C 49
C 67
C 82
C 88
C 97
C115
C130
C136

Wk.	Rig	Wheel	Sect.	Rod No.	Pos. on Rod	Take off Pairs V-F	Put on Pairs RV-RF
12	A	1	C	5	3-4	15	483
		2	B	3	3-4	33	484
		3	A	1	3-4	51	485
		4	D	7	3-4	93	486
	B	1	C	5	3-4	111	487
		2	B	3	3-4	129	488
		3	A	1	3-4	147	489
		4	D	7	3-4	189	490
	C	1	C	5	3-4	207	491
		2	B	3	3-4	225	492
		3	A	1	3-4	243	493
		4	D	7	3-4	285	494
13	A	1	D	8	3-4	22	495
		2	B	4	3-4	34	496
		3	C	6	3-4	64	497
		4	A	2	3-4	76	498
	B	1	D	8	3-4	118	499
		2	B	4	3-4	130	500
		3	C	6	3-4	160	501
		4	A	2	3-4	172	502
	C	1	D	8	3-4	214	503
		2	B	4	3-4	226	504
		3	C	6	3-4	256	505
		4	A	2	3-4	268	506
14	A	1	A	2	3-4	4	507
		2	D	8	3-4	46	508
		3	B	4	3-4	58	509
		4	C	6	3-4	88	510
	B	1	A	2	3-4	100	511
		2	D	8	3-4	142	512
		3	B	4	3-4	154	513
		4	C	6	3-4	184	514
	C	1	A	2	3-4	196	515
		2	D	8	3-4	238	516
		3	B	4	3-4	250	517
		4	C	6	3-4	280	518
15	1	1	B	4	3-4	10	519
		2	C	6	3-4	40	520
		3	A	2	3-4	52	521
		4	D	8	3-4	94	522
	2	1	B	4	3-4	106	523
		2	C	6	3-4	136	524
		3	A	2	3-4	148	525
		4	D	8	3-4	190	526
	3	1	B	4	3-4	202	527
		2	C	6	3-4	232	528
		3	A	2	3-4	244	529
		4	D	8	3-4	286	530
16	1	1	C	6	3-4	16	531
		2	A	2	3-4	28	532
		3	D	8	3-4	70	533
		4	B	4	3-4	82	534
	2	1	C	6	3-4	112	535
		2	A	2	3-4	124	536
		3	D	8	3-4	166	537
		4	B	4	3-4	178	538
	3	1	C	6	3-4	208	539
		2	A	2	3-4	220	540
		3	D	8	3-4	262	541
		4	B	4	3-4	274	542

Field Test for Estimating Service Life of Corrugated Metal Pipe Culverts

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In 1959 the California Division of Highways reported on findings of a corrosion survey of 7,000 corrugated metal culverts located in one area of California. Based on those data supplemented by additional information collected during statewide investigations, a survey technique has been developed to estimate the corrosion potential of proposed culvert sites.

The results of the 1959 study indicated that the corrosion rate of metal culverts was a variable depending on the environment, with certain factors exerting greater influence than others. It was therefore considered worthwhile to attempt to determine the relative influence of the individual factors that comprise the corrosion environments.

By utilizing the information from the first study and supplementing it with data from different types of watersheds located in various parts of the State, it has been observed that the major factors influencing the corrosion rate are the hydrogen ion concentration (pH) and the electrical resistivity of the soil and waters. A standard test method has been developed and is reported in detail. It is relatively simple to perform, taking about 5 min for water and not more than 20 min for soils. As a check on the test method, it has been found to correlate with the published corrosion rates of metal pipe reported by the National Bureau of Standards in its nationwide studies.

• MANY TYPES of materials have been used for the small structures needed to carry water beneath highways. Wood, brick, stone, glazed and unglazed clay pipe, cast iron, and steel have all been used as pipe materials for well over a century. In recent years the majority of the pipe culverts under roads have been of either reinforced concrete or corrugated metal. The corrugated metal is zinc-coated iron or steel.

If the hydrologic and special requirements of a specific site can be met by either material, the choice between the two types of culvert pipe usually depends on one of three factors: (a) the character and stability of the underlying foundation material, (b) the effect of loading (either from high embankments or from traffic under shallow earth cover), and (c) an estimate of the annual cost (*1*). Insofar as hydrologic, foundation, and embankment data are concerned, means are available (*1*) to determine such information to whatever degree might be considered necessary for the particular installation. However, up to the present time, the only way an accurate estimate of the annual cost could be made was to compare the past service records of the two types of pipe materials in the same watershed and apply this information to current prices. In new locations or where watershed land use has been changed, such comparisons are not usually available.

The effects of an aggressive (*i.e.*, acid, sulfate, sea water, etc.) environment on concrete and reinforcing steel have been reported in the literature. Although an annual cost is still difficult to develop for reinforced concrete pipe, sufficient information is available to determine its suitability for a particular location providing the chemical environment is known. In broad terms this also has been the case when considering corrugated metal pipe. However, it has not been possible to predict the years of service life of a zinc-coated steel corrugated metal pipe in any specific site. This report describes a test procedure used by the California Division of Highways to predetermine the corrosion rate (and thus the annual cost) of corrugated metal culverts.

This study is the result of a series of investigations performed during the past 35 years by the Materials and Research Department of the California Division of Highways during which time more than 12,000 corrugated metal highway culvert pipes have been evaluated throughout the State highway system.

DEVELOPMENT OF TEST METHOD

Life of Metal Culverts

Corrugated metal pipes used by the State of California conform to AASHO Specification M36. All pipes in this study conformed in general to this specification with the minor changes made during the past 40 to 50 years.

Considering the various climates and geographic features found in California, it is not surprising that figures purporting to show the average life of metal culverts are of little use to a designer dealing with individual installations inasmuch as the service life of metal culverts depends on their locations.

The best criterion for estimating the service life of metal culverts is considered to be the proven tenure of an existing pipe in the same location as the proposed facility; that is, using existing culverts for estimating the economic life of a proposed metal pipe. All others are a compromised estimate, the accuracy of which depends on the experience and judgment of the engineer.

Although the past history of a culvert in a particular location is considered to be the best criterion for estimating future life, this method of analysis is not infallible. Experience has shown that the use of the land in a watershed may be altered; *e.g.*, from a timber or brush land to an agricultural use. In such a case, the previous service life of the culverts may not reflect the future corrosion rate of the pipes. It is also true that any other corrosion test method or other means for estimating culvert life would be subject to the same error due to a possible change in the culvert environment.

There is also speculation as to whether several culverts that would be repeatedly placed at the same site would have the same service life. The corrosion rate of steel is difficult to reproduce even in the same environment. This fact is brought out by Romanoff (*2*, p. 18) who gives reasons that steel specimens buried in one end of a trench might corrode at a somewhat different rate from those at the other end. The National Bureau of Standards stated concerning the reason for dispersion of corrosion rate data found in its testing program, "The lack of reproducibility of results is common to all corrosion tests, particularly to nonlaboratory tests." Because an existing metal culvert can be considered as a test site when its corrosion rate is determined, the corrosion rate of an existing pipe consequently may or may not be duplicated by a subsequent installation. Therefore, it appears that the life of existing metal culverts may be a general indication whose reliability depends on the number of inspected culverts that are exposed to the same environment. Also, an example of the performance of a single culvert installation may not accurately represent the corrosion rate of a number of culverts in the same general area of environment.

Corrosion Theory

It is generally accepted that the corrosion of steel is electrochemical in nature; that is, there is an electrolyte and a flow of electricity accompanying the corrosion or conversion of the metal to its oxide or to its most stable compound in the particular environment. Even though the theory of corrosion is well established, the variables that influence the rate of corrosion are so numerous that the corrosion rate of metals is generally established by an environmental test rather than from a calculation based on a corrosion theory.

The basic cause of corrosion of metal culverts is moisture; however, the presence of moisture does not necessarily mean that the corrosion rate will be rapid. The rate of corrosion depends not only on the frequency of the wetting of the metal but also on the formation of corrosion-inhibiting films and by the types and quantities of aggressive salts that are contained in the waters.

In an unreported study made in 1925 of 5,000 metal culverts throughout California, it was found that when the average annual rainfall increased from 10 to 80 in. per yr, the average life of metal culverts appeared to decrease by about 65 percent. One result was the suggestion that the magnitude of annual rainfall could be used to predict corrosion rate. However, it was also observed that in areas of equal rainfall there were extreme differences in rates of corrosion of individual culverts. It was apparent that factors other than the relative amount of rainfall were involved in the corrosion rate of the pipes.

In a 1955 study of 7,000 metal culverts in the northwestern section of California (*3*), it was observed that when a certain type of soil bacteria appeared to be present at a culvert location, the corrosion rate of the culvert was rapid. The apparent presence of these bacteria (*Sporovibrio desulfuricans*) were assumed to be indicated by the observation of hydrogen sulfide gas in the soil. However, among other variables, the propagation of these bacteria required that the soil be relatively flooded with moisture so as to restrict the ingress of air. It became apparent that the presence of these anaerobic bacteria was limited to areas of high rainfall or ponded water conditions and was of minor importance in many geographic areas of the State.

Corrosive Factors

Numerous factors could influence the corrosion rate of metal culverts. If all the elements that could influence corrosion were investigated, the investigation would be prolonged for several years. Rather than investigate all possibilities, only those factors that a search of the literature indicated to be of the greatest probable influence were recorded and analyzed for their relative contribution to the observed corrosion rates of metal culverts:

1. The presence of flowing water;
2. Average annual rainfall;
3. Probable presence of bacteria (*3*);
4. pH or hydrogen ion concentration;
5. Minimum resistivity of soil;
6. Sulfate (SO_4) concentration;
7. Total alkalinity as $CaCO_3$;
8. Dissolved solids; and
9. Total calcium as $CaCO_3$.

By means of mathematical and graphical analyses it was determined that all of the considered factors had an influence on the corrosion rate of metal culverts. However, it was also observed that the influences of most of these elements were isolated to certain geographic areas. Some of these factors were combined by means of the Langelier (*4*) and also the Ryznar (*5*) indices and compared to the corrosion rate of the culverts. As such, all the considered factors were not found usable as a general and economical test procedure. The exceptions were the pH or hydrogen ion concentration, the electrical resistivity of the soil or water, and the indirect effect of rainfall on these two variables.

Influence of Salts

There are numerous salts in soils or water which affect the corrosion rate of culverts. For instance, it is well known that the salts in sea water will cause the rapid corrosion of steel. Also, the salts in "alkali" soils, effluents from mines and numerous other sources have been observed to be highly corrosive to iron and steels.

When corrosive salts are present, other conditions being equal, the corrosion of steel will generally vary as their concentration (*1, 6*). Up to certain limits, the greater the concentration of salts, the more rapid the corrosion rate (*7*).

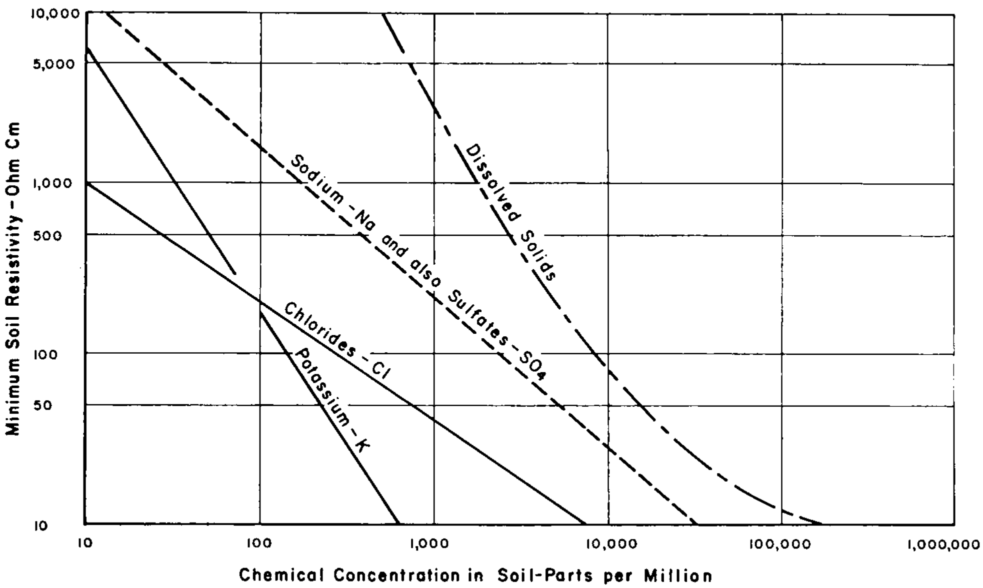


Figure 1. Salts in a soil *vs* minimum soil resistivity. Curves developed from analysis of soils found in Colorado Desert, Calif., and are not necessarily representative of soils found in other California areas.

With other factors equal, an increasing quantity of salts will result in the lowering of the specific electrical resistance of a soil or water. Figure 1 shows the relationship between the natural variations in the concentrations of salts in a particular soil series compared to its measured minimum soil resistivity. The salt-resistivity relationship represents a specific geographic area of about 1,000 sq mi near the Salton Sea. This relationship of the data is not necessarily applicable to other areas, as the proportions and the presence of the particular salts have been found to vary throughout the State.

Figure 2 shows that the electrical resistivity of a soil influences the corrosion rate of metal highway culverts. The service life of metal culverts is less in a soil of low electrical resistivity than in soils of higher values. This correlation of corrosion rate and electrical resistivity compares with the general observations of other investigators in the field of the underground corrosion of pipe lines (*8, 9, 10*).

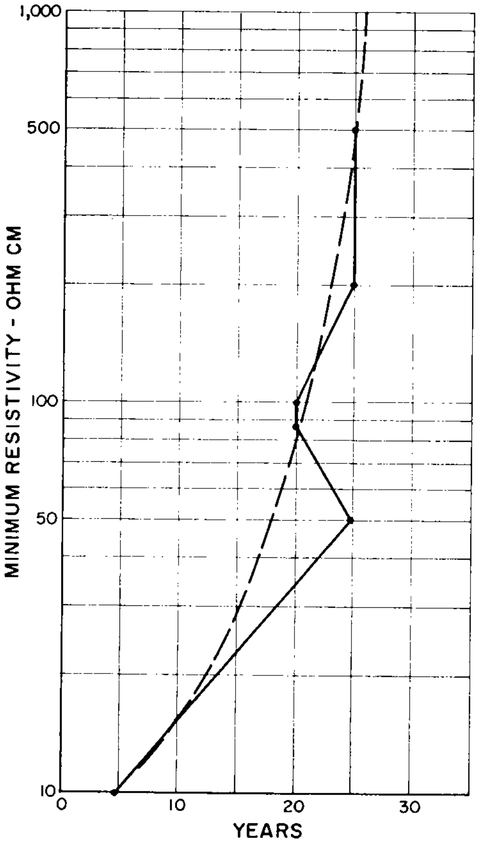


Figure 2. Resistivity *vs* service life of 16-gage CMP. To maintain but one variable, pH limited between 8.0 and 8.5; annual rainfall limited to less than 5 in. per year.

Influence of Electrical Resistivity

Numerous methods for measuring the electrical resistivity of a soil or water are described by Romanoff (*2*).

The method employed in this investigation for measuring the electrical resistivity of a soil is a derivation of one described by the U. S. Department of Agriculture (*11*). In its test method, a Wheatstone bridge and a "Bureau of Soils" soil cup were utilized to measure the resistivity of a water-saturated soil at a corrected temperature of 60 F.

In duplicating the "Bureau" test method, it was found that the water saturation point of some soils would lead to disagreement as to when the saturation point occurred. As a result, the method for measuring the resistivity of soils was investigated to determine a standard means for testing.

Figure 3 shows the measured resistivity of soils will decrease with continued additions of distilled water to a minimum value—a characteristic of all soils. With the addition of more water, the soil solution with its contained salts becomes diluted in respect to free ions and the resistivity increases. In this test method, the soil resistivity value that is utilized is the minimum value obtainable.

Normally the minimum resistivity of a soil does not duplicate what

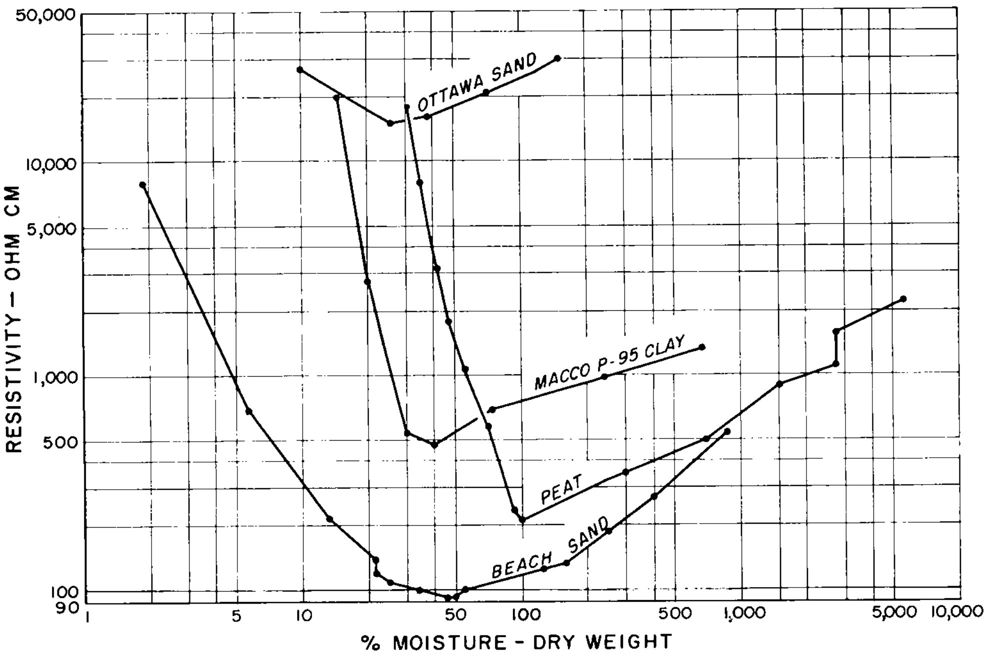


Figure 3. Moisture content *vs* resistivity of soils.

might be measured under field conditions, as the moisture content of the soil is a controlling factor. The moisture content of an in-situ soil could be a variable depending on the time of year and the rainfall. As all tested soils were found to have a minimum resistivity at the optimum moisture content, it is considered that this value is a common denominator and is a basic characteristic of soils.

It was observed that the soil resistivity values seem to be related to their hydrologic location. In areas of high rainfall, the soil resistivity was high; conversely, it was low in areas of meager rainfall.

As indicated by this relationship between soil resistivity and the average annual rainfall (Fig. 4), the presence of soluble salts in surface soils appears to be primarily controlled by the average annual rainfall.

Influence of Soil pH

When other factors are equal, the pH or hydrogen ion concentration has been found to be an indication of corrosive soils or waters (*10, 12*). Normally, as the pH of the solution increases, the corrosion rate of steel decreases. A pH of less than 7.0 is an acidic condition, a pH of 7.0 is neutral, and a pH of greater than 7.0 is an alkaline solution.

In this study the pH values were determined by an electrometric method involving the simultaneous use of a glass and a calomel electrode.

The pH measurements of the soil solutions generally were obtained by mixing 1 part of soil to 1 part of distilled water by volume in a glass beaker or waxed paper cup. There is no absolute or consistent proportion of water or soil to be mixed for measuring the pH value. It has been reported that ratios of 1 part soil to 5

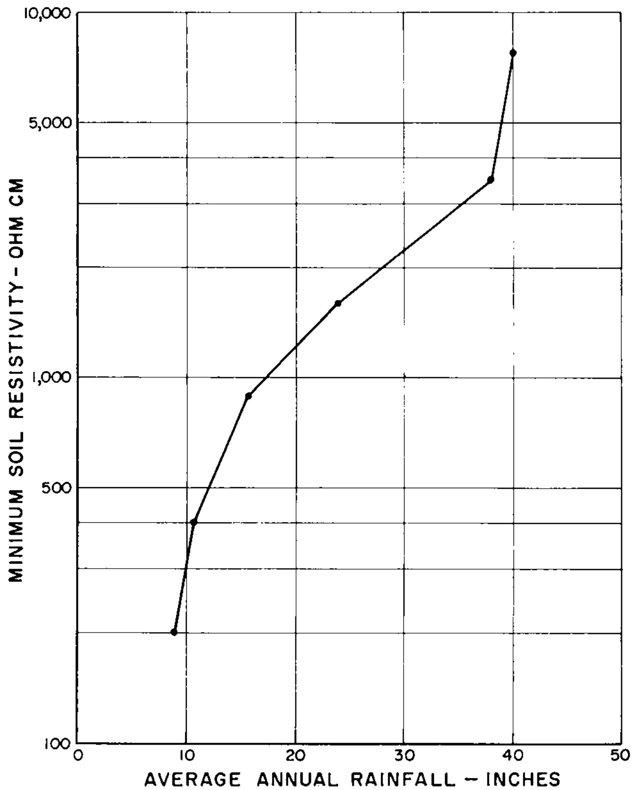


Figure 4. Soil resistivity *vs* average annual rainfall.

parts water have been used when the soil is high in organic content (*2, 11*).

During this investigation, the pH of a soil appeared to vary according to the hydrologic area. In general, alkaline soils were found in areas of low rainfall, and acidic soils were found in areas of high rainfall. Figure 5 shows this trend of the variations of the average pH of surface soils and rainfall values. Also, all the soils located in areas of the same rainfall did not have the same pH or soluble salt content. This appears to be the result of the variation between drainage, vegetation, and the usage of soils in various locations. The measured soil pH is a value that could be the result of the variables affecting the land in a particular location; the average annual rainfall is considered to be indicative of an expected over-all average condition.

The culvert corrosion rate data obtained during this study were analyzed to determine if there was a correlation between the average annual rainfall and the life of metal culverts. A direct correlation between rainfall and culvert life was not obtained; therefore, it was concluded that the higher rainfalls serve to create the corrosion-causing environment rather than to establish the actual corrosion rate of culverts directly.

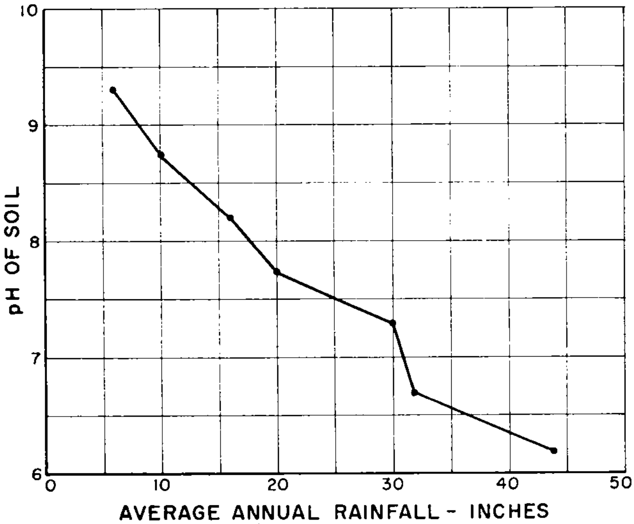


Figure 5. pH *vs* average annual rainfall.

Derivation of Chart

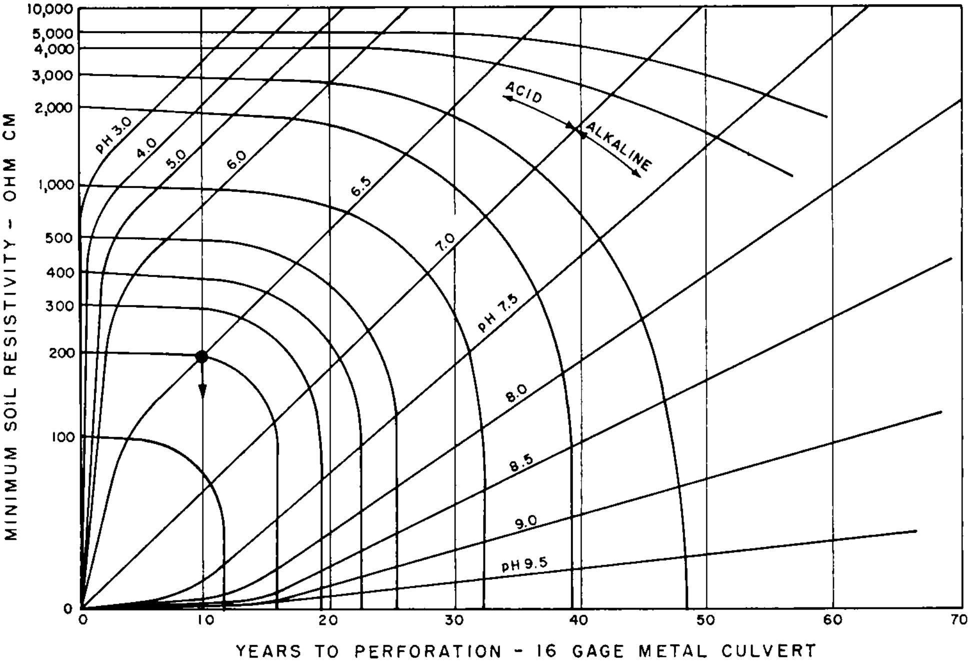
The three variables found to be possible major indices of the corrosion rate of metal culverts were average annual rainfall, pH, and the electrical resistivity of the soil.

Initially a linear equation was evaluated by the method of least squares (*13*) to determine the relative influence of the three environmental variables on the performance of metal culverts. The resulting equation had a correlation coefficient of 0.219, which was indicative of a correlation at the 5 percent level of significance. However, when this linear equation for correlating the life of metal culverts to the considered variables was checked by computing the actual data in the equation, it was apparent that the relationships were not linear nor accurately expressed by the equation, except at the mean values.

Therefore, the data were arranged into graphs on which culvert life was plotted against rainfall, resistivity, and pH. Then the same environmental factors were plotted against one another to examine a number of different relationships by trial and error.

Figure 6 is a graphical solution of the data and represents the influence of the average pH and resistivity values that were measured at each culvert channel and then compared to the corrosion rate of the existing culverts. The degree of correlation between the mathematically computed theoretical life as determined from Figure 6 and the actual life of the inspected metal culverts was computed by the method of least squares. There was a correlation coefficient of 0.344, indicative of about 0.08 percent level of significance. The standard error of estimate of the theoretical life for culverts was found to deviate by 12 years of the actual service life. The investigation included culverts that were bituminous coated for corrosion protection; however, only those culverts without the coatings were analyzed in the original calculations.

The estimated years-to-perforation of a metal culvert (Fig. 6) does not necessarily mean that the culvert will



EXAMPLE: Given, pH = 6.5 & Resistivity = 200 ohm cm
Then 16 gage CMP perforated in 10 years.
For a culvert metal gage of 12 multiply
years by factor below. i.e. 1.8 x 10 = 18 years

Gage	14	12	10	8	6	2	0	000
Factor	1.3	1.8	2.3	2.8	3.3	4.3	5.0	6.0

Figure 6. Chart for estimating metal culvert corrosion rate.

collapse or that its usefulness as a carrier of water will cease. Instead, this terminology of years-to-perforation is used as a common yardstick for all culverts. If the arching action of a fill is sufficiently substantial to warrant disregard of a perforation or loss of the culvert invert, then the arching action of the fill could be considered in the mechanics of the design.

For the present, it is concluded that there is a linear relationship between service life and the thickness of metal. For example, it is assumed that in the same environment a metal culvert of 8-gage thickness will last about three times longer than a culvert of 16-gage thickness. It is believed that there are locations in which the time to perforation of the culvert metal will not be proportional to the metal thickness. It is not known, however, whether the deviation between the corrosion rate and the metal thickness will consistenly favor an added or reduced perforation time. For instance, Romanoff (*2*, Figs. 11, 52, and 56) shows that the underground corrosion rate of steel and galvanized steel generally is relatively linear with time after approximately 2 years of exposure. In most of the soils, the corrosion rate reduces with time and is considered to be the result of the formation of a more or less protective rust or oxide coating on the surface of the metal.

In the case of highway culverts, there are numerous examples in which a corrosion-inhibiting film may not be substantial or remain on the metal surface that is in contact with a flow that carries abrasive particles. These particles can and do abrade the corrosion-inhibiting oxide film from the surface of the metal.

Inasmuch as Figure 6 is based on the data from culverts that have been in service from 10 to 40 years, the average effect of abrasion on the corrosion rate of the culverts is considered to be included in the chart. Because the effect of the thickness of the culvert metal on the corrosion rate is still open to speculation, the direct ratio method may be the most reasonable assumption for the present time.

Bituminous Coating

The theoretical life of plain, galvanized culverts and the estimated service life for bituminous-coated pipes in the same locations was statistically analyzed to determine whether the life attributed to the coated pipes would affect the theoretical value. The results of the statistical analysis for the 68 asphalt-coated pipes resulted in a correlation coefficient of 0.324, which is indicative of about 0.08 percent level of significance. The standard error of estimate of the theoretical from the actual data was found to deviate by 14.5 years. This analysis indicated a greater deviation between the theoretical and the actual life of the bituminous-coated culverts than was found for galvanzied culverts. It appeared that the coating definitely increased the life of culverts.

Because the average life of the plain, galvanized culverts was found to be about 6 years less than the life of the bituminous-coated pipes, it was decided to determine statistically whether the average difference in life as indicated by the two sets of calculations could be the result of chance.

The statistical t-test demonstrated that there was a difference in the average life of the coated and uncoated pipes at the 2 percent significance level, indicating that the difference was not likely to be due to a chance sampling. Therefore, the over-all gain in service life of galvanized pipe in California is about 6 years. This difference is significant, but it should not be assumed that the service life of all pipes will be extended 6 years by the coating as a highly abrasive flow could remove the coating during one period of flow. Conversely, it has been observed that in the desert areas of infrequent flow, the coating could increase culvert life by at least 20 years. (The asphalt coating referred to in this study was in place on old existing pipes and was an uncontrolled commercial type of dip. The presently specified bituminous coating and lining material is superior and should result in a greater increase in life).

OPERATIONAL TEST PROCEDURE

Two environmental factors are combined for estimating the service life of metal culverts. These environmental factors are the hydrogen ion concentration (pH) and the electrical resistivity of the site and backfill materials. The hydrogen ion concentration (pH) of the soils and waters indicates the degree of acidity or alkalinity, and the resistivity measurements indicate the relative quantity of soluble salts. The probable service life of a metal culvert in a given location is estimated by using the chart in Figure 6.

This information, combined with

observations of existing culverts, if any, provides a basis for (a) estimating the service life of galvanized metal culverts and (b) estimating the additional life obtained by coating the culverts to reduce the corrosion rate. The test method is divided into five steps:

1. Field resistivity survey and sampling for corrosion tests,
2. Preliminary field determination of pH of water samples,
3. Determination of pH of soils,
4. Laboratory determination of minimum resistivity, and
5. Estimation of service life of metal culverts from test data.

Field Resistivity Survey and Sampling for Corrosion Tests

The field resistivity test is an indication of the soluble salts in the soil or water and is used primarily as a guide for selecting samples that will be further tested in the laboratory to obtain data for estimating the service life of culverts. The natural soil in each channel or culvert location and the structural backfill material are tested by a portable earth resistivity meter, and samples are selected on the basis of these tests.

A. Apparatus
   1. Portable earth resistivity meter, suitable for rapid in-place determinations of soil resistivity.
   2. Field probe.
   3. Steel starting rod, for making hole (in hard ground) for inserting probe.
   4. Sledge hammer (4 lb).

B. Materials
   Distilled, de-ionized or other clean waters that measure greater than 20,000 ohm cm.

C. Recording Data
   Record test data in a field notebook for use in selecting samples and also for use as needed in analyzing laboratory test data.

D. Test Procedure
   1. In the channel of a proposed culvert site, insert the field probe into the soil for a depth between 6 and 12 in. and measure resistivity. Remove the field probe and pour about 2 oz of clean water into the hole.
   2. Re-insert the probe, while twisting to mix the water and soil, then measure the resistivity. Follow manufacturer's instructions for correct use of meter.
   3. Withdraw the field probe and add an additional 2 oz of clean water.
   4. Re-insert the probe and again measure the resistivity of the soil.
   5. Record the lowest of the readings as the field resistivity of the soil.

E. Selection of Soil Samples for Laboratory Tests
   1. Make sufficient resistivity determinations at various locations in the channel or culvert site area to represent adequately the entire area.
   2. If the resistivity is reasonably uniform within the limits of the project, three soil samples from different culvert locations will be sufficient. If, however, some locations show resistivities that differ significantly from the average of the determinations for the area being surveyed, additional soil samples should be taken to represent these locations—particularly those with resistivities significantly below the average.

      For example, if the soil resistivities throughout the surveyed area are all at or near an average value of 2,000 ohm cm, three samples will be enough. If any of the locations tested have resistivities markedly below this average (for example, 800 ohm cm) then these "hot spots" should definitely be represented by additional samples. Scattered locations of higher resistivity (for example, 3,000 ohm cm or more) do not necessarily require additional samples.

Judgment must be exercised both in the field testing and sampling and in evaluating the laboratory tests. In all cases, do not take less than 3 samples.

F. Precautions

In field testing and sampling, follow very carefully the test method instructions and also the manufacturer's instructions for use of meters.

Notes.—If the minimum resistivity of a soil is determined to be less than 3,000 ohm cm in the laboratory, a representative sample weighing at least 2 lb which passes the No. 8 sieve will be needed for a sulfate (SO_4) analysis. This should be taken into account in field sampling and is to be used for evaluating the effect of the environment on the stability of normal concrete.

Preliminary Field Determination of pH of Water Samples

This method is suitable for use in the field or laboratory for determining the pH of water samples.

A. Apparatus and Materials
1. Two ounce or larger widemouth container, *e.g.*, glass jar, beaker, or dry wax paper cup.
2. pH meter, suitable for either field or laboratory testing.
3. pH standard solution of pH 7.

B. Recording Data

Record test data in a field notebook.

C. Method of Sampling
1. Dip the wide-mouth container into the water to be tested. Swirl to rinse and pour out contents to avoid contamination from container.
2. Dip into the water again for obtaining a sample.
3. Pour off any film that is on the surface of the sample before testing.

D. Standardizing pH Meter

Follow the instructions provided with the type of pH meter being used.

E. Use of pH Meter to Determine pH of Water

Follow the instructions provided with the type of pH meter being used.

F. Precautions

Follow the manufacturer's instructions for use of the meter and observe the usual precautions for making chemical tests.

Notes.—pH readings may be taken at any period other than flood flow. All waters having a pH of less than 6 should be sampled for further analysis, in 1-qt bottles.

Method of Determining pH of Soils

This method is suitable for use in determining the pH of soil samples.

A. Apparatus and Materials
1. Paper cups, 2 oz, wax-coated type.
2. Teaspoon or small metal scoop.
3. Wash bottle containing distilled water.
4. pH meter suitable for field or laboratory testing.
5. pH standard solution of pH 7.

B. Recording Data
 Record data in a field notebook.

C. Preparation of Test Specimens
 1. Place 2 rounded teaspoons of the soil to be tested into a 2-oz paper cup.
 2. Add about two teaspoons of distilled water to the sample in the cup.
 3. Disperse soil in water by stirring. The specimen is now ready for testing.

D. Standardization of pH Meter
 Follow the instructions provided with the pH meter.

E. Use of pH Meter to Determine pH of Soil
 Follow the instructions provided with the pH meter.

F. Precautions
 Carefully follow the above procedure and the manufacturer's instructions. If the pH reading is unstable when the electrode is immersed in the soil slurry, leave the electrode immersed until the pH reading has stabilized. In some cases this waiting period for the stabilization of the pH reading may take 5 min.

Laboratory Determination of Minimum Resistivity

This method covers the procedure for determining the minimum resistivity of soil or water samples selected as indicated previously. These resistivity values are used in estimating culvert life as described subsequently.

A. Apparatus
 1. Resistivity instrument suitable for laboratory testing.
 2. Soil box calibrated for use with resistivity meter (See Fig. 7 for details).
 3. No. 8 sieve.
 4. Round tin pans, 12-in. diameter and 2 in. deep.
 5. 200 F oven.

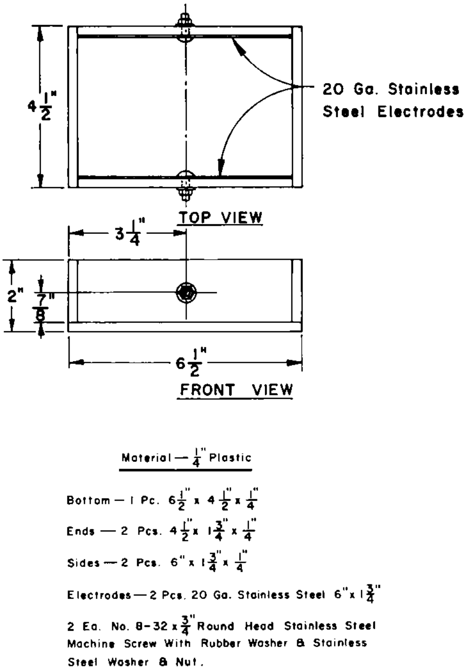


Figure 7. Soil box for laboratory resistivity determination.

 6. One balance, 5 kg capacity, accurate to 10 g.

B. Materials
 Distilled or de-ionized water.

C. Recording Data
 Record data in notebook.

D. Preparation of Soil Samples
 After thoroughly mixing sample, screen it through a No. 8 sieve. If the sample is too moist to be sieved, it may be dried and crushed. Do not crush rocks. Only the natural material that passes the No. 8 sieve is to be used for the test.

E. Measuring Resistivity of Soil Sample
 1. Quarter or split out about 1,300 g of the passing No. 8 material.
 2. If the sample was dried, add about 150 g of distilled water to the 1,300 g of soil and mix thoroughly.

3. After the soil sample is thoroughly mixed, place and compact it (moderate compaction with the fingers is sufficient) in the soil box.
4. Measure the resistivity of the soil in accordance with the instructions furnished with the meter.
5. Remove the soil from the soil box, add about 100 more grams of distilled water and again mix thoroughly.
6. Again place and compact the soil in the soil box and measure its resistivity.
7. Repeat this procedure.
8. If the resistivity of the soil has not followed a trend of high resistivity, low resistivity, and then an increase in resistivity for the preceding additions of distilled water, continue to add water in about 50-g increments to the soil; mixing, placing, compacting, and measuring resistivity for each increment until the minimum resistivity is obtained.
9. If the sample was not dried, begin the test procedure by adding 50 g of water in lieu of 150 g specified previously. Continue to add 50-g increments of water followed by mixing, placing, compacting, and measuring until a minimum value of resistivity is measured.
10. Record the test value that is the minimum value of soil resistivity at any moisture content.

F. Measuring Resistivity of a Water Sample
1. Thoroughly clean the soil box of all soil particles and rinse the soil box at least three times with distilled or de-ionized water.
2. Fill the soil box with distilled water and measure its resistivity.
3. If the distilled water in the soil box measures infinite resistivity, empty the soil box of distilled water, fill with the test water, measure its resistivity, then record the measured value.
4. If the distilled water in the soil box did not measure infinite resistivity, continue to rinse the box with distilled or de-ionized water until the box is thoroughly clean, which is indicated by an infinite resistivity measurement.

G. Recording Data

Record data in notebook.

H. Precautions

Follow the above instructions very carefully.

Estimating Service Life of Metal Culverts from Test Data

Using the minimum resistivity and the pH values of the soils or waters, obtained as described in Steps 2, 3, and 4 of this test method, determine the estimated service life (years to perforation) from Figure 6.

TEST CORRELATION

Operational Test Accuracy

As previously pointed out, this investigation indicated that the accuracy of the test method when compared to existing facilities is ± 12 years. Since the time of the original findings, the test method has been distributed to each of the eleven California Highway Districts and has been used on an operational basis for approximately two years. The District personnel have been given the responsibility of investigating all existing culverts on each section of highway that is to be realigned or reconstructed. Generally the age and

TABLE 1

SUMMARY OF TWO YEARS OF ROUTINE OPERATIONAL USE OF TEST RESULTS VS EXISTING CULVERT CONDITIONS[1]

Proj.	Exist. Culverts with Inspected Perforation Time in Excess of 50 Years[2]			Existing Culverts with Inspected Perforation Time Less than 50 Years		
	Actual Service All Culverts (yr)	No. of Insp. Culv.	Est. Time to Perf. by Test (yr)	No. of Insp. Culv.	Average Time to Perf. by Culv. Insp. (yr)	Est. Time to Perf. by Test (yr)
1	10–20	7	60	4	25	20
2	12–30	9	60	5	40	40
3	30	2	50			
4	33	7	50			
5	20			3		35
6	30			15		20
7	11			4		40
8	20–40	12	60	1	40	45
9	12			1		25
10	23	27	60			
11	35			6		40
12	20			2		35
13	25			4	30	35
14	20			6	15	20
15	30			3	45	45
16	37	3	35	3	40	35
17	35			2		50
18	28			10	25	30
19	6			4	10	20
20	30	4	70			
21	15	7	45			
22	38	1	20	4	40	35
23	30			1		13
24	25			3	40	35
25	20			3	40	40
26	20	5	45	1	20	15
27	9			2		50
28				1	20	25
		84[3]	55[4]	88[3]	30[4]	30[4]

[1] Culvert perforation time rounded off to nearest 5-yr increment of time.
[2] Inspected condition of these culverts clearly indicated they would not be perforated by corrosion in less than 50 yr of exposure.
[3] Total.
[4] Average.

condition of each pipe have been determined and the necessary tests performed so that a determination may be made in advance of a new project for preventing the accelerated corrosion of metal or the deterioration of concrete culverts, which is indicated by the pH and the sulfate (SO_4) of the soils; or the feasibility of extending or replacing the existing pipes.

Table 1 gives the results of 28 highway project investigations by the District personnel. These investigations represent the studies of various sections of highways that are being considered for realignment and reconstruction. This table contains the results of a total of approximately 170 corrosion tests performed at the sites of the same number of inspected pipes. The number of investigations given in the table does not represent the total number of projects or culverts that the Districts have investigated. Numerous projects have covered highways that were on such drastic realignment that the condition of existing culverts beneath nearby highways could be misleading, or were not representative of the environment in the new highway location. Therefore, in these cases the accuracy in the use of the test method could not be compared to the corrosion rate of an existing culvert.

Table 1 shows the culvert test favorably compares to the actual culvert corrosion rate observed in the field. For example, on the highway projects that were investigated by the Districts, 84 inspected culverts had an estimated culvert perforation time that is in excess of 50 years. The culvert test method that was performed at these same culvert sites indicated an average perforation time of 55 years.

The reason that the culvert data were not projected beyond the 50-year period was that some of the culverts had an apparent life that bordered on infinity and so could not be used to obtain a mathematical average. For instance, some culverts that have been in place for 30 or 40 years still had the zinc coating intact and were anticipated to resist perforation by corrosion beyond any reasonable limit of calculated time.

At the 88 culvert installations inspected by District personnel and listed under the general heading of culverts with less than 50 years to perforation, the average perforation time of these culverts was estimated to be 30 years by inspection, and by the test method the average perforation time was also 30 years. Thus far, the test method appears to be a reasonable tool when in the hands of trained personnel.

Comparison to Work of Others

Generally, if a new test method can be verified by means of independently acquired data, then it is considered that the significance of the test is increased. A search of the literature indicated that such culvert corrosion rate data are not available for comparison to this test method. Therefore, it was compared with results obtained by the National Bureau of Standards in its nationwide studies of underground corrosion. The culvert corrosion rate chart (Fig. 6) when converted to underground corrosion rates was found to have a high degree of correlation to the Bureau's data. Also, when this test method was compared to the reported data of seven different types of corrosion test methods, it was observed that it was one of the more accurate means for determining relative soil corrosivity (*14*).

Because of the indicated high degree of correlation of this test method to independently acquired data, a search of the literature was undertaken to determine the reason.

Romanoff (*2*, p. 14), in a discussion of the various corrosion tests and the factors considered to be a cause of corrosion, stated, "However, these factors are interrelated, and it is difficult to control conditions so that there is only one variable. In the absence of such control, the correlation may be difficult or indefinite, ..." As the corrosion test method described in this paper evaluates two variables (pH and resistivity), it is apparent that this is the circumstance that is responsible for its relatively high degree of correlation. Also, when additional variables are related to the corrosion phenomenon, there will be a substantial increase in the accuracy of this or any other test.

FINDINGS

1. The life of metal culverts is a variable depending primarily on the presence of moisture and the chemicals contained in the watershed.
2. The chemical environment of a specific culvert site is directly influenced by the soils, vegetation, rainfall, and drainage characteristics of the watershed; also by the frequency and the volume of the flow of water as well as the character of the culvert backfill soil used at the site. The chemical environment of a watershed directly affects the pH and electrical resistivity of a culvert site.
3. A relatively accurate estimate of the corrosion rate of galvanized metal pipe in a specific location can be made by using pH and resistivity values of the soils and the waters. The relationships are given in Figure 6.
4. A procedure for determining the

pH and resistivity of a culvert site is outlined under Operational Test Procedure in this report.

5. Bituminous coatings were found on the average to add six years to the life of CMP culverts. However, this benefit cannot be indiscriminately applied as the actual added life varied from nearly zero in areas of continuous flow carrying heavy debris to over 20 yr in arid areas with infrequent runoff. The bituminous coatings covered in this study were of a lower quality than that presently used by California. It is too early to report on the effects of the new coating; however, its characteristics should result in a marked improvement over the old.

6. The usefulness of a bituminous coating lies in its ability to serve as a moisture barrier. Therefore, its protective life is limited in streams where it will be subjected to an abrasive flow that will result in the rapid removal of the coating from the culvert invert.

SUMMARY

Generally the estimation of culvert perforation time by using the test method when compared to field inspections of existing facilities has been remarkably close. Even so, it is considered that the culvert test method is a substitution, and at most a verification of the evaluation of an existing facility. It should be relied on solely only when existing facilities are not available for inspection.

All the investigations by the California Division of Highways have shown that the perforation time of a zinc-coated corrugated steel culvert usually can be extended by means of a protective coating. The previous work has also shown that further study is required to determine how many additional years of protection will be gained by the various modern means of protecting metal pipe from corrosion.

This study has indicated that the bituminous coating of metal culverts will extend the perforation time of the culvert by approximately 6 yr when the pipe is subjected to a corrosive flow. It is known that the bituminous coating will protect the metal from corrosion on the backfilled side of the pipe for more than 6 yr. It has been assumed that the asphalt coating will protect the surface of the pipe that is in contact with the backfill soil for at least 20 yr. This is primarily speculation, but is also based on the observation of the condition of numerous coated pipes that have been removed after 20 to 30 yr of service.

The corrosion rate chart (Fig. 6) was derived from the comparison of the observed corrosion rate of existing galvanized corrugated metal culverts in California to the described environmental factors. As a result, Figure 6 is an average value of the effect of frequency of flow, abrasion, bacterial action, and other known and unknown variables on the perforation rate of the culvert wall.

As indicated by operational use of this culvert test method, inaccuracies can and do occur when culverts are subjected to greater than normal periods of water flow or excessive abrasion or lack of normal soil moisture. The degree of error in the test method when compared to the inspected life of such unusual installations is being studied and will be evaluated at a later date. Irrespective of these few exceptions it appears that the accuracy of the test method is sufficient to warrant its continued use. Discrepancies in its accuracy can be evaluated because there is now a means by which unusual circumstances can be recognized as such, and it is possible that they may be given a numerical value.

The use of the test method by the California Division of Highways when considered with proper hydraulic and foundation information has placed the use and the method for protecting corrugated metal culverts on an engineering basis. Previously the judgment of the effect of the environment on corrosion was generally controlled by experience or prejudice or a combination of the two.

ACKNOWLEDGMENTS

This investigation, which resulted in the development of a test for estimating the corrosion rate of metal culverts, was conducted as one of the activities of the Materials and Research Department of the California Division of Highways.

The authors wish to express their appreciation to F. N. Hveem, Materials and Research Engineer, for his advice and direction during this study; also to the numerous personnel of the California Division of Highways and those of the Materials and Research Department who extended their aid and cooperation during this study.

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A Practical Method for Constructing Rigid Conduits Under High Fills

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This paper gives a brief account of the design and construction of reinforced concrete pipe culverts in Humboldt County, Calif. These culverts were constructed as convential positive projecting conduits, but with a layer of baled straw placed directly on the pipe to create a condition wherein the central prism of soil above the conduit would settle more than the soil adjacent, thereby insuring the development of upward shearing forces; that is, an arching action, which greatly reduces the load on the structure. The resulting action is comparable with that obtained for negative projecting conduits (California Method B) or for imperfect ditch conduits.

Favorable results were obtained by this straw method of construction in the case of two 54-in. and one 66-in. concrete pipe culverts under fills 38 to 65 ft in height. Performance of the pipes was measured by observing the distortion of these pipes compared with the distortion of similar pipes installed conventionally. Distortion is defined as the difference between the horizontal and vertical diameters of the pipe, expressed as a percentage of the theoretical pipe diameter.

The performance of the culverts in which the baled straw method was used was much better than those in which it was not used.

• THIS PAPER is a brief account of the design and construction of reinforced concrete pipe culverts in Humboldt County, Calif. The culverts were constructed as conventional positive projecting conduits with a layer of baled straw placed directly on the pipe to obtain the loading advantages of negative projecting conduits.

The first test pipe was placed in July 1958 and is in excellent condition. This pipe and two subsequent ones have been field-checked along with three other conventional pipes on similar foundations. Figure 1 shows a comparison by distortion, which was the only practical method available without special funds for pressure plates.

In recent years there have been a number of reinforced concrete pipes in this area that have deflected and cracked seriously. It is not the purpose of this paper to discuss these installations and attempt to find factors that contributed to the failure. In general, these pipes have been placed on firmer foundations than the adjacent native soils and the foundation has usually been improved for a width of one diameter on each side of the pipe. The pipe had been produced by an established company and sample tested. Construction has been performed both by California method A (positive projecting), wherein placement consists of placing embankment to 30 in. or ⅓ diameter,

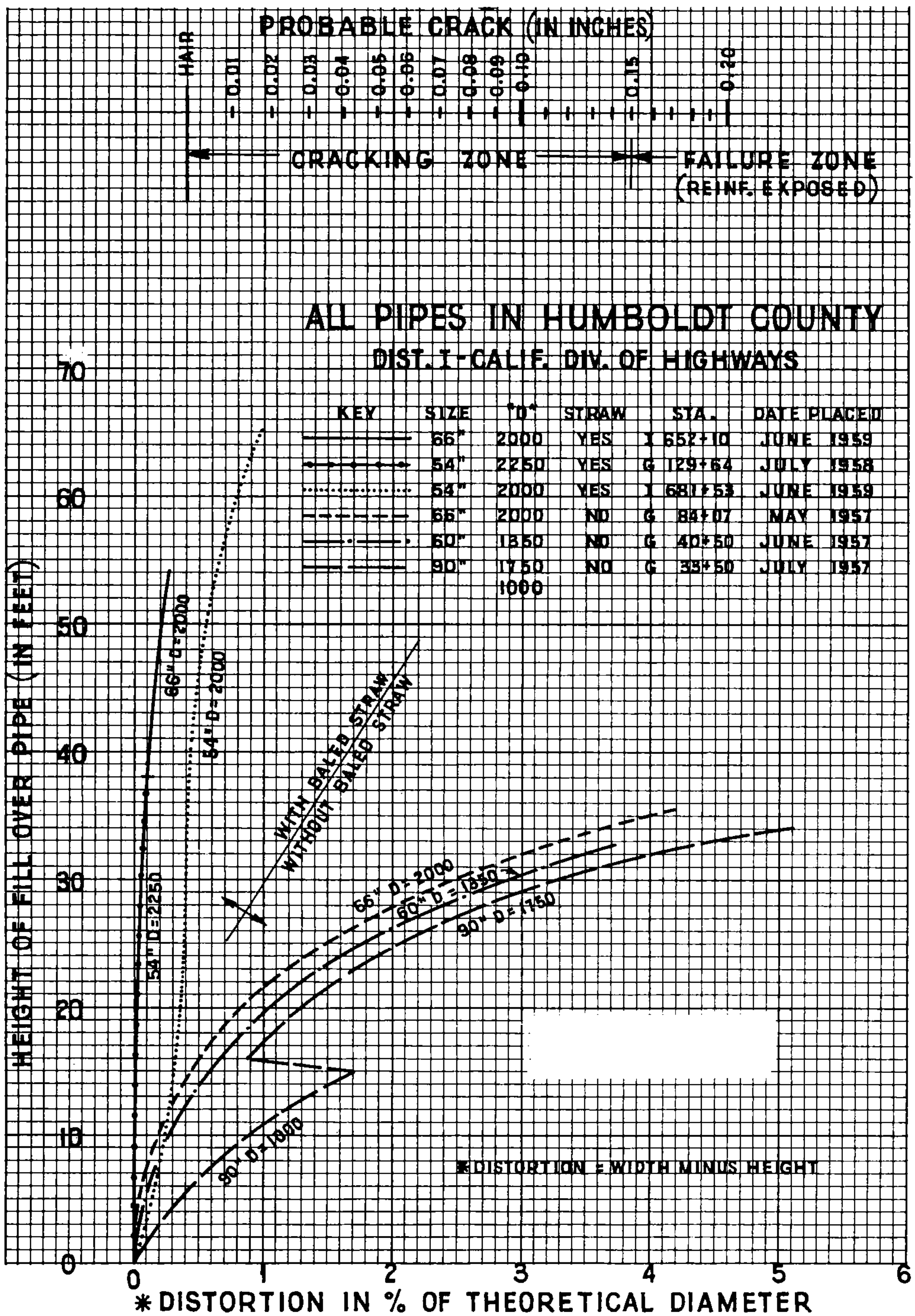

Figure 1. Report on in-place reinforced concrete pipes.

whichever is greater, above flow line, excavating a trench, placing the pipe, and backfilling with granular material; and by Method B (negative projecting with loose earth cover only), which calls for pipe placement in an excavated trench 2 diameters deep with vertical sides and with backfill compacted to the top of the pipe and remainder placed in the loosest possible condition. In all cases the cracking occurred immediately after the fill was placed and has not reappeared noticeably after repairs were made.

Only one of the cracked pipes was placed by Method B (negative projecting with loose earth backfill in a trench for 1 diameter above the pipe). The failure is generally attributed to a misunderstanding of the method of denoting the top of the pipe during placement. Most engineers seem to accept the idea that Method B placement does reduce the load on culverts under high fills. The only factors limiting its general acceptance are "practical" in nature:

1. Excavation quantities are about 3 to 4 times greater;
2. Expensive shoring is often required to preserve vertical slopes and protect workmen;
3. Modern fast-moving grading operations are often delayed by a relatively slow Method B installation;
4. An inspection (or enforcement) problem arises if the trench sides cave in—it is virtually impossible to restore material to a vertical slope; and
5. A trench filled with loose earth is difficult to maintain in that condition with heavy earth-movers and water trucks crossing it constantly.

To provide the benefits of Method B and eliminate its undesirable characteristics, the use of baled straw placed directly on a pipe that has been installed by the conventional Method A was proposed. Figure 2 showing this type of installation was distributed to designers in District 1 on November 13, 1957.

The theory of this design is summarized as follows:

1. In baled form, straw is easy to handle and resilient enough to prevent compaction of earth immediately above.
2. The sides of baled straw are vertical and provide a form to support the adjacent fill laterally during compaction. This eliminates the necessity of constructing the fill above the pipe, excavating, and backfilling with loose material.
3. Baled straw is compressible under load and will allow for settlement on both sides of the culvert without adding to the load directly on the culvert. Presuming Marston's theory to be correct, the baled straw will reduce the initial static load on the culvert to about one-third of the normal load imposed by the weight of the fill directly over the culvert.
4. Straw is organic and will therefore rot with the passing of time to produce a void over the pipe.

TEST INSTALLATIONS

The first test pipe was placed July 18, 1958, at Station G 129+64 in Humboldt County, Calif. (Fig. 3) on US 101, about 16 mi south of Eureka. It is a 54-in (2250 *D*-Load*) by 280-ft R.C.P. under a 40-ft fill. The pipe has been checked on several occasions, with a recorded inspection on May 13, 1959. This fill (40 ft to finished grade) is at the maximum allowable under current design standards for this pipe if Type A backfill is used. As reported May 26, 1959, no cracking or distress of any kind could be found in the pipe.

The second and third test installations were placed in June 1959. They

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* Actual test load (in pounds per linear foot of pipe per foot of inside diameter) under the 3-edge bearing method which produces a 0.01-in. crack throughout a length of 1 ft.

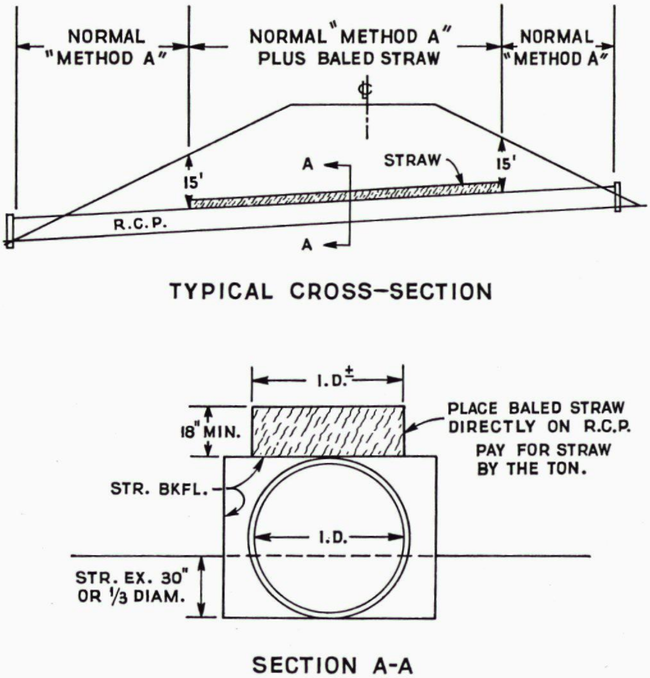

Figure 2. Proposed baled straw installation.

Figure 3. First known installation (July 1958) of reinforced concrete pipe with baled straw protection. Baled straw being placed directly on top of pipe after conventional granular backfill completed.

were a 54-in. (2000 *D*-Load) by 344-ft R.C.P. at Station I 681+53 on US 101 about 20 mi north of Eureka under 65 ft of fill, and a 66-in. (2000 *D*-Load) by 299-ft R.C.P. at Station I 652+10 under 54 ft of fill. The installations are under fills that exceed design limitations for Method A by 30 ft and 20 ft, respectively.

After discussing the actual construction details with the resident engineers, the following observations were made:

1. The 54-in. R.C.P. at Station G 129+64 was placed carefully to insure compaction of the fill on both sides of the straw. A sheepsfoot

roller was operated parallel to the pipe on both sides until the fill was about 2 ft above the straw (Fig. 4).

2. Although none of the pipes with straw cover shows any real signs of distress, the 54-in. R.C.P. at Station I 681+53 does indicate that an unequal foundation (yielding with rock ridges crossing the pipe transversely) does have a tendency to cause some light cracking in spite of the straw backfill. The two rock ridges encountered in the foundation at Station 681+53 were located in the 20- to 30-ft fill range and caused the distortion curve for the pipe to make an erratic bend, even though the rock was excavated to a depth of 4 ft below the pipe.

The 66-in. R.C.P. at Station 652+10 passed very close to a rock ridge at joints 18 and 19. (The pipe was curved to miss the rock.) The only significant cracks in the pipe occurred at this location on the side opposite the rock. A case of this sort demonstrates the effect of variable backfill conditions.

CONSTRUCTION DETAILS

1. Bales should be turned on edge or sides to avoid splitting for a particular width. The nearest dimension ±6-in. of the inside diameter should be satisfactory.

2. The binders on the bales, should be cut when the embankment is even with the top of the bales, but it does not seem to be of great importance if forgotten at the crucial moment.

3. The straw should never be placed to the extremities of the fill. This would permit water to pipe through above the culvert.

4. Compaction adjacent to the straw is important. Ramping up and over the straw for hauling equipment should be allowed only at the ends. When the fill height has reached the top of the straw, the culvert location should be marked with lath and compactive effort held to a minimum over the straw for an additional height of ½ in. in diameter. This means that rollers should continue to turn parallel to the culvert and work the fill in two sections. Earth-movers should

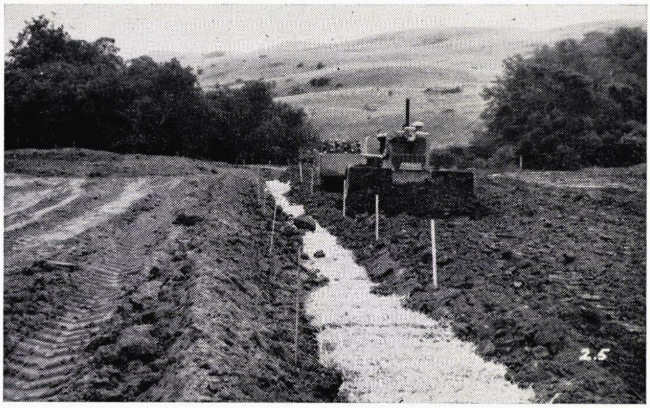

Figure 4. Embankment being placed adjacent to straw. This method was used until fill was about 2 ft above straw. Earth-moving equipment crossing pipe only at ends and dumping parallel to pipe; inconvenience to contractor slight. No problems encountered in this first installation.

also be encouraged to make their unloading pass adjacent and parallel to the culvert.

5. An economical means of checking the loading conditions on a pipe culvert is to measure the horizontal and vertical axis of each joint (or at intervals of 8 ft± for C.M.P.) with no load, full static load, and again about two years after construction.

6. The construction details should be covered in the special provisions for the project as the method will probably be unfamiliar to both the contractor and engineers. If possible, the designer should visit the project just before construction of the culverts to make certain that the basic principles are understood.

Figure 5. Trenching and straw placement above existing 36-in. R.C.P. (D-load=1,750 lb) to be covered with 62-ft fill to convert existing expressway to full freeway. Trench cave-in will do no serious harm, even though it is desirable to maintain vertical sides. Excavation being performed in unsuitable material from previous construction project.

7. The baled straw method of construction works very well where a culvert is already in place (Fig. 5) and the fill is to be increased beyond the allowable limit. A trench is excavated to within 3 ft± of the pipe, straw is placed immediately (so trench cave-ins do not have to be re-excavated) and the trench filled with loose earth. Only one installation of this type has been completed. It was placed in September 1960 and shows no signs of failure, even though the pipe was located in a disposal area for unsuitable material on a previous project and the fill exceeds the "allowable" under Method A by some 35-ft.

CONCLUSIONS

1. It is now reasonably conclusive that a layer of baled straw does produce the desired load reduction on the pipe. The amount of this reduction will vary with foundation conditions and degree of compaction of the material immediately adjacent to the straw. It would seem that fills of about three times the height allowable for normal Method A installation could be supported without serious damage to the pipe under average conditions (*1*). Under favorable conditions (uniform foundation under pipe and adjacent area, plus proper design and installation), it is not unreasonable to assume that fills of unlimited height can be safely constructed over rigid R.C.P. with straw cover.

2. The depth of straw required for a particular installation should be based on size of culvert, relative differential in foundation under and adjacent to the pipe, the height of fill to be supported, and the type of material used to construct the fill. As much as 5 ft of baled straw is being proposed on a 14-ft reinforced concrete arch under 180 ft of fill in Mendocino County at Mallo Pass Creek. The use of straw on an arch

was not previously considered to be of value but a recent 14-ft arch installation under 80 ft of fill at Luffenholtz Creek in Humboldt County is settling and separating at the lower portions of the expansion joints (pivoting about the crown). It is hoped that the use of baled straw will prevent this type of settlement.

3. Experiments are being conducted with C.M.P. under fills in excess of that normally permitted for a particular gage and bolt spacing. If successful, this will allow construction of fills that were previously avoided because of the cost of providing a suitable drainage facility. Today's higher traffic volumes and speeds require better alignment, which in turn means heavier grading. Mountainous terrain often requires maintenance of all cut sections in side-hill conditions on both sides of a steep canyon that runs perpendicular to the desired direction of travel. Advantages in user benefits, earthwork balance, and future maintenance are apparent if the canyon can be crossed without swinging upstream into a switch-back situation just to keep the fill to a particular limiting height.

REFERENCE

1. SPANGLER, M. G., and SCHLICK, W. J., Iowa State College Bull., Eng. Rept. 14 (1952).

DISCUSSION

JOHN G. HENDRICKSON, JR., *Director of Engineering-Research, American Concrete Pipe Association.*—The author is to be complimented for having found an easy, practical, and an apparently effective method for reducing the load on culverts due to high fills.

The California Method B procedure has probably been more frequently used than any other method in obtaining a negative projecting culvert installation. That it is correct in theory and in practice also if properly carried out appears to be proved. Many alternate methods have been used and are generally classified as imperfect trench methods of construction. The difference is that in the imperfect trench the fill is built up over the pipe. A trench is excavated above the pipe and backfilled with compressible material.

Each procedure has certain advantages and disadvantages. From the contractor's point of view Method B procedure is often preferred. This allows the contractor to build the lower portion of the fill up to 1 diameter over the pipe unhampered by the presence of the pipe. This often may give him greater freedom in the movement of his equipment. However, in an area subject to sudden and severe storms, there is some risk of a washout unless some alternate drainage can be provided. The use of baled straw with the Method A construction decreases this risk.

Placing the pipe in a trench with vertical sides results in a load that is proportional to the trench width. The standard imperfect trench or the use of the baled straw results in a load proportional to the outside pipe diameter. Other things being equal, this load will be somewhat less.

The writer has always been somewhat skeptical of whether the backfilling around the pipe in a trench is generally as effectively carried out as when the fill is built up around the pipe. Consequently, lateral pressure may not be fully developed for the trench installation. The question then arises as to whether some of the problems with Method B construction have occurred out of a failure to develop lateral pressure on the pipe fully. Certainly if this has been a fault of Method B construction, the use of Method A with baled straw should be less susceptible.

The use of baled straw over the pipe effectively provides a compressible prism and avoids the need for the trench used in the standard imperfect trench procedure. The construction of this trench is not a major problem provided the contractor uses the proper equipment. A backhoe seems to work best in maintaining a straight trench with vertical sides, although a dragline also seems to work well. A trench 5 ft wide and 5 ft deep and about 250 ft long was completely excavated and backfilled with loose topsoil in about 5 hr on an installation in Illinois.

The thought of placing compressible material which may eventually decay in the middle of a highway fill has been accepted reluctantly by some highway engineers. Yet the use of leaves, straw, etc., is not entirely new. In his paper "A Practical Application of the Imperfect Ditch Method of Construction," M. G. Spangler describes the use of leaves, straw, and a quantity of surplus Christmas trees together with loose soil. This project was completed in the fall of 1956 and no settlements on the top of the embankment have been observed to date.

A portion of Interstate 74 in Vermilion County, Ill., in the Kickapoo Park Area crosses a ravine with a fill 37.5 ft above the top of a 48-in. reinforced concrete culvert. An imperfect trench method of construction was chosen. The trench excavated over the pipe was filled to about one-third of its depth with loose straw which was then covered with loose soil. The fill was brought up to grade, being completed in November 1959. Periodic examinations of the site since then have found no signs of distress in the pipe. There has been no settlement in the fill and none is anticipated.

The relocation of US 69 in Cherokee County between Tyler and Jacksonville, Texas, required a number of small drainage structures under embankments exceeding the maximum allowable for standard reinforced concrete pipe culverts. The imperfect trench method of construction was used for ten pipe culvert installations on this project. The bottom of the trench excavated over the pipe was filled with sawdust to one-third of its depth. The remainder of the trench was filled with compressible soil material. This project has recently been completed and is being closely observed. To date no signs of failure or settlement have been found.

The preceding installations are examples of the use of the imperfect trench method of construction where organic material that may eventually decompose has been used. To date no adverse effects of using this material have been observed. The same results would be expected from the use of the baled straw in the California method. The utilization of the shearing strength of the soil to support part of the load over a culvert is perfectly justified and is good engineering practice. It is also proving to be an economical means of constructing high fills over standard reinforced concrete pipe. These and other advantages already discussed by Larsen make the California procedure appealing to highway engineers.

What Can Be Expected from Treated Wood in Highway Construction

J. OSCAR BLEW, JR., *Technologist,*
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The important factors influencing the performance of treated wood are the quality of the preservative and the quantity and distribution obtained during treatment. These factors are discussed, and the service data from Forest Products Laboratory tests and those of various railroads and highway departments on treated posts, piles, and timbers are presented.

• HIGHWAY engineers are naturally interested in wood structures, for many wood items have demonstrated their value and economy for highway use. Guardrail posts, fence posts, guide posts, signs and sign posts, foundation piling, lighting poles, culverts, and bridges of wood have given excellent service.

Because all of these uses are under conditions that favor decay and frequently insect attack, the wood requires preservative treatment. Such treatment is also needed in contact with salt water to protect causeway timbers, bridges, and piling against marine borer attack. The low cost of wood structures has a particular appeal to the highway engineer, but he wants to know what to expect about the performance of treated wood in highway construction.

One point the engineer should recognize immediately is that age alone does not affect the strength properties of wood. When not subject to deterioration by decay fungi or insects, wood structures have performed satisfactorily for centuries. These forms of deterioration, when not controlled, cause a reduction in the strength properties. That such control is possible, and variable in quality, is illustrated by the data on breaking strength from tests at the General Motors Proving Ground (*6*). Posts that were pressure treated with a preservative and put in service for 5 years showed breaking strength values higher and less variable than those obtained on pressure-treated posts in service for 4 years. Furthermore, strength values from the 5-year-old treated posts compared favorably with those reported for new posts without preservative treatment. The poorest results, and the greatest variation in breaking strength on the wood posts, occurred with posts that had been treated superficially by dipping in a preservative and then put in service for 6 years. In this case the low quality of the treatment rather than the 1-year difference in the age of the posts was unquestionably the important factor involved.

[1] Maintained at Madison, Wis., in cooperation with the University of Wisconsin.

PERFORMANCE OF TREATED WOOD

The user of wood treated with preservatives usually expects and is entitled to maximum protection. It is true that some highway structures, such as guardrails and direction signs, have a limited life due to accidental breakage. It is questionable, even in these cases, that less than maximum protection of the treated wood should be expected, because on this basis the accident survivors as well as the victims would give limited service.

Several factors are important in getting maximum service from wood treated with preservatives. The quality of the wood preservative and quantity and distribution of the preservative obtained during treatment are most important. Factors relating to the wood and its preparation for treatment (such as species, sapwood and heartwood content, seasoning or conditioning, and machining) influence the quality of preservative treatment and thereby influence performance. Inspection or quality control during and after treatment and the proper care of the wood after treatment are other factors that should not be overlooked.

Wood Preservatives

Table 1 summarizes the wood preservatives and retentions recognized in AASHO, Federal, and American Wood-Preservers' Association specifications.

Standard preservatives have been evaluated in laboratory tests. Their

TABLE 1

WOOD PRESERVATIVES FOR HIGHWAY STRUCTURES AND MINIMUM RETENTIONS RECOMMENDED IN FEDERAL SPECIFICATION TT-W 571g

Preservative	Minimum Retention (pcf): Lumber, Plywood, and Timber for Use: Not in Contact with Ground or water	In Contact with Ground or in Fresh Water	Piles: Foundation	Marine [a]	Poles	Posts
Oil:						
Coal tar creosote	6	10	12	17 [b]	8–10 [c]	6
Creosote-petroleum solution (50-50)	7	12	14	—	—	7
Creosote-coal tar solution	6	10	12	17–20 [b]	—	6
Pentachlorophenol, 5 percent in petroleum oil	6	10	12	—	8–10 [c]	6
Waterborne:						
Acid copper chromate (Celcure)	0.50	1.00 [d]	—	—	—	1.00
Ammoniacal copper arsenite (Chemonite)	0.30	0.50 [d]	—	—	—	0.50
Chromated copper arsenate (Erdalith)	0.35	0.75 [d]	—	—	—	0.75
Chromated zinc arsenate (Boliden salts)	0.50	1.00 [d]	—	—	—	1.00
Chromated zinc chloride	0.75	1.00 [d]	—	—	—	1.00
Copperized chromated copper arsenate (copperized Boliden salt)	0.50	1.00 [d]	—	—	—	1.00
Copperized chromated zinc chloride	0.75	1.00 [d]	—	—	—	1.00
Fluor chrome arsenate phenol:						
Type A (Tanalith)	0.35	0.50 [d]	—	—	—	0.50
Type B (Osmosalts)	0.35	0.50 [d]	—	—	—	0.50

[a] Round piles, lumber, and timbers for marine use are treated with minimum retentions of either 14 pcf creosote or 20 lb of creosote-coal tar solution for Pacific Coast-type Douglas fir and southern pine.

[b] Determined by extraction from boring: 17 lb for Douglas fir and 20 lb for southern pine (20 and 25 lb, respectively, for use where limnoria are active).

[c] High retentions required for Group B poles (over 37.5 in. in circumference, 6 ft from butt) and for severe service.

[d] For conditions where leaching is moderate.

comparative effectiveness from the standpoint of the highway engineer, however, can be best shown from Forest Products Laboratory field studies on stakes or posts. In these studies, the wood is southern pine and mostly of sapwood, which can be well penetrated with the preservative, thus helping to eliminate this highly important variable that influences preservative performance. Figure 1 shows the results of a test on southern pine posts at the Harrison Experimental Forest, Miss., started in 1936 (*4*). One hundred posts were installed for each preservative, and the bar graphs (Fig. 1) show average life on the basis of post failures at the time of the December 1960 inspection. Several of the standard preservatives are included in Figure 1, although retentions in the case of the waterborne preservatives are slightly below the minimums for posts in Federal Specification TT-W-571g. Data from a more recent post test, installed in Mississippi in 1949, are given in Table 2 (*4*). These include various standard preservatives and, except for two waterborne preservatives, with rententions approximating those recommended in Federal Specification TT-W-571g. Table 3 gives the results of 2- by 4-in. stake tests, also installed in Mississippi and including standard preservatives with several retentions (*1*).

Regular coal tar creosote, with retentions as low as 6 pcf, has shown good protection under conditions highly favorable to decay and termite attack. An average life of 33 years is given in Figure 1 for test posts treated with that preservative. Several of the special types of coal tar creosote, particularly English coke oven, have not performed as well in stake tests with retentions of approximately 8 pcf as the same creosotes with retentions of 6 pcf in the post test, but this is due to the nature of small stakes as a test medium.

Pentachlorophenol (4.8 percent) in used crankcase oil has shown somewhat better protection than coal tar creosote in the 1936 post installation (Fig. 1). From the stake test results in Table 3, pentachlorophenol in the lighter solvents and fuel oils, particularly with lower rententions, has somewhat less preservative value than is obtained from solutions with oils of a selected type. American

TABLE 2

RESULTS OF FOREST PRODUCTS LABORATORY STUDY ON SOUTHERN PINE POSTS PRESSURE TREATED WITH STANDARD PRESERVATIVES[1]

Preservative	Average Retention (pcf)	Condition After 11½ Years (% failure)
Ammoniacal copper arsenite (Chemonite)	0.34	0
Chromated zinc arsenate (Boliden salts)	0.70	4
Chromated zinc chloride, copperized	0.98	8
Coal tar creosote:		
Various types	5.6 to 6.3	0
Low in fraction 235-270 C., crystals removed	6.1	4
High residue, crystals removed	6.0	0
Low temperature	6.3	0
70 percent and coal tar (30 percent)	6.1	0
Medium residue, low in tar acids and naphthalene (50 percent) and petroleum oil (50 percent)	6.9	0
Pentachlorophenol (5 percent) in:		
No. 2 fuel oil	6.3	0
No. 4 aromatic residual oil	5.9	0
Wyoming residual oil	6.0	0
Untreated control posts		—[2]

[1] Exposed at Harrison Experimental Forest, Miss.
[2] 2.3-yr average life.

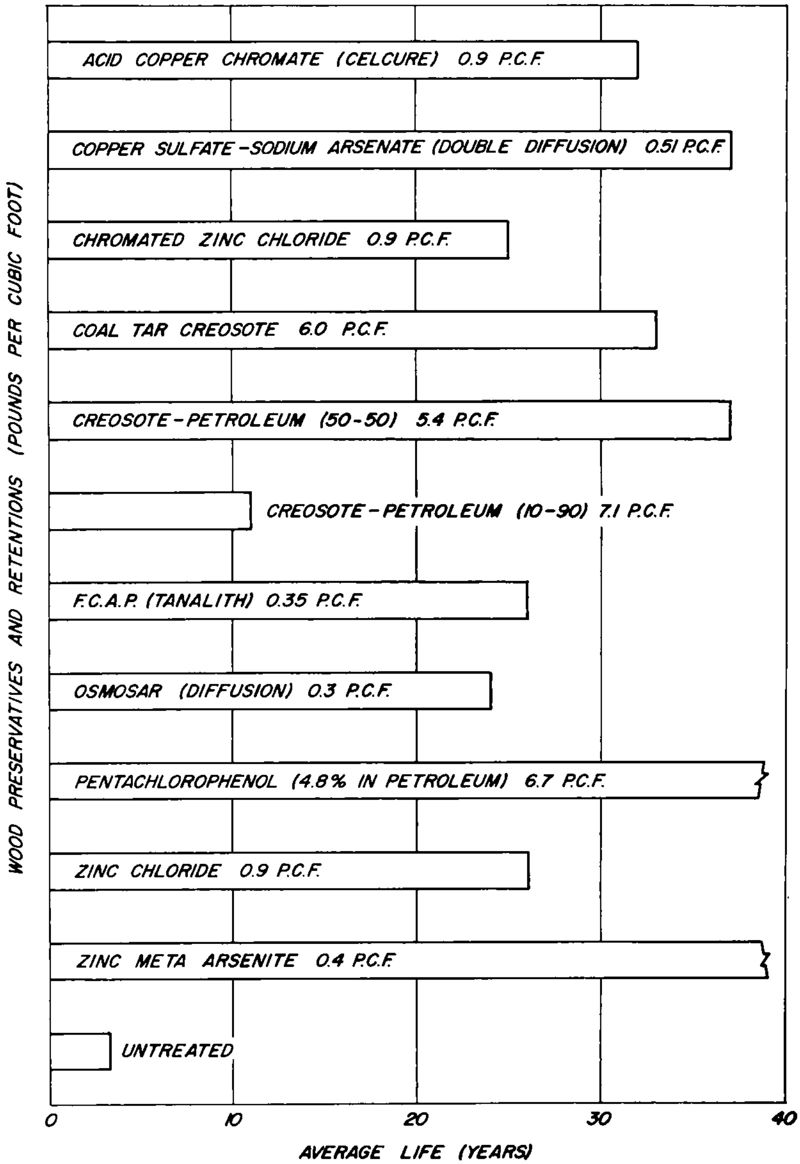

Figure 1. Average life of treated southern pine posts determined in December 1960 at Harrison Experimental Forest, Saucier, Miss. Posts pressure treated except for two preservatives, applied as indicated, by diffusion methods.

Wood-Preservers' Association Standard P9 lists the properties of a heavy petroleum oil for pentachlorophenol solutions to be used for the treatment of such items as posts, poles, piles, and structural timbers. The lighter petroleum solvents are used in pentachlorophenol solutions principally where cleanliness is the important requirement.

TABLE 3

RESULTS OF FOREST PRODUCTS LABORATORY STUDY ON STAKES PRESSURE TREATED WITH STANDARD WOOD PRESERVATIVES[1]

Preservative	Average Retention (pcf)	Condition, Dec. 1960 Failure (%)	After (yr)
Acid copper chromate (Celcure)	0.26	80	15
	0.52, 0.75	0	15
Ammoniacal copper arsenite (Chemonite)	0.28, 0.59, 1.12, 1.45	0	15
Chromated copper arsenate (Erdalith)	0.26, 0.50, 0.78	0	15
Chromated zinc arsenate (Boliden salts)	0.33, 0.44, 0.58, 0.78, 1.06	0	20
Chromated zinc chloride	0.49	100	14.2[2]
	0.76	50	22
	1.02	50	22
Coal tar creosote (regular type)	4.2	60	20
	8.0, 11.8, 16.5	0	20
	4.6	30	20
	10.0, 14.5	0	20
	4.1	40	12½
Coal tar creosote (special types):			
Low residue, straight run	8.0	20	12
Medium residue, straight run	8.0	0	12
High residue, straight run	7.8	0	12
Medium residue, low in tar acids	8.1	0	12
Medium residue, low in naphthalene	8.2	0	12
Medium residue, low in tar acids and naphthalene	8.0	10	12
Low residue, low in tar acids and naphthalene	8.0	10	12
High residue, low in tar acids and naphthalene	8.2	0	12
English, vertical retort	5.3, 8.0, 10.1, 15.0	0	12
English, coke-oven	4.7	20	12
	7.9	50	12
	10.1, 14.8	0	12
English, vertical retort (50 percent) and horizontal retort (50 percent)	8.1	20	12
Low temperature (tar acids not removed)	5.0, 10.2, 15.4	0	8
Low temperature (with high percentage of tar acids removed)	5.0, 9.8, 15.2	0	8
Coal tar creosote (medium residue, low in tar acids and naphthalene, 70 percent) and coal tar (30 percent)	8.1	0	12
Coal tar creosote (medium residue, low in tar acids and naphthalene, 70 percent) and petroleum oil (30 percent)	8.1	10	12
Fluor chrome arsenate phenol (Tenalith)	0.2	100	10.2[2]
	0.3	70	22
	0.61	30	22
Pentachlorophenol (5%) in various oils:			
Light fuel	4.7	10	20
	9.6, 15.3, 20.0	0	20
Stoddard solvent	4.0	40	12½
No. 2 fuel	4.0	30	12½
Light aromatic solvent	4.2	80	12½
	12.1	30	17
No. 300 fuel	4.0	20	12½
No. 400 fuel	4.2	40	12½
Light gas	4.0	20	12½
No. 200 Diesel	4.1	20	12½
Heavy thermal side cut	4.0	30	12½
Catalytic gas-base	4.1	10	12½
	8.0, 12.0	0	12½
Denver No. 3 blend	4.0	0	12½
Heavy gas oil	4.1, 7.9, 12.0	0	12½
Lube oil extract	4.2	0	12½
Untreated control stakes		100	1.8-3.2[2]

[1] Southern pine sapwood, 2- by 4- by 18-in. stakes installed at Harrison Experimental Forest Miss.
[2] Average life.

The preservative oils, such as creosote and pentachlorophenol solutions, generally have high resistance to leaching and are therefore used for outdoor exposures and for wood used in contact with the ground and water. They help protect the wood from weathering but may adversely influence its cleanliness, odor, color and paintability (*15*).

The waterborne preservatives are considered best suited to uses where the wood will not be in contact with the ground, or under wet conditions where it will be subject to leaching. Such waterborne preservatives as acid copper chromate (Celcure), ammoniacal copper arsenite (Chemonite), chromated copper arsenate (Erdalith), and chromated zinc arsenate (Boliden salts), particularly with high retentions, have shown protection in contact with the ground comparing favorably with that furnished by coal tar creosote and pentachlorophenol in selected oils.

The results of field tests, particularly with stakes treated with several retentions of the same preservative, show improvement in performance when the preservative retention is increased.

Properties other than preservative effectiveness are important in treated wood used in some highway construction. Paintability and cleanliness are sometimes required for guardrail posts, sign posts, and lighting poles, although the wide use of light-reflective materials now makes these properties of less importance. Waterborne preservatives are generally best suited for uses where paintability and cleanliness of the treated wood are important. Waterborne preservatives also leave the wood free from objectionable odor. The absorption of water by the wood during treatment may involve problems of swelling, distortion, and grain raising of the wood. These effects, however, can be minimized through careful handling and seasoning after treatment.

Preservative Penetration and Distribution

Colley and Amadon (*7*) of the Bell Telephone Laboratories, in referring to causes of early failures in creosoted poles in the Bell System, comment: "The evidence from the field data showed poor penetration to be by far the most important cause of fungus infection and failure by decay. As a matter of fact, the effect, if any, of geographical location or of the type of creosote used was completely masked by the penetration factor."

Poor penetration is the most frequent cause of early failures in treated wood in field tests of the Forest Products Laboratory and also in the cases of premature failure of treated poles, piles, posts, and timbers furnished to the Laboratory for examination. Well-treated posts and poles should have a life of at least 35 to 40 years (Fig. 2).

Figure 3 shows cross-sections of 2- by 4-in. poorly penetrated test stakes that failed in Panama through decay or termite attack in the unpenetrated centers (*9*). The life of these poorly penetrated stakes varied from 3 to 6 years, while others better penetrated lasted from 9 to 26 years. Stake 1 failed in 4 years, while another well-penetrated stake in the same installation with the same preservative and retention lasted 26 years. Stake 2 failed in 4 years, while one better penetrated with the same preservative and a lower retention lasted over 20 years and another with a slightly higher retention lasted 24½ years. Stake 4 failed after 6 years as compared with a 25-year life from one better penetrated with the same preservative.

Treating specifications generally require penetration of from 85 to 100 percent of the sapwood depth. Heartwood penetration requirements are often overlooked in specifications. Most woods are difficult to penetrate in the heartwood, but most are easily penetrated in the sapwood. Round posts and piles with a higher proportion of sapwood are better penetrated and thereby better protected with the preservative than square posts and piles. For woods difficult to penetrate, impregnation under pressure provides better penetration than nonpressure application (*3*). In the case

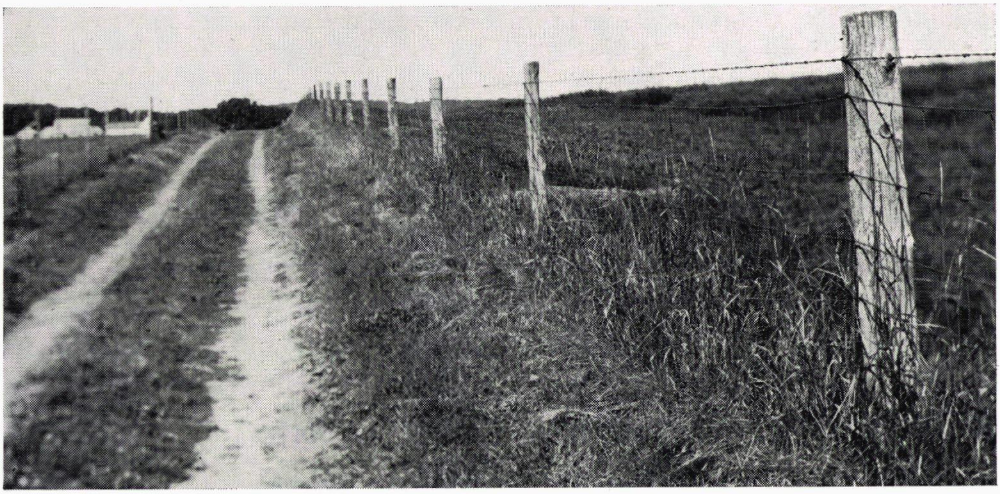

Figure 2. Test posts of various hardwoods and northern white cedar near Madison, Wis., pressure treated with creosote-petroleum solution, in service for 44 years.

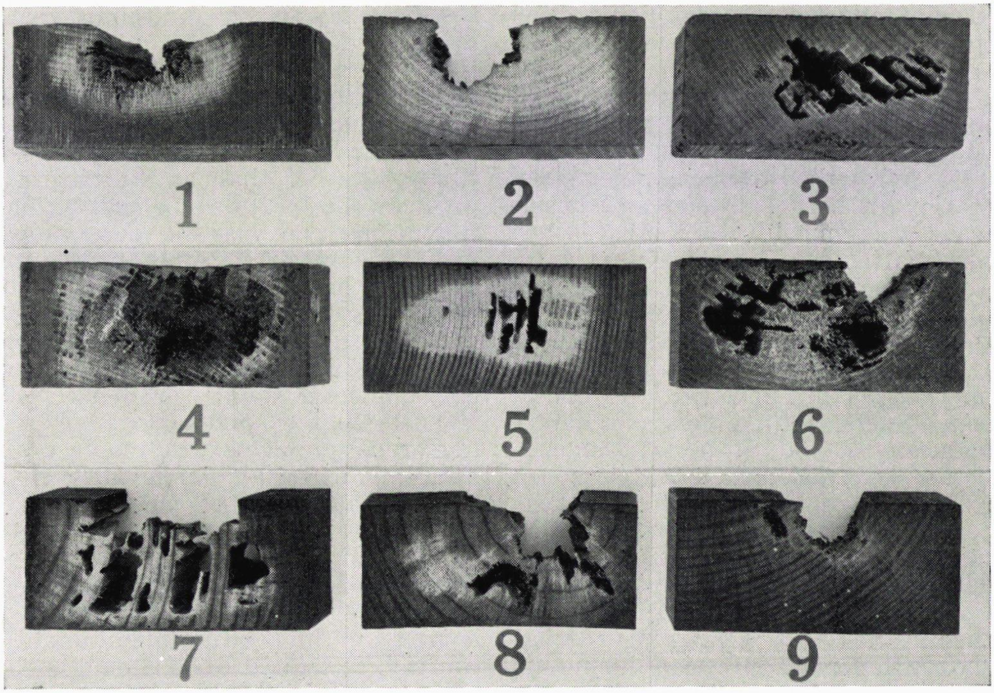

Figure 3. Cross-sections of poorly penetrated 2- by 4-in. test stakes that failed within 3 to 6 yr due to decay fungi or termite attack in unprotected wood.

of some woods, particularly the commonly used Pacific Coast-type Douglas fir, heartwood penetration can be substantially improved by incising (*3*). With heartwood of very high resistance to penetration, such as that of Rocky Mountain Douglas fir (*2*) and white oak, incising does not significantly improve penetration. For important wood structures that require maximum penetration of the preservative, the user should be selective of the wood species and form of the product as well as the method of treatment. Whenever incising is recommended or required in treatment specifications, the performance of the treated product will be improved and the user should follow the recommendations in placing his order. It is also important that cutting and framing of the product be required before treatment rather than be done after treatment.

The condition of the wood, particularly its moisture content at the time of treatment, frequently has considerable influence on the penetration and distribution of preservative. This was shown in a study made by the Forest Products Laboratory on treating 4- by 6-in. seasoned and unseasoned Douglas fir and southern pine (*3*). With the southern pine, penetration in both sapwood and heartwood was much better in seasoned material at a moisture content from 17 to 20 percent than in unseasoned material at a moisture content of 59 to 79 percent—particularly with a waterborne preservative. In the case of Douglas fir, however, the unseasoned lumber with a moisture content between 33 and 37 percent showed better penetration than that seasoned to a moisture content of 15 to 19 percent. Poor preservative penetration in southern pine poles and posts and wide variation in preservative retention can be caused by excessive moisture in the wood. Lindgren (*12*) has shown that mold infection during air seasoning substantially increases the water absorptiveness of southern pine sapwood, and this again is a factor that needs consideration in producing any quality treated product.

Checking after treatment and the exposure of unpenetrated wood to infection can cause difficulties in both sawed timbers and round poles, particularly with poles of species with limited sapwood thickness such as lodgepole pine and Douglas fir. The corrective measure in such cases is to have the wood adequately seasoned before treatment so that it will not lose appreciable moisture, shrink, and check open after it is treated and installed. Checking in large timbers can be reduced by selection to avoid timbers with boxed heartwood.

Inspection and Quality Control

Careful inspection of the wood is necessary both before treatment and immediately after it. Some specifications now include requirements that the adequacy of pole and piling treatment be determined from preservative retention from borings taken after treatment. This would enable the purchaser to check preservative retentions and penetrations at destination rather than at the treating plant as the treatment is carried on. Retentions can also be determined more accurately by this procedure than by the conventional method of measuring preservative by tank gage readings. Requirements for retention by boring analysis are still in the developmental stages for items such as posts, lumber, and sawed timbers; numerous problems are involved in sampling material, particularly that with both heartwood and sapwood faces.

Even though retention and penetration of preservative on some treated products can be determined away from the treating plant, quality control for treated wood should begin with the inspection and selection of fabrication of the material at the plant before treatment. At this time, defects such as decay, insect holes, stain, excessive knots, spiral grain, and compression wood are more easily distinguished and culled out than after the preservative has been applied.

Until preservative retentions can be determined from borings, the treating plant is the only place where the quality and quantity of the preservative specified can be determined. This means that the inspector, after checking the quality of the untreated wood, should be there (a) to take a sample of the preservative for analysis, (b) to observe the temperatures and pressures used during treatment, (c) to take preservative gage readings and determine the total quantity of preservative used, (d) to determine the volume of the material treated and compute retentions on a volumetric basis, (e) to take borings for penetration measurements, (f) to examine the material for any defects that might have been caused by the treatment, and (g) to see that the material is handled properly as it is unloaded from the treating cylinder trams and loaded on cars or trucks for shipment. Piles and large timbers should not be subjected to heavy stresses through dropping or rough handling, and no treated product should be handled with sharp-pointed hooks or tongs that will injure or break open the penetrated shell.

Users of large quantities of treated wood (such as the railroad, telephone, and power companies) often have their own inspectors to carry on this work. Commercial inspection service is available for those wishing to use it. Some treating companies furnish their own inspection and provide an affidavit that indicates conformance with specifications. A new assured warranty service is also available for pole purchasers in which a bonding company replaces poles that fail prematurely.

SERVICE EXPERIENCE

Some of the important factors influencing the performance of treated wood, particularly preservative effectiveness and the penetration of preservative obtained during treatment, are given along with actual service experiences of users of treated wood where the installations are either on highways or under other conditions related to highway uses.

Railroads

It would be much easier to talk on what can be expected from treated wood in railway construction than on treated wood in highway construction. The U.S. railroads have used treated wood for approximately 100 years and have made somewhat closer observations on performance than have highway engineers. There is, nevertheless, a similarity in many railroad and highway uses of treated wood. A review of railroad experience therefore may indicate what can be expected from highway structures.

Although the railroads started using treated ties around 1860, from that year to 1899 only 1.38 percent were treated. Annual replacements had dropped from 359 ties per mi in 1898 to an average of 56 per mi during the 5-year period of 1956 to 1960 when few ties were used without treatment (*8*). The treated tie renewal cost per mile during this 5-year period was $202 divided by 56, or $3.61 per tie. If the number of replacements for 1899 and for 1956 to 1960 are compared on the basis of present-day costs, these values for treated and untreated crossties result:

	Untreated	Treated
Annual replacements per mile (3,000 ties)	359	56
Average life (yr)	8	54
Renewal cost ($)	1.98	3.61
Annual cost per tie at 5 percent interest ($)	0.306	0.195
Annual savings per tie ($)	0.111	
Annual savings on 955, 275, 352 ties in track ($)	106,036,000	
Savings per day ($)	290,000	

Untreated posts of some woods have greater durability than others, but posts of southern pine and hardwoods of limited decay resistance could be expected to last without treatment only 2 to 5 years throughout most of the United States. Treated posts, except those treated with chromated zinc chloride, can be expected to give service at least 35 years. In connection with railroad experience, here are a few examples of service test records from various areas on railway right-of-way fencing posts that were pressure treated with more commonly used preservatives (*11*).

Chicago, Burlington, and Quincy Railroad near Galesburg, Ill.—No failures were reported in 26 years for 68 round pine posts treated with 8 pcf of coal tar creosote, and no failures in 27 years for 69 half-round pine posts similarly treated.

Missouri Pacific Railroad near Poplar Bluff, Mo.—There have been no failures in 31 years for 50 round pine posts treated with 80-20 creosote-coal tar solution (retention not reported).

New York Central Railroad near Rome, N.Y.—No failures have been reported in 30 years for 50 round beech posts treated with 7.9 pcf of coal tar creosote. Only one failure was reported in 30 years for 51 round maple posts treated with 6.5 pcf of coal tar creosote, and none reported in that time for 14 round oak posts treated with 5.1 pcf of coal tar creosote.

Atchison, Topeka, and Santa Fe Railroad at Cleveland, Tex.—Round southern pine posts showed an average life of 12 years when treated with 1.01 pcf of chromated zinc chloride and 18 years for a rate of 1.25 pcf. Similar posts showed an average life of 35 years when treated with 5.3 pcf of coal tar creosote, or with 4.9 pcf of creosote-coal tar (70-30) solution. The average life of similar posts with 5.4 pcf of creosote-petroleum (40-60) solution was 39 years.

No failures have been reported in 29 years for 15 southern pine posts treated with 8.1 pcf of creosote-coal tar (70-30) solution or for 15 posts treated with 5.1 or 7.8 pcf of creosote-petroleum (70-30) solution. In 20 years of service no failures have been noted for 9 and 10 southern pine posts, respectively, treated with 5.0 and 8.3 pcf of 4.9 percent pentachlorophenol in gas oil.

Missouri-Kansas-Texas Railroad at Denison, Tex.—In 38 years of service, only 3 percent failures have been noted for 200 round southern pine posts pressure treated with 6.0 pcf of coal tar creosote.

Foundation Piles.—Many of the railroads have used treated wood foundation piles, but three experiences should be of special interest to highway engineers. The Southern Pacific Railroad has successfully used such piling for more than 65 years, the Illinois Central for 56 years, and the Atchison, Topeka, and Santa Fe Railroad for 55 years. After installation there is seldom an opportunity to inspect foundation piles, but Frank R. Judd, Engineer of Buildings of the Illinois Central Railroad, in 1932 (*10*) reported on an examination of piles in a 21-year-old installation supporting an 85-ft balanced deck turntable. Mr. Judd reported, "... the general condition was good, in fact, the piles looked as if they had just been driven. Borings from these piles showed that both the untreated heart and the outer treated portion were apparently as sound as new piles."

Highway Experience

Starting in 1934, the Forest Products Laboratory has made an effort to obtain information on the performance of treated wood in highway structures. Service records on treated wood in highway installations are less available than those of the railroads, but a few cases have been selected to show highway experience.

U.S. Bureau of Public Roads Guardrail Posts.—The U.S. Bureau of Public Roads selected a number of installations of guardrail posts and has since supplied information periodically to the Forest Products Laboratory on the condition of those posts. Many of these posts have now been removed because of highway changes, and the preservatives used at that time are no longer employed in commercial treatment. A comparison of the preservatives used (zinc meta-arsenite and zinc chloride) with those in commercial use is given in Figure 1, however. Douglas fir guardrail posts, 8 by 8 or 9 by 9 in. and treated with approximately 0.25 pcf of zinc meta-arsenite, have been in service in Montana for 21 to 29 years with very few removals due to deterioration of the treated wood (Table 4). Locally available species such as balsam fir, jack pine, and northern white cedar have been used in North Dakota and Minnesota as round guardrail posts and, when treated with approximately 0.5 pcf of zinc chloride, have given good service for 15 to 21 years.

Connecticut State Highway Posts.—Henry W. Hicock, the Connecticut Agricultural Experiment Station, started in 1940 a service study on treated posts in highway fences of the Connecticut State Highway Department. A 1960 progress report on this study by A. R. Olson (*14*), covering 8,977 posts (mostly round), indicated that red pine and southern pine posts pressure treated with 6 pcf of creosote were in excellent condition, with over 95 percent serviceable after 20 years. Serviceability after 20 years for round hardwood posts similarly treated was 82 percent for white oak, 73 percent for red oak, 55 percent for birch, and 34 percent for maple. Olson suggests poorer preservative distribution as a reason for the less impressive performance of the local hardwood posts. Posts of oak and maple pressure treated with Tanalith (Wolman salts) compared favorably with those of the same species treated with creosote when all were exposed for 15 years; after 20 years the Tanalith-treated posts deteriorated more rapidly.

Full-length, hot-and-cold-bath treatment with creosote provided somewhat less protection than pressure treatment to oak posts. However, somewhat greater protection was obtained in the hot-and-cold-bath treatment of maple and birch, possibly due to a heavier retention of creosote than that of the 6 pcf for the pressure-treated posts.

Wisconsin State Highway Commission Sign Posts.—During the years 1954 and 1956, the Wisconsin State Highway Commission installed a large number of 4- by 4- and 6- by 6-in. sign posts treated with different preservatives. These were of southern pine pressure treated with preservatives and retentions as follows:

	Pcf
Coal tar creosote	6.0
Chromated copper arsenate (Erdalith)	0.5
Pentachlorophenol, 5 percent in petroleum	6.0
Chromated zinc chloride	1.05 to 1.39

Also installed were posts of local woods—jack pine, eastern white pine, and sugar maple—treated by the Osmose diffusion process with 0.19 to 0.33 pcf of Osmosalts. A total of 181 of these posts, including from 16 to 30 posts for each species and treatment, were selected in the Madison area and identified by the Forest Products Laboratory for test purposes. An inspection of the posts during October 1961, after 5 to 7

TABLE 4

SERVICE DATA ON TREATED GUARDRAIL POSTS [1]

Species and Size	Location	Preservative	Retention (pcf)	Installation		Last Report		No. Removed	Cause
				Number	Year	Years of Service	Number Serviceable		
Douglas fir (8 by 8 or 9 by 9 in. by 6 ft)	Montana								
	FAP 239–E	Zinc meta-arsenite	0.24	154	1931	29	150	4	Snow plow damage and decay
	FAP 242–C	Zinc meta-arsenite	0.25	1,215	1931	29	1,215	0	
	FAP 254–B	Zinc meta-arsenite	0.23	1,000	1931	29	424	576	Reconstruction
	FAP 255–A	Zinc meta-arsenite	0.24	800	1931	28	800	0	
	FAP 220–B	Zinc meta-arsenite	0.24	693	1931	28	693	0	
	FAP 191–C	Zinc meta-arsenite	0.24	470	1932	28	122	348	Reconstruction, traffic breakage
	FAP 142–A	Zinc meta-arsenite	0.24	147	1931	28	8	139	Reconstruction
	FAP 117–A	Zinc meta-arsenite	0.26	1,100	1932	24	1,100	0	
	FAP 108–C	Zinc meta-arsenite	0.26	917	1931	29	917	0	
	FAP 153–B	Zinc meta-arsenite	0.52	221	1931	21	221	0	
Balsam fir and jack pine (round, 6 in.)	North Dakota	Zinc chloride	0.52	20,351	1932	17	20,351	0	
Northern white cedar (round, 6 to 7 in.)	Minnesota								
	FAP 1–C	Zinc chloride	0.50	6	1927	15	0	6	Reconstruction
	FAP 1–C	Zinc chloride	0.50	8	1927	21	0	8	Reconstruction
	Maple Lake—Buffalo								
	FAP 345–B	Zinc chloride	0.50	12	1927	17	8	4	Broken
	FAP 339–B	Zinc chloride	0.50	8	1927	19	8	0	

[1] Reported by U.S. Bureau of Public Roads.

years of service, showed only one failure due to decay. This was in a group of 25 sugar maple posts treated with Osmosalts. Of particular interest, however, is the fact that of 181 posts selected for test, 99 have been removed, during the limited test period, for reasons other than decay and eliminated from the test. More than one-half of these removals were due to breakage by vehicles. Others have been removed for road improvement or unknown causes.

Mississippi State Highway Department Posts.—Service tests on treated posts were started in 1931 by the Mississippi State Highway Department (*13*). At that time 8- by 8-in. southern pine guardrail posts, pressure treated with 12 pcf of creosote or with 0.25 pcf of Tanalith (Wolman salts), were installed. Five of 72 creosoted posts were removed because of automobile wrecks, and the remaining 67 were reported as serviceable in late 1959 after nearly 29 years of service. The estimated average life of the Tanalith-treated posts was 14 years (*5*). Since 1931, other preservatives and treatments were included for 8- by 8-in. guardrail posts and for round guardrail posts 8 to 10 in. in diameter. In 1933 a test garden was established in Jackson, Miss., to include 32 pressure and nonpressure treatments with various wood preservatives. More detailed information on these installations is given in Table 5.

TABLE 5
SERVICE STATUS OF MISSISSIPPI TEST POSTS

Type and Treatment	Serviceability (%)	After (yr)
Guardrail posts:		
Pine (8 by 8 in.) treated with coal tar creosote, 8 pcf	100	20
Pine (8 by 8 in.) treated with zinc meta-arsenite, 0.25 pcf	24	12
Untreated eastern red cedar (round)	58	18
Untreated western red cedar (round)	24	9
Untreated black locust (round)	75	17
4- by 4-in. posts[1]:		
Untreated	0	5
Tanalith (Wolman salts), 0.32 pcf	12	15
Zinc meta-arsenite, 0.26 pcf	15	15
Chromated zinc chloride, 0.96 pcf	20	17
Pentachlorophenol solution, 10.7 pcf	98	17
Creosote-coal tar solution (80-20), 16 pcf	100	22
Chemonite, 0.42 pcf	92	21

[1] In test garden (southern pine).

Marine Piles

Service data on treated wood marine piles show variable results depending on marine borer activity, the severity of limnoria attack, and preservative retentions (*15*). Only coal tar creosote and creosote-coal tar solutions are recommended for the treatment of marine piles and the highest possible retentions of these preservatives are suggested, particularly for installation on the Southern California, Gulf of Mexico, and southeastern coasts where limnoria attack is severe.

Well-treated wood piling has given service for 22 to 48 years in and around the San Francisco Bay area and somewhat less service in the Los Angeles area. In areas of severe marine borer activity in the South Atlantic and Gulf of Mexico, creosoted piles have been estimated to have an average life of 10 to 15 years, although they frequently last much longer. Along the North Atlantic coast considerably longer life can be expected. Recently improved Federal Specifications should greatly lengthen the service life to be expected from treated marine piles.

U.S. Forest Service Highway Bridges

Wood pressure treated in accordance with Federal Specification TT-W-571 is approved as a permanent construction material in bridges and culverts in the National Forest road

TABLE 6

RECORDS ON U.S. FOREST SERVICE BRIDGES OF TREATED DOUGLAS FIR IN SERVICE IN ROCKY MOUNTAIN AREA

Bridge Location	Species	Volume of Treated Wood (bd ft)	Retention of Creosote (pcf)	Date Installed	Last Inspection Date	Condition Reported
Slide Gulch, Boise National Forest, Idaho	Douglas fir (coast type)	53,233	10	1933	1955	Considerable checking, no replacements; some badly checked timbers reinforced with straps in 1940; bridge to be replaced as inadequate for present traffic conditions
Big Greys River, Bridger National Forest, Wyo.	Douglas fir (coast type)	38,957	10	1934	1953	No replacements
Panther Creek, Salmon National Forest, Idaho	Douglas fir (coast type)	16,297	10	1934	1943	Some wear and checking but no replacements
Camas Creek, Boise National Forest, Idaho	Douglas fir (coast type)	32,554	10	1934	1945	Good condition but bridge to be removed because of area flooding
Fall River, Targhee National Forest, Idaho	Douglas fir (coast type)	37,628	10	1936	1945	Chords badly checked and bridge has sagged 2 to 3 in. in middle of each span; no replacements
Near Big Smoky Guard Station, Sawtooth National Forest, Idaho	Douglas fir	18,376	7.5	1936	1954	Severe checking; no decay or replacements
Antelope Creek, Challis National Forest, Idaho	Douglas fir	5,365	11.2	1936	1945	No replacements
Farmington Canyon, Wasatch National Forest, Utah	Douglas fir	16,900	10	1936	1954	Deep checking; filled with hot tar; tops of chords covered with light galvanized sheet metal; runners replaced 1954 due to wear
Three Fork, Uinta National Forest, Utah	Douglas fir	6,825	10	1937	1950	No replacements except for untreated run planks

system of the U.S. Forest Service. Service records on several bridges in Idaho, Wyoming, and Utah are shown in Table 6, and in Mississippi, Texas, Florida, and South Carolina in Table 7, to indicate the performance of piles and various bridge components in these areas representative, respectively, of the less severe and the more severe exposure conditions of the United States.

In the dry Rocky Mountain region, checking is common in bridge timbers and could be reduced somewhat by avoiding boxed-heartwood timbers. Decking replacement due to wear has been necessary, but no decay has been noted after 18 to 22 years. In the southern region, checking is accompanied by decay and is therefore more serious than in drier areas. Decking replacement has been due principally to wear, followed by some decay. Treated wood, for the most part, has performed satisfactorily for 20 to 25 years in bridges under the severe climatic conditions in the southern region.

CONCLUSIONS

The performance of treated wood is determined by the quality and quantity of the preservative used and, to a great extent, by the penetration and distribution of the preservative in the treated wood. This distribution is influenced by the species of wood itself, the proportion of sapwood and heartwood, the moisture condtion of the wood, and by the method of treatment.

The results of field tests for the evaluation of wood preservatives, and those of service tests representative of highway installations, can be summarized as follows:

1. Treated crossties used by class A railroads, on the basis of average replacements from 1956 to 1960, have an average life of 54 years or 6½ times the life of untreated ties. This added crosstie life through treatment results in a daily saving of $290,000 to those railroads.

2. Round southern pine posts and poles properly treated with recommended retentions of such preservative oils as coal tar creosote, creosote solutions, and 5 percent pentachlorophenol in a selected petroleum oil, and of the better waterborne preservatives, can be expected to give service for 30 to 35 years, even under severe exposure conditions. The service obtained from sawed posts or from round posts of less easily treated species is somewhat below these figures, particularly in severe exposure conditions in the South or when treated with the less effective waterborne preservatives.

3. Treated wood foundation piles have been widely used by the railroads with good results for over 60 years. Apparently, failure of encased treated foundation piles has been no particular problem, so that average life is unknown.

4. Treated wood bridge members, except where subject to severe mechanical wear, have given good service for 20 to 25 years of severe exposure in the Southeastern United States and have lasted much longer under more favorable climatic conditions. Improvement in service can be expected from exposed treated bridge timbers by avoiding boxed heartwood, by proper seasoning prior to treatment and other measures to reduce checking, and by special attention to the protection of cutoff ends of piles and posts.

5. Properly treated wood, when used for guardrail posts, fencing, guide posts, sign posts and signs, foundation piling, lighting poles, culverts, and bridges, has proved to be economical and, on the basis of its long and satisfactory performance, can help materially in the rapidly expanding highway program.

TABLE 7

RECORDS ON U.S. FOREST SERVICE BRIDGES OF TREATED SOUTHERN PINE, IN SERVICE IN THE SOUTH

Bridge and Location	Species (pine)	Vol. of Treated Material		Retention of Creosote (pcf)	Date Installed	Last Inspection Date	Condition
		Bd Ft	No. of Piles				
Piney Woods Creek, De Soto National Forest, Miss.	Southern yellow	12,774	28	12, 16[1]	1934	1961	Untreated handrails and posts replaced with treated material in 1941; 31 pieces of decking, 3 wheel guards, and 1 post replaced in 1954 and 17 pieces of decking in 1956
Whiskey Creek, De Soto National Forest, Miss.	Southern yellow	13,155	32	12, 16[1]	1934	1957	Untreated handrails and posts replaced with treated material in 1940; deck replaced in 1949. Some decay in piles, stringers, and wheel guards, where nailed or bolted. (Creosote originally applied to bolt holes.) No replacements of treated wood
Chalk Creek, Angelina National Forest, Texas	Shortleaf	10,997	624[2]	12, 16[1]	1935	1960	Some decay in treated handrail and guardrail posts. No replacements
Bay Creek, Apalachicola National Forest, Fla.	Shortleaf	48,146	—[3]	10,[4] 12,[5] 16[1]	1936	1961	Considerable decay in decking and runways and some decay in stringers; 36 treated pieces replaced in 1959
French Quarter Creek, Francis Marion National Forest, S.C.	Longleaf and shortleaf	4,140	—[3]	10, 14, 16[1]	1936	1961	Some decay in treated handrail posts. No replacements
Cromer Road, Sumter National Forest, S.C.	Longleaf and shortleaf	3,773	6	10, 14, 16[1]	1936	1961	Good. No replacements
Flint Road, Sumter National Forest, S.C.	Longleaf and shortleaf	3,696	12	10, 14, 16[1]	1936	1957	Good. No replacements
Piney Creek, Davy Crockett National Forest, Tex.	Shortleaf	6,588	252[2]	10, 14, 16[1]	1936	1960	25 percent of planks in headwall replaced due to decay in 1945. Tops of guardrail posts show some decay.
Sandy Creek, Sabine National Forest, Tex.	Shortleaf	8,787	432[2]	10, 14, 16[1]	1936	1960	Some decay in treated handrail posts No replacements
San Jacinto River (West Fork), Sam Houston National Forest, Tex.	Shortleaf	15,081	768[2]	10, 14, 16[1]	1936	1957	Some decay in treated handrail posts and 1 pile replaced due to decay
Turkey Hen Creek, formerly Chochtawhatchee National Forest, now Eglin Air Force Base, Fla.	Shortleaf	5,462	15	10, 14, 16[1]	1936	1953	No replacements
McHenry Road, No. 401, De Soto National Forest, Miss.	Southern yellow	13,409	24	10, 14, 16[1]	1936	1957	Untreated handrails and posts replaced in 1942 with treated material. All treated material in good condition and no replacements
No. 67 on Trenton Road, Bienville National Forest, Miss.	Southern yellow	17,941	36	12, 16[1]	1936	1951	Bridge rebuilt. Some decay in 10- by 12-in. cap sill noted in 1944; 6 stringers replaced in 1945 due to damage by falling tree
Homochitto River on Copaih Road, Homochitto National Forest, Miss.	Southern yellow	19,035	33	14	1937	1954	Good condition except for wear and decay on decking, which needs replacement
Wagner Creek on Chickasaw Road, Holly Springs National Forest, Miss.	Southern yellow	5,242	17	12, 16[1]	1937	1960	All wheel guards and decking replaced due to wear and checking, also 25 percent stringers, 30 percent of posts, and 50 percent handrails replaced in 1955 due to decay following checking

Toxaway Creek, Sumter National Forest, N.C.	Shortleaf	1,215	38[6]	14	1937	1961	Stringers replaced in 1956 due to decay
Panther Creek, Conecuh National Forest, Ala.	Longleaf	10,000	15	14, 16[1]	1937	1960	24 pieces of 2- by 10-in. deck run planks replaced in 1941 due to excessive wear; 1 cap sill replaced in 1954 and 4 wing piles replaced in 1957. Some decay in decking, run planks, handrails, posts, and guardrails but no replacements
Walden Creek, Talladega National Forest, Ala.	Longleaf	19,270	36	14, 16[1]	1938	1956	Untreated white oak decking, wheel guards, and run planks replaced with treated material in 1949; run planks again replaced in 1956 due to wear. No other replacement of treated wood

[1] Piles. [2] Lineal feet. [3] Unspecified number. [4] Handrails. [5] Timbers. [6] Stringers.

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Viscoelastic Properties of Asphalt Concrete

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A continuing research program is currently under way at the University of California for the purpose of establishing the usefulness of viscoelastic analysis as applied to an understanding of the rheologic characteristics and structural responses of asphalt concrete. As a preliminary step, the asphalt concrete mixture to be used in preparing the test slabs was subjected to an extensive program of triaxial compression testing, covering a range of load types including creep, stress relaxation, constant rate-of-strain, and repeated load lateral pressures from 0 to 250 psi and temperatures from 40 to 140 F. From the results of triaxial tests at a specific temperature and lateral pressure, viscoelastic constants for a four-element rheologic model were derived; values for these constants were obtained for the entire range of ambient conditions studied. This phase of the research thus produced a basic system of data relating the general stress-strain-time characteristics of the test mixture.

A comparison of the deflections predicted by viscoelastic theory with those measured during the slab tests indicated that the slab profiles from both sources had the same general shape and time dependence, but that the measured values had magnitudes considerably greater than those given by theory. The deviations between the two sets of values increased with increased temperature.

Also included is a discussion of the probable source of the differences found between the slab test data and theoretical predictions.

• A CONTINUING RESEARCH program is currently under way at the University of California for the purpose of establishing the usefulness of viscoelastic analysis as applied to an understanding of the rheologic characteristics and structural behavior of asphalt concrete. This avenue of approach may aid in the understanding of the behavior of asphalt concrete pavements inasmuch as (a) the load conditions that may be imposed on such pavements cover a wide range in time—from the essentially static condition associated with vehicle parking areas to the rapidly applied repeated loads occurring on heavy-duty highways and airfield taxiways—and (b) the stress vs strain characteristics of asphalt concrete are time-dependent.

Although elastic theory may prove useful in the design of the pavements under fast-moving wheel loads, the effect of very slowly applied or static wheel loads cannot be considered by this approach. Moreover, the accumulation of small deformations under repeated load application and the subsequent rutting of asphalt concrete also cannot be accounted for by elastic theory. Thus, an approach wherein the time dependence of the stress vs strain characteristics of asphalt concrete can be considered would seem appropriate for analysis of certain aspects of pavement behavior.

A number of papers have already been presented dealing either with the basic viscoelastic properties of asphalt concrete or with the development of possible theoretical relationships for the behavior of flexible pavements (*1, 2, 3, 4, 5*). Recently attention has been directed to an attempt at joining these two avenues of research through a study of the reactions of actual paving slabs subjected to static loading on an idealized elastic foundation.

This paper is concerned with a presentation and discussion of two types of test data: (a) the results of four types of triaxial compression tests used to evaluate the viscoelastic properties of an asphalt concrete mixture over a range in temperatures and lateral pressures, and (b) the results of deflection measurements performed on test slabs of the same mixture under static loading on a large spring base. Included as an essential part of the discussion is a presentation of the mathematical relationships necessary to define the rheologic properties of the mixture in terms of a simple linear viscoelastic model both for the triaxial compression tests and for the slab tests. An important feature of this paper is a comparison of actual test data with similar results predicted from theory.

MATERIALS

The scope of the investigations made it necessary to restrict the laboratory investigation to tests on a single asphalt concrete mixture. The physical properties of the asphalt and aggregate used in this mixture have been described in detail elsewhere (*1*); therefore, only a brief description is included here.

One 85- to 100-penetration asphalt cement was used in the investigation. Results of standard tests on this asphalt are given in Table 1. Based on absolute viscosity measurements by sliding plate microviscometer (*6*) and capillary viscometer over a range in temperatures, the asphalt would be classed as a material approaching Newtonian behavior. These results are presented elsewhere (*1*).

The aggregate used in the mixtures was a crushed granite from Watsonville, Calif. (*7, 8*), with a uniform specific gravity of 2.92. It was originally intended that one dense gradation of the material (a ⅜-in. maximum size gradation conforming to the 1954 State of California standard specifications) be used in both the triaxial compression and the slab tests. For the triaxial compression test specimens, this was readily ac-

TABLE 1
IDENTIFICATION TESTS ON ASPHALT

Test	Result
Pen. at 77 F., 100 g, 5 sec	96
Pen. at 39.2 F., 200 g, 60 sec	24
Penetration ratio	25
Flash point, Pensky-Martens (°F)	445
Viscosity at 275 F (SSF)	138
Heptane-xylene equivalent	20/25
Softening point, ring and ball (°F)	110
Thin film oven test, 325 F, 5 hr:	
Percent weight loss	0.51
Percent pen. retained	53
Duct. of residue, cm at 77 F	100+

complished by screening the material then recombining each of the fractions in the amount required to pro- into individual-size fractions and duce single specimens. The grading curve for these specimens is shown in Figure 1, together with the specification limits.

As described later, the slabs were prepared in the Richmond, Calif., laboratories of the California Research Corporation, and the aggregate was supplied from the stocks of that organization. Due to the large quantities of aggregate required, these stocks could not be combined to duplicate exactly the grading curve used for the aggregate in the triaxial compression specimens. The grading finally accepted as the best compromise is also shown in Figure 1. By comparing the two materials, this gradation appears deficient in sizes near the No. 8 sieve. The effect of this disparity was evaluated on additional specimens prepared using the second gradation.

PREPARATION OF TEST SPECIMENS

Two different types of specimens were required for the laboratory work associated with this investigation. For the triaxial compression tests, cylindrical specimens 2.8 in. in diameter and 6.5 in. in height (nominal dimensions) were used. Details of the method of preparation using kneading compaction (*9*) are also included elsewhere (*1*). For these specimens, one asphalt content of 6 percent (by dry weight of aggregate) was selected, this amount approximating the design value for the type and gradation of aggregate used, based on the State of California mix design procedure. The ingredients for the specimens were mixed at 230 F. The density of the specimens as determined by water displacement was approximately 150 pcf with only minor variations between individual samples.

The slab tests used thin "plates" of asphalt concrete approximately 40 by

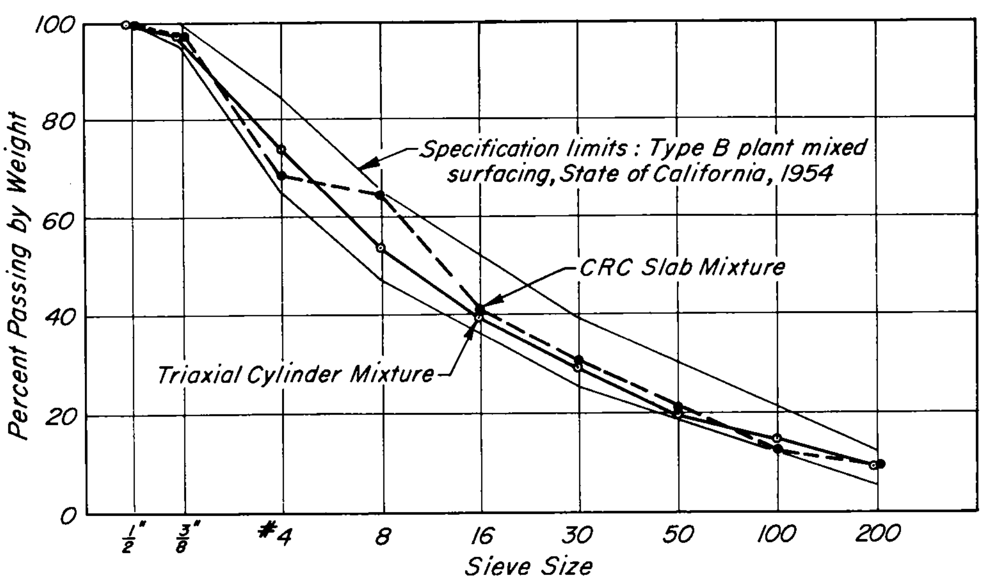


Figure 1. Grading curves for Watsonville aggregate.

40 in. square and 1.5 in. deep. The slab specimens were prepared at the Richmond laboratories, where equipment of the size necessary for such an operation was available. This process involved the production of all the required slabs simultaneously by the construction of a small (approximately 4 ft wide by 25 ft long) paving strip with the desired thickness (about 1.5 in.) Upon cooling, the pavement was broken into segments approximating the size of the test slabs and transported to the University of California laboratories for final trimming and testing. Each segment was supported during this entire process by a heavy plywood form to prevent possible damage from accidental bending.

Inasmuch as a complete description of the functions of the California Research Corporation paving laboratory has been published by that organization (*10*), no attempt made to detail the methods used to produce the test strip from which the slabs were taken. However, the mixing temperature employed was 300 F (slightly above the 230 F used for the triaxial cylinders, due to the heat losses involved in handling the paving mixture), and considerable compactive effort was applied to the strip by alternate applications of steel and pneumatic rollers.

Also, the results of the procedure described were gratifying, as the test slabs were quite uniform in texture and thickness. Samples cut from the slabs after all test measurements had been completed gave a unit weight by water displacement of 152.5 pcf.

Unfortunately, extraction tests on the slab mixture (performed as a routine check by the California Research Corp.) disclosed that the actual asphalt content of the compacted slabs was only 5.1 percent, instead of the desired 6.0 percent, increasing the necessity for checking the comparative properties of the original triaxial test mixture and of the modified slab mixture.

INSTRUMENTATION AND TEST TECHNIQUES

Four types of triaxial tests were performed to determine the rheologic behavior of the asphalt concrete mixture: (a) creep tests, (b) stress relaxation tests, (c) constant rate-of-strain determinations, and (d) repeated axial load tests. The tests were conducted at three temperatures, 40, 77, and 140 F, with three lateral pressures being used at each temperature, 0 (unconfined), 43.8, and 250 psi. These values of temperature and pressure were selected to cover the range of practical interest. The apparatus and procedures used in these tests have been described elsewhere (*1*).

For the creep tests, three levels of stress were employed for each temperature and lateral pressure combination. In the repeated load tests (essentially, repeated creep tests), only one level of stress was employed due to the relatively long time required for a particular test. The cycle selected for the repeated load determinations involved application of the desired stress for 1 sec and at the rate of 20 applications per min. For the stress relaxation tests, three levels of initial strain were tested for each set of ambient conditions. The constant rate-of-strain determinations were performed at load rates of 0.01, 0.10, and 1.0 in. per min for the various conditions.

The satisfactory completion of the triaxial test program required the preparation and testing of over 250 specimens. Thus, it is believed that these test data can be accepted as an accurate portrayal of the stress-strain-time characteristics of this test mixture over the wide range of test conditions.

The slab tests required the design and construction of considerable special equipment. Essentially, the proposed structural analog required apparatus by which the slab could be placed under static load on a foundation of constant elastic properties. In addition, this apparatus had to include some means for controlling the environmental temperature within fairly rigid limits. Provision also had to be made for measuring the deflected shape of a test slab and for observing changes in that shape with time.

A major portion of the device constructed to satisfy these requirements is shown in Figure 2. The flexible foundation was provided by approximately 1,600 ⅞-in. diameter coil springs, arranged in a pattern of equilateral triangles in such a way that each spring would account for 1 sq in. of the surface area. Because each spring was required by specification to have a constant of 200 lb per in., such a configuration thus provided a foundation with a vertical resistance k of 200 psi per in. A $\frac{1}{32}$-in. rubber membrane was placed on the surface of the springs to protect them from the entrance of foreign material and to provide an improved bearing surface for the asphalt concrete slabs. Plate bearing tests performed on the spring base justified the use of a theoretical foundation constant of 200 psi per in.

Loads for the various slab tests were provided by the use of pneumatic cells and are similar to those used in the creep and repeated load tests (*1*). Inasmuch as a rather large range of loads was involved, it was necessary to employ several sizes of these cells to obtain sufficient sensitivity. A typical intermediate size is shown in Figure 2, attached to the frame used to provide a suitable reaction surface.

The entire spring base (as well as the reaction frame) was securely

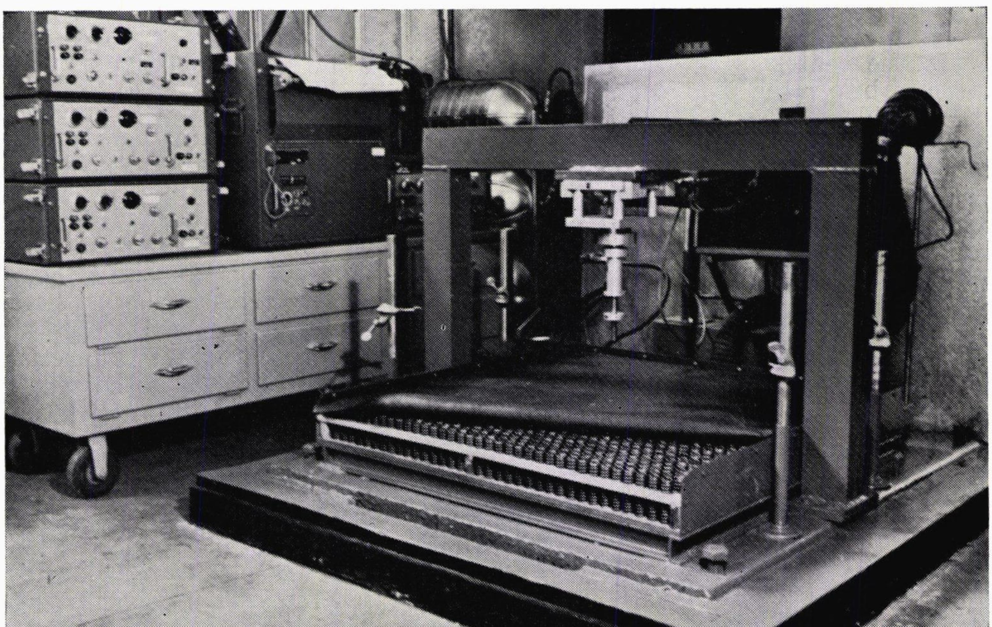


Figure 2. Slab-testing device with rubber membrane stripped back to show spring base (note pneumatic load cell hanging from the center of the reaction frame; electronic recorder for use with differential transformers appears in background).

fixed to a large block of cast iron. This provided an extremely level and rigid support for the unit, as well as an excellent heat-stabilizing sink for temperature control.

The test unit (Fig. 2) was designed to fit under a heavily insulated plywood cover, which was set on wheels to facilitate its movement. When locked in place over the spring base, the cover completed a nearly airtight enclosure with very low thermal conduction. Temperature control within the enclosure was obtained by the use of a large refrigeration unit in combination with an electric heater; both were attached to a master thermostat. A battery of electric blowers was used to provide a high degree of air circulation. These devices combined to permit temperature control within ±1 F, over a range from +30 to +150 F.

Records of deflected shapes of loaded slabs were provided through the use of a system of linear variable differential transformers. A single small transformer was rigidly mounted in the center of the coil spring, directly below the load center, with its movable core attached to the rubber membrane. This device was used to record the center deflection of a given slab. Deflections at locations away from this point were indicated by several larger transformers above the slab, spaced along a radial line from the center. A typical test setup of this type is shown in Fig. 3. The control wires from these gages were run through the insulated cover to an electronic chart recording unit (background Fig. 2).

In performing a given slab test, the slab was first carefully slipped from its plywood form onto the spring base. The deflection gages were then positioned as shown in Fig. 3 and calibrated. Next, the cover was rolled over the slab and the temperature controls were actuated. As soon as the temperature of the slab reached equilibrium at the desired level (usually about 24 hr) the test could be performed. Load was applied by means of the pneumatic cell, using a rubber-surfaced plate to simulate a flexible bearing surface. The load was maintained either until the deflections as (indicated on the recorder) reached equilibrium or until the capacity of the transformers was exceeded.

This test procedure was carried out at the same three temperatures employed in the triaxial testing, 40, 77, and 140 F. Two slabs were loaded at each temperature, with differing stress levels. A 1-in. diameter rubber loading foot was used at 40 F and 77 F; and a 2-in. diameter foot was used at 140 F. These limitations were imposed by the necessity for maintaining an approximation of infinite boundary conditions for the slab. The resultant data from the slab tests are discussed in detail and compared with that predicted by viscoelastic theory in a later section.

BASIC MODEL THEORY

The use of model systems composed of purely elastic springs and purely viscous dashpots to aid in derivations associated with the use of linear viscoelastic concepts is by this time a fairly familiar feature of such analyses (*1, 2, 11, 12*) and no attempt is made to amplify this subject here.

The viscoelastic model (Fig. 4) was selected for study in connection with the investigations discussed in this paper. It should be noted that this is the same four-element model suggested by Kühn and Rigden (*13*) for use in research with asphalt cements. The authors (*1*) have also suggested its possible value for describing the rheologic behavior of asphalt concrete. This model is capable of accounting for instantaneous elastic deformation, retarded elastic deformation, and viscous flow. The first two types of deformation are

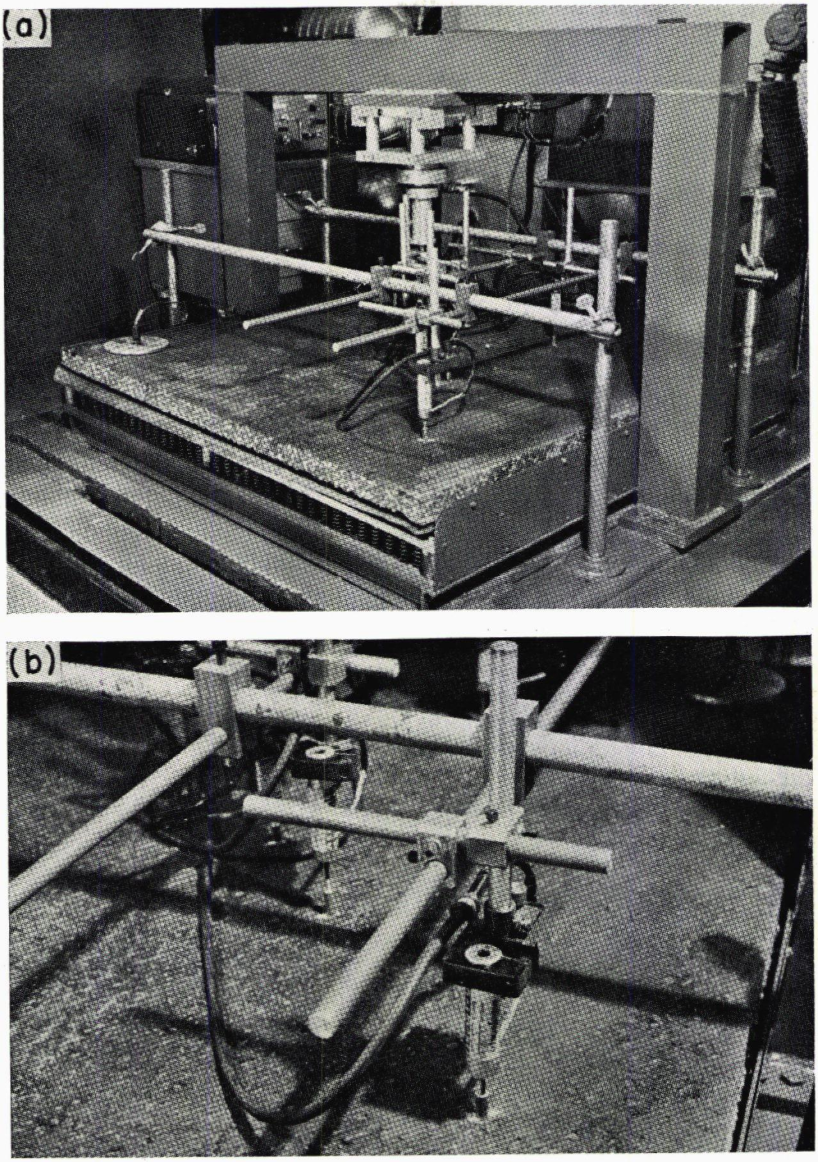


Figure 3. Slab test apparatus ready for operation, showing (a) instrumented slab ready for testing and (b) closeup of differential transformer installation.

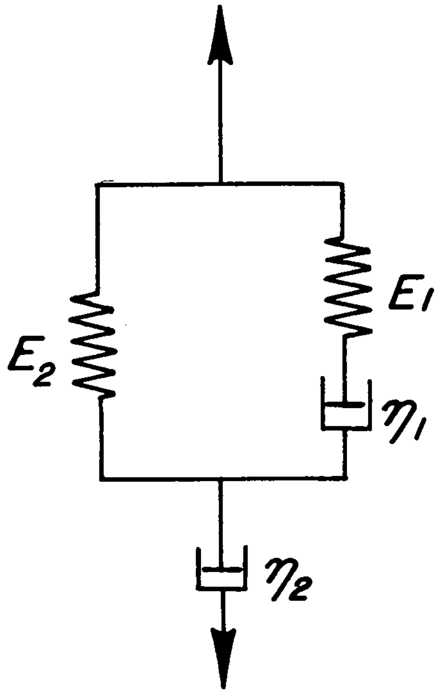


Figure 4. Four-element model.

recoverable, whereas the viscous flow is, of course, irrecoverable. It can be seen that these are precisely the properties often suggested as being characteristic of asphalt paving mixtures (*14, 15*).

Mathematically, the four-element model shown in Figure 4 is equivalent to the Burgers model, a system often mentioned in discussions of viscoelastic analysis; the Burgers model also has the three types of response previously mentioned. The basic differential equation governing the stress-strain-time characteristics of the four-element configuration can be stated as

$$\left[\frac{d^2}{dt^2}+\left(\frac{E_1}{\eta_1}+\frac{E_1+E_2}{\eta_2}\right)\frac{d}{dt}+\frac{E_1\,E_2}{\eta_1\,\eta_2}\right]\sigma\ (t)=$$

$$\left[\left(E_1+E_2\right)\frac{d^2}{dt^2}+\left(\frac{E_1\,E_2}{\eta_1}\right)\frac{d}{dt}\right]\varepsilon\ (t) \quad (1)$$

This equation can be simplified:

$$\left[\frac{d^2}{dt^2}+C_1\frac{d}{dt}+C_2\right]\sigma\ (t)=$$

$$\left[C_3\frac{d^2}{dt^2}+C_4\frac{d}{dt}\right]\varepsilon\ (t) \qquad (2)$$

in which

$$C_1=\frac{E_1}{\eta_1}+\frac{E_1+E_2}{\eta_2}$$

$$C_2=\frac{E_1\,E_2}{\eta_1\,\eta_2}$$

$$C_3=E_1+E_2$$

$$C_4=\frac{E_1\,E_2}{\eta_1}$$

Eqs. 1 and 2 can be written in terms of differential operators $Q(t)$ and $P(t)$ as

$$Q(t)\,\sigma=P(t)\,\varepsilon \qquad (3)$$

For the creep test, where $\sigma_1(t)=\sigma_0$ (a constant) and $\varepsilon_1(0)=\frac{\sigma_0}{E_1+E_2}$ and σ_1 and ε_1 refer to axial stress and strain, the solution of Eq. 1 becomes

$$\varepsilon_1(t)=\sigma_0\left[-\frac{E_1}{E_2(E_1+E_2)}\exp\left(-\frac{t}{\tau^*}\right)+\frac{1}{E_2}+\frac{t}{\eta_2}\right] \qquad (4)$$

in which

$$\tau^* = \eta_1 \left[\frac{E_1 + E_2}{E_1 E_2} \right]$$

For creep recovery on removal of the stress σ_0 at $t = t_0$, the solution of Eq. 1 for $t > t_0$ is

$$\epsilon_1(t) = \sigma_0 \left\{ \frac{E_1}{E_2(E_1+E_2)} \left[\exp\left(-\frac{t-t_0}{\tau^*}\right) - \exp\left(-\frac{t}{\tau^*}\right) \right] + \frac{t_0}{\eta_2} \right\} \qquad (5)$$

For the relaxation test, where $\epsilon_1(t) = \epsilon_0$ (a constant) and $\sigma_1(0) = \epsilon_0 (E_1 + E_2)$, the solution of Eq. 1 is

$$\sigma_1(t) = \frac{\epsilon_0}{R_2 - R_1} [(C_3 R_2 - C_4) \exp(-R_2 t) + (C_4 - C_3 R_1) \exp(-R_1 t)] \qquad (6)$$

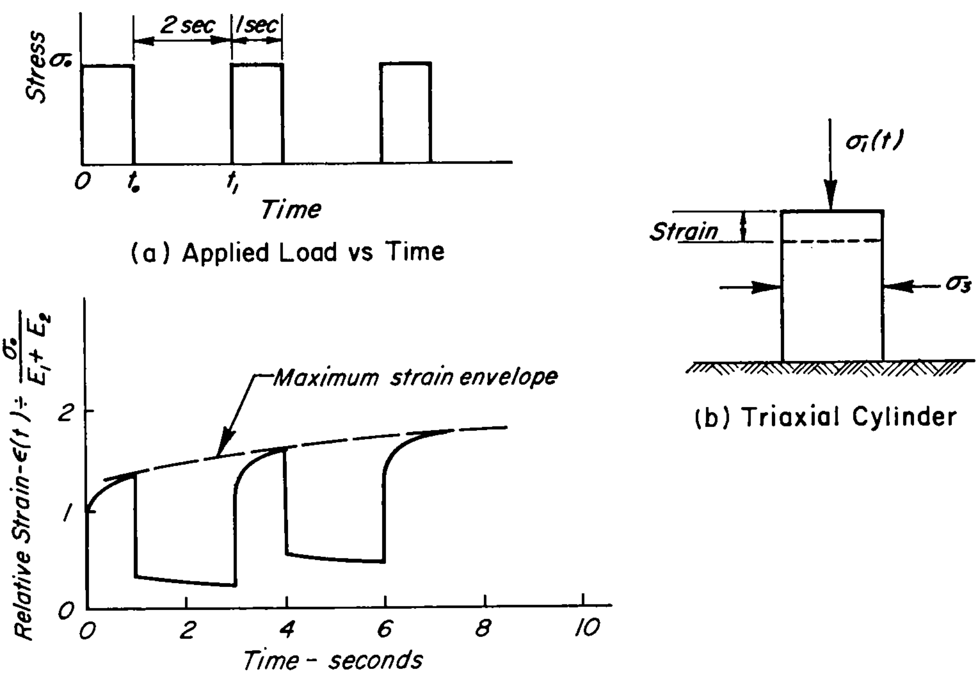
in which C_1, C_2, C_3, and C_4 are the same as for Eq. 2,

$$R_1 = \frac{C_1}{2} + \frac{1}{2}\sqrt{C_1^2 - 4C_2}$$

$$R_2 = \frac{C_1}{2} - \frac{1}{2}\sqrt{C_1^2 - 4C_2}$$

For the constant-rate-of-strain test, where $\epsilon_1(t) = C\,t$ and $\epsilon_1(0) = 0$, the solution of Eq. 1 is

$$\sigma_1(t) = \frac{C}{(R_2 - R_1) R_1 R_2} [R_2 (C_3 R_1 - C_4) \exp(-R_1 t) + R_1 (C_4 - C_3 R_2) \exp(-R_2 t) + C_4 (R_2 - R_1)] \qquad (7)$$



(a) Applied Load vs Time

(b) Triaxial Cylinder

(c) Response of Four-Element Model

Figure 5. Creep and repeated-load relationships for the four-element model.

in which all constants are the same as for Eq. 6.

Eqs. 4 and 5 can be used to deal with repeated-load applications by the simple expedient of shifting the time scale so that these two relationships fit each successive cycle of loading. This process may be visualized by inspection of Figure 5. For the first load cycle, Eqs. 4 and 5 can be applied directly, because this load condition is identical to creep loading. The four-element model, if undisturbed during creep recovery, will rebound until only the irrecoverable viscous flow due to the free dashpot remains. If, however, a second cycle of load is applied before rebound is complete, cumulative buildup of strain in addition to viscous flow may occur. Eqs. 4 and 5 may be used to study the second cycle of load in Figure 5, because the value of strain at $t=t_1$ can be taken as the new initial value. By this same approach, load cycling may be examined indefinitely.

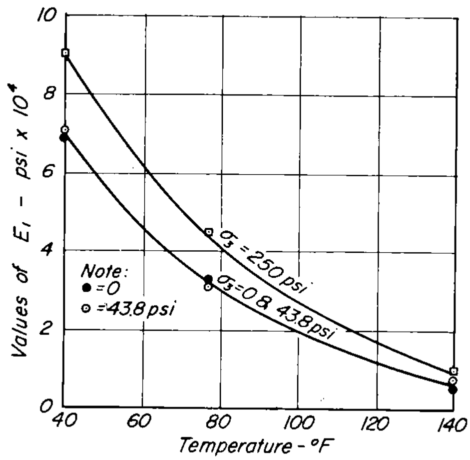


Figure 6. Relation of E_1 to temperature and lateral pressure.

APPLICATION OF MODEL THEORY TO ANALYSIS OF TRIAXIAL TEST RESULTS

Eqs. 4 through 7 completely establish the mathematical relationships necessary for employing the four-element model in connection with the various types of triaxial tests discussed herein. These relationships were used in conjunction with test data to compute values for the four constants, E_1, E_2, η_1, and η_2 over the range of ambient conditions selected for study. Figures 6 through 9 show plots of these numerical values versus temperature and lateral pressure.

The viscoelastic constants for the four-element model are most easily approximated from the results of a creep test, although they can be estimated, with some difficulty, from relaxation test data. Figure 10 shows how such processes may be carried out. In general, the values plotted in Figures 6 through 9 were estimated, for a particular set of ambient conditions, by first considering creep data from tests under those conditions. Four constants providing a satisfactory agreement between theory and data were selected and then applied to the equations governing the constant-rate-of-strain and relaxation tests, for further comparisons between theory and test results. The constants were then adjusted, if necessary, to provide the best possible over-all agreement for all three test types. To reduce the work involved in performing these numerical manipulations, Eqs. 4 through 7 were

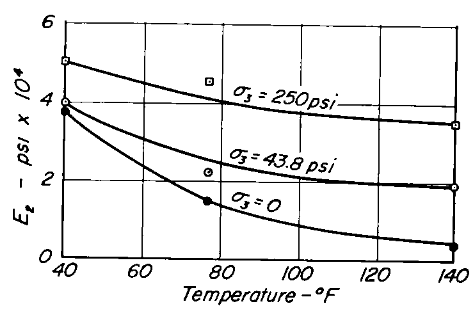


Figure 7. Relation of E_2 to temperature and lateral pressure.

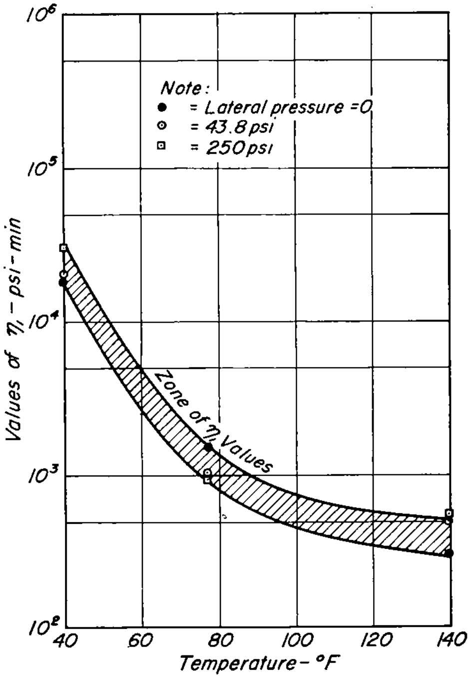


Figure 8. Relation of η_1 to temperature and lateral pressure.

programed for solution on a digital computer.

The values plotted in Figures 6 through 9 were obtained by considering each set of ambient conditions independently; because tests at three temperatures (40, 77, and 104 F) were involved, with three lateral pressures at each temperature (0, 43.8, and 250 psi), nine individual determinations were made. However, when plotted in final form the constants seemed to indicate reasonably clear trends, which may be summarized as follows:

1. E_1 (Fig. 6). Values of E_1 indicated a marked decrease with increases in temperature. Lateral pressure increases produced a corresponding increase in E_1, although a considerable increase in lateral pressure was required to produce an appreciable effect (note that values for $\sigma_3=0$ and $\sigma_3=43.8$ psi lie almost on the same curve).

2. E_2 (Fig. 7). This plot shows the same general characteristics as Figure 6. However, values of E_2 seemed less susceptible to changes in temperature than those for E_1, and more susceptible to changes in lateral pressure.

3. η_1 (Fig. 8). Values of η_1 displayed a marked decrease with increases in temperature. The effect of lateral pressure, however, was not clear cut.

4. η_2 (Fig. 9). This constant showed considerable increases in value with increases in either temperature or lateral pressure.

It should be emphasized that these values of E_1, E_2, η_1, and η_2 are simply arbitrary constants selected for use in connection with Eq. 1. It would probably be a mistake to attempt any

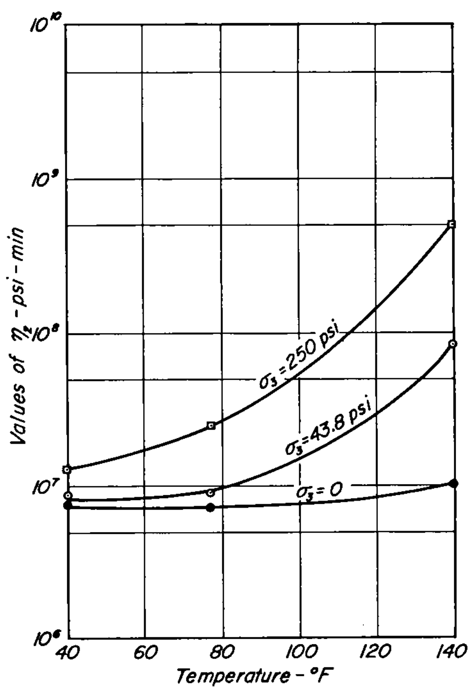
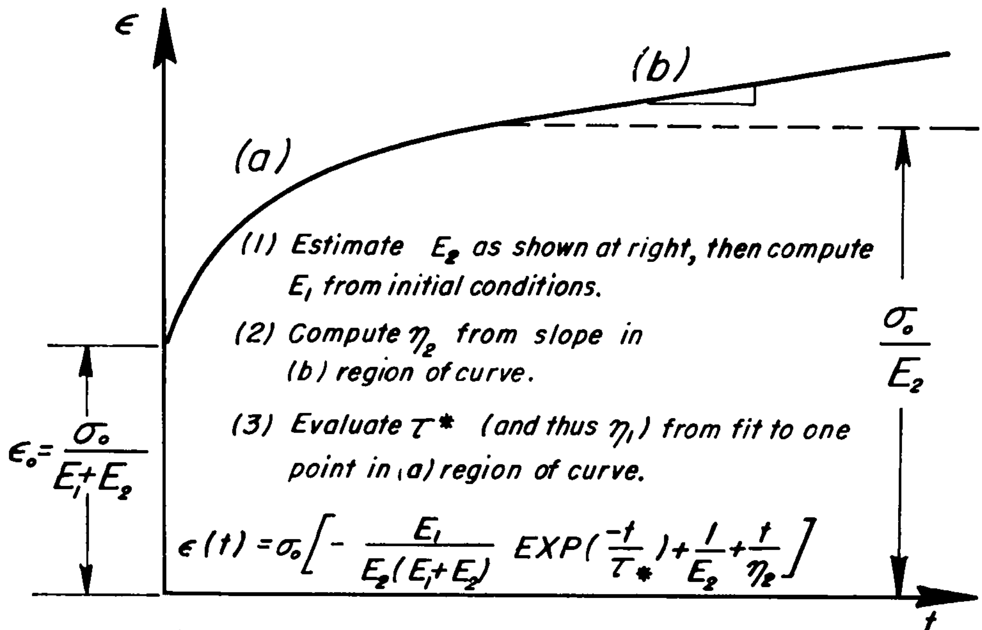
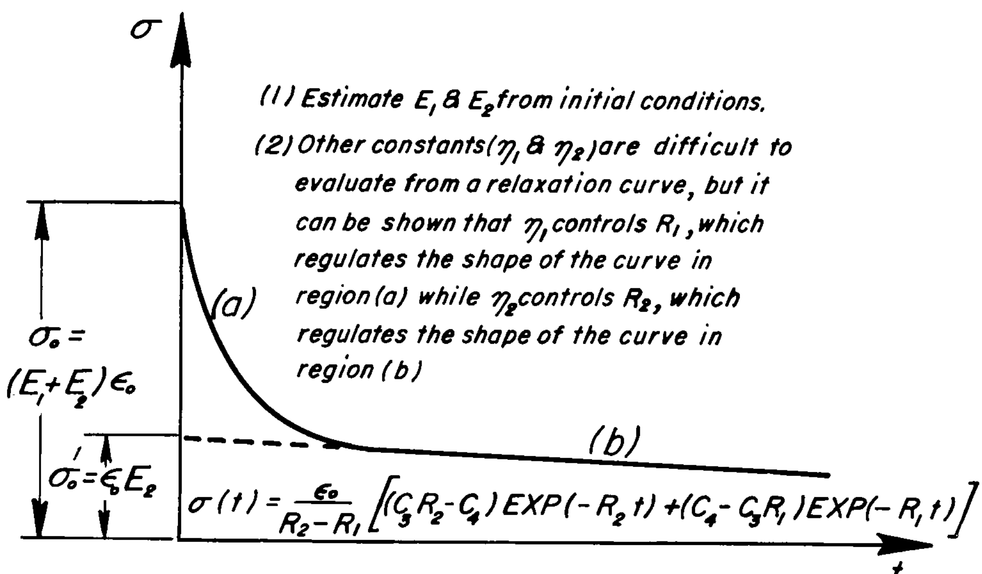


Figure 9. Relation of η_2 to temperature and lateral pressure.



(a) Four-Element Model Relationships for Creep Tests.



(b) Four-Element Model Relationships for Relaxation Tests.

Figure 10. Determination of four-element model constants from triaxial compression test results.

significant connection of the suggested trends with recognized rheologic criteria.

To demonstrate the ability of the four-element model to reflect the characteristics of the paving mixture selected for study, Figures 11 through 14 are presented. These figures show comparisons of unconfined test data at 40 F with similar data as predicted by the four-element model relationships. Although the agreement between actual and predicted data was not perfect, the model appeared to reflect the characteristics of the material to a marked degree over a wide range of load situations. The comparisons given in Figures 11 through 14 are fairly typical of those obtained for the other ambient conditions studied; the disagreement between test data and theoretical predictions seldom exceeded 30 percent and was usually much less. Only for the case of repeated loading at high temperature and low lateral pressures, where some unexplained deviations occurred, did the four-element model fail to achieve satisfactory expression of the rheologic characteristics of the paving mixture.

It should be noted in Figure 14 that no attempt has been made to correct the deformation under the first stress application for the results presented. (This initial load increment can vary due, for example, to seating and apparatus continuity problems.) The shapes of the theoretical and actual curves however, are quite similar up to about 10^4 stress applications.

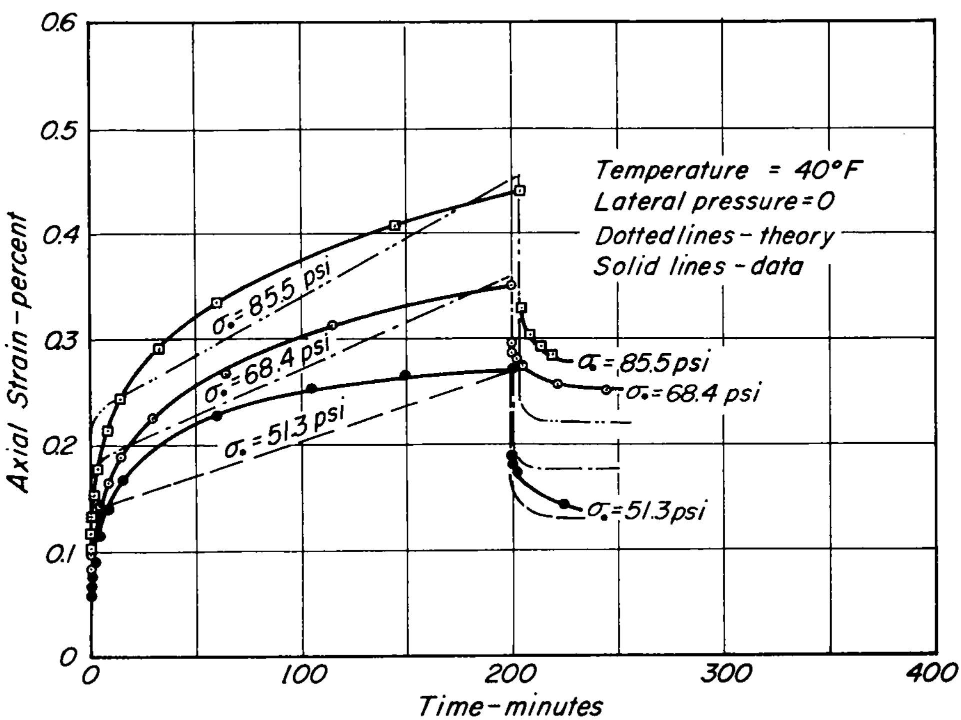


Figure 11. Comparison of creep test data with data predicted by four-element model.

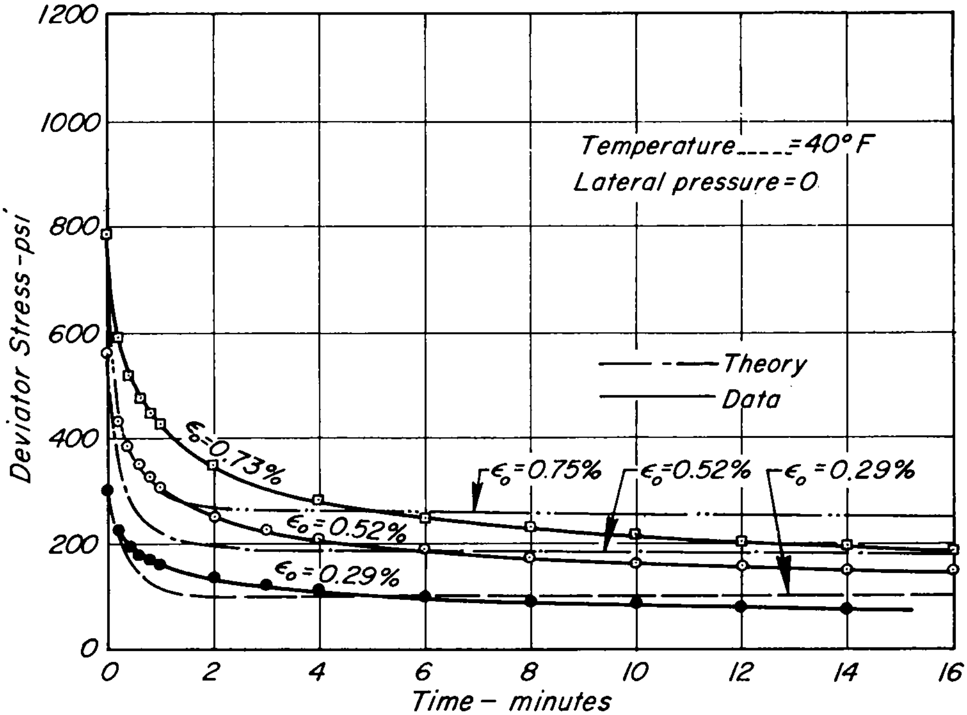


Figure 12. Comparison of relaxation test data with data predicted by four-element model.

VISCOELASTIC SLAB THEORY

The viscoelastic equations and coefficients presented in the preceding discussion were used to produce theoretical predictions of the deflections measured during slab tests on the spring base. For the purpose of this paper, it was assumed that a paving slab resting on the spring base could be closely approximated by the structural analogy of a thin viscoelastic plate on a Winkler foundation (approximated by a set of independent springs).

Hertz (*16*) has presented a solution for the structural responses of a thin elastic plate on a Winkler foundation. He showed that the deflections of such a plate could be obtained, in the case of an infinitely large plate in a state of axial symmetry, from

$$D\nabla^4 w(r) = q(r) - p(r) \qquad (8)$$

in which

$$D = \frac{E\,h^3}{12(1-\nu^2)};$$

E = elastic modulus;

h = thickness;

ν = Poisson ratio of plate,

$$\nabla^2 = \frac{\partial^2}{\partial r^2} + \frac{1}{r}\,\frac{\partial}{\partial r};$$

$w(r)$ = vertical deflection of plate, as a function of radius, r, from center of load;

$q(r)$ = surface loading; and

$p(r)$ = foundation reaction.

It has already been mentioned that the basic differential equation for the

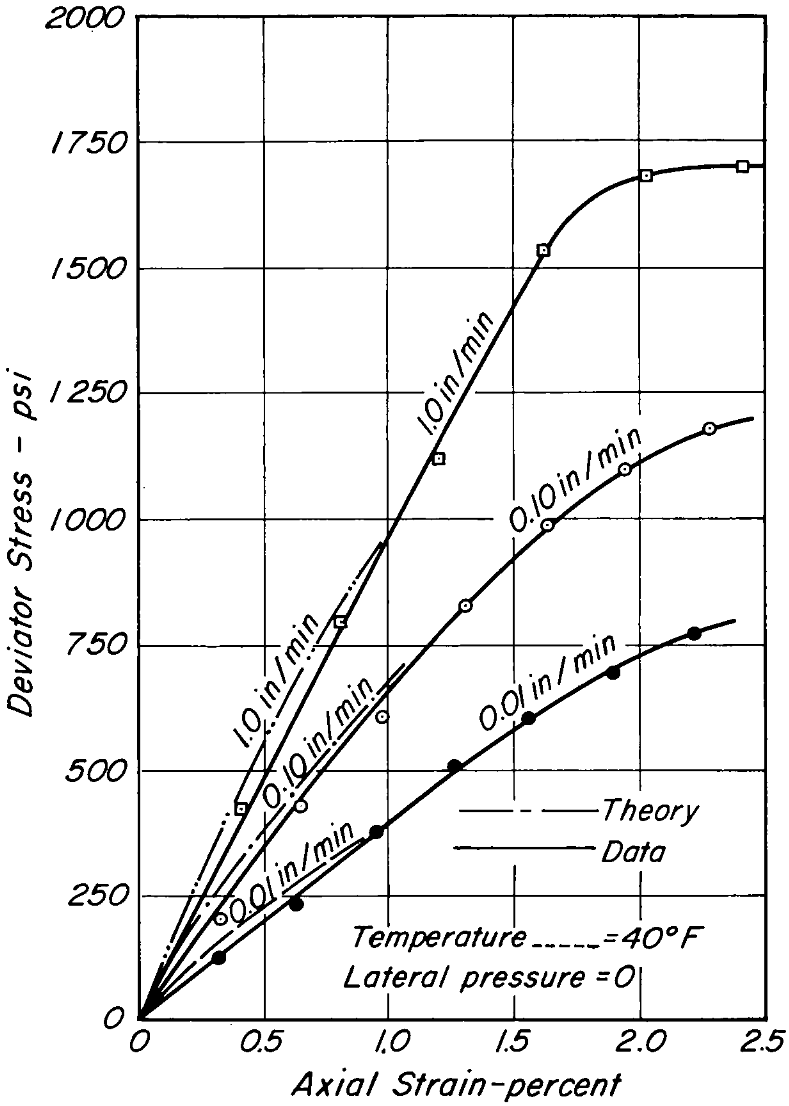


Figure 13. Comparison of constant-rate-of-strain test data with data predicted by four-element model.

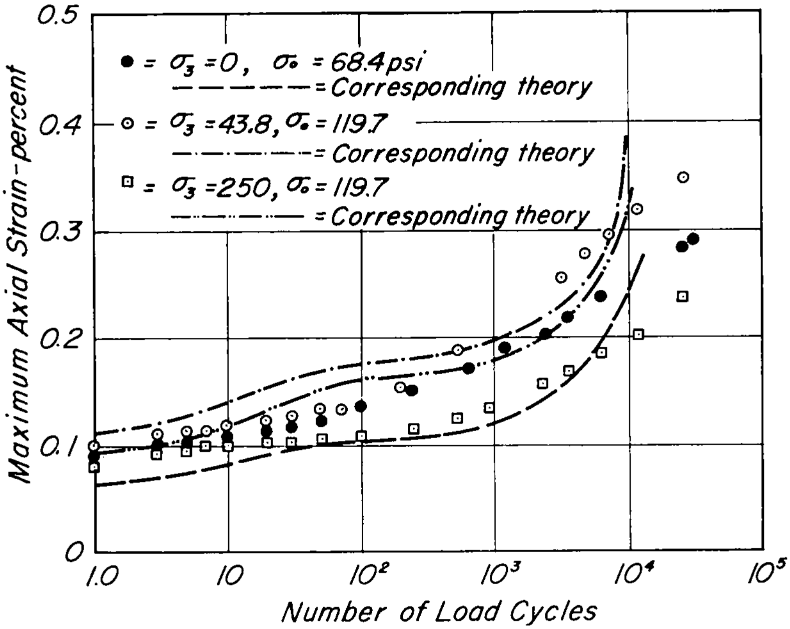


Figure 14. Comparison of repeated-load test data with those predicted by four-element model—40 F.

four-element model can be expressed in terms of the differential operators $P(t)$ and $Q(t)$. By replacing the modulus of elasticity in the D-term of Eq. 8 with the ratio of these operators, $P(t)/Q(t)$, an expression for the viscoelastic behavior of the slab can be obtained:

$$D(t)\nabla^4 w(r,t)=q(r,t)-p(r,t) \tag{9}$$

It can be seen that the time-dependence of w, q, and p is also implied by this process. The solution of this differential equation, utilizing an application of the correspondence principle relating elastic and viscoelastic boundary value problems, was carried out through the use of integral transform methods (*4*). The result for the deflections due to a uniform load of radius r_1 applied at $t=0$ and held constant is:

$$w(r,t)=\frac{q_0}{k}\int_0^\infty \frac{f(a,b)}{\left[\left(\frac{l_0}{r_1}\right)^4\lambda^4+1\right]\left[a\,b(a-b)\right]} J_1(\lambda)J_0\left(\frac{\lambda}{r_1}r\right)d\lambda \tag{10}$$

in which

λ = an integration variable;

q_0 = unit load on area of radius r_1;

k = foundation modulus;

C_1, C_2, C_3, *and* C_4 = basic viscoelastic constants, Eq. 2;

$$l_0^4=\frac{D^*}{k};$$

$$D^*=C_3\left[\frac{h^3}{12(1-\nu^2)}\right];$$

$$a = \frac{-\left[A\left(\frac{l_0}{r_1}\right)^4 \lambda^4 + C_1\right] + \left[A^2\left(\frac{l_0}{r_1}\right)^8 \lambda^8 + B\left(\frac{l_0}{r_1}\right)^4 \lambda^4 + F\right]^{\frac{1}{2}}}{2\left[\left(\frac{l_0}{r_1}\right)^4 \lambda^4 + 1\right]};$$

$$b = \frac{-\left[A\left(\frac{l_0}{r_1}\right)^4 \lambda^4 + C_1\right] + \left[A^2\left(\frac{l_0}{r_1}\right)^8 \lambda^8 + B\left(\frac{l_0}{r_1}\right)^4 \lambda^4 + F\right]^{\frac{1}{2}}}{2\left[\left(\frac{l_0}{r_1}\right)^4 \lambda^4 + 1\right]};$$

$$A = C_4/C_3;$$

$$B = 2\frac{C_1C_4}{C_3} - 4C_2;$$

$F = C_1^2 - 4C_2$; and

$$f(a,b) = \{\exp(at)\,[a^2b + c_1ab + C_2b] - \exp(bt)\,[ab^2 + C_1ab + C_2a] + C_2(a-b)\}$$

The foregoing equation is tedious to evaluate by desk computational methods; therefore, it was programed for the digital computer.

The derivation of Eq. 10 for use in the analysis of the deflections of the asphalt paving slabs tested for this paper involved certain important assumptions, as follows:

1. The thin-plate theory defined by Eq. 8 was derived by neglecting the effects of transverse normal and shearing stresses.
2. The basic theory also required the assumption that the plate (slab) material had equal properties in tension and compression.
3. Poisson's ratio for the plate (slab) material was considered to be independent of time, in order to achieve an important (although not absolutely essential) mathematical simplification.
4. The lateral dimensions of the plate were taken as infinite, in relation to the size of the area affected by the imposed load.

The importance of these assumptions in influencing the ability of the theory, as exemplified by Eq. 10, to describe the deflections of the test slabs is discussed later.

COMPARISON OF VISCOELASTIC SLAB THEORY WITH TEST DATA

The results of deflection measurements on a typical test slab at 40 F are shown in Figures 15 and 16. Figure 15 shows deflection measured under the center of load versus time; Figure 16 shows the profiles of the slab for various times after load application. Similar data were obtained for each of the slabs tested. The loads at each temperature were selected arbitrarily from theory as being those which would produce deflections roughly corresponding to values obtained from field measurements on typical pavement installations.

Also shown in Figures 15 and 16 are the theoretically predicted curves produced by the application of Eq. 10. These curves were computed for each of the six test slabs by applying the viscoelastic coefficients (C_1, C_2, C_3, C_4) obtained from the unconfined triaxial compression tests at the same temperatures. The constants derived from unconfined tests were used because, except for the region directly below the loaded area, lateral pressures in the slabs were estimated as

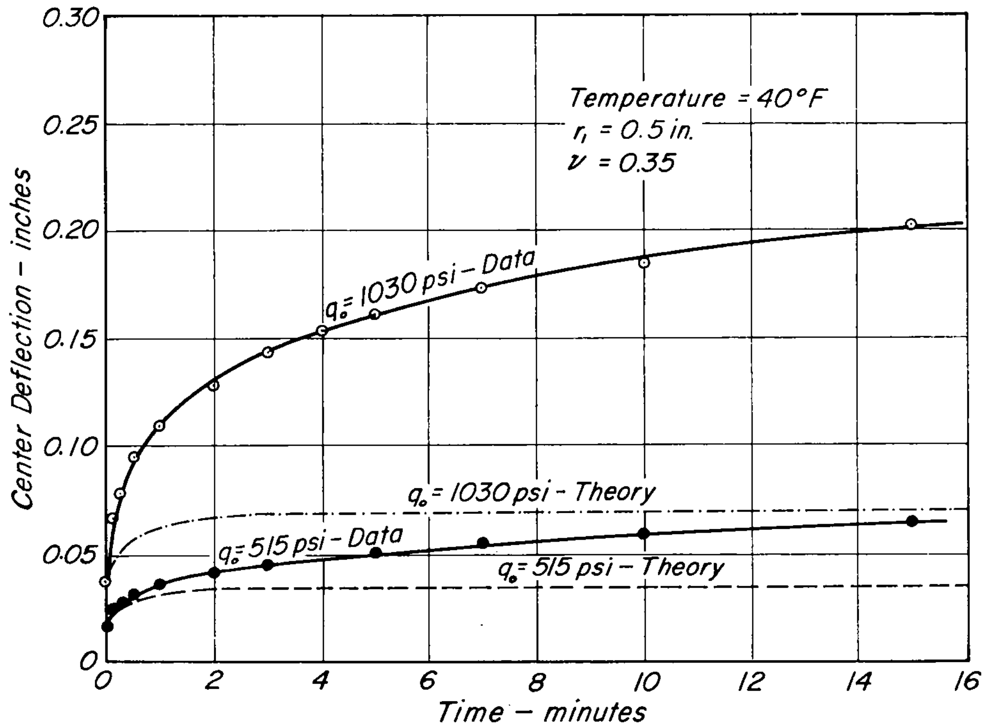


Figure 15. Comparison of measured deflections under centers of loaded slabs with deflections predicted by viscoelastic theory.

being negligible. Poisson's ratio for each temperature was estimated from the results of volume change measurements previously reported (*1*).

A large disparity between theory and test data is apparent from an inspection of Figures 15 and 16; similar disagreement was found for all of the comparisons made. It is interesting to note, however, that the disagreement was affected to some extent by temperature. The test results at 40 F, for example, show a ratio of measured deflections to those predicted by theory of almost 2:1 after 15 min of load application (Figures 15 and 16); at 77 F the ratio was more on the order of 3:1.

Although data such as those shown in Figures 15 and 16 might raise a question as to the value of viscoelastic analysis as applied to problems such as the one under consideration, there are a number of important factors which should be cited in argument against such an early, and possibly mistaken, conclusion. These factors are considered in the following.

It was mentioned earlier that slight differences in asphalt content and aggregate gradation existed between the triaxial test cylinders and the slab specimens. A comparison between the results of triaxial compression tests in creep, stress relaxation, and constant-rate-of-strain loading for the two mixtures failed to disclose any significant differences in their rheologic properties. Thus it was believed that the explanation of

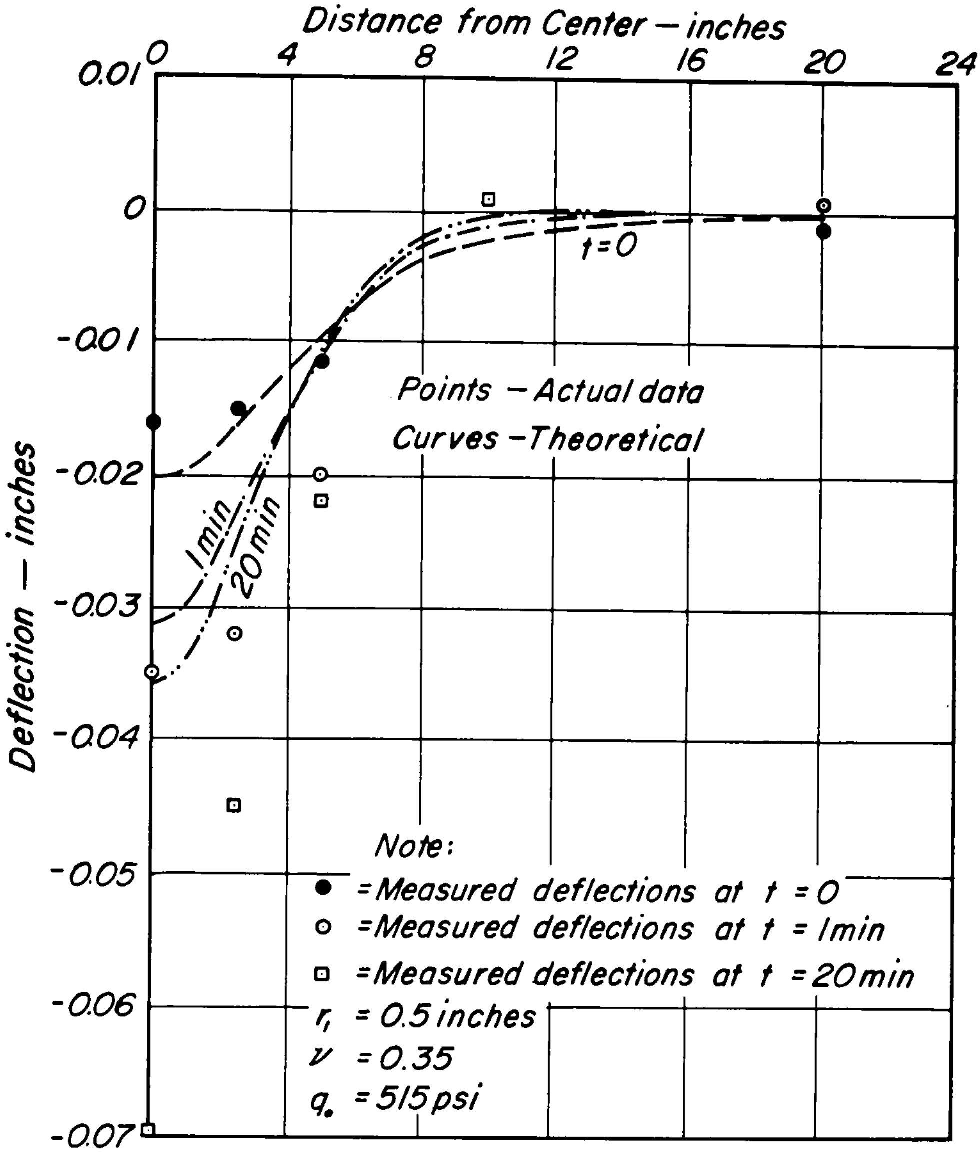


Figure 16. Comparison of actual and theoretical deflection profiles of a test slab for various times after load application—40 F.

the disagreement between test results and the viscoelastic thin-plate theory found in the slab studies might be afforded by a critical examination of the assumptions made in deriving and applying that theory, as follows:

1. The basic assumption that transverse normal and shearing stresses could be neglected is open to question; however, it is unlikely that the errors introduced by this simplification would be of the order of magnitude discussed here.

2. The assumption that asphalt paving mixtures have the same properties in tension and compression is almost certainly in error. Unfortunately, few data are available on this subject at present. Quite probably, the stiffness of asphalt mixtures in tension is less than in compression; this difference might also be time-dependent. These factors could easily cause errors in this theory, as given by Eq. 10, of the magnitude shown in Figures 15 and 16. Research is currently under way to evaluate this possibility.

3. The assumption of a constant Poisson's ratio for a viscoelastic material can also be questioned. However, for the purpose of this paper it was found expedient to employ this assumption, after careful consideration of the possible resultant effect on theoretical calculations. Values for the Poisson's ratio of the slab material at the desired temperatures were estimated from the results of volume change measurements during unconfined constant-rate-of-strain tests, as has already been mentioned. The results are given in Table 2. As can be seen, the assumption of a constant ratio at each temperature is nearly correct.

4. The infinite boundary conditions assumed for Eq. 10 were apparently satisfied at all temperatures used during slab testing. In all cases the deflections measured at a radius of 20 in. from the center of load (*i.e.*, the edge of the slab) were relatively negligible, as suggested by theory.

TABLE 2
POISSON'S RATIOS COMPUTED FROM VOLUME CHANGE MEASUREMENTS
(CONSTANT-RATE-OF-STRAIN TESTS)

Temp. (°F)	Load Rate (in./min)	Poisson's Ratio
40	0.01	0.371
40	0.10	0.358
40	1.00	0.305[1] (approx.)
77	0.01	0.492
77	0.10	0.484
77	1.00	—[1]
140	0.01	0.495
140	0.10	0.498
140	1.00	—[1]

[1] Volume changes difficult to record at this load rate.

5. In addition to the formal assumptions previously listed, use of a linear viscoelastic model in deriving Eq. 10 implied that the levels of stress and strain imposed on the test slabs be kept in the linear range. This range was not well defined in the triaxial compression test phase of this research, due to the time demands required for such definition. If the loads imposed on the test slabs were such that the linear material range was exceeded, the disagreement shown in Figures 15 and 16 might easily be explained in part. Further investigation of this problem is also under way.

SUMMARY AND CONCLUSIONS

The ability of an analytical system founded on a simple, linear, viscoelastic model to express the rheologic characteristics of an asphalt concrete was demonstrated by the data presented herein. The comparisons made here to triaxial compression test results through the use of the four-element model clearly showed that it could provide a general approximation of the properties of the material under study over a wide range of load types and ambient conditions. The errors obtained during such comparisons in most cases were less than 30 percent.

The success obtained in the application of viscoelastic analysis to the prediction of the deflections of the slabs of the test mixture placed under static loading on an elastic foundation was less marked. However, further research into areas such as the comparison of relative properties of asphalt mixtures in tension and compression and the study of the linear limits of these properties holds promise that such analysis can be vastly improved. It must be remembered that elastic techniques, as they might be applied to such a problem, would only be capable of giving one set of deflections under a specific load; the necessary variations in those deflections with time would not be available. Thus, with further attention to some of the considerations mentioned in the foregoing discussion, it would seem that viscoelastic theory might yet serve to provide a better analytic framework for application to specific loading conditions for asphalt concrete pavements.

ACKNOWLEDGMENTS

The authors would like to thank the staff of the Institute of Transportation and Traffic Engineering of the University of California for their efforts in the preparation of this paper. Thanks are also due to the staff of the California Research Corporation in regard to the preparation of the asphalt paving slabs tested for this work and to the Asphalt Institute for the grant-in-aid supporting the investigation. The authors also wish to acknowledge the assistance of R. A. Westmann in performing some of the mathematical operations presented herein.

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DEPARTMENT OF MAINTENANCE

Calcium Chloride-Salt Snow and Ice Control Test, Winter 1960-61

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The Connecticut State Highway Department, for many years has endeavored to maintain bare pavements throughout the winter. It is extremely difficult and in some areas impossible to accomplish the desired results using sand-salt mixture or salt alone.

A further consideration was the conservation of sand. In many areas of Connecticut, sand deposits have been depleted or zoned out of existence, resulting in the necessity of importing sand. The resulting increase in the cost of sand tends to make chemical control of snow and ice more attractive from an economic point of view.

The results obtained by the use of calcium chloride-salt mixtures on the New York Thruway were cited to gain permission to conduct a test of chemical control of snow and ice on a section of Conn. 15 in the towns of Willington, Ashford, and Union during the winter of 1960-61. This report covers the test in considerable detail.

As pointed out in the report, mechanical failures and unfamiliarity with procedures resulted in higher costs than would normally be expected. Nevertheless, the cost appears to be substantially the same as standard methods of snow and ice control.

• THE CONNECTICUT State Highway Department is constantly seeking to improve its techniques and methods in snow and ice control. The successful use of calcium chloride-salt mixtures by the New York Thruway Authority and the Massachusetts Department of Public Works prompted the Connecticut Highway Department to conduct controlled tests on a section of its highway system during the winter of 1960-1961.

In addition to improving service to the public, it is necessary to give serious consideration to the increasing cost of abrasives and their increasing scarcity. In many areas of the State, deposits of sand have been depleted or zoned out of existence.

LOCATION

The test section, on Conn. 15 (the Wilbur Cross Highway, a 4-lane divided highway) in the towns of Willington, Ashford, and Union, started at the east junction of US 44 and ran easterly to the Massachusetts State line, a distance equal to 27.8 mi of 2-lane road. The control section (also on Conn. 15) was in the towns of Tolland, Vernon, South Windsor, and Manchester and started at the east junction of US 44 and extended westerly to the Tolland Turnpike overpass, a distance equal to 29.1 mi of 2-lane road.

The test section is considerably higher in elevation than the control section and contains more and longer grades. Because of the difference in elevation, temperatures are generally lower in the test section.

PROCEDURES

Storage facilities at the highway maintenance garage in the town of Union were limited. Two carloads of bulk flake calcium chloride were stored in a salt shed from which all salt had been removed. Two more carloads were piled on the ground and covered with sand and polyethylene sheeting. Figure 1 shows calcium chloride storage early in the season. Unfortunately, children living in the vicinity of the storage area punched holes in the covering and some dampening and lumping of the calcium chloride resulted.

All the salt was stored outside and covered with tarpaulins and polyethylene sheeting.

The general procedure was to have available enough of the calcium chloride-salt mixture for one storm, about 30 to 35 tons. Sometimes it was necessary to mix additional amounts during the storm.

The mixing was done at the storage area on a dry bituminous surface. A front-end loader was used. Two buckets of salt were put down, then a bucket of calcium chloride, then two more buckets of salt and another bucket of calcium chloride. This was then thoroughly mixed by the front-end loader. The process was repeated until the required amount of the mixture had been prepared. The material was stored on a bituminous surface at the storage area and covered with polyethylene sheeting until needed for storm use.

The front-end loader was used for loading the mixture into Good Roads spreaders. These spreaders can be easily installed or removed

Figure 1. Calcium chloride storage shed with surplus stored outside.

from a regular dump truck body. Normally they would remain on the truck during the winter season. Figure 2 shows the spreader being loaded.

At the end of the storm season what remained in the separate stockpiles of calcium chloride and salt was covered with sand and polyethylene sheeting. A recent inspection showed that the materials stood up very well. A slight crust had formed on both the calcium chloride and the salt but it crumbled easily on handling.

Figure 3 shows the calcium chloride storage shed buttoned up between storms after the outside storage pile had been used.

Except where otherwise indicated the test mixture proportions were one part calcium chloride to two parts salt by volume.

The rate of application of the calcium chloride-salt mixture varied from 400 to 1,200 lb per mi of 2-lane roadway with 800 lb being the rate most frequently used. The mixture was spread in the center of the traveled way except on banked curves where it was spread on the high side.

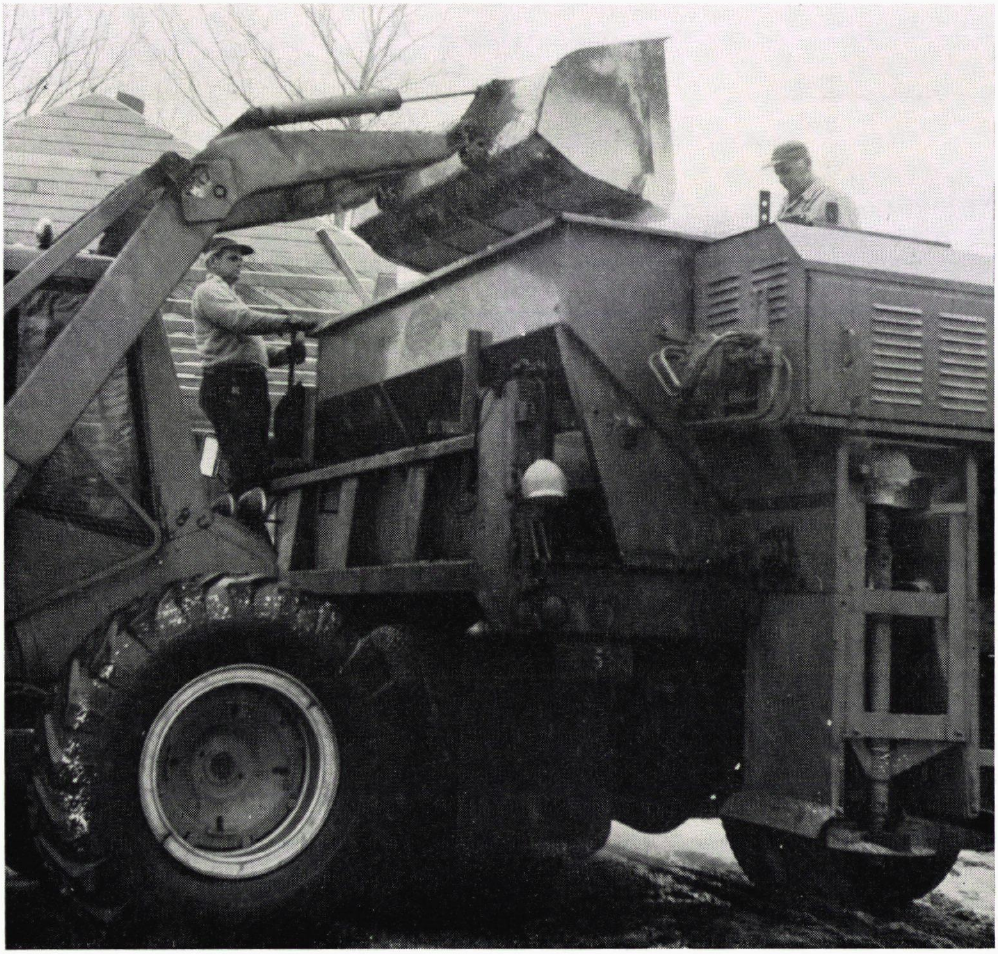

Figure 2. Loading demountable sander with mixture.

Figure 3. Calcium chloride storage shed closed by tarpaulin between storms.

At first the mixture was spread from 12 to 14 ft wide. However, as experience was gained, this was gradually reduced to a width of 3 to 5 ft. This proved to be the most effective width of spread.

In the control section, salt was spread from 3 to 4 ft wide in the center of the traveled way. The rate of application varied from 500 to 1,200 lb per mi of 2-lane roadway depending on the temperature and the amount of precipitation, with 1,000 lb being the rate most frequently used.

The calcium chloride-salt mixture was applied one or more times in the test section during 17 storms. In the control section, straight salt was applied one or more times in 15 storms and treated sand (1 part salt to 8 parts sand) was used in two storms when temperatures were high and precipitation was light.

In several of the storms, when the temperature was very low, a mixture of one part calcium chloride to one part salt was applied to those areas in the test section that did not clear up fast enough with the one to two mixture.

Normal plowing operations when necessary were carried on in both sections during each storm, and hills and curves were sanded from time to time if they became slippery.

Figure 4 shows the condition of the pavement in the test section during test 3, 1½ hr after spreading at the rate of 800 lb per mi. The off-ramp has had only normal application of treated sand.

OBSERVATIONS

Table 1 was compiled from reports submitted by the foremen of both the test and control sections at the end of each storm and shows that the test section was in better condition during 8 storms. The control section was better during 2 storms and both sections were the same during 7 storms. The test section was clear of snow and ice sooner in 4 storms, the control section in 1 storm and both sections were cleared at about the same time during 12 storms, although, in general, temperatures were lower and snowfall heavier in the test section.

Total snowfall for all 17 storms was approximately 87.5 in. for the test section and 80.0 in. for the control section. The average temperature in the test section was 26.1 F and in the control section 29.7 F. Duration of the 17 storms was 209.75 hr for the test section, and 209.25 hr for the control section. Total amount of sand used was 730 cu yd in the test and 632 cu yd in the control section. Over-all cost was $24,238.49 in the test section and $21,823.67 in the control. This does not include sand cleanup.

The cost for rock salt was about $13 a ton and that of bulk calcium chloride about $36 a ton. This brought the mixture cost to approximately $19 a ton.

Because one of the objectives in testing the calcium chloride-salt mixture was to reduce the amount of sand used, the greater use of sand in the test section needs an explanation. The shed for the storage of salt was not completed until almost the end of the season. The salt got wet several times and when mixed with the calcium chloride caused the mixture to become lumpy and clog the spreaders.

During storms 7, 14, and 17 considerable difficulty in starting equipment and caking of the mixture was encountered. As a result it was necessary to use substantial amounts of sand, making the chemicals less effective when applied. If the sand used during these three storms is deducted from the total amounts used in each section, it then appears that the test section used 340 cu yd and the control section 514 cu yd of sand. Comparison of the amounts of sand used during the individual storms indicates less sand was generally used in the test section than in the control section. An attempt was made to determine what the probable costs, including spring cleanup, would be if treated sand, straight salt, or calcium chloride-salt were used. Because no detailed costs are available for previous years it was decided such speculative costs would not be significant.

Those who have been closely involved in maintenance operations are

Figure 4. Southbound roadway at Conn. 198, 1½ hr after application of mixture.

TABLE 1

CALCIUM CHLORIDE-SALT TEST, CONN. 15, WINTER OF 1960-61

Test No.	Date	Test Section[1]								Control Section[2]							Section		
		Duration of Storm (hr)	Total Snowfall (in.)	Avg. Temp. During Storm (°F)	Calcium Chloride Used (tons)	Salt Used (tons)	Sand Used (cu yd)	Total Cost ($)	Cost per Mile ($)	Duration of Storm (hr)	Total Snowfall (in.)	Avg. Temp. During Storm (°F)	Salt Used (tons)	Sand Used (cu yd)	Total Cost ($)	Cost per Mile ($)	In Better Condition During Storm	Cleared Sooner: Section	Cleared Sooner: Hr
1	12-11-60	16½	16	13	7.5	25	—	1,966.27	70.78	25	15	18	6	12	1,726.62	59.29	Test	—	—
2	12-16-60	18	2	31	10	25	—	1,123.32	40.44	20	2.5	31.5	36	40	1,091.28	37.48	Test	—	—
3	12-19-60	5½	1	26	14	42	20	1,234.18	44.43	6½	1	26	33	17	853.16	29.30	—	—	—
4	12-21-60	3½	1.5	29	9	27	—	1,047.72	37.71	4	1	34	12	25	693.76	23.82	—	—	—
5	12-29-60	16½	3.5	24	28	84	55	1,749.92	62.99	13½	3	26	30	71	2,037.50	69.97	Test	Test	3½
6	1- 1-61	6	3	30	7	21	30	1,573.60	56.65	6½	2.5	34	12	40	1,123.04	38.57	—	—	—
7	1-15-61	24½	12	26	12.4	30.8	275	1,807.10	65.05	25	10	31.5	44	60	2,184.96	75.03	Test	Control	1½
8	1-19-61	18	14	11	9	27	50	2,559.60	92.14	20	13	19	12	40	2,152.36	73.91	Test	Test	1
9	1-26-61	12	3	12	7	21	5	1,176.49	42.35	10½	2.5	18	3	39	1,296.38	44.52	Test	Test	1
10	2- 3-61	25	14	24.5	15	45	55	3,044.33	109.59	21	15	28	24	30	2,277.82	78.22	Test	Test	½
11	2-14-61	3½	0.5	33	2	6	20	388.04	13.97	4	0.5	37	—	20	293.68	10.09	—	—	—
12	3- 1-61	12¾	3	32	9	25	20	1,231.10	44.32	9½	1	34	30	30	1,070.05	36.75	—	—	—
13	3- 8-61	14½	3	29.5	9	21	35	1,540.58	55.46	15¼	3	32.5	36	50	1,586.27	54.47	Test	—	—
14	3- 9-61	6½	0.5	29	2.5	7	60	541.13	19.48	2	0.5	32	—	18	214.50	7.37	—	—	—
15	3-13-61	16½	5.5	34	10	27	35	1,509.75	54.35	14½	5	35	48	40	1,601.40	54.99	Control	—	—
16	3-19-61	7	2	29	8	21	15	985.41	35.47	7	2	33	24	60	1,067.79	36.67	—	—	—
17	3-19-61	3½	3	31	3	10	55	759.95	27.36	5	2.5	35	6	40	553.10	18.99	Control	—	—
Total	—	209¾	87.5	—	162.4	464.8	730	24,238.49	—	209¼	80.0	—	356	632	21,823.67	—	—	—	—
Avg.	—	12.34	5.15	26.12	9.55	27.34	42.94	1,425.79	51.32	12.31	4.71	29.68	20.94	37.18	1,283.75	44.08	—	—	—

[1] Conn. 15, from east junction of US 44, easterly to Massachusetts State Line (27.78 mi of two-lane road).

[2] Conn. 15, from east junction of US 44, westerly to the Tolland Turnpike Overpass at Buckland (29.12 mi of two-lane road).

well aware that there is no substitute for experience. It must be remembered that the crews in the control section have had 15 years experience in the use of treated sand and straight salt. On the other hand, the crews in the test section were totally unfamiliar with the storage, mixing, and use of the calcium chloride-salt mixture. As might be expected, mistakes were made. However, knowledge gained from these mistakes should help to reduce costs in the future.

Both the foreman and general foreman in the test section were somewhat skeptical of the use of the mixture, particularly the possible reduction in the use of sand. This attitude was completely reversed at the end of the snow season. The general foreman's and the foreman's reports are contained in the Appendix. The crew that worked on the test section can now pass on to other crews the experience gained in the use of the mixture during the 1960-61 season.

During the 17 storms of the test period the average temperature was above freezing in only 2 storms in the test section, but in 8 storms in the control section. There were 3 storms when the average temperature was below 20 F in both sections.

Neither calcium chloride nor salt was used during the first storm on December 11 and 12. On the day after the storm there were approximately 8.5 mi of hard-packed snow, 2 to 4 in. thick, covering or partially covering one or both lanes in the test section. Although the temperature did not go above 6 F, all the packed snow was removed with the calcium chloride-salt mixture and snowplows in a period of 6.5 hr (10:30 AM to 5:00 PM). In the control section even though the temperature rose to 12 F, it took as long to clear 6.5 mi of highway covered with the same thickness of hard-packed snow. However, in addition to salt and snowplows, graders were also used. The test section was in safer condition throughout the storm, although the cost per mile was greater.

In storm 8 (January 19-21, 1961) the snowfall in the test section was 14 in. and in the control it was 13 in. The average temperatures were 11 F and 19 F, respectively. The traveled way in the test section cleared approximately 6.5 hr earlier than that in the control. The test section was in better condition during the storm and during the cleanup. Only calcium chloride-salt mixture and snowplows were used except for several small areas where a grader was used for a short time. In the control section, in addition to salt and snowplows, graders were used nearly all day January 21 and until 10:30 PM that night. The test section cost more per mile, however, it was safer during the storm and cleared 6½ hr ahead of the control section.

In storm 9 the snowfall was approximately 3 in. in the test section and 2.5 in. in the control. Average temperatures were 12 F and 18 F, respectively. The test section cleared approximately one hour sooner than the control and was in better condition during the storm. The cost of snow removal and sanding was approximately 10 percent less in the test section.

In storm 16 the snowfall and duration of the storm were the same. The temperature was 4 F higher in the control section. Both sections were substantially the same during the storm and cleared up at the same time. The test section used 15 cu yd of sand and the control 60 cu yd. Here also the cost per mile for snow removal and sanding was lower on the test section.

During the three storms when the average temperature was below 20 F in both sections the calcium chloride-salt mixture worked much better than straight salt. Also during storm 16, when the average temperature in the test section was 29 F and that in the control section 33 F, the use of the mixture was more economical.

It has been the practice in Connecticut to include the cost of spring cleanup in the total cost of snow and ice control.

The original plan for the experiment contemplated detailed costs for this operation. Because scheduled cleanup in the control section would only include shoulders, gutters, and catch basins and the test section would include these items plus two or three years' accumulation of sand back of the shoulder as well as other areas, a fair comparison could not be made.

The Highway Department has expanded its program to include 8 test and 8 control sections in various areas of the State during the winter of 1961-62. It is anticipated that cleanup cost can be included in subsequent reports. Fortunately, the test section is an area where special maintenance costs have been tabulated as part of a Bureau of Public Roads maintenance cost study. The control section was not a part of this study. Table 2 gives data for the test section for the winters of 1959-60 and 1960-61. The control section was not part of this study.

In the winter of 1960-61 when the calcium chloride-salt mixture test began, although the snowfall was more than 80 percent greater than that of the previous winter and the temperatures somewhat lower, the cost for snow and ice removal was only slightly more than 3 percent greater. This increase could be partially accounted for by annual increments in wages and possibly increases in the costs for materials and equipment.

Caution should be used in drawing any conclusions from Table 2. Depth of snowfall and temperature are only two factors of many that influence cost. For example, the storm of test 1 had 16 in. of snow, had 13 F average temperature in the test section, and cost $70.78 per mi for control. The storm of test 2 had 2 in. of snow, had 31 F average temperature, and cost $40.44 per mi for control. On the basis of snowfall and temperature alone one might expect a cost of $9 or $10 per mi for test 2.

CONCLUSIONS

Based on the results of this test, the following conclusions appear valid:

TABLE 2

TEST SECTION, WINTERS OF 1959-60 AND 1960-61[1]

Period	Snowfall		Avg. Temp.		Sand Used		Salt Used		Calcium Chloride Used		Total Cost Snow Removal and Sanding	
	In.[2]	%	°F[2]	%	Cu Yd	%	Tons	%	Tons	%	Dollars	%
Winter 1959-60	48.32	—	29.99	—	2,904	—	614.5	—	—	—	38,651.59	—
Winter 1960-61	87.5	—	27.41	—	1,697	—	464.8	—	162.4	—	39,912.77	—
Excess:												
Winter 1959-60 over winter 1960-61	—	—	2.58	9.42	1,207	71.13	149.7	32.2	—	—	—	—
Winter 1960-61 over winter 1959-60	39.18	81.05	—	—	—	—	—	—	162.4	100	1,261.16	3.26

[1] 24 storms each winter.

[2] Data obtained from Bradley Field Weather Station. It was assumed snowfall and average temperature ratios were same in test section and at Bradley Field.

1. The calcium chloride-salt mixture works faster at all temperatures. At 20 to 25 F the mixture starts to work in 5 to 10 min. Straight salt takes 30 to 45 min.
2. The mixture works at lower temperatures than salt.
3. The mixture keeps roads safer during storms.
4. The use of the mixture reduces the amount of sanding and plowing.
5. The mixture is more expensive than straight salt.
6. Mixing and handling costs are greater. Of course, this applies to the salt-sand mixture also.
7. An additional storage shed is required at each storage area.
8. The use of the mixture initially requires closer supervision.
9. Dampness in the spreader motors presents a greater problem when the mixture is used.
10. Depth of snowfall and temperature are only factors in the cost of snow and ice control.
11. Substantial reduction can be achieved in the amount of abrasives used.

RECOMMENDATIONS

Despite the additional cost, the use of the calcium chloride-salt mixture appears to be justified. This cost differential may have been due to lack of experience in the use of the mixture and more severe storm conditions in the test section. However, the superior performance of the mixture at all temperatures has been effectively demonstrated.

A test and control section should be established in each of the four maintenance districts for the winter of 1961-62. These sections should be in substantially the same condition at the start of the test in order to compare cleanup costs.

Construction of separate sheds for calcium chloride and salt would protect both materials from becoming wet. Protection of the ignition systems of spreader motors can be effectively maintained by adopting a procedure followed by the New York Thruway Authority. Equipment is washed with hot water after storm use. Then ignition wires and spark plugs are sprayed with a silicone lubricant preservative.

Based on the results obtained on the test section during 1960-61 the Connecticut State Highway Department will expand the test to include two test and two control sections in each of the four districts during the winter of 1961-62.

ACKNOWLEDGMENTS

The author is particularly grateful to John O. Wilson, Engineer of Research and Development, for help and guidance in conducting the tests and organizing this report; to William J. O'Brien of the Calcium Chloride Institute for suggestions and photographs; to the Division of Highways of the Massachusetts Department of Public Works for its assistance; to all members of the Connecticut Highway Department who participated in the test and especially Munroe Usher and William Busse for cooperation and support; and to Joseph J. Levin for help in revising and editing this paper.

APPENDIX

OBSERVATIONS ON USE OF SALT AND CALCIUM CHLORIDE DURING THE WINTER OF 1960-61

WILLIAM BUSSE, *Foreman, Connecticut Highway Department.*—Use of the salt and calcium chloride mixture on the road is advantageous if used correctly at the right time, in this writer's opinion.

Last winter, mistakes were made in applying it, for example, it was applied when it was snowing very hard and, even though some action resulted, this is not advisable, as the road cannot be kept bare. On the other hand, regardless of the amount of snow coming down it is advisable to get one coat of the mixture on the entire road immediately, as it does help in the final cleanup. It was best also to touch up the hills at intervals during the storm, this cut down tremendously on the amount of sand required.

After the big storms of Dec. 12, 1960, and Jan. 20, 1961, when the road was completely plowed off, a mixture of the chemicals would bare the road completely in 2 to 3 hr even with the temperature around zero.

Storage of chemicals is, or was, a major concern. It must be stored properly and under cover if possible. Too much should not be mixed in advance—just enough to cover the entire road. Material that is stored out-of-doors should be well covered with dry sand and a cover of waterproof paper. The salt used in the mixture should also be kept dry as there was considerable trouble with this until a salt shed was acquired. If salt or calcium chloride tends to get lumpy an even distribution of the chemicals cannot be attained as they will not feed evenly through the jets; this also would call for a second application where one should have done the job properly.

The mixing of the chemicals should be done with care. Improperly mixed materials will result in rapid action when the calcium chloride is concentrated and slow action where there is more salt. Correct blending takes a little longer, but it pays off. Because no two storms are alike, mixing too much in advance is expensive. A mixture suitable for one condition might not be right for the next storm. Remixing for proper proportioning or making up a new mixture wastes both time and money.

Considerable difficulty with spreader motors resulted from the use of the mixture; they were continually getting wet. A motor shut off for a period of time would have to be dried out before it would start, even though all motors were properly cared for before and after the storm.

The center strip has a much better appearance this year, and there is less sand on shoulders because of the use of the mixture.

There were no tie-ups on the hills, because the mixture was used, sparingly, and plows kept the snow closely plowed down.

No special equipment had to be used to remove hard packed snow. At one time a grader could have been used, but before it was available, an application of the mixture and plowing did the job.

Having used clear salt in previous years and having used a calcium chloride-salt mixture this year, there is no comparison if the mixture is correctly used. It will give quick action, where clear salt will not, and it will make the road safe for travel much sooner.

MUNROE USHER, *General Foreman, Connecticut Highway Department.*—The Chemical Test Section begins at the junction of US 44 in Tolland and extends northeasterly to the Massachusetts State Line, a distance of 14.0 mi of divided highway. A mixture of calcium chloride with salt was used for snow and ice control, and abrasives (sand) were applied when the mixture was not available, or heavy snowfall and cold temperatures were not favorable for the use of chemicals.

The effectiveness of this mixture (about 3 parts salt to 1 part calcium chloride by weight, or 2 parts salt to 1 part calcium chloride by volume) was far superior to straight salt in all cases when the temperature was below 28 F. Above this temperature the mixture was more rapid in action but the salt acted so quickly the traffic experienced no difficulty with either one. The mixture was applied at the rate of 800 to a 1,000 lb per mi (one side of the highway), using three jet spreaders which would cover the entire section without reloading. With this equipment the entire section could be treated in about 30 min, and at temperatures of 20 to 25 F the pavement would be completely bare in about 30 min. If snow or ice cover was very thin in spots, the pavement would be bare of snow or ice in 5 or 10 min. If snow or ice cover was heavy and temperature lower, an additional application was necessary in a few cases. With this quick and positive action no sanding was necessary.

Obtaining a bare pavement in this experimental section was very important as the route is heavily used by trailers and is generally on a grade; and the section between Conn. 32 and Conn. 89 is subject to about a 500 ft change in elevation in a distance of 7.7 mi, so that eastbound trailers are slowed down to the extent that anything except bare pavement can cause a poorly loaded vehicle to stall and cause a tie-up.

Before an application, it is important to have the road well plowed, as the calcium chloride apparently triggers the salt into instant action and the effectiveness is expended quickly, whereas salt will lie in the snow inactive until favorable moisture conditions prevail.

Several times during the last winter trucks were borrowed from this section for use on other roads. This was not possible before, as with any snow or ice on the pavement the heavy trailer traffic was so critical, that sanding operations were constant and all equipment was needed.

The amount of sand used on the experimental section was about one-fourth that used during the previous winter. Sand was used during cold, hard snow storms when traffic was generally having little difficulty because of the dryness and consistency of the snow.

After one severe and very cold snow storm, on both the experimental section and the control section, each had a layer of cold, hard-packed snow on most grades, where the slow moving truck traffic had packed the snow in spite of the temperature of around 10 F. The mixture was applied on the experimental section and salt on a few test sections within the control section. The mixture worked to the extent that within a few hours the pavement was plowed bare. Where the salt was applied it lay inactive on the packed snow until traffic whipped it off onto the shoulders. Graders were used on the control section until it was bare, about 12 hr later than the experimental section. During this comparison the temperature was about 3 F lower on the test section.

The mixture was prepared about one storm ahead of its use and generally at the 3:1 ratio by weight described previously, as experience proved this to be generally the most effective mix. A lower ratio of calcium, possibly 4:1, would work at temperatures above 27 F, but the saving in material costs would be offset by the extra work and the additional space required to store and handle the two mixes. Further, the hilly and northern parts of the State are usually too cold for the lighter mix, except perhaps, at the beginning and end of the snow and ice season.

The control section used for comparison was a section of the same route, commencing at the junction of Tolland Turnpike, in Manchester, and extending northeasterly to the beginning of the experimental section at the junction of US 44 in Tolland.

Comparison of the snow and ice removal costs between the two sections, can be of only limited value for the following reasons: Temperature differences of 2 to 4 degrees colder on the Union section frequently necessitated longer and more expensive operations. During each of the past three winters there were five instances when partial or full operations for snow and ice work were required on the experimental section, when no work, or merely skeleton forces, were needed on the control section.

Greater care of the rear-end motors must be exercised when spreading, or after using the calcium chloride mixture. Apparently the fine dust given off by the calcium chloride will penetrate all motors and ignition parts and as soon as moisture is available, it is absorbed and causes ignition shorts. A thorough washing and drying, and a more frequent starting of the rear-end motor during periods of non-use practically eliminated this trouble.

The extra difficulties caused by the calcium chloride are so minor and the improvement in pavement conditions so great that giving up the use of the mix is not recommended even if the savings in spring cleanup operations were not considered.

DEPARTMENT OF TRAFFIC AND OPERATIONS

Street Travel as Related to Local Parking

MATTHEW J. HUBER,
Bureau of Highway Traffic, Yale University

Some percentage of travel on local urban streets is related solely to the time and distance expended in searching for a parking space. This paper attempts to measure the amount and characteristics of such "search" travel. Field interviews were conducted in New Haven and Waterbury, Conn. Each interview was conducted at the parking site (curb or off-street) as soon as the driver parked. The driver was asked to retrace the route he followed in going from his last origin to his parked destination. The usual information on sex of driver, trip destination, and trip purpose was also obtained. Comparisons of search patterns as related to type of parking (curb vs off-street), sex of driver, trip purpose and origin, are included. The influence of traffic volume and availability of spaces on search patterns are also noted. Finally, data that may aid in simulation of downtown traffic on electronic computers are presented.

• THIS STUDY sought to determine and measure the characteristics of search patterns by drivers seeking parking spaces at curbs or off-street facilities in the central business district (CBD). The relationship of terminal to expressway, the influence of demand and location on search patterns, and the magnitude of travel on streets by vehicles searching for parking were some of the items for which answers were sought.

STUDY METHOD

Locations Studied

The studies were made in the cities of New Haven and Waterbury, Conn., where all curb spaces were metered with limits ranging from 15 min. to 1 hr.

Off-street facilities in New Haven included two self-park municipal lots, a pigeonhole garage and a privately operated attendant parking lot. Waterbury off-street facilities included two self-park municipal lots and one privately operated attendant lot.

The curb spaces in both cities were those with the greatest turnover in the core of the CBD. The off-street facilities are located throughout the core area on various approaches to the two "downtown areas."

Waterbury had a population of 107,130 and New Haven a population

TABLE 1
SURVEY DATES AND TIMES

City	Day	Date	Time
New Haven	Thursday	July 7, 1960	1 PM—9 PM
	Friday	July 8, 1960	8 AM—5 PM
	Wednesday	Nov 2, 1960	9 AM—5 PM
	Thursday	Nov 3, 1960	8 AM—5 PM
	Monday	Dec 19, 1960	9 AM—5 PM
	Tuesday	Dec 20, 1960	9 AM—5 PM
Waterbury	Friday	July 15, 1960	9 AM—5 PM
	Thursday	July 21, 1960	12 NOON—8 PM
	Wednesday	Nov 9, 1960	9 AM—5 PM
	Friday	Nov 11, 1960	9 AM—5 PM
	Thursday	Dec 22, 1960	9 AM—5 PM
	Friday	Dec 23, 1960	8 AM—4 PM

of 152,048 in 1960. The metropolitan area populations were 141,626 and 278,794, respectively.

Date and Time of Study

The dates and hours of the parking study are given in Table 1. Three seasons—the relatively quiet summer period, an average fall season, and the pre-Christmas rush—are included. During the summer, night shopping hours were surveyed; during the remaining periods, only daylight hours were included.

Sex of driver M F Date ______________ Time ________________

1. From where did you start this trip ______________________________

2. Did you make any stops before coming here? NO YES, last stop: ________

3. On what street did you come into the downtown area?
(Show map to driver)

4. a) CURB Where did you begin looking for a place to park?
What streets did you follow while looking for a space?

b) OFF-STREET Did you look for a place to park before coming to this lot? YES NO

if YES (Where did you begin looking for a space?
(What streets did you follow while looking for a space?

5. Before you came downtown, did you have a place in mind where you thought you could find a place to park? NO if YES: ______________________

6. Where are you making your first stop now? ______________________

7. For what purposes did you make this trip?
Work Personal Business Shopping Sales & Service Other __________

REMARKS:

Figure 1. Questionnaire form.

Collection of Data

Information on parking search characteristics was obtained by interviewing the drivers as soon as they had parked. Data were collected for each driver on a questionnaire (Fig. 1).

A map of the CBD, with approaches, was included with each interview form. Information on the search pattern was obtained by asking drivers to trace their route to the actual parking space. Some help was required by drivers in orienting themselves on the map, but by knowing the interview location and the origin of this trip the interviewer was able to assist the driver in tracing his route. Most drivers were able to recall landmarks or particular intersections that served as a check on the route followed. Also, proper use of one-way streets was a further check on drivers' routes.

Questions 1 and 2 were intended to eliminate any confusion and ambiguity in defining the last immediate origin. Questions 3 and 4 required information on the routes followed.

Drivers identified the point where search began as "in this block" or "at the corner of Elm" or by some particular building. Of course there is no certainty that drivers would have accepted a parking space had there been one at the indicated start of search. This is a source of error in interpretation of searching distance. The questions on search patterns were asked only of those off-street parkers who indicated they searched for curb space before entering the lot or garage.

Questions 6 and 7 were specifically related to trip purpose, and to get walking distance.

Definition of Terms

The following terms are used in this study:

Walking distance.—The distance, in feet, from the location at which the vehicle is parked to the driver's destination.

Search-walk distance.—The distance, in feet, from the point at which a driver begins to look for a parking space to the driver's destination.

Search distance.—The distance, in feet, along the route the driver travels between the point he begins to search and the point where he parks.

Total travel distance.—The distance, in feet, along the route the driver travels between the point the driver enters the study area and the point where he parks.

PARKING CHARACTERISTICS

Trip Purpose Distribution

The distribution of trip purpose by type of parking is given in Table 2 for New Haven and Waterbury and

TABLE 2
TRIP PURPOSE BY PARKING TYPE

Purpose	New Haven				Waterbury			
	Curb		Lot		Curb		Lot	
	No.	%	No.	%	No.	%	No.	%
Work	127	9	152	10	144	7	70	5
Personal business	664	48	375	25	596	29	252	19
Shopping	388	28	852	57	1036	51	901	67
Sales and service	126	9	61	4	102	5	28	2
All others	91	6	65	4	153	8	102	7
Total	1396	100	1505	100	2031	100	1353	100

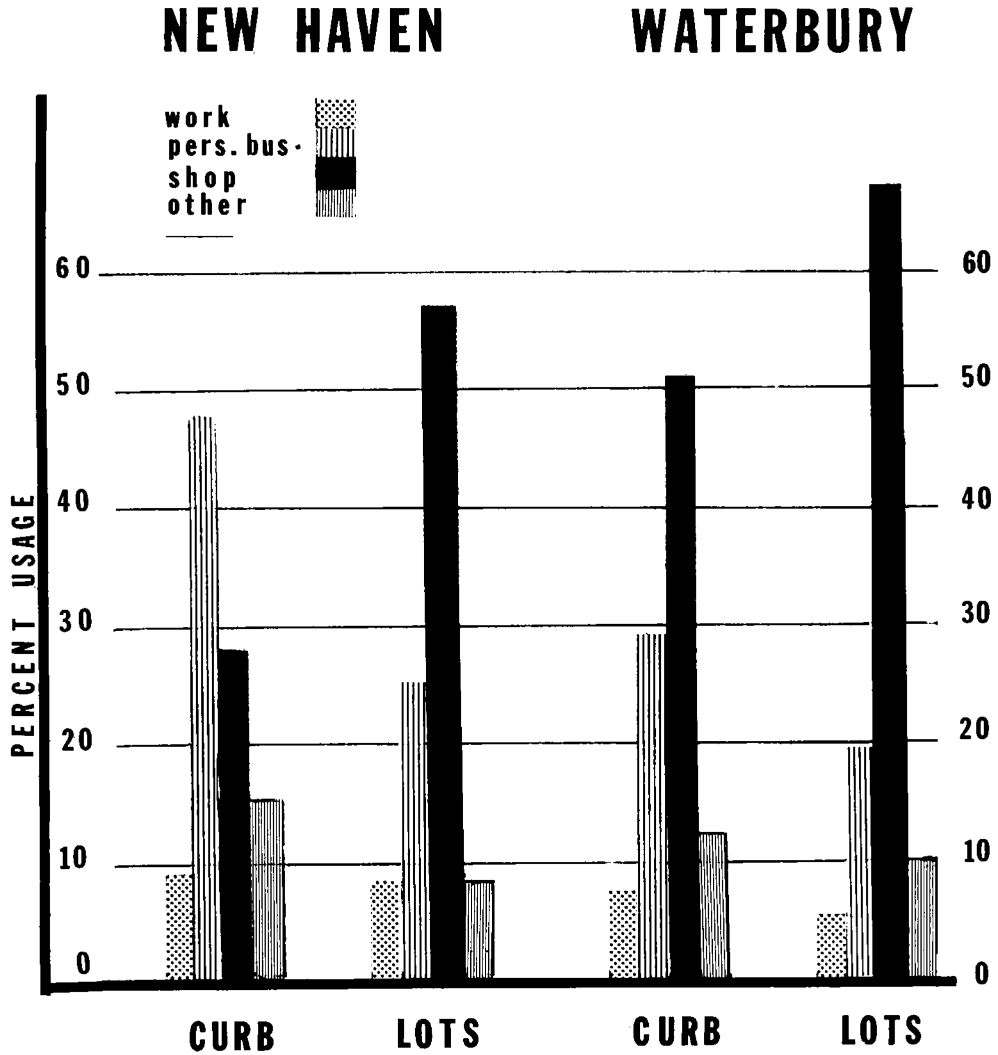

Figure 2. Trip purpose by location.

summarized in Figure 2. Shopping was the predominant trip purpose at off-street facilities in both cities and at curb sites in Waterbury. The principal trip purpose for curb parkers in New Haven was personal business. In both cities, on all dates, there was a greater percentage of shopping trips observed at off-street locations than at curb locations.

In order, the trip purposes were shopping, personal business, work, and sales and service. No other purposes were of great significance.

The greater use of lots by women is shown in Table 3 and Figure 3. Approximately one-fourth of all parkers interviewed at curbs were women; the percentage of women parkers was over 50 percent in lots.

TABLE 3
USE OF LOTS AND CURBS, MEN AND WOMEN

City	Date	Curb				Off-Street			
		Men		Women		Men		Women	
		No.	%	No.	%	No.	%	No.	%
New Haven	July 7	181	76	57	24	54	49	56	51
	July 8	193	74	68	26	162	46	187	54
	Nov 2	158	76	47	24	85	46	100	54
	Nov 3	150	69	67	31	104	38	169	62
	Dec 19	147	79	40	21	132	49	140	51
	Dec 20	173	75	59	25	107	40	159	60
	All	1,002	75	338	25	644	44	811	56
Waterbury	July 15	315	78	87	22	46	43	60	57
	July 21	250	74	86	26	77	37	131	63
	Nov 9	127	72	49	28	83	37	144	63
	Nov 11	185	63	110	37	40	29	99	71
	Dec 22	222	73	81	27	98	38	157	62
	Dec 23	266	77	80	23	192	54	162	46
	All	1,365	73	493	27	536	42	753	58

TABLE 4
TRIP PURPOSE BY SEX OF DRIVER [1]

Purpose	New Haven				Waterbury			
	Men		Women		Men		Women	
	No.	%	No.	%	No.	%	No.	%
Work	211	13	54	5	180	9	26	2
Personal business	723	44	269	24	621	32	175	14
Shopping	445	27	759	66	855	45	936	76
Sales and service	170	10	10	1	105	6	16	1
All others	93	6	51	4	155	8	86	7
Total	1642	100	1143	100	1916	100	1239	100

[1] Totals all dates.

Trip Purpose vs Sex of Driver

Trip purpose as related to sex of driver in the two cities is summarized in Table 4 and Figures 4 and 5. (Minor variations in total interviews were due to incomplete recording on the part of the interviewers, recording all but one item on the interview sheet. These incomplete interviews were retained in tabulations where the remainder of the data was valid.)

The principal trip purpose for women in both cities was shopping; the next category of importance was personal business. Shopping was also the most important trip purpose for men in Waterbury but in New Haven the primary trip purpose was personal business. Work, sales, and service trips and all other trips were about one-fourth of all trips made by men in both cities.

Interviews were made on July 7 in New Haven and July 21 in Waterbury on shopping nights and influence the percentage of shoppers observed on the two dates. The high percentage of shoppers in both cities in December was a reflection of the pre-Christmas rush.

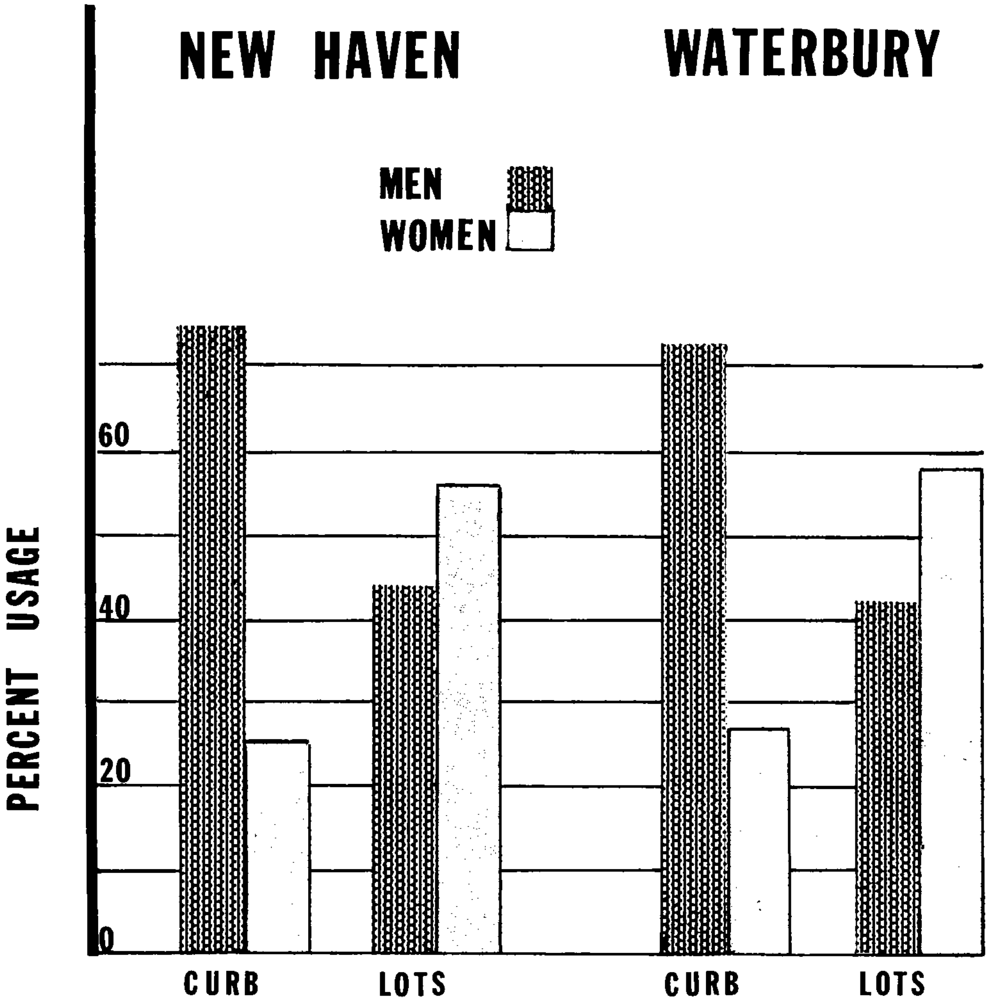

Figure 3. Curb-lot use by men and women.

Walking Distance

Walking distances, by sex and by location parked, are given in Table 5 and summarized in Figure 6.

Off-street walking distances were substantially greater than curb walking distances, varying from 212 ft for men in New Haven to 393 ft for men in Waterbury.

TABLE 5
WALKING DISTANCES

City	Parking	Walking Distance (ft) Men	Women	Grand Avg.
New Haven	Curb	389.7	343.4	
	Off-street	601.9	590.8	
				492.2
Waterbury	Curb	307.8	323.1	
	Off-street	701.7	684.0	
				474.2

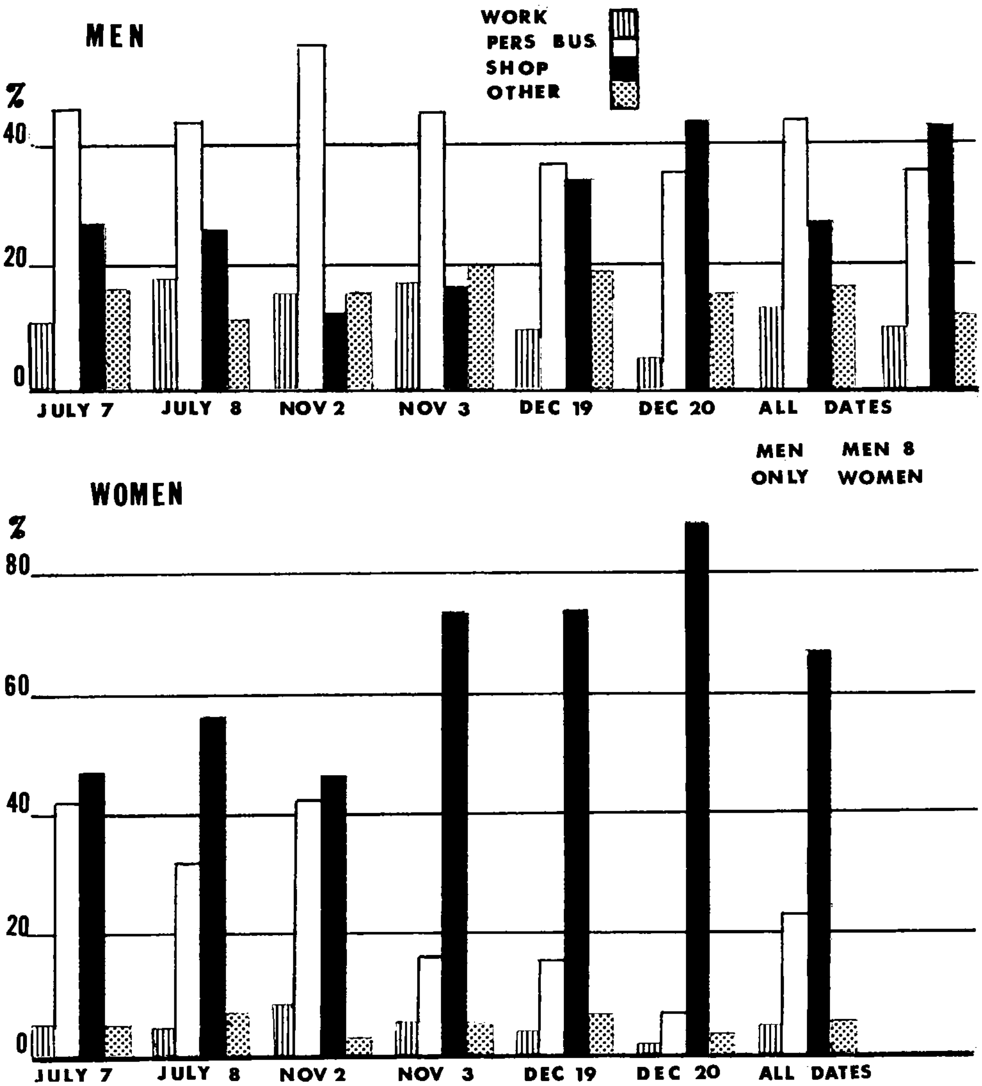

Figure 4. Trip purpose, New Haven.

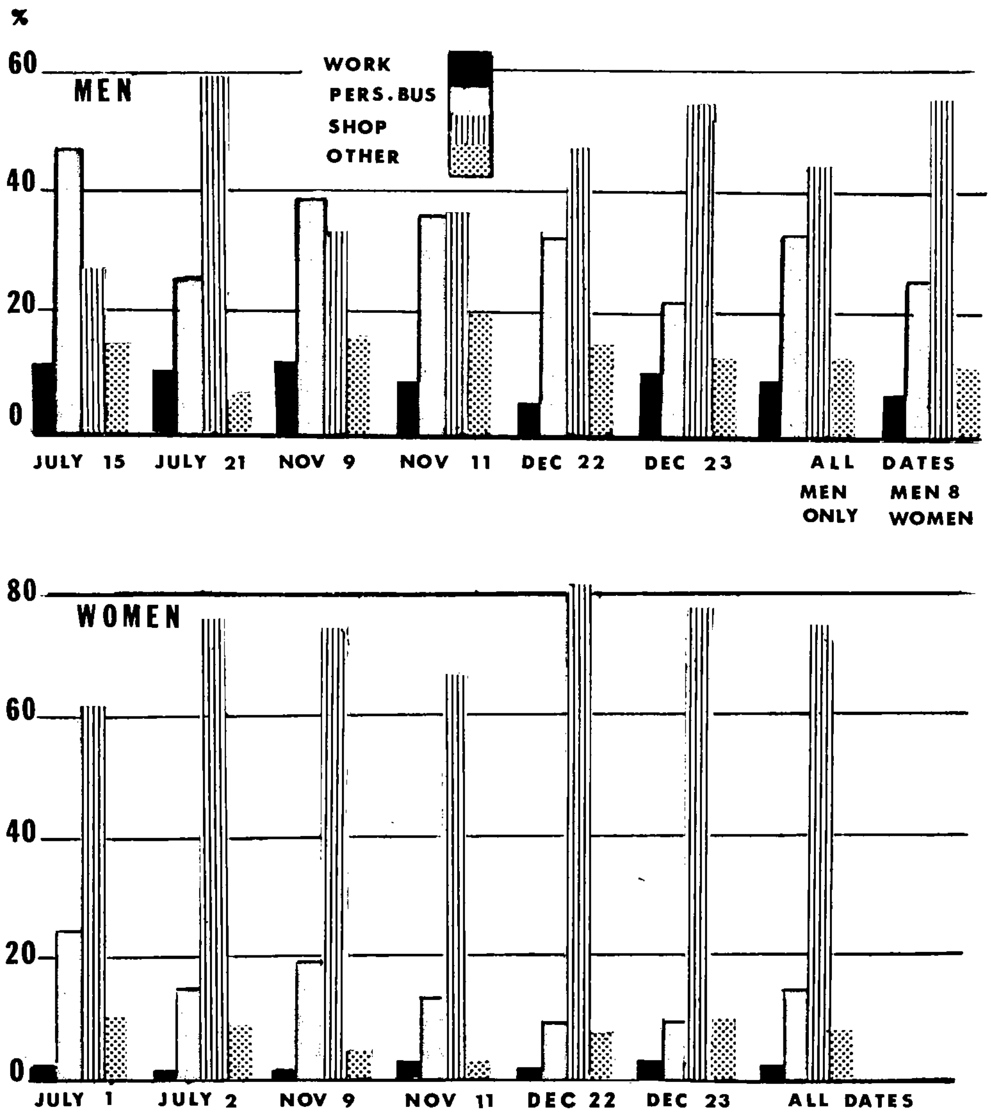

Figure 5. Trip purpose, Waterbury.

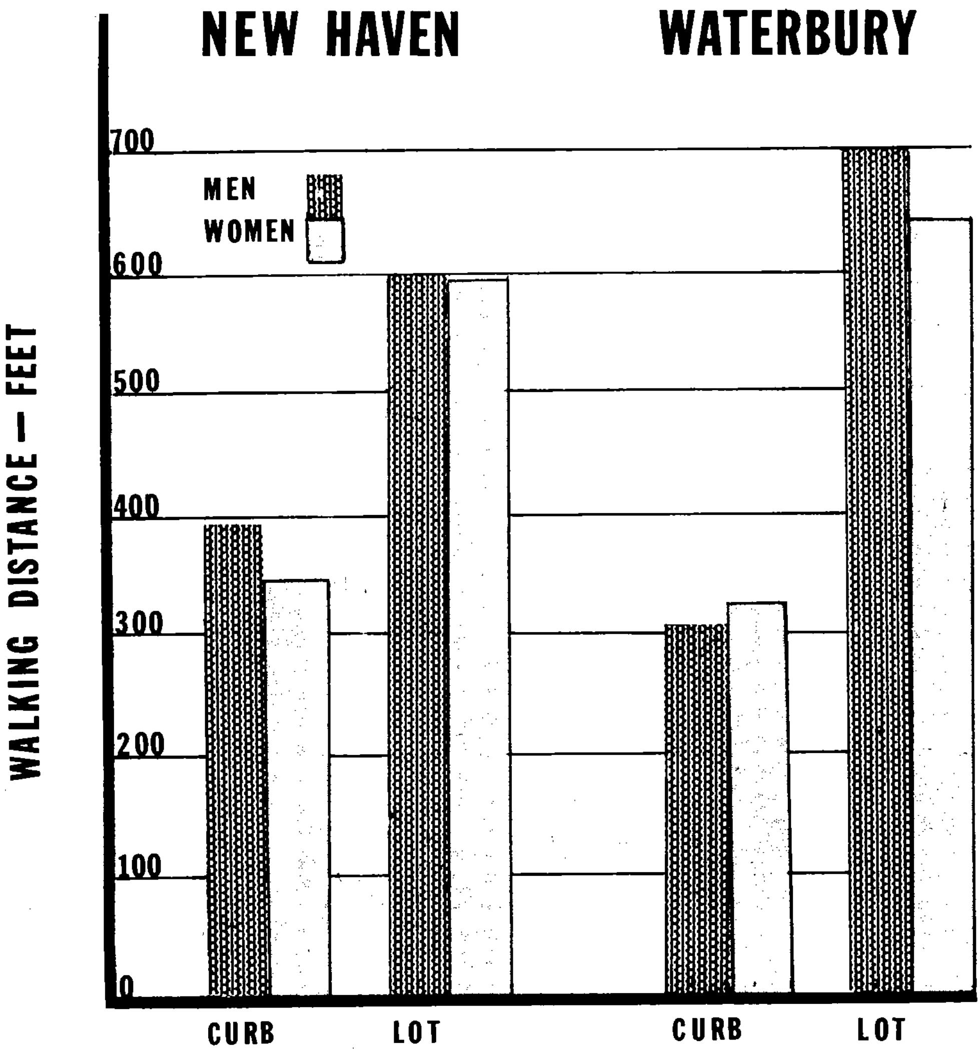

Figure 6. Walking distance.

Women, except at curbs in Waterbury, managed to park closer to their destination than men, but the differences are not statistically significant at the 0.05 level.

Finally, parkers at the curb in New Haven tended to have greater walking distance than curb parkers in Waterbury, though for off-street parkers New Haven walking distances were shorter. The shorter walking distances at lots in New Haven were a reflection of the large off-street facility very close to some of the prime generators.

Comparison to Previous Studies

Although no attempt was made to get a sample of all trip purposes and of all parking facilities in the two study areas, a comparison of the results with previously reported values may be of interest. Both communities had traffic and parking studies conducted by a firm of consulting traffic engineers during 1953. Table 6 gives the comparison for New Haven and Waterbury.

The difference in percentage of work trips between the two studies again reflected the type of interviews made for this project. Private lots or lots catering primarily to all-day parkers (workers mostly) were not included in this survey; therefore, more trips of the shopping and miscellaneous categories were observed.

Walking Distance

Average walking distances for the previous studies and the present study in the two cities were as follows:

	New Haven	Waterbury
Previous study	436 ft	471 ft
This study	492 ft	474 ft

There is no immediate explanation as to why the average walking distance in New Haven differed in the two studies or why they agreed so well for Waterbury.

SEARCH CHARACTERISTICS

Anticipation of Parking Location

One hypothesis of this study was that drivers who parked at curb locations were more likely to "take a chance" in finding a site at which to park. On the other hand it was reasoned that lot users tended to have a place in mind before starting and proceeded directly to that site. Furthermore, it was reasoned that persons with a particular curb site in mind were not as likely to be satisfied as those drivers who anticipated finding an off-street facility.

Drivers were asked, "Did you have a (parking) place in mind before you started this trip?" The results of the answers to this question are shown in Figures 7 and 8. Parkers using

TABLE 6
COMPARISON OF PRESENT AND PREVIOUS STUDIES OF TRIP PURPOSE

Purpose	Distribution (%)			
	New Haven		Waterbury	
	This Study	1953 Study	This Study	1953 Study
Work	9.6	30.4	6.3	16.7
Personal business	35.8	35.5	25.1	37.0
Shopping	42.7	30.4	57.2	27.5
All others	11.8	1.3	11.4	18.8

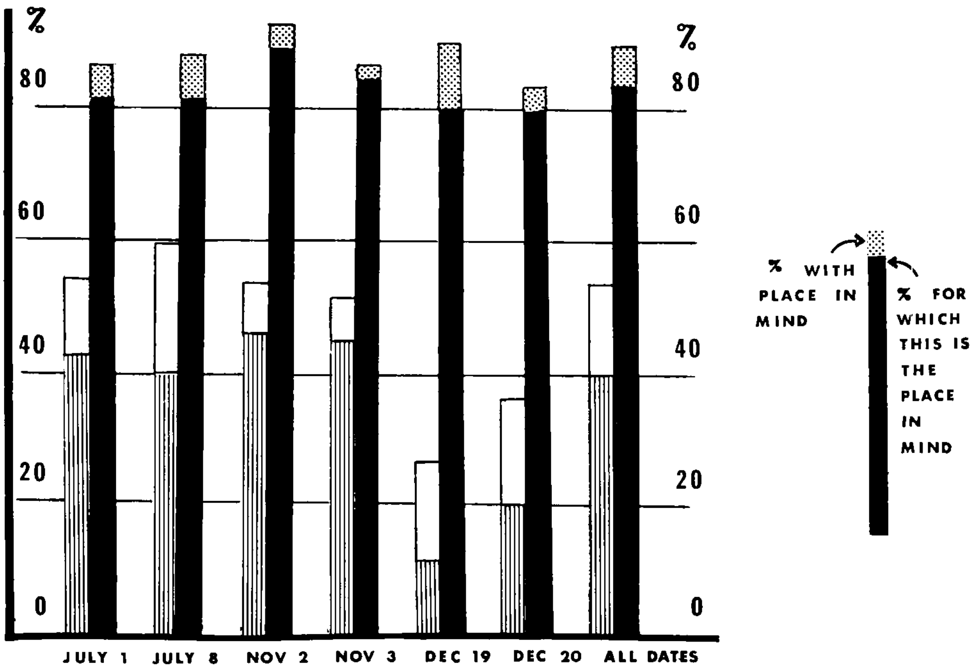

Figure 7. Place in mind, New Haven.

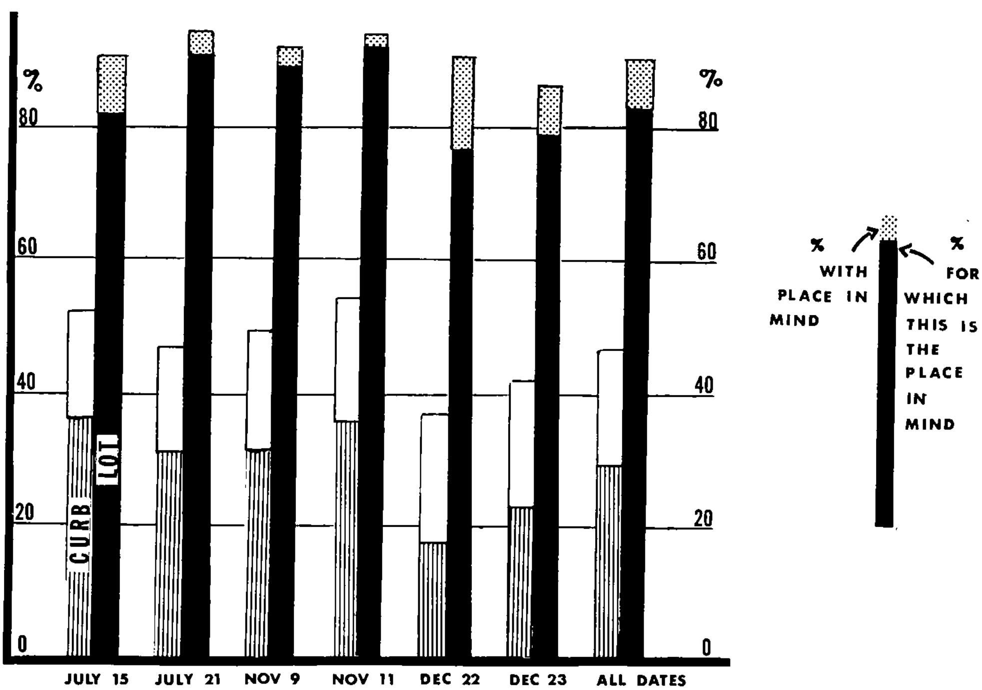

Figure 8. Place in mind, Waterbury.

lots consistently showed a high percentage with a place in mind as compared with those parking at curb sites.

For all dates in New Haven 89 percent of the lot parkers had a particular place in mind as compared to 53 percent of curb parkers. In Waterbury comparable percentages were 90 and 46 percent.

The percent of New Haven lot parkers answering Yes to this question varied from a low of 83 percent on December 20 to a high of 92 percent on November 2. Curb parkers answering Yes varied from a low of 26 percent on December 19 to a high of 59 percent on July 8.

The lowest percent of Waterbury lot parkers answering Yes was 85 percent during the pre-Christmas rush on December 23 and the highest was 94 percent on July 21 and November 11. For curb parkers the lowest percentage was 36 percent on December 22 and the highest was 54 percent on November 11.

In general, there was no great difference in the way men and women replied to these questions. Those parkers who replied Yes to the question about having a place in mind were then asked if the site of the interview was the site they had in mind.

Again, lot parkers outranked curb parkers, when measured in terms of percentage who were able to park at the particular site they had in mind. For all dates in New Haven 71 percent of curb parkers and 94 percent of lot parkers were able to park at sites of their own choosing; in Waterbury 61 percent of curb parkers and 92.5 percent of lot parkers were comparable figures.

Variations in percent of successful curb parkers ranged from a low of 43 percent on December 19 to a high of 86 percent on November 3 in New Haven and from a low of 48 percent on December 22 to a high of 68 percent on July 15 in Waterbury. For lot parkers the range for New Haven was a low of 89 percent on December 19 to a high of 98 percent on November 3. Waterbury percentages ranged from a low of 85 percent on December 22 to a high of 98 percent on November 11.

The results of answers to the two questions "Did you have a place in mind?" and "Is this it?" indicated that when parking demand is great, in this case the pre-Christmas rush, fewer drivers had a pre-conceived idea of where to park and of those who had a site in mind fewer were successful. This was especially true for those parking at the curb.

Persons parking at the curb appeared to be ready, in about one-half of all instances, to accept whatever sites were available along a general route, without a specific block face in mind, and of those that did have a site in mind about one-third finally located at some other curb site.

About 85 to 90 percent of lot parkers, on the other hand, had a specific lot in mind and proceeded directly to that lot. In over 90 percent of the observations they were successful in this endeavor.

Relationship Between Parking Site and Destination

Curb parkers were asked to designate the point at which they began to look for a place to park. This location was then related to the final destination of the driver. The location where the driver parked was also related to the final destination. Drivers who passed their destination before searching or parking were classified as "Yes, did pass." This category included those drivers who were unable to park at their destination because of parking restrictions. Persons who began to search or who parked within 200 ft of their destination were classified as parking "at" the destination. Finally, those drivers whose routes did not pass their destination or began their search or

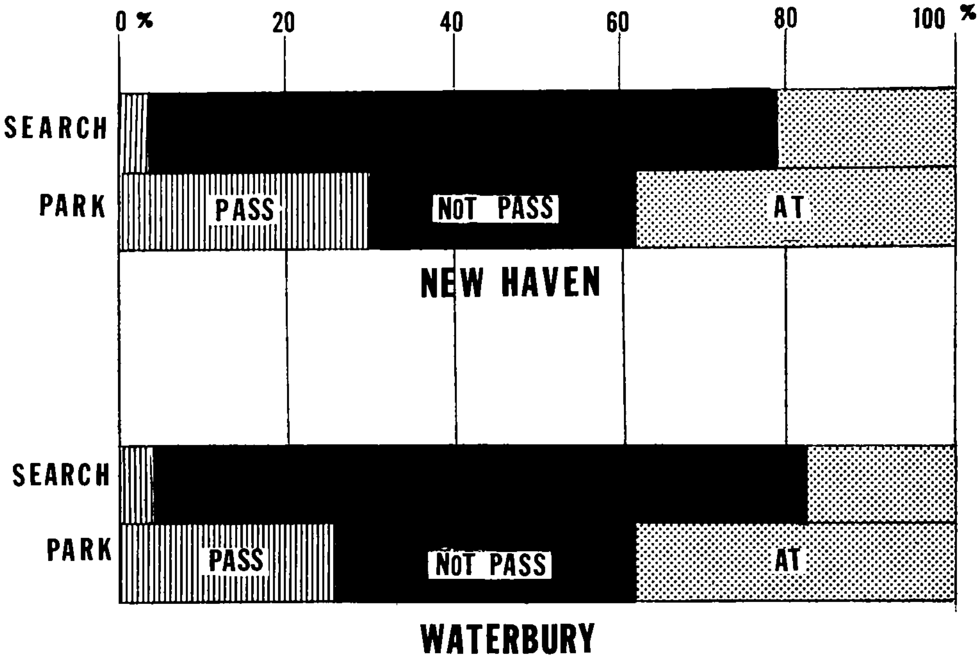

Figure 9. Destination and search-park.

parked before reaching their destination were classified as "No, did not pass."

Results are shown in Figure 9. As might be expected, few drivers passed their destination before starting to look for a space. For all dates 4 percent of drivers in Waterbury and New Haven indicated they did pass their destination before beginning to search. Eighteen percent of drivers in Waterbury and 21 percent in New Haven began to search for a space within 200 ft of their destination. The greater numbers of parkers—78 percent in Waterbury, 75 percent in New Haven—began to look for a space before reaching their destination.

The relationship between the parking site actually used by the driver and the destination are also shown in Figure 9. Thirty percent of all drivers in both cities passed their destination before parking, 38 percent parked within 200 ft of their destination, and the remaining 32 percent did not pass their destination before parking.

No readily discernible pattern was seen in the day-to-day differences in destination as related to start of search or parking site. Even during the Christmas-rush period, over 30 percent of the drivers were able to park within 200 ft of their destination.

Search Distance

The distance between the point where the driver began to search and where he parked is given as the search distance in Table 7 and shown in Figure 10. The search distance for off-street parkers applied only to those drivers who looked for a space at the curb before entering the lot.

TABLE 7
SEARCH DISTANCES

City	Date	Men: Curb Parked No.	Men: Curb Parked Avg. Distance (ft)	Men: Off-Street Parked No.	Men: Off-Street Parked Avg. Distance (ft)	Women: Curb Parked No.	Women: Curb Parked Avg. Distance (ft)	Women: Off-Street Parked No.	Women: Off-Street Parked Avg. Distance (ft)
New Haven	July 7	177	1,181.5	10	1,980.0	57	1,050.6	8	4,008.7
	July 8	192	856.7	24	1,877.1	67	910.7	28	1,814.3
	Nov 2	156	1,194.4	15	1,123.3	47	1,159.1	15	1,620.0
	Nov 3	149	699.6	5	2,234.0	67	759.9	5	3,052.0
	Dec 19	143	1,132.4	20	2,500.5	40	1,644.5	37	3,202.2
	Dec 20	172	1,010.2	33	2,264.5	59	1,499.8	27	3,000.0
	All	989	1,010.9	107	2,033.7	337	1,129.2	120	2,682.6
Waterbury	July 15	303	1,038.2	17	1,447.1	83	1,078.2	12	1,565.8
	July 21	242	840.0	9	1,914.4	84	897.7	5	2,146.0
	Nov 9	140	477.5	14	1,293.1	49	675.5	11	976.4
	Nov 11	181	758.3	7	1,972.0	107	817.8	29	2,055.9
	Dec 22	220	1,154.0	38	1,535.5	80	1,379.2	41	1,503.7
	Dec 23	263	947.8	59	1,702.5	80	1,044.1	26	1,465.4
	All	1,349	908.2	144	1,614.8	483	992.5	124	1,609.9

Those drivers who searched at curbs before parking in lots tended to have the longest search distance. For the most part, these represented people who drove around the streets looking for a parking space without success and then entered lots. Average search distances for men at lots in New Haven exceeded search distances for men at curbs by 1,010 ft, for women the excess search distance by lot parkers was 1,553 ft.

In Waterbury the search distance for men at lots exceeded search distance at curb sites by 706 ft and for women the excess search distance by lot parkers was 617 ft. Again, data on search distances at lots apply only to those relatively few parkers who entered the lots after attempting to find a curb space without success.

Women, in most instances, searched a greater distance than men, but the differences are not significant for the sample size measured.

Day-to-day differences in search distances were rather large when related to the mean search distance because of several factors. First, some evening shopping hours were included on one day of the July studies in both cities and had some influence on the results for those days. Second, the same sites were not checked on each day in each month, although in effect there were two sets of study sites in each city and these were studied during three periods—July, November, and December. Finally, such items as the increased shopping activity at Christmastime had an influence on results.

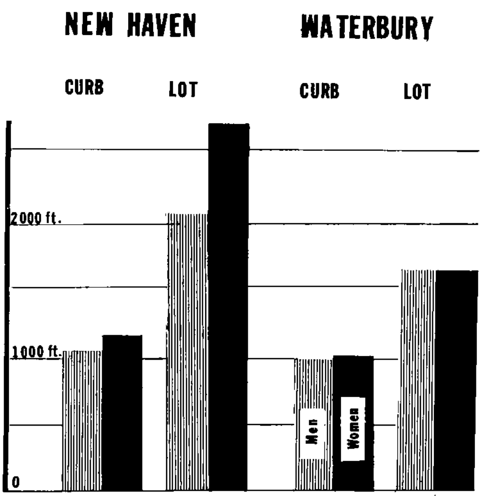

Figure 10. Search distance.

Walking Distance vs Search-Walk Distance

Table 8 compares walking distance and search-walk distance.

Drivers indicated a willingness to walk about 150 to 200 ft further than they acutally did. Search-walk distances ranged from 455 to 631 ft in New Haven and from 407 to 569 ft in Waterbury.

If the days are ordered by magnitude of variable, the maximum walking distance and search-walk distance are not coincidental. For example, the greatest mean walking distance in New Haven occurred on December 20 and the longest search-walk distance occurred on December 19. For Waterbury the longest walking distance occurred November 11 and the longest search-walk distance was observed on December 23.

TABLE 8

WALKING DISTANCE VS SEARCH-WALK DISTANCE (CURB PARKERS)

City	Date		Walking Distance (ft)	Search-Walk Distance (ft)
New Haven	July	7	371.2	550.8
	July	8	292.4	503.4
	Nov	2	312.8	454.7
	Nov	3	316.4	494.2
	Dec	19	404.9	630.6
	Dec	20	567.7	502.8
Waterbury	July	15	299.3	550.3
	July	21	278.6	462.1
	Nov	9	262.5	406.5
	Nov	11	357.1	464.8
	Dec	22	306.0	496.9
	Dec	23	346.2	568.6

Apparently days when drivers were unable to park close to their destination did not induce the individual driver to begin his searching at a greater distance from the destination than might otherwise be the case.

Walk Distance, Search-Walk Distance, and Search Distance

It was anticipated that those factors which influence walking distance would also influence search-walk distance and search distance. Analysis of the 6 days in the two cities for these relationships were not successful. The maximum walking distance did not necessarily occur on the same date as the maximum search distance or maximum search-walk distance. The same variables were grouped by the three months, rather than the 6 days, but there was not a consistent trend. Further analysis, with particular attention to locations, will be required to detect any common denominator for these variables.

Relationships to Volume and Parking Accumulation

Another assumption was that as vehicular volumes increased on the streets or as parking accumulation increased there would be an associated increase in walking distance and search distance.

Hourly volume counts were made at certain key locations in and around the study areas of the two cities. Parking occupancy counts on selected streets and in selected lots were made at 1-hr intervals. These data formed a basis on which checks could be made.

The search characteristics, walking distance, search-walk distance, and search distance were also calculated for each hour of the day. Hourly vehicular volumes and parking accumulations were then compared with the aforementioned search characteristics. The results to date have shown no consistent relationship. It does not follow that peak volume or parking accumulation occurs during the same time interval as maximum walking distance or maximum search distance. As for days or months, the search characteristics are not related to each other by hours of the day; the maximum walking distance does not occur at the same hour as the maximum search distance or maximum search-walk distance.

It is not surprising that vehicular volumes are unrelated to search characteristics. Relatively few parkers were located at the study sites during peak volume hours. Work trips contribute much to peak hour volumes and such trips were not a target of this study.

The poor correlation between parking accumulation and search characteristics was not anticipated. Perhaps the search characteristics observed at 10:00 AM, for instance, should be related to parking accumulation at 9:30 AM, because the space occupied at that time may have had some influence on the driver's search. This point is being explored.

Total Street Usage

The final measure of street usage was the total distance parkers drove on city streets within the limits of the study area, including searching distance. Each trip was traced from the point it entered the cordon to the location of the parking site.

This section of the analysis is incomplete because proper computer facilities were not available, but data have been calculated for 3 days—2 days in New Haven and 1 day in Waterbury. Results are given in Table 9 and Figure 11.

On all 3 days, the over-all average total travel distance for off-street facilities was slightly less than the average travel distance at the curb. The difference was most pronounced in Waterbury, particularly because of the short travel distance to the private lot. Almost all users went directly to this lot along the main approach street to the town, and did

TABLE 9

TOTAL TRAVEL DISTANCE

City	Date	Parking	Men No.	Men Avg. Distance (ft)	Women No.	Women Avg. Distance (ft)	All No.	All Avg. Distance (ft)
New Haven	Nov 3	Off-street:						
		Private lot	33	3,465	54	3,362	87	3,401
		Municipal lot	63	3,142	105	3,268	168	3,221
		Total	96	3,253	159	3,300	255	3,282
		Curb:						
		Orange St.	83	3,317	31	3,334	114	3,353
		Chapel St.	26	3,419	26	3,847	52	3,633
		Church St.	38	3,282	12	2,791	50	3,244
		Total	147	3,353	69	3,378	216	3,379
	Dec 20	Off-street:						
		Private lot	36	4,283	63	4,170	99	4,211
		Municipal lot	68	4,016	102	3,634	170	3,787
		Total	104	4,109	165	3,839	269	3,937
		Curb:						
		Orange St.	107	4,128	30	4,492	137	4,208
		Chapel St.	30	3,832	20	4,689	50	4,175
		Church St.	35	3,940	8	5,416	43	4,215
		Total	172	4,038	58	4,688	230	4,202
Waterbury	Nov 11	Off-street:						
		Private lot	23	1,740	56	1,679	79	1,697
		Municipal lot	21	2,500	39	2,324	60	2,386
		Total	44	2,102	95	1,944	139	1,994
		Curb:						
		Bank St.	36	2,591	17	3,145	53	2,768
		S. Main St.	19	2,579	24	2,555	43	2,565
		Leavonworth St.	54	2,357	24	2,391	78	2,368
		Center St.	22	2,635	24	2,712	46	2,675
		W. Main St.	49	2,897	20	2,496	69	2,781
		Total	180	2,608	109	2,635	289	2,618

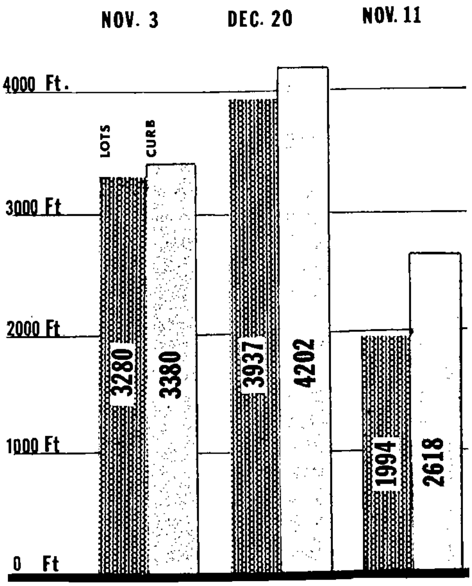

Figure 11. Total travel distance.

very little search driving before entering the lot.

The over-all differences—265 ft (December 20), 97 ft (November 3) and 624 ft (November 11)—represented the difference in street usage generated by off-street facilities vs curb facilities. The results suggest that off-street users generate less "street mileage" than curb users but further analysis is needed to prove if the difference is significant.

The results for New Haven were influenced by the location of the Oak Street Connector, an expressway leading from the Connecticut Turnpike to the CBD. Although the Connector is within the study cordon zone, travel distances were not included in this analysis because it is not a city street and vehicles cannot begin a parking search while on this expressway. Since users of the turnpike were more likely to use the municipal lot than other facilities, the exclusion of expressway travel distance had a greater impact on off-street-parker travel distances than on curb-parker travel distances.

The difference between the November and December data for New Haven is of interest. Off-street parkers drove an average of 655 ft further in December as compared to November. For curb parkers the difference was 823 ft, a definite indication that parkers had to do more driving during the Christmas shopping rush period.

Some idea of the total magnitude of travel induced by this extra travel may be assumed as follows:

A 1953 study showed 31,750 vehicles with destinations in the New Haven CBD during a 10-hr period in July. Assuming that 70 percent of these might have been making trips of the type made for this study, allowing for no growth between 1953 and 1960, and making no correction for December over July, it can be assumed that there are at least 22,000 trips to the CBD that must search for a space. Assuming their trips are increased by 655 ft, as developed in this study, this is equivalent to an additional 14,410,000 ft or 2,729 mi of travel on the city streets.

Dividing the 14,410,000 by 3,379 (the average travel distance in November), this becomes the equivalent of adding 4,264 vehicles to the assumed 22,000 vehicles on shopping, personal business, or work trips.

Parking Location and Trip Origin

One method advocated for relief of congestion in the CBD is to supply parking facilities at locations that "intercept" trips at the edge of the congested area, before vehicles contribute to the congestion. If such a system were to work, there would be no trips that pass through the downtown area. Instead, each parking facility would receive traffic exclusively from origins located in the same direction from the downtown area as the facility itself.

At the other extreme, it may be assumed that each parking site is equally attractive to all origins. In this instance the number of parkers from each origin at any parking site will be present in the same proportion as that origin is to all origins. For example, if 10 percent of all origins come from direction A and 15 percent come from direction B, then 10 and 15 percent of all the vehicles at a given facility might be expected from direction A or B.

Table 10 gives the origins and destinations of the study data in New Haven. The figures in parentheses represent the number of parkers expected from each origin if it is assumed that each site is equally attractive to all origins.

A χ^2 test was applied to the hypothesis of equal attractiveness and the hypothesis was rejected. As might be expected, certain origins are more likely to park at some destination than at others.

If the municipal lot located on Church Street in New Haven (Facility 9) is considered, all vehicles entering by way of the Oak Street Connector and Church Street (Origin 12-13) pass this facility when entering the downtown area. Most trips from the southeast side of the study area (Origin 14-18) are also funneled down Church Street and past the municipal lot. The result is that a disproportionate number of persons from these two directions parked in this lot. There were 208 destinations from the Oak Street Connector where 115.5 might be anticipated and 103 persons parking from the southeast as against an expected 76.

The most pronounced disparity on the negative side occurred for origins

TABLE 10

ORIGIN AND DESTINATION, NEW HAVEN [1]

Destination	Origin								
	1–2–3	4	5	6	7	8–11	12–13	14–18	19–21
1	(97.8) 135	(34.8) 25	(40.3) 70	(27.1) 23	(16.1) 19	(19.2) 12	(63.2) 35	(41.6) 31	(44.8) 35
2	(73.2) 102	(26.1) 21	(30.1) 62	(20.3) 22	(12.1) 16	(14.4) 15	(47.3) 18	(31.1) 14	(33.5) 18
3	(67.1) 53	(23.9) 16	(27.6) 8	(18.6) 12	(11.1) 8	(13.2) 25	(43.3) 49	(28.5) 52	(30.7) 41
4–5–6	(48.0) 42	(17.1) 12	(19.8) 17	(13.3) 14	(7.9) 11	(9.4) 15	(31.0) 37	(20.4) 28	(220.0) 13
7	(36.6) 57	(13.0) 11	(15.1) 20	(10.1) 10	(6.0) 2	(7.2) 16	(23.6) 18	(15.6) 4	(16.8) 6
8	(77.5) 70	(27.6) 33	(31.9) 28	(21.5) 60	(12.8) 34	(15.2) 11	(50.0) 28	(32.9) 24	(35.5) 19
9	(179.0) 117	(63.7) 78	(73.7) 24	(49.6) 25	(29.5) 12	(35.2) 19	(115.5) 208	(76.0) 103	(81.9) 118
10	(74.2) 76	(26.4) 22	(30.6) 50	(20.6) 19	(12.3) 11	(14.6) 19	(47.9) 38	(31.5) 26	(34.0) 31
11	(24.4) 29	(8.7) 8	(10.0) 4	(6.8) 5	(4.0) 0	(4.8) 1	(15.8) 9	(10.9) 6	(11.2) 34
12	(18.5) 16	(6.6) 22	(7.6) 4	(5.1) 3	(3.1) 2	(3.6) 4	(12.0) 10	(7.9) 8	(8.5) 4

[1] Figures in parentheses are expected number of trips if all destinations are equally attractive to all origins.

[2] *Destination Code:*

1—Curb parking, east side of Orange St. from Elm to Court Sts.; 2—Curb parking, east side of Orange St. from Court to Chapel Sts.; 3—Curb parking, west side of Church St. from Chapel to Court Sts.; 4–5–6—Curb parking, both sides of Church St. from Court to Elm Sts.; 7—Municipal lot, Elm St. between Orange and State Sts.; 8—Pigeon-hole municipal garage, State St. between Elm and Court Sts.; 9—Municipal lot, Church St. between Crown and Chapel Sts.; 10—Private lot, Orange St. between Chapel and Center Sts.; 11—Curb parking, south side of Chapel St. between College and Temple Sts.; 12—Curb parking, north side of Chapel St. between College and Temple Sts.

[3] *Origin Code:*

1–2–3—Broadway St., Ashmun St., and Prospect St.; 4—Temple St.; 5—Orange St.; 6—State St.; 7—Grand Ave.; 8–11—Chapel St., Court St., and Wooster St.; 12–13—Oak St. Connector (to Connecticut Turnpike); 14–18—Orange St., Congress St., and southwest New Haven area; 19–21—George St.

1-2-3 and 5. In the case of origin 5, a driver is on a direct route past curb sites and a private lot and must cross the CBD to arrive at destination 9. Though 73.7 destinations were expected there were only 24 observed at the subject lot. Persons from origin 1-2-3 do not pass any of the survey sites in taking the most direct route to destination 9, but they are required to follow a devious route (because of one-way streets and turn restrictions) that takes them across the CBD. By proportion, 179 trips were expected and 117 observed.

There were more trips than expected at destination 9 from origins 4, 12-13, 14-18, and 19-21. All but origin 4 are on the same side of the CBD as the facility. Origin 4 is on a one-way street that leads directly to lot 9. Those zones that had less than the expected parking volume were not on a direct route or were on the opposite side of the CBD from lot 9.

Parking locations 1 and 2 are curb sites on Orange Street, one of the important streets leading downtown. Persons entering the CBD on this street can be expected to pass these locations if they follow the most direct route. There were 132 drivers entering the area along Orange Street who parked along this same street as compared to 70.4 who were expected to park. Origin 1-2-3 contributed 237 parkers as against an expected 171. Drivers from this direction are required to make only one turn in order to get to a parking site and it is the most accessible facility of those covered in this study.

Greatest negative differences were contributed by origins 12-13 and 14-18. Drivers from these origins pass facilities 9, 3, 4, and 5 before arriving at destinations 1 and 2. Fifty-three drivers parked on Orange Street as against an expected 110.5 from the Oak Street Connector. The southeast area contributed 45 parkers against an expected 72.7. Origin 19-21 on the east side of the CBD also showed fewer than expected destinations (53 observed vs 78.3 expected).

Because of one-way streets and other restrictions destination 4-5-6 is not accessible without passing some other location and is not directly on any route. In this instance the assumption of equal attractiveness was borne out. The χ^2 test indicated that arrivals from all origins cannot be shown to be disproportionate. Only the private lot at location 10 approached the curb area 4-5-6 in being equally attractive to all parkers. The greatest source of disparity was the traffic entering along Orange Street directly on the route to the facility. From all other directions the arrivals were nearly in proportion to all origins from that direction.

All remaining facilities tended to be a "favorite" destination of one origin or another. The evidence of this analysis is that parking sites located on routes do tend to attract vehicles using that route and have a tendency to intercept trips bound for the CBD.

Those parking locations that are not on a direct route tend to attract vehicles from all directions and the influence of location is less pronounced.

CONCLUSIONS

1. Drivers who park at lots tended to have a particular location in mind when taking a trip to the CBD. Curb parkers were less inclined to look for a particular site, and more likely to "take a chance" in finding a parking location.

2. Parkers at lots were successful over 83 percent of the time in finding room at the facility of their choice. Curb parkers were successful only between 26 and 59 percent of the time.

3. The percent of drivers with a place in mind and the percent of drivers successful in their search

were at a minimum during the Christmas rush.

4. Approximately 75 percent of all curb parkers began to look for a space before passing their ultimate destination. Twenty percent began to seek a space within 200 ft of the ultimate destination and the remainder passed the destination before searching.

5. Before reaching their parking location 30 percent of the drivers passed the ultimate destination, 38 percent parked within 200 ft and 32 percent did not pass their destination before parking.

6. Persons who looked for spaces at the curb before entering off-street facilities tended to have longer search distances than people who were successful in seeking a site at the curb.

7. Drivers indicated they began searching for a space (search-walk distance) at a distance of 150 to 200 ft further than the distance actually walked.

8. Magnitudes of walk distance, search-walk distance, and search distance did not correlate by rank. Those days of maximum or minimum walk distance were not the same as days of maximum or minimum search distance or search-walk distance.

9. None of the variables, walk distance, search-walk distance or search distance were correlated to volume or parking accumulation when measured by hours of the day.

10. Average total travel by curb parkers was slightly greater than for off-street parkers on the same date.

11. The influence of the Christmas rush period was to increase off-street parker average total travel by 655 ft and curb parker total travel by 823 ft over November measures. The net effect was equivalent to increasing vehicle travel—by about 20 percent even without the addition of further vehicles.

12. Parking facilities located on routes to the CBD tended to intercept trips entering along that route, both for curb and off-street locations. Drivers showed a preference for the nearest parking space and the most direct route.

13. Those parking locations not on a direct route tended to generate traffic from all origins in proportion to all vehicles entering at that origin.

A Parking Study Designed for Downtown Planning

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At present many cities are undertaking comprehensive planning studies of their downtown areas. These studies are concerned with how the areas might be redeveloped to carry out their basic functions more adequately.

In such a study it is necessary to know the traffic and transportation requirements related to the area, as well as the transportation linkages between the various land uses. In Ithaca, N. Y., a special parking survey was conducted to answer these questions. This parking survey, in addition to obtaining the usual information on primary destination of the trip, also established what other stops were made in the downtown area.

Analysis of the data indicated that most of the persons who come downtown primarily for shopping reasons seldom take advantage of other activities. About 55 percent of the people who come mainly to shop visit more than two stores. However, 40 percent visit only one store. It appears that many of the persons coming downtown for such activities as banking, dental appointments, and other personal business do combine such trips with shopping. For example, about one-half the people who come for these purposes made other stops in the downtown area for shopping. These relationships would indicate that in developing a plan for the downtown area it is important to make these activities convenient to the shopping area so that the downtown area will be desirable from a visitor's point of view.

• IT HAS recently become stylish to develop plans for the downtown area. Undoubtedly, this is due to downtown areas feeling the competition of outlying centers or beginning to realize that they must improve their services and facilities if they are to compete with outlying areas and remain the dominant centers in cities. In planning for these areas, however, adequate techniques and procedures have not been developed to evaluate the transportation problems related to these areas and how the downtown area should best be arranged to make it convenient from a transportation point of view.

Recently the author undertook an analysis of the transportation problems related to downtown Ithaca. The analysis aimed at developing a program for meeting future needs in the downtown area and recommending how the downtown area might be planned to make it more convenient from the transportation standpoint.

Ithaca is a city of about 25,000 population with an additional 15,000 in the trading area. It is a college town with two universities—Cornell and Ithaca College. Very few suburban centers have developed in the area primarily because Ithaca is a small city and its growth has been slow, tending to discourage speculative development of shopping centers.

In light of the characteristics of downtown Ithaca and the particular objectives that were sought, a special parking survey was devised that would give access to some of the fundamental problems. The survey interviewed people as they returned to their cars after visiting the downtown area. A procedure similar to that outlined in the Bureau of Public Roads standard parking survey for placing interviewers in and around the downtown area was used. However, in this case, the person was approached as he parked and was told about the parking study and that there would be a few questions asked him when he returned to his car. At that time a card was placed under his windshield wiper indicating the time of arrival. This card also acted as a reminder to the parker of the survey when he returned. This device also gave the interviewer a little more time to get to the car before the driver left his parking stall.

After noting the time of arrival and departure, the interviewer would then proceed with the interview. Women interviewers were employed in the survey. The interviews took as long as 10 min, but it was found that the women were able to hold attention for this length of time. It appears essential to have women interviewers in this type of study. Their approach is necessary in obtaining the proper response from those being interviewed.

The first question dealt with the main reason for coming downtown, as indicated on the sample interview form (Fig. 1). This follows very closely the purposes used in connection with the Bureau of Public Roads standard procedure. With the second question, the street or building where the person was going for the described purpose was sought. Then in Question 3 all stops made in the downtown area were ascertained. In effect, a complete itinerary of all stops made in the downtown area was obtained. Question 4 was used as a reminder or a check to see if all downtown stops had been given.

Questions 5 and 6 dealt with where the person had come from and where he was going, to find if he was planning to make or had made any other stops in the downtown area, and what business was transacted at such stops. Unfortunately, it was felt that this would make the form too involved and a compromise was made with the questions indicated. However, this gave a great deal of insight as to where people came from in coming to the downtown area, and how many of them were going to other sections of the downtown area.

Question 7 was used to establish the trading area for Ithaca and Questions 8 and 9 were aimed at obtaining the person's attitude about the downtown area. Question 9 was open-ended, and it produced many interesting comments that were helpful in developing the plan for downtown Ithaca.

Although this questionnaire was not field tested in advance, it proved to work very well. No major changes in this form are suggested, although changing the order of some of the questions might improve reporting. In obtaining the information on this survey, the interviewers were not stationed at a given location for a whole day, but were moved every 4 hr. This seemed to work well. In fact, it appears that 2-hr observations would have been satisfactory in areas where the parking limit was 1 hr.

The parking survey was run during the hours that the downtown stores were open. This included Thursday and Friday evenings.

Date ______________________ Station ______________________

TRAFFIC AND PARKING STUDY FOR DOWNTOWN ITHACA, NEW YORK

Mode of Travel: Pass. car Truck Bus Walk

Type of Parking: Meter curb Free curb Customer Lot Private Lot Other

Time of Arrival: ______________ Time Parked: ______________

Time of Departure: ______________ Parking Cost: ______________

1. What was your main reason for coming downtown today?

A. Work	C. Bank	E. Pay Bills	G. Social Activity	I. Business related to work	J. Other personal business
B. Shop	D. Meal	F. Recreation	H. Medical-Dental		

2. What was the name of the store or building where you were going for this purpose?

____ Code

3. After you parked here

		Purpose	Code
where did you go first?			
And from there to			
And from there to			
And from there to			
And from there to			

4. Have we covered all your purchases and all the stops that you made in connection with your trip downtown today? If not, please add to above list.

5. Where did you come from prior to parking here? Home

Other Where? ______________________________
(Name of place or address)

Downtown Where? ______________________________
(Name of place or address)

6. Do you plan to visit any other stores or places of business in or around the downtown area today? Yes __ No __ If so, where? ______________
Are you going there now? Yes __ No__

7. What is your home address? ______________________________ Code

8. Was this parking space conveniently located for this particular trip? Yes __ No__

9. Comments: ______________________________

Figure 1. Interview form.

TABLE 1
MAIN PURPOSE OF COMING DOWNTOWN

Purpose	Percentage
Shop	42.7
Bank	13.5
Meal	8.7
Pay bills	3.6
Social-recreation	1.8
Medical-dental	6.0
Business related to work	7.8
Other personal business	15.9
Total	100.0

TABLE 2
PURPOSE OF OTHER STOPS MADE IN RELATION TO SHOPPING TRIP

Purpose	Percentage
Shop	85.0
Bank	4.6
Meal	2.8
Pay bills	1.5
Social-recreation	—
Medical-dental	1.3
Business related to work	0.5
Other personal business	4.3
Total	100.0

TABLE 3
PURPOSE OF OTHER STOPS MADE IN RELATION TO BANK, MEDICAL-DENTAL AND PERSONAL BUSINESS TRIPS

Purpose	Percentage
Shop	67.5
Bank	7.8
Meal	4.8
Pay bills	3.6
Social-recreation	—
Medical-dental	1.8
Business related to work	1.3
Other personal business	13.2
Total	100.0

TABLE 4
RELATIONSHIP OF PRIMARY PURPOSE OF TRIP AND NUMBER OF STORES VISITED

Primary Purpose of Trip	Percent Visiting						
	0 Stores	1 Store	2 Stores	3 Stores	4 Stores	5 Stores	Over 5 Stores
Shop	0	45	24	15	10	4	2
Other	64	22	11	1	—	—	—

Table 1 gives the main purpose of the non-work trips made to the downtown area. This analysis indicates that over 40 percent of all the trips made by persons parking in the downtown area were for shopping purposes. However, a considerable number of people came downtown for purposes of banking (13.5 percent) and for other personal business (15.9 percent). The study also revealed that nearly 9 percent of the parkers came down primarily to eat.

Table 2 shows that most who came downtown primarily for shopping reasons seldom took advantage of other downtown activities, such as banking or eating. However, 55 percent of those coming downtown primarily to shop visited two or more stores. This, of course, meant that a large number visited only one store. On the other hand, people who came downtown for banking or medical-dental appointments, or other personal business, seemed to combine such trips with shopping (Table 3). About one-half of those coming downtown for these purposes made other stops in the downtown area for some other purpose. Thus it would appear that people plan in advance their trips for banking or medical-dental purposes to the downtown area, and therefore often find it advantageous to do other things at the same time.

Table 4 shows that a large number of these people who came downtown for purposes other than shopping visited stores. In fact, over one-third of them visited at least one store and about 12 percent visited two or more stores. On the whole, one out of every two who came downtown for purposes other than shopping visited a store. This is certainly an important factor to be considered in any downtown plan, particularly because the number of trips for these other purposes outweighs the special shopping trips.

Generally, in Ithaca, these other trips were about twice as numerous as those for shopping. In many other parking studies completed throughout the country, the proportion of these other trips is even higher.

Of course, the people who came downtown primarily to shop, as shown by Table 4, tended to visit more stores than those who came downtown for other purposes. The average person coming downtown to shop in Ithaca stopped at slightly more than 2 stores. The number of stores that people will visit depends on the type of shopping trip involved. A shopping trip for furniture, for example, may include only one or two stores, whereas a woman looking for a new dress will often go to 3, 4, or even more stores.

The importance of shopping to the downtown activities can be seen more clearly in Table 5, which gives the purpose at the beginning and end of all walk trips made by people coming to downtown Ithaca by car. It shows that over one-half of all trips were between one store and another, and over 80 percent of all the trips were involved in shopping at one end or the other. This far outweighed any other trip purpose. The next in line was banking, which was about 16 percent, followed by "other personal business," which was about 14 percent. This table, of course, does not take into consideration the pedestrian movements of people who work in the downtown area. This information could have been obtained by sampling a number of the downtown workers to determine their pedestrian movements on a given day. However, this was not done because of the limited budget and time. However, if possible, this additonal information should be developed so that a complete picture of all pedestrian movements in the downtown area could be determined.

Table 6 shows that over one-third

TABLE 5

PURPOSE INVOLVED AT EITHER END OF TRIP MADE WITHIN DOWNTOWN AREA

From	To (%)						
	Shop	Bank	Meal	Pay Bills	Medical-Dental	Business Related to Work	Other Personal Business
Shop	51.0						
Bank	8.2	—[1]					
Meal	5.6	2.0	—[1]				
Pay bills	4.6	1.3	0.3	1.0			
Medical-dental	4.6	0.3	—[1]	0.3	—[1]		
Business related to work	1.0	0.7	0.3	—[1]	0.3	1.3	
Other personal business	6.1	3.9	0.3	—[1]	1.3	1.0	0.3

[1] Less than 0.1 percent.

TABLE 6

ORIGIN OF TRIPS RELATED TO TRIP PURPOSE

Origin	Purpose of Trip (%)							
	All Purposes	Shop	Bank	Meal	Pay Bills	Medical-Dental	Business Related to Work	Other Personal Business
Home	63.4	28.2	6.0	4.4	2.2	6.4	3.2	12.0
Downtown	11.4	5.4	2.4	—[1]	—[1]	0.9	1.8	0.9
Other	25.2	11.4	2.6	3.6	0.9	0.6	2.7	3.4

[1] Less than 0.1 percent.

of the trips made to the downtown area were not related to the home. About one-fourth of them started from other places, such as place of employment or a friend's house, and over 10 percent started from some other place in the downtown area. The number of people who had started within the downtown area may appear to be high, but it was also found that over 10 percent of the people leaving their parking spaces in the downtown area were going to move to some other parking space within the downtown area to carry out the rest of their business or shopping. This would indicate that a considerable number of people were moving their cars instead of walking around the downtown area.

In light of these results, an analysis was made of the walking pattern within the downtown area (Table 7). It showed the percentage of people who walked various distances. About 80 percent walked less than 500 ft. The walking trip from the parking space was generally somewhat shorter than those made within the downtown area. However, there really was a high degree of comparability in the walking patterns for the distances of these various trips.

It appears that the differences that occurred are largely related to the distribution of various activities in the downtown area and their relationship to other activities or parking facilities. This probably explains why there was a small number of people walking less than 200 ft to eat meals. In Ithaca, parking spaces suitable for people who are going to be gone long enough to eat a meal are some distance away from the restaurants, and at the same time the restaurants are some distance away from the major stores. The walking limitation of people in the downtown area is very short—less than 750 ft—and this distance is about the same for all walk trips in the downtown area. In large cities where the walking distance to the various activities is somewhat longer, the walking distance between downtown activities would probably be comparable.

This whole pattern indicates the

TABLE 7

LENGTH OF WALK TRIP IN DOWNTOWN AREA

Purpose	Percent Walking: 0 to 250 Ft	250 to 500 Ft	500 to 750 Ft	750 to 1,000 Ft	Over 1,000 Ft
Shopping trips:					
All	27	52	18	3	—
From parking space	26	55	19	—	—
Banking trips:					
All	16	69	13	2	—
From parking space	35	60	5	—	—
Meals:					
All	13	43	40	4	—
From parking space	7	83	10	—	—
Business trips:					
All	30	48	22	—	—
From parking space	53	40	7	—	—
Personal business:					
All	30	52	11	3	4
From parking space	39	42	14	3	3
Average:					
All	26	52	18	3	1
From parking space	31	56	11	1	1

need to keep shopping and various other activities in the downtown area as compact as possible. A radius of 750 ft should include most of the activities linked together in the downtown area. Therefore, in planning the downtown area these linkages should be recognized and the walking distances between these linkages be kept less than 750 ft.

The results of this survey indicated that, for every employee in retailing in the downtown area, one auto driver shopping trip was made to downtown. Of course, because the average person who came downtown primarily to shop visited a little over two stores and because the people who came downtown for other reasons visited, on an average, a store on every other trip, it might be said that there are three visits to a store for every retail employee in the downtown area.

About three auto driver trips were made to the downtown area for every four employees in non-retailing activities. Because shoppers occasionally made non-shopping stops in the downtown area, it could be concluded that, on an average, every non-retailing employee generated about one trip. Of course, trips made by downtown workers are not included. Because of the nature of the study, information on these trips was not obtained.

In considering the destination of trips in the downtown area, two analyses were made—one for shopping trips only, and the other for all other trips not related to work. The pattern of these two types of trips was quite different, as shown by Figure 2. This type of summary should be developed for any city for which a downtown plan is being prepared, because it shows where close-in spaces are needed—those most essential for the health of the downtown area. Similarly, it is important to explain the parking requirements for the all-day parkers. This was done, as shown in Figure 3. Because few parkers for work purposes were observed, this is based on employment because the bulk of the people working in the downtown area depend on the auto.

As already indicated, the attitudes of the parkers were very helpful. A summary of these attitudes is given in Tables 8 and 9. They indicated several things. First of all, they ob-

TABLE 8

SUMMARY OF COMMENTS ON PARKING FACILITIES IN DOWNTOWN ITHACA

Comment	No.	Comment	No.	Comment	No.
Parking facilities adequate:	39	Parking facilities inadequate:	93	Meter rates too high	17
No worse than other cities	2	Lucky to find space	87	Longer time limit allowed:	13
Don't mind walking a few blocks	7	Must circle several times to find a space	26	½ hr should be available on meters	2
Adequate if you come downtown early	6	Especially bad on weekends	15	Enforcement too strict	6
Parking facilities better after universities close	2	Avoids downtown shopping due to poor parking condition	12	More uniformity in meter prices	4
Total	56	More municipal lots needed	3	Too many truck loading zones	4
		More space needed near P.O.	2	Parking spaces too small for large cars	3
		More space needed on State, Cayuga and Seneca Streets	3		
		More evening parking	3		
		Total	194		

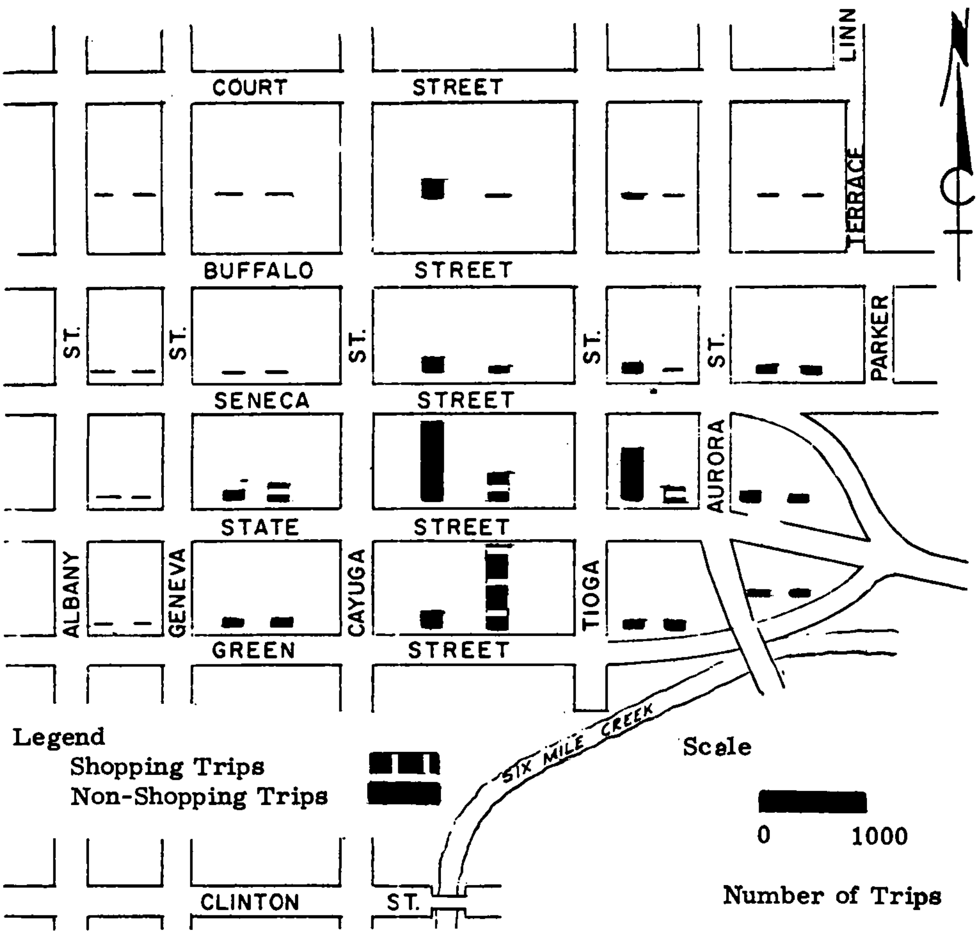

Figure 2. Destination of shopping and non-shopping trips in downtown Ithaca.

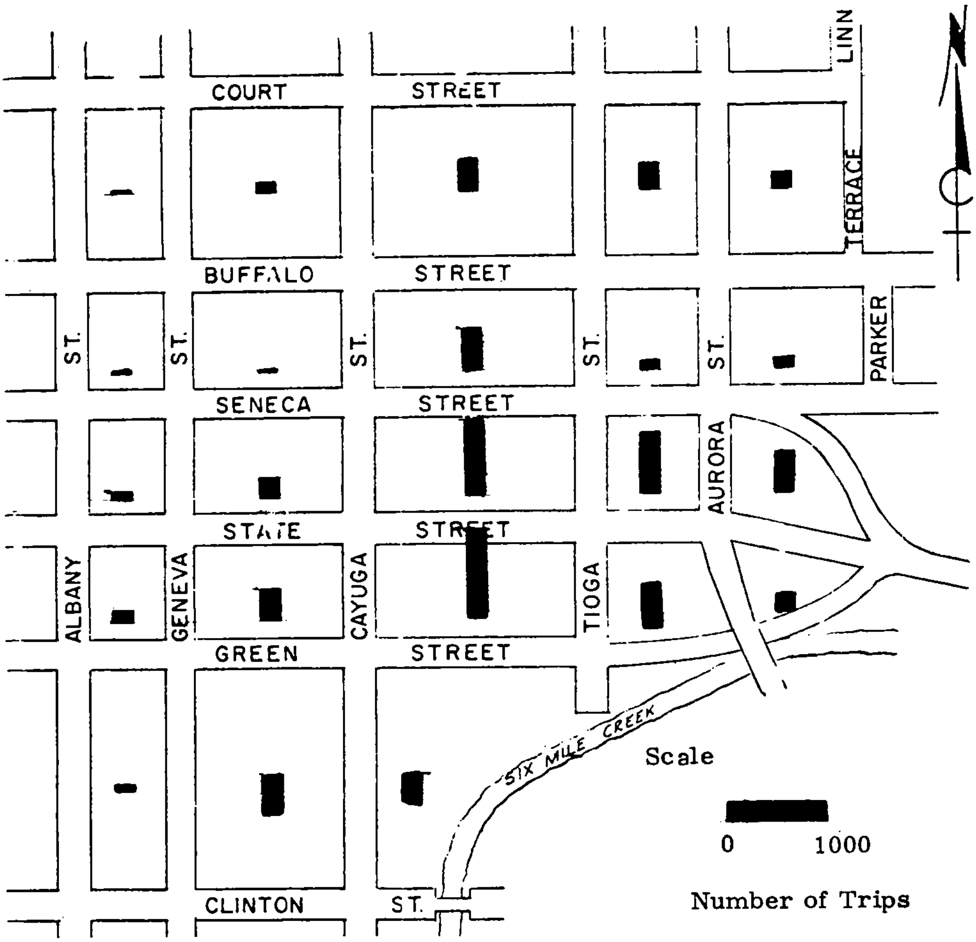

Figure 3. Destination of working trips in downtown Ithaca.

TABLE 9

MISCELLANEOUS COMMENTS ON PARKING FACILITIES IN DOWNTOWN ITHACA

Telephone company needs employee parking lot.
Ten-minute parking limit around post office should be more strictly enforced.
Bank parking lot time limit should be extended to 1 hr.
Necessary to walk great distance for quick shopping.
Ithaca needs more good shops with better selection of merchandise.
State Street should be for pedestrians only.
More municipal parking lots needed; those now available are inconvenient.
Too much double parking in downtown area.
Meters are chasing shoppers away. Instead, shoppers should be encouraged.
More parking lots needed on Cayuga and Seneca Streets.
More overnight parking space needed near Cornell University.
Store owners park all day in front of their stores.
Too many empty truck spaces that shoppers could use.
Convenient night parking needed.
Need convenient bus station with parking lot.
Parking lot on Green Street is mass confusion.
Merchants should provide free parking for their customers.

jected to the meter rates and felt that the lots and garages should be paid for by merchant-validated programs rather than by meters. People seemed to detest the cost of parking, even though it is only a nickel or a dime. They greatly preferred customer lots where available. It was apparent from these attitude studies that most people were very cognizant of the traffic and parking problems in the downtown area. In fact, several of them indicated that there were too many truck loading zones, because they knew that as a general practice many people would pull into a truck loading zone and wait in their car until a space became available down the street, and then would hurry to move into that space.

The insight gained from this study was very valuable to the team that was preparing the plan for downtown Ithaca. It gave a better understanding of the primary destination in the downtown area, the activities that were the backbone of the area in terms of attraction, and the linkages between various activities in the downtown area. It also established the pedestrian patterns and helped in providing for the over-all framework for the downtown plan. In addition to these advantages, it helped to establish parking needs and transportation requirements for the downtown Ithaca area.

Some Mathematical Aspects of the Parking Problem

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The comparison between a parking lot and a telephone switchboard is established, and some of the pertinent results for switchboards are applied to parking. Some new data are analyzed from this point of view.

• ONE interesting aspect of the mathematical theories on road traffic that have grown up in the last decade is that they are very largely new. Although considerable effort has been spent in trying to establish meaningful analogies between road traffic and other physical phenomena, none of it has been very successful. On the contrary, students of traffic flow theory have more and more been compelled to introduce new concepts, define new parameters, and obtain new theorems.

Parking is an exception: fortunate if one wishes to employ classical results quickly, unfortunate if one wishes to develop a new theory. The problem of a parking lot is basically the same as the problem of the telephone switchboard. Inasmuch as the latter has been studied intensively for half a century, engineers interested in the design of parking lots would be well advised to consult the procedures of their colleagues in telephony; for example, Erlang (*1*) and Fry (*2*).

This paper mentions several of the ideas on which the statistical theory of telephone traffic is based, quotes some useful results and shows the significance of these results in the design of parking facilities. It also gives some possibilities for improving the operation of parking meters and concludes with a brief mathematical treatment of a new parking problem.

ANALOGY WITH A SWITCHBOARD

A parking lot consists of a number of parking slots, each of which may be either full or empty. Vehicles arrive from time to time and, if a slot is empty, fill it. After a parking time, the vehicles empty the slot and go away. Each vehicle operates independently of the others; when all slots are filled, vehicles may wait and form a queue (system with queueing) or go away (system with loss). In the terminology of automatic telephony, the vehicles are equivalent to calls, the slots to lines, and parking time to holding time. In both fields, systems with loss are more common, but systems with queueing sometimes occur.

A complete statistical description of such a system consists of a distribution of interarrival times (that is, times between successive arrivals) and a distribution of parking times. In case of a system with queueing, it would also be necessary to specify the queue discipline; usually "first come, first served." Following traditional notation, the mean interarrival time is denoted by $1/\lambda$, the mean parking time by $1/\mu$ and the number of slots by N. The quantity $\rho=\lambda/N\mu$ is called the relative traffic intensity and measures how well the lot is able to deal with the demand. When $\rho<1$, the situation is said to be stable.

The simplest assumption is that the demand for parking occurs at random and that departures are also at random. In a later section, the few data available support this conjecture. If arrivals to the parking lot occur at random with mean rate λ, then the distribution of interarrival times is known to be the negative exponential:

$$\lambda e^{-\lambda x},\ 0<x<\infty \tag{1}$$

which can also be written in cumulative form:

$$\text{Prob (time between consecutive arrivals}>x)=e^{-\lambda x} \tag{2}$$

or in "counting" form:

$$\text{Prob (exactly } n \text{ arrivals in unit time)} = \frac{e^{-\lambda}\lambda^n}{n!},\ n=0, 1, 2, \ldots . \tag{3}$$

the Poisson distribution.

Three similar expressions with λ replaced by μ, characterize random parking times and random departures from the lot. Of course, the parameters λ and μ will vary considerably from hour to hour during a typical day, so that the parking process is only temporarily homogeneous. It can be shown (*3*, p. 377) that under the assumptions of random arrivals and random departures, the probability that exactly n of the slots are occupied (assuming temporary equilibrium) is also given by the Poisson distribution:

$$\text{Prob (} n \text{ slots occupied)} = \frac{e^{-\lambda/\mu}(\lambda/\mu)^n}{n!},\ n=0, 1, 2, \ldots . \tag{4}$$

assuming an infinitely large lot ($N\rightarrow\infty$). This result will be approximately valid whenever the value of ρ is small enough to insure that the possibility of all slots being filled is very unlikely.

In case the traffic intensity is so large that the lot may be quite possibly overflowing, the Poisson distribution (Eq. 4) needs only to be truncated; that is, stopped at the value N. However, in order that the probabilities add to unity, it is also necessary to divide by their sum, so that

$$\text{Prob (} n \text{ slots occupied)} = \frac{\dfrac{(\lambda/\mu)^n e^{-\lambda/\mu}}{n!}}{\sum_{j=0}^{N}\dfrac{(\lambda/\mu)^j e^{-\lambda/\mu}}{j!}},\ n=0, 1, 2, \ldots, \text{N}$$

$$= \frac{\dfrac{(N\rho)^n}{n!}}{1+N\rho+(N\rho)^2/2!+\ldots+(N\rho)^N/N!} \tag{5}$$

This result is also proved by Feller (*3*). A particular case ($n=N$) of Eq. 5 gives the probability that the lot is full, and hence is the probability of a loss to the system in a system with loss, or an increment to the queue in a system with queueing. It is called "Erlang's loss formula," and can be written

$$L_N(\rho) = \frac{\dfrac{(N\rho)^N}{N!}}{1+N\rho+\dfrac{(N\rho)^2}{2!}+\ldots+\dfrac{(N\rho)^N}{N!}} \tag{6}$$

Eq. 6 has been used very extensively in the design of telephone switchboards. If it is decided in advance how probable the loss of a call is to be; *i.e.*, the choosing of L, and if there is a value for the demand ρ, then the correct number of lines N can easily be determined from existing tables of the Poisson distribution.

Mori (*4*) and Kometani and Kato (*5*) have analyzed the operation of several parking lots in Japan, with a view to testing the validity of the hypotheses of random arrivals and departures on which Eq. 6 is based. Using appropriate statistical procedures, they confirm the propriety of these assumptions for the parking lots studied.

However, in Japanese parking lots, the customers catered to are persons making short business stops, rather than workers and shoppers. This fact helps explain how the random arrival and random departure model obtains.

SANTA MONICA STUDY

A study was carried out in Santa Monica, Calif., to obtain extremely detailed data on parking. The gross fluctuations in parking are already well known, but some interesting conclusions might be forthcoming from a short period of precise observation. Therefore, a parking lot of 150 slots was observed for 10½ hr during a typical weekday (Wednesday, June 15, 1960) and every significant movement recorded. The results (*6*) of this investigation can be briefly summarized. The principal document consists of a master list of each car arriving and departing, and each person feeding a meter, together with the slot number, the time of day, the amount of money deposited, the time showing on the meter, the number of passengers, and the total lot occupancy at that time. It is then easy to deduce the amount of time used by each car, this time being divided into three categories: "green time," *i.e.*, time paid for; "red time," *i.e.*, time illegally parked; and "blue time," *i.e.*, time inherited from the previous occupant of the slot.

This particular parking lot was also used by certain cars holding parking permits, sold for a monthly fee, which exempts them from feeding the parking meters. There were two categories of parkers, of which one (permit holders) always used green time.

The first problem in the analysis of the master list is to see if the random switchboard model previously described is valid. In doing so, it was necessary to deal separately with the two types of parkers, as well as to make allowance for varying values of λ and μ. It turned out that it was sufficient to divide the period under study into three parts: growth, stability, and decay, as shown in Figure 1. If cash customers and permit holders are handled separately, and only one of the three time periods referred to, then the assumption of random arrivals and departures seems fairly well justified. Figures 2 and 3 show these results for departures, and the fit with the Poisson distribution for arrivals is given in Table 1.

TABLE 1
OBSERVED AND THEORETICAL NUMBER OF DEPARTURES IN TEN-MINUTE PERIODS
(10:30 to 6:00)

Time Period	No. of Departures	
	Observed	Theoretical
0	0	0
1	2	0
2	2	1
3	3	3
4	3	5
5	7	7
6	7	7
7	5	7
8	6	5
9	2	4
10	4	2
11 or more	4	4

VALUE OF PARKING

During the period of observation, a total of $37.20 was deposited into

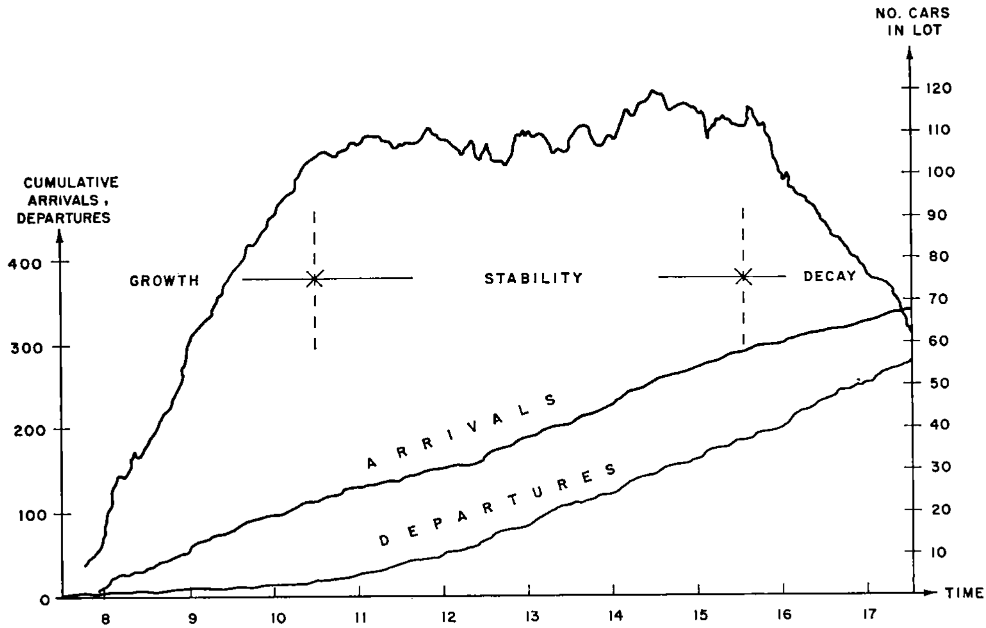

Figure 1. Number of cars in lot, cumulative arrivals and departures.

the meters, and 34,622 min. of meter time were used by non-permit holders. This represents about $0.07 per hr paid for parking, neglecting the slight effect of cars parked before the experiment began or remaining after it finished. Figure 4 shows arrivals classified by amount deposited. Permit holders consumed 8,737 min of parking. It is difficult to calculate their financial contribution. If 21 working days per month during which the permit is used are assumed, the daily cost is about $0.30. (A permit sells for $6.00.) Inasmuch as 33 permit holders were seen, the daily value would be $9.90. However, the monthly income from the 65 permits current was known to be exactly $390.00. It appears that many permit holders did not use the lot during the period under investigation; possibly they store their cars in this lot at night.

One of the most interesting practical consequences of this particular study is the exact evaluation of blue time and red time, and the consequent improvement in municipal revenue if these could by some means be reduced or eliminated. The income from all parking meters in Santa Monica for the year preceding the study is given in Table 2, and the lot being studied is part of the first category, "Downtown and Pier."

The income from this parking lot consists of not only meter receipts, but also permit sales and "bail forfeiture" for parking tickets issued.

TABLE 2

NUMBER AND REVENUE OF VARIOUS GROUPS OF METERS

Area	No. of Meters	Revenue [1] ($)
Downtown and pier	1,077	87,448.70
Wilshire	137	8,962.35
Ocean Park	79	6,317.79
Douglas Aircraft	159	5,373.02
Ocean Avenue	358	16,066.00
Beach	204	21,295.53
Total	2,014	145,463.39

[1] For year ending June 30, 1959.

During the period of observation, the police issued two parking tickets, which one assumes produced $2.00 in revenue at the current price of $1.00 a ticket (subsequently raised to $2.00 a ticket). This is far less than possible; Figure 5 shows the number of cars illegally parked during the entire day.

Therefore, the income from this day's parking can be considered to be $37.20 in cash, $9.90 from permit

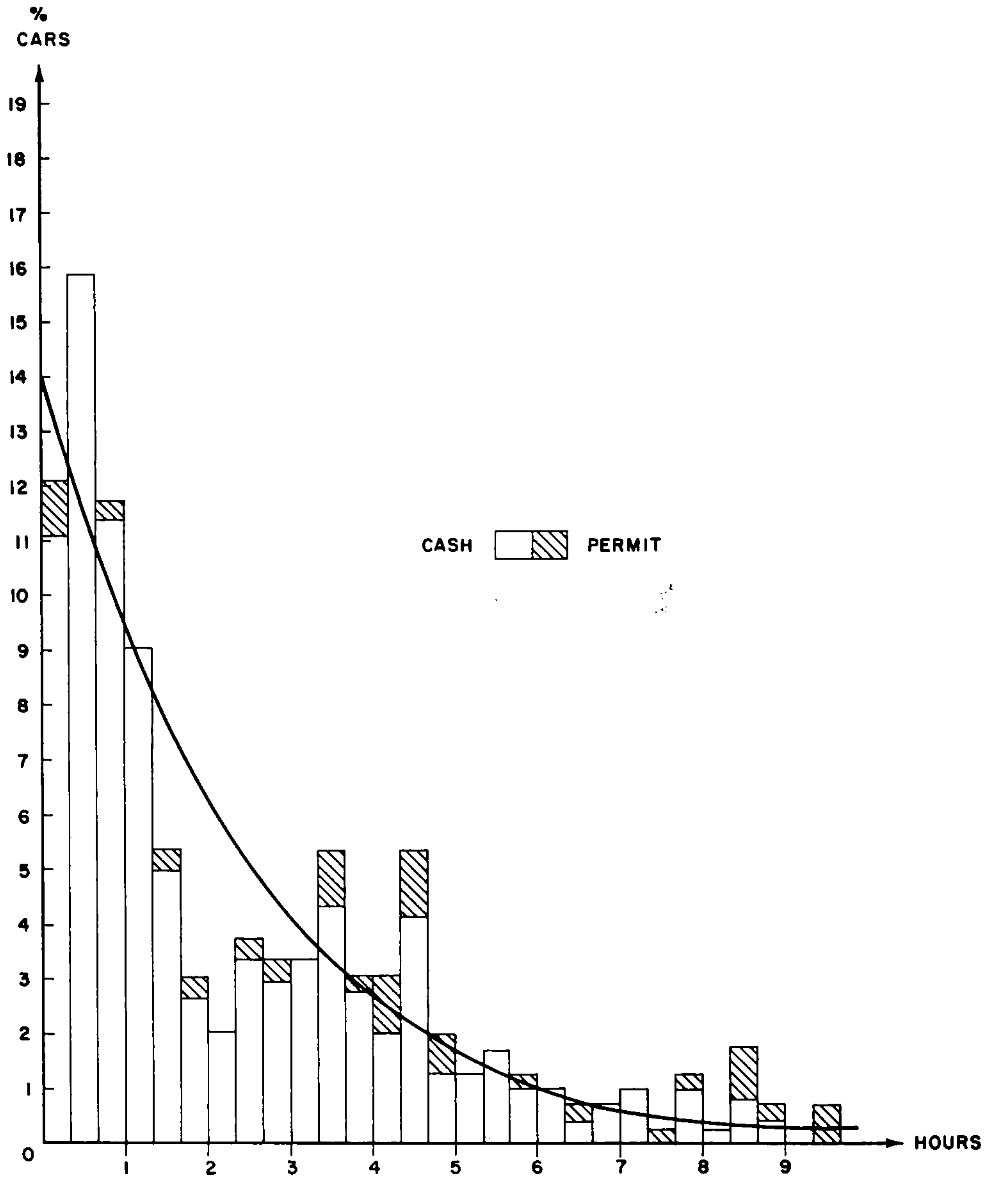

Figure 2. Percentage of cars parking various lengths of time; all departing cars included.

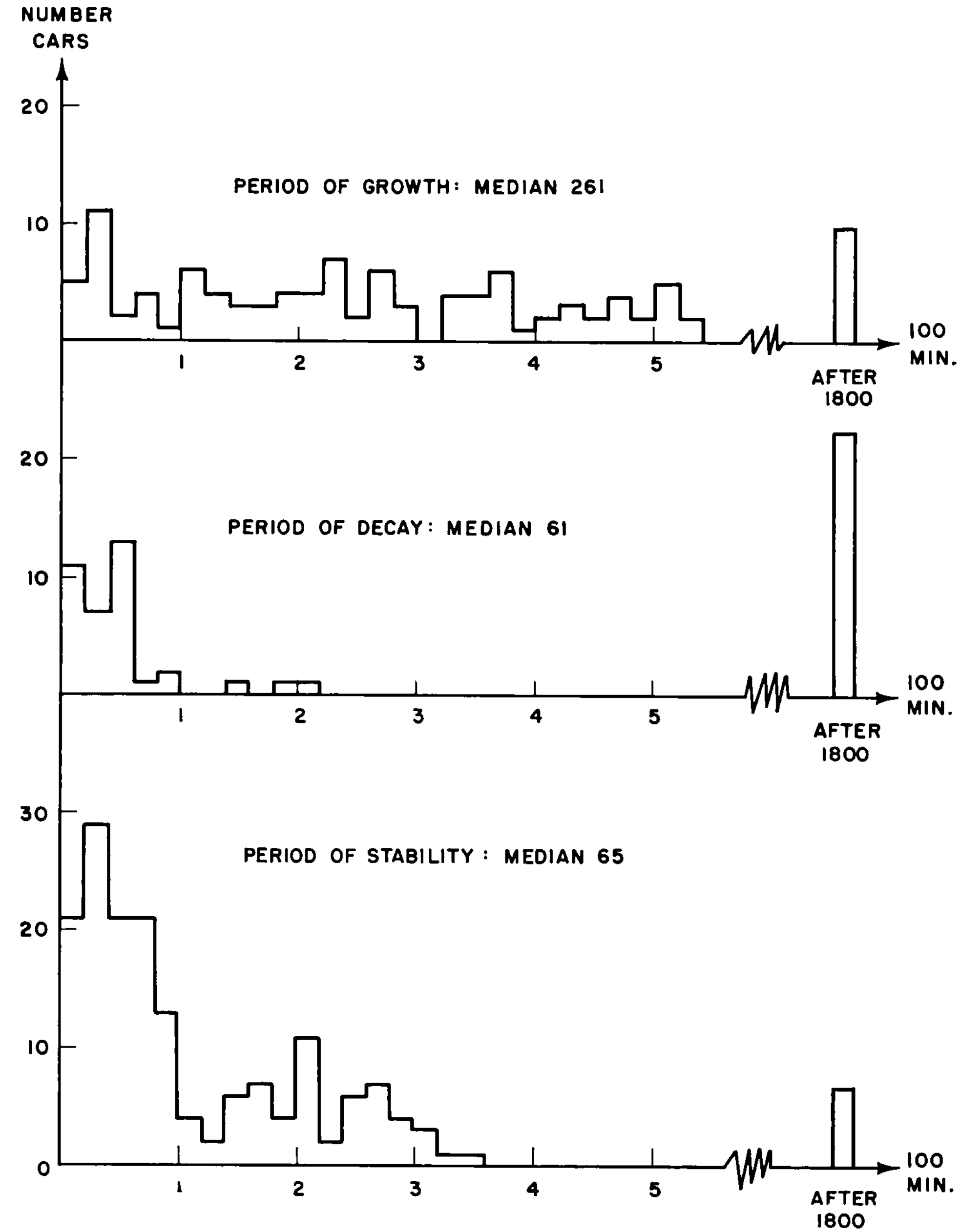

Figure 3. Number of cars parking various lengths of time; all cars arriving in various periods.

holders, and $2.00 from bail forfeiture—a total of $49.10.

The maximum possible for the operation of 150 meters over a $10\frac{1}{2}$ hr period is $78.75 (at $0.05 per hr), of which $29.65 was lost. The loss consists of red time and blue time parking, and of times when slots were empty. The loss proportions shown by the study are $20.10 from empty slots, $4.05 from red time, and $4.50 from blue time—a total of $29.65.

These figures are very likely not quite typical, because the presence of investigators on the lot was observed in many cases to make arrivals change their mind and put money in the meters. In some instances they would feed the meters when their subsequent behavior showed this to be absolutely unnecessary, and in one case even offered to pay us a fine for some small amount of red time used.

Thus, nearly one-third of the

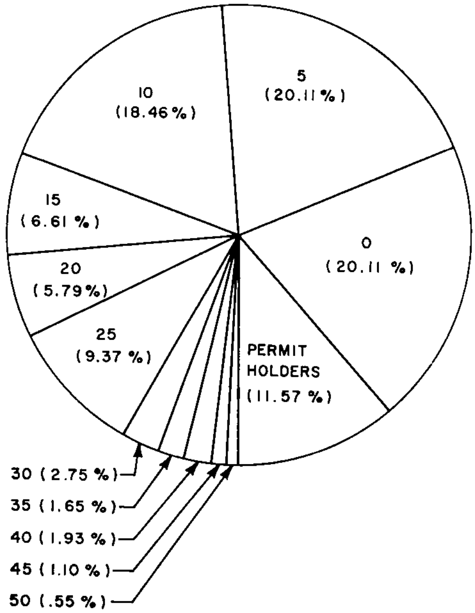

Figure 4. Arrivals classified by amount deposited.

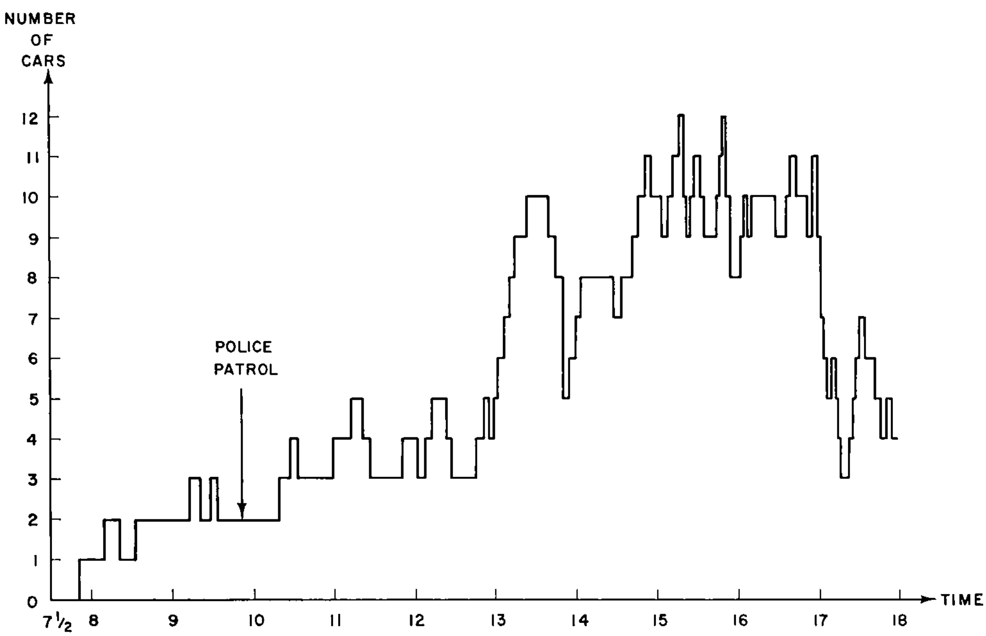

Figure 5. Cars illegally parked.

amount lost during the day could have been recovered. If this experience had been typical (and from the remarks, can hardly be much less) then the increase in revenue to Santa Monica from the elimination of red and blue time would be $33,436.36 for the year.

It seems, therefore, that some serious effort should be made to induce persons using the parking facilities to pay for their time. The question of red time is probably one of enforcement, in which parking revenue is balanced against enforcement costs in a sensible way.

Is it possible to zero each meter before a new car is parked? The suggestions in this direction usually involve electric and mechanical systems, magnets, road tapes, photoelectric cells, or similar devices. It seems that these have two principal disadvantages: cost and reliability. Power cables must be laid to each meter, and the whole area booby-trapped with devices to tell whether a car is coming in or going out or shuffling back and forth.

There are two other possible sources of power for meter zeroing; namely, the arriving driver and the departing driver. It might be possible to invent parking meters that would have to be zeroed before use or after use, and yet which could not be meddled with by passing busybodies; for example, one for arriving driver, and one for departing driver.

Arriving Driver

A driver arrives and finds enough green time already showing on the meter. How can he be persuaded to drop in a coin (assuming that the instrument could be constructed so that when a coin is dropped, the flag first falls to zero, and then records the correct amount) ? When the coin is dropped, the meter feeds out a small gummed piece of paper with the meter number printed on it. The driver pastes this on his windshield and is liable to a ticket without this, even if the meter shows green. Because the meter number is printed on the paper, this driver cannot go all over town using up green time.

Departing Driver

It would be desirable to have the departing driver throw a switch to deprive his successor of green time. He must be offered some incentive, and guard against anyone else doing so. The incentive will be money, and the guard a key. Each parking meter could be provided with a key such as are now used in luggage checking lockers. He must deposit $0.50; then the key can be removed. When he returns, he puts the key back and receives his change from parking—at least a dime. As he puts back the key and collects his change, the same force zeroes the meter.

OFF-STREET PARKING

The basic formulation of a parking lot was shown to correspond closely to that of a telephone switchboard. This does not mean, however, that there are no individual variations on that premise. A mathematical model illustrating one sensible difference between the two systems can be sketched. The difference referred to is that the choice of a telephone line is done mechanically according to some predetermined principles, while the choice of a parking slot is made by the individual driver to suit his own purposes.

Assuming that a driver wishes to park as near as possible to some particular objective and is offered parking lots $P_1, P_2, \ldots$ in increasing order of distance (or inconvenience) from his objective, if he finds a slot in P_n he can park at once. If the lot P_n is full, he may wait and watch P_n, and will be able to take supposedly the first available slot. Which lot should he watch? In order for this to be a sensible question, some restrictions must be placed on the problem. Clearly the lower numbered lots are more desirable in that they are nearer his objective. Assume that the remote lots are more desirable in that there is a greater chance of finding a parking place, that is, if he wishes to attend a movie on zeroth street, shall he search first street, which is congested, or second street, which is less so, or third street . . . .?

Suppose the number of slots in P_n is N_n and the traffic intensity in P_n is $\rho_n=\lambda_n/\mu_n$. Then the probability of P_n being full is given by Eq. 6 with parameters N_n and ρ_n, abbreviated

$$L_n=L_{N_n}(\rho_n) \tag{7}$$

Consequently, there is zero delay with probability $1-L_n$ in the n^{th} parking lot. Otherwise let the delay be t, with density function $f\ (t)$ and cumulative density $F\ (t)$. Then

$$\begin{aligned} F(x) &= \text{Prob}(t\leqq x)=1-\text{Prob}(t>x) \\ &= 1-\text{Prob}(\min[x_1,\ldots,x_{N_n}]>x) \end{aligned} \tag{8}$$

in which $x_1, \ldots, x_{N_n}$ are the N_n random variables of parking time for the N_n slots in the n^{th} lot.

Each of these variables is assumed to have density function,

$$\mu_n e^{-\mu_n x}$$

so that Eq. 8 can be written

$$\begin{aligned} F(x) &= 1-\text{Prob}[x_1>x, \ldots, x_{N_n}>x] \\ &= 1-\left[e^{-\mu_n x}\right]^{N_n} \\ &= 1-e^{-\mu_n N_n x} \end{aligned} \tag{9}$$

Differentiating Eq. 9, the continuous density for delay in the n^{th} lot becomes

$$f(x)=\mu_n N_n e^{-\mu_n N_n x} \tag{10}$$

Therefore, the whole probability distribution for delay in the n^{th} lot can be written in terms of the Dirac delta function for the discrete component and Eq. 10 for the continuous component.

$$f_n(x)=[1-L_n]\,\delta(x)+L_n\mu_n N_n e^{-\mu_n N_n x} \tag{11}$$

Multiplying Eq. 11 by x and integrating, the average delay in the lot P_n becomes

$$D_n=L_n\mu_n N_n \int_0^\infty x e^{-\mu_n N_n x}\, dx=L_n/\mu_n N_n \tag{12}$$

There are several applications that might be made of Eq. 12. For example, if explicit expressions for the various traffic intensities were given, the value of D_n could be compared with the inconvenience of parking in P_n, and some optimum obtained. To illustrate this for a simple but plausible set of assumptions, assume all the lots to be the same size, so that the subscript on N can be dropped. Next, the parking times are assumed independent of the lot parked in, but the demand is inversely proportional to the nearness of the lot; namely, the other parkers are also going to the same movie. This would permit dropping the subscript on μ, and to write $\rho_n = (C/n)$. Therefore,

$$L_n=\frac{(C/n)^N/N!}{\sum_{j=0}^{N}\frac{(C/n)^j}{j!}} \tag{13}$$

and with the constants C and N specified, L_n could be computed as a function of n. If the disadvantage of the n^{th} parking place consisted of a simple linear constraint (*i.e.*, if the streets in this town are equally spaced and if it takes time T to walk a block), then the values of Eq. 13 would only have to be compared with nT, and an optimum difference obtained.

ON-STREET PARKING

In this section a new model is presented to analyze some features of on-street, or curb parking. Figure 6 shows how a typical downtown area can be broken up into zones. Each square represents a city block and the broken lines outline the zones. The property of each of these zones is that no more than one block need be walked to pass from one zone to the next. Here, to prevent any ambiguity, the city block, referred to before, will henceforth be called a square, and the full distance from corner to corner along one side of the square will be called a block. All zone outlines are contingent on the center of attraction, or destination designated by an X in the diagram, and each destination will determine a different zone pattern. (Because it is unlikely that all persons parking in a downtown area will have the same destination, it is therefore unlikely that all persons who happen to know the following procedures will concentrate their parking efforts in a single specific zone, thus upsetting the results of the procedure.) Then, the maximum distance to be walked from zone n to the destination is n blocks.

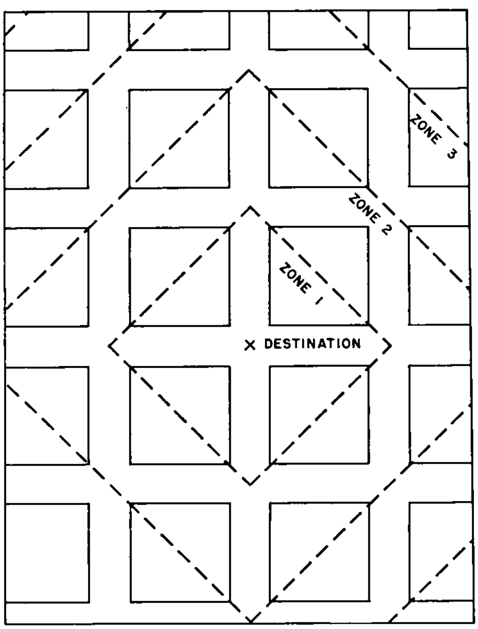

Figure 6. Rectangular street network divided into zones.

This problem is an optimal choice of zone in which to try to park. The following approximations will be made to simplify the presentation of this method. The streets shall be considered to have very small width. The squares are regular and of uniform dimensions, and each is surrounded by the same number of parking slots, the slots running from corner to corner with no omissions.

Let N be the number of parking slots along one side of any square. Then the number of parking slots surrounding each square is $4N$. From the diagram, in zone 1, there are $8N$ slots; in zone 2, $24N$ slots; in zone 3, $40N$ slots; and in zone n, there are $(2n-1)\,8N$ slots. If the number of slots in zone n is defined as N_n, then,

$$N_n = (2n-1)8N \qquad (13)$$

Let the length of one block be A, and the length of each parking slot be a. Then $A = Na$. Now, the average distance of zone n from the destination is $(2n-1)A/2$, or $(2n-1)$ $Na/2$ blocks. This, of course, considers only travel along the perimeter of a square to be allowed. If v is the walking speed, the average time it takes to walk from zone n to the destination is

$$t_n' = (2n-1)\, Na/2v \qquad (14)$$

If now it could be estimated how long, on the average, it would take to find a parking slot in any particular zone, then the zone could be picked for which the total average time involved in reaching the destination would be a minimum.

The problem is now to find out how long it will take to find an empty parking slot in any zone, or equivalently, how many parking slots in zone n can be expected to be passed before there is an empty one? Let this number be $\bar{k}_n$ (the bar indicating that it is a mean value) so that if the search speed is V, the average time it will take to find a parking slot is

$$t_n = \bar{k}_n\, a/V \tag{15}$$

and the total average time involved in reaching the destination by parking in zone n is the sum of Eqs. 14 and 15,

$$T_n = t_n + t_n' = \frac{\bar{k}_n a}{V} + \frac{(2n-1)Na}{2v} \tag{16}$$

Now, for given n, what is the expected $\bar{k}_n$? Assuming equilibrium conditions, the probability that j slots are filled in lot n is given by Eq. 5. It seems reasonable that ρ is constant for each zone. Certainly $1/\mu$, the mean parking time, can be considered constant; but λ, the mean arrival rate, will vary because the zones are of different sizes. From the earlier discussion on the variety of destinations, no area will be more in demand than any other. Then, λ should be approximately linearly proportional to the size of the zone, or synonymously to the number of slots in each zone, so that $\lambda_n = CN_n$, where C is a constant. Then,

$$\rho = \lambda_n / N_n\mu = CN_n / N_n\mu = C/\mu = \text{constant} \tag{17}$$

If the j-filled slots in zone n are independently distributed, then the probability that the first k_n slots searched are all filled is, according to Fry (*2*),

$$\frac{\binom{j}{k_n}}{\binom{N_n}{k_n}}, \quad k_n = 0, 1, 2, \ldots, j \tag{18}$$

and the probability of having to test more than k_n is found by

$$p(>k_n) = \sum_{j=k_n}^{N_n} \frac{\binom{j}{k_n}}{\binom{N_n}{k_n}}$$

$$\text{Prob } (j \text{ slots occupied}) \tag{19a}$$

or

$$p(>k_n) = \sum_{j=k_n}^{N_n} \frac{j!(N_n-k_n)!}{N_n!(j-k_n)!} \frac{(N_n\rho)^j/j!}{\sum_{i=0}^{N_n} \frac{(N_n\rho)^i}{i!}}$$

$$= \frac{(N_n-k_n)!}{N_n!} \frac{\sum_{j=k_n}^{N_n} \frac{(N_n\rho)^j}{(j-k_n)!}}{\sum_{i=0}^{N_n} \frac{(N_n\rho)^i}{i!}} \tag{19b}$$

Letting $i = j - k_n$,

$$p(>k_n) = \frac{(N_n\rho)^{k_n}(N_n-k_n)!}{N_n!} \frac{\sum_{i=0}^{N_n-k_n} \frac{(N_n\rho)^i}{i!}}{\sum_{i=0}^{N_n} \frac{(N_n\rho)^i}{i!}} \tag{19c}$$

Now, the probability of having to search exactly k_n slots before finding an empty one is

$$p(k_n)=p(>k_n-1)-p(>k_n) \qquad (20)$$

and the average number of slots expected to be searched in zone n is

$$\bar{k}_n=\sum_{k_n=0}^{N_n} k_n p(k_n)$$

$$=p(1)+2p(2)+3p(3)+\ldots$$
$$=p(>0)-p(>1)+2p(>1)-$$
$$2\,p(>2)+\ldots-(N_n-1)p(>N_n-1)+$$
$$N_n p(>N_n-1)-N_n p(>N_n) \qquad (21)$$

This model does not allow for the search to involve more than N_n slots, so that if N_n slots are searched unsuccessfully, then the driver drops the search in the most favorable zone, moves to the next most favorable, and initiates a new search. Hence, the term $N_n p(>N_n)$ in Eq. 21 is zero, and the equation becomes

$$\bar{k}_n=\sum_{k_n=0}^{N_n-1} p(>k_n) \qquad (22)$$

Then, the average time it will take to find a parking place, if the time of entering zone n is $t=0$, is

$$t_n=\frac{\bar{k}_n a}{V} = \frac{a}{V}\sum_{k_n=0}^{N_n-1} p(>k_n) \qquad (23)$$

and the total average time involved in reaching the destination is, from Eq. 16,

$$T_n=\frac{a}{V}\sum_{k_n=0}^{N_n-1}\frac{(N_n\rho)^{k_n}(N_n-k_n)!}{N_n!}\;\frac{\displaystyle\sum_{i=0}^{N_n-k_n}\frac{(N_n\rho)^i}{i!}}{\displaystyle\sum_{i=0}^{N_n}\frac{(N_n\rho)^i}{i!}}+\frac{(2n-1)Na}{2v} \qquad (24)$$

Since t_n' is an increasing function, and t_n a decreasing one, it can be expected that there is some zone n for which T_n is a minimum. That would be the zone in which to seek a parking slot in order to reach one's destination in the least time.

Each square has a zone boundary cutting it diagonally, so that if it were elected to circle a square, rather than concentrate all one's efforts in a single zone, it would be best to choose the two zones for which the times were less than the times involved in parking in any other zone.

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Use of Safety Rest Areas

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The improvement of motor vehicles and highways and the increased use of highways by motor vehicles during recent years has presented an increasingly critical safety problem, particularly with respect to handling vehicles parked adjacent to the traveled way of rural arterial highways. To reduce this hazard, many sections of these highways have been posted to limit parking for emergency purposes only. These restrictions and the desire to eliminate parking hazards require that provision be made to assist motorists who need a rest or other type of stop in finding adequate facilities for their needs.

This study was undertaken to provide data on the amount and character of use of the existing rest areas on the traveled route of the Interstate System, so that information would be available to serve as a basis for determining the need and space for rest areas as well as facilities to be provided in rest areas.

The seven rest areas studied in 1960 were all on non-access controlled portions of the traveled route of the Interstate System. Some of the major findings resulting from the studies were:

1. From 2 to 4 percent of the traffic on the adjacent highway used the rest area. (Limited studies on access-controlled sections have shown this relationship to be as high as 10 percent.)
2. The use of rest areas is inversely related to the portion of highway traffic that is local short-haul traffic and directly related to the remoteness of the rest area with respect to other locations providing the same facilities.
3. The major facilities used by motorists are rest rooms, drinking water, and picnic tables.

• RECENT IMPROVEMENTS in automotive and highway design, accompanied by heavy increases in highway traffic and vehicle speeds, have created a need for the control of and planning for emergency and rest stops on rural arterial highways. Freeways are known to encourage the extension of trips beyond customary lengths and create driver fatigue. The importance of breaking long trips by frequent stops is recognized by safety experts (*1*). Leaving the traffic stream on freeways at other than designated access points, however, has been recognized as a cause of accidents. To decrease this hazard, policies of prohibiting parking on freeway shoulders except in emergencies and providing rest areas as safe stopping places have been adopted. It has become evident that highways must be designed not only for the moving vehicle but for the parked vehicle (*2*).

The current major task in this field

has been the planning of facilities for the emergency, rest, and service stops on the hundreds of miles of full-access control freeway on the Interstate and Defense Highway System. A comprehensive statement of rest area policy and design for the Interstate System was published by AASHO in July 1958 (*3*). However, this publication states that reliable estimates of the amount and type of rest area use are not available and emphasizes the need for such data for the intelligent planning and justification of rest areas and their facilities.

The primary purpose of this study* was to provide data on rest area use in Oregon along Interstate routes. These data then would serve as a basis for determining not only the need for safety rest areas on the Interstate System in Oregon but the spacing and facilities to be provided. Secondary purposes of the study were (a) to obtain information on use of existing rest areas that would be helpful in planning and operating the extensive system of roadside rest areas operated on State highways by the Oregon State Highway Department; and (b) to develop and test procedures for conducting studies of rest area use for the "Rest Area Study Procedure Guide."

This paper presents an analysis and summary of data on rest area use and related information at seven roadside rest areas on US 30 and US 99 during the summer of 1960. No safety rest areas were in operation on full-control access sections of the Interstate System in Oregon in 1960; therefore, existing roadside rest areas on non-access controlled sections of the traveled way of the Interstate System were selected for study.

* The study reported here was based on and tested the "Rest Area Use Study Procedure Guide" prepared by the HRB Committee on Shoulders and Medians and to be published in HRB Bulletin 359.

METHODOLOGY

The studies were conducted essentially with the procedures outlined in "Rest Area Use Study Procedure Guide." This guide was prepared at the request of Committee No. 3 on Shoulders and Medians, Department of Traffic and Operations, Highway Research Board. Although some of the data collected and reported herein do not conform in minute detail to the guide, nevertheless, the basic concepts were adhered to. Stated conversely, the procedure guide was prepared based on experience gained in this study.

Study Sites

A total of 19 Oregon rest areas and roadside parks on US 30 and US 99 (the traveled routes of Interstate 80N and 5) were initially considered as potential study sites. These included 13 completed and operational roadside rest areas, 5 roadside State parks, and 1 new rest area site that had parking and access completed but no facilities or rest area signing installed. A field inspection of each potential study site was made before the final selection. The three major factors considered in the selection of study sites were (a) location, (b) suitability of the site for collection of traffic-volume, interview, and observation data, and (c) the manpower and equipment available for the study.

Location was a prime factor for several reasons. First, it was desired that the study sites be distributed geographically throughout the State so that they would be representative of the various terrain, weather, and population characteristics encountered on the Interstate Highway System in Oregon. Second, it was desired to study locations similar to those that would be encountered in new construction on Oregon's Interstate System.

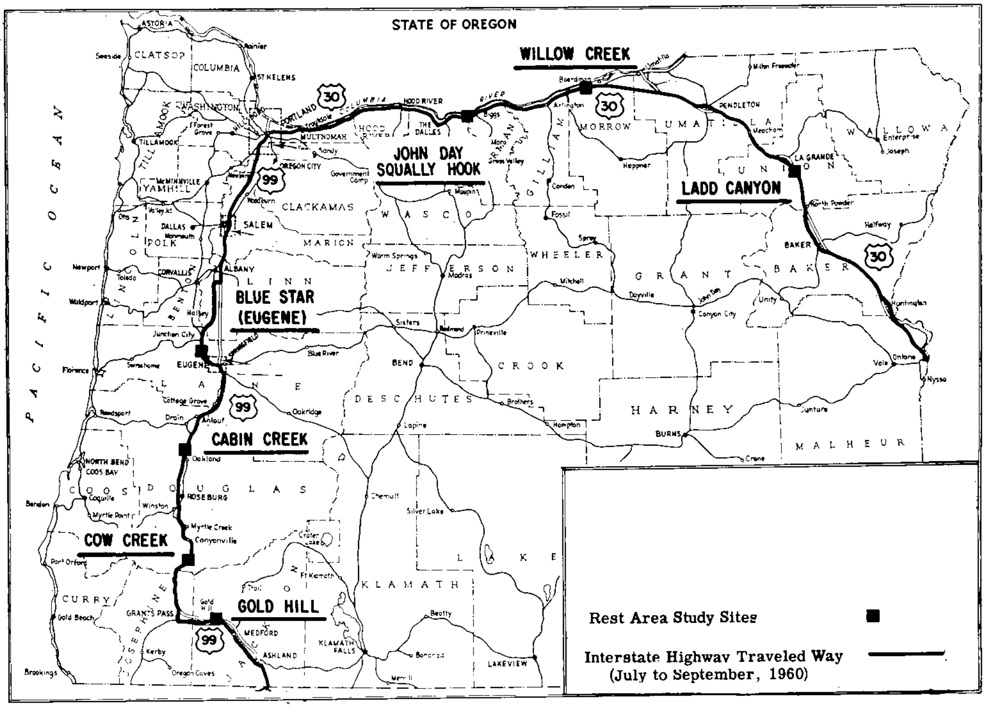

Figure 1. Location of roadside rest area study sites.

The type of access and location of rest area roads were important in determining whether the rest area was suitable for obtaining accurate counts of rest area traffic by traffic recorder equipment. Sites that had highway shoulder access or several exits or entrances were not considered because of the impracticality of the installation of traffic recorder equipment. Rest areas on frontage roads were also considered unsuitable sites.

A limiting factor in sites chosen was the number of automatic traffic recorders and men available for data collection. The physical layout of the rest area was important in determining whether the site was suitable for observation and interviewing by interviewer-observers stationed in the rest area. Figure 1 shows the location of the seven rest area study sites selected. Unfortunately, no completed and operational safety rest area* sites of modern design on completed full-access control sections of the Interstate System were available for study. All study sites available were on non-access control sections of US 99 and US 30, were designed primarily for light vehicle use, were without parking areas and access designed for heavy trucks, had no acceleration or deceleration lanes, and had no signing comparable to Interstate System standards. The sites selected and pertinent data for each are given in Table 1. A case history of a typical site is contained in the Appendix.

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* The term "safety rest area" used in this report refers to rest areas of modern design on Interstate Highways as described in "A Policy for Safety Rest Areas for the National System of Interstate and Defense Highways," AASHO, 1958.

TABLE 1
ROADSIDE REST AREA STUDY SITES

Name and Route	1960 ADT	Location	Facilities
John Day-Squally Hook, US 30, M.P. 115.8	2,950	32 mi east of Dalles City, 21 mi west of Arlington in dry, open country in north-central Oregon	5 picnic tables, 2 dry pit restrooms; shoulder parking on rest area road; no drinking water, fireplaces, or artificial lighting
Willow Creek, US 30, M.P. 159.7	2,600	22 mi east of Arlington, 48 mi west of Pendleton in dry, open country in north-central Oregon	4 picnic tables, 4 dry pit restrooms, gravel parking area, piped drinking water, 2 sun shelters, a small creek; no fireplaces or artificial lighting
Ladd Canyon, US 30, M.P. 268.3	1,950	7 mi southeast of LaGrande, 36 mi northwest of Baker in northeast Oregon in Blue Mountains	8 picnic tables, 2 dry pit restrooms, piped drinking water, paved parking area; no fireplaces or artificial lighting
Blue Star (Eugene), US 99, M.P. 114.8	9,500 [1]	8 mi north of Eugene, 5 mi south of Junction City in the Willamette Valley of west Oregon	8 picnic tables, 2 dry pit restrooms, shoulder parking on paved rest area road, small lake; no drinking water, fireplaces, or artificial lighting
Cabin Creek, US 99, M.P. 175.0	4,500	30 mi south of Cottage Grove, 18 mi north of Roseburg in hills of west Oregon	4 picnic tables, 2 dry pit restrooms, 2 gravel parking areas adjacent to paved rest area road; no fireplaces, drinking water or artificial lighting
Cow Creek, US 99, M.P. 234.1	3,800	14 mi south of Canyonville, 28 mi north of Grants Pass in southwest Oregon	5 picnic tables, 2 dry pit restrooms, gravel parking area and circulatory road; no fireplaces, drinking water, or artificial lighting
Gold Hill, US 99, M.P. 297.2	4,700	18 mi south of Grants Pass, 12 mi north of Medford in southwest Oregon and 1 mi west of Gold Hill on spur of State Secondary 271 in Rogue River Valley	6 picnic tables, 2 dry pit restrooms, drinking water, graveled parking area and road; no fireplaces or artificial lighting

[1] Before rerouting of Interstate 5 traffic on new alignment, November 1960.

Sources of Data

The two primary data sources were automatic traffic recorder counts of rest area and adjacent highway traffic volumes and interview and observation data on rest area use collected by study crews during the summer of 1960.

Traffic Volume Data.—Counts of the number of vehicles entering the rest areas were obtained by installing automatic traffic recorders on the rest area access roads for a period of about two months from about July 1 to September 5, 1960. Counts of traffic volumes on the highway adjacent to the rest area, needed for relating rest area and highway traffic, were obtained from permanent automatic traffic recorder installations or automatic traffic recorders temporarily installed on the adjacent highway. Counts from permanent traffic recorders were used when the rest area was located a short distance from the permanent recorder and there was no significant difference in the traffic volume between the rest area and the permanent counter.

All of the study sites, with the exception of Blue Star (Eugene) Rest Area which had a single entrance, had two entrances which also served as exits for two-way traffic on the rest area roads. Where there was no appreciable difference in the amount of use of either of the two rest area entrances, a single traffic recorder was installed on the rest area road near an entrance. When there was a definite tendency for the rest area occupants to use one entrance (or exit) in preference to the other, as at Cow Creek and Gold Hill Rest Areas, a traffic recorder was installed near each entrance, and the counts were averaged.

The counts from the road tube type of traffic recorder were adjusted to compensate for overcounting due to multiple-axle vehicles such as passenger car-trailer coach combinations or truck-trailer combinations. The adjustments were made on the basis of axle overcount factors derived from classification counts of vehicles by number of axles. These adjustment factors ranged from 0.92 to 0.95.

Interview-Observation Data.—The rest area use interview and observation data were collected at all study sites on 7 days (5 weekdays, a Saturday, and a Sunday) from 10:00 AM to 5:00 PM. The days were distributed over a period starting June 14 and ending August 28, 1960. The interview days for each rest area were scheduled on a staggered basis during the summer to avoid possible bias that a specific set of weather conditions, special events, or seasonality may have on data collected during a limited time period.

The data were collected by a vehicle classifier and an observer-interviewer stationed at the rest area during the survey periods. The latter interviewed an occupant (usually the driver) of vehicles stopping and made observations of rest area use, and the former made visual classification counts of highway traffic by vehicle type and direction of travel. These duties were alternated approximately every 1½ hr. The interview form (Fig. 2) shows the questions asked during the interview. In some instances a log of vehicles entering and leaving the rest area was kept by the vehicle classifier to aid the interviewer. A visual count of restroom use was taken at 5 of the 7 study sites. The lack of time and the location of restroom facilities did not permit making accurate counts of restroom use at the two remaining study sites.

It was not possible or necessary to interview all vehicle-parties stopping in the rest area due to the large volume of rest area stops. However, an interview form was prepared for all vehicles entering the rest area, whether the vehicle-party was interviewed or not. In those instances where no interview was possible data on the interview form was obtainable by observation only; such as the times of entry and departure, vehicle type, vehicle registration, number of occupants, facilities used, and often the purpose of stop.

Persons requesting information pertaining to travel routes, parks, and rest areas were furnished copies of the Oregon State Highway Map or the Oregon Outdoor Guide which showed and described rest areas and park locations and facilities. Pertinent comments from rest area occupants on the operation of the immediate rest area or rest areas in general were noted on interview forms.

After the field data were collected, the interview forms were reviewed, coded, and punch cards prepared. These punch cards served as the primary data source for the tabulations and computations in the analysis.

ANALYSIS

Traffic Volumes

Comparison of Traffic Volumes at Study Sites.—The average daily summer rest area and adjacent highway traffic volumes at the 7 sites are given in Table 2. The 24-hr daily summertime rest area traffic averages ranged from a low of 76 vehicles at Gold Hill Rest Area to a high of 186 at Cabin Creek Rest Area. The average number of vehicles entering rest area study sites on US 99 (132) was not appreciably lower than that of rest areas on US 30 (137). However, the rest areas on US 30 had appreciably higher percentages of highway traffic entering (4.1 percent) than did those on US 99 (2.0 percent). The percentage entering the different rest areas on US 30 was also much more consistent, ranging

TABLE 2
COMPARISON OF HIGHWAY AND REST AREA TRAFFIC VOLUMES

Rest Area	Route	Average Daily Summer Traffic		
		Highway	Entering Rest Area (No.)	(%)
John Day-Squally Hook	US 30	3,591	146	4.1
Willow Creek	US 30	3,533	129	3.7
Ladd Canyon	US 30	3,129	137	4.4
Avg.	US 30	3,418	137	4.1
Blue Star (Eugene)	US 99	10,346	175	1.7
Cabin Creek	US 99	5,932	186	3.1
Cow Creek	US 99	5,239	92	1.8
Gold Hill	US 99	5,800	76	1.3
Avg.	US 99	6,829	132	2.0
Avg. (all sites)		—	—	2.9

from 3.7 to 4.4 percent, whereas the range at the four study sites on US 99 was from 1.3 to 3.1 percent.

Rest areas with the largest amounts of traffic in terms of numbers of vehicles entering were not necessarily those with the highest volume of highway traffic.

The factors apparently responsible for the higher rates of rest area use on US 30 were remoteness and the low proportions of local traffic. The three rest areas on US 30 were located in north-central and eastern Oregon where the daytime temperatures usually exceeded 90 F, and there were long stretches of road in open and sparsely populated country with relatively few opportunities for rest or service stops. The proportions of local traffic on US 30 (*i.e.*, traffic with short trip lengths, and origins and destinations in the vicinity) were low, and conversely, the proportions of intercity and interstate traffic high.

These factors of remoteness and local traffic were also a major influence on the percent of highway traffic entering rest areas on US 99. For example, Cabin Creek Rest Area, which had the highest rate of entering traffic (3.1 percent) of all US 99 study sites, was located 30 mi from Cottage Grove and 18 mi from Roseburg, the nearest cities on US 99. Both cities had been bypassed by the new highway location. Also, the highway north of Cabin Creek to Eugene was on the whole full-access control. Study sites other than Cabin Creek on US 99 were located at much less remote distances from alternate facilities and had considerable local traffic.

It was apparent that the substantial differences in the percent of traffic entering the various study sites tended to reflect effects of differences in the physical location, design, and facilities available at the rest area. A χ^2 test on the numbers of vehicles entering the 7 study sites during the 7-day sample period, taking into account differences in highway traffic volumes, showed the differences between study sites to be highly significant.

There were numerous reasons for the differences in the percent and number of vehicles entering the various study sites, the chief of which appears to be the factor of remoteness. It is, however, difficult to isolate them. The extremely low volume of entering vehicles at Gold Hill Rest Area was attributed to the following:

1. The difficulty of access to the rest area encountered by northbound traffic which was required to use the Gold Hill interchange overcrossing.
2. The lack of signing of the rest area for northbound traffic.
3. The considerable amount of local traffic.
4. The numerous alternate places for stopping by southbound traffic which was routed directly through the business district of Grants Pass only 18 mi north. This southbound

OREGON STATE HIGHWAY DEPARTMENT
Traffic Engineering Division
Planning Survey Section

REST AREA INTERVIEW FORM

Identification:

Serial Number

Highway No.

Rest Area

Day

Date

A. TIME: Entered rest area

Departed rest area

Stay (minutes) ____
(Office Entry)

B. REGISTRATION:
1. Oregon
2. Washington
3. California
4. Idaho
5. Montana
6. Utah
7. Nevada
8. British Columbia
9. Other States & Counties
0. Unknown
Y. Out of state, unidentified

C. DIRECTION:
1. Eastbound
2. Westbound
3. Northbound
4. Southbound

D. NUMBER OF OCCUPANTS:

E. VEHICLE TYPE:
1. Light Vehicle
2. Light Vehicle & Trailer Coach
3. Light Vehicle & Other Trailer
4. Truck or Bus
5. Truck and Trailer Combination

F. LAST STOP:

Time ____

Location ____

Type:
1. Start of Day's Trip
2. Rest Stop
3. Vehicle Service
4. Eat
5. Shopping
6. Other ____ (Explain)

COMMENTS:

____

G. TRIP PURPOSE:
1. Business
2. Driving to or from work
3. Vacation
4. Social, recreation
5. Moving
6. Other ____ (Explain)

H. TYPE AND PURPOSE OF STOP:
1. Rest or Nap
2. Checking Map
3. Changing Drivers
4. Eating
5. Car Sickness or Illness
6. Recreation (Picnic, Fishing, etc.)
7. Restroom or Latrine
8. Deposit Litter or Garbage
9. Other ____ (Describe)
0. Drive through (no stop)

I. FACILITY USE:
Drinking Water

Restrooms

Tables and Benches

Fireplace or Cooking Facility

Shelters

Other ____ (Describe)

J. WAS THIS A STOP ENROUTE?
1. Yes 2. No

K. WHERE WOULD YOU HAVE STOPPED HAD THIS REST AREA NOT BEEN HERE?
1. Shoulder of the Highway
2. Next Service Station Available
3. Next Rest Area or Park
4. City or Town of ____
5. Other ____ (Describe)
6. Do not Know
7. Would not have stopped, no alternative

L. HOW DID YOU LEARN OF OR LOCATE THIS REST AREA?
1. Located from Road Map
2. Located from Road Signs
3. Known from Previous Visits
4. Other ____ (Describe)

Observer ____

Figure 2.

TABLE 3

REST AREA TRAFFIC VOLUME COUNTS [1]

Rest Area	Week Beginning	Traffic Volume						
		Sun.	Mon.	Tues.	Wed.	Thur.	Fri.	Sat.
John Day-Squally Hook	June 26	...	...	...	...	118	183	208
	July 3	153	181	188	167	152	146	149
	July 10	173	145	152	126	121	144	180
	July 17	182	198	136	140	142	152	160
	July 24	151	149	114	149	129	...	...
	July 31	...	...	141	115	125	150	195
	Aug. 7	160	161	156	137	149	140	169
	Aug. 14	170	125	144	145	131	150	152
	Aug. 21	169	125	133	134	123	117	125
	Aug. 28	118	116	96	84	124	143	165
	Sept. 4	123	159	...	...	...	...	...
Ladd Canyon	June 26	...	...	...	...	135	160	179
	July 3	153	174	150	155	163	162	152
	July 10	155	138	128	132	128	124	177
	July 17	174	155	132	162	154	139	131
	July 24	143	138	140	118	141	143	138
	July 31	145	128	149	114	90	131	160
	Aug. 7	140	137	140	130	139	111	169
	Aug. 14	167	124	102	115	149	145	151
	Aug. 21	126	113	135	116	111	94	100
	Aug. 28	127	127	107	86	103	127	139
	Sept. 4	114	153	...	...	...	...	...
Blue Star (Eugene)	June 26	131	156	128	112	136	199	221
	July 3	220	213	207	204	182	203	168
	July 10	177	160	148	159	149	227	188
	July 17	211	194	171	186	224	187	219
	July 24	183	166	173	174	196	181	202
	July 31	213	191	187	181	187	181	194
	Aug. 7	222	196	169	164	136	217	192
	Aug. 14	219	198	178	179	172	178	190
	Aug. 21	189	152	115	153	160	150	169
	Aug. 28	155	129	135	123	131	116	163
	Sept. 4	144	149	...	...	...	...	...
Cabin Creek	June 19	...	...	...	...	...	178	191
	June 26	197	169	186	159	163	207	220
	July 3	209	208	221	249	213	193	231
	July 10	299	213	184	160	157	207	204
	July 17	195	224	176	161	168	172	228
	July 24	181	171	184	147	172	185	168
	July 31	233	199	191	161	182	172	219
	Aug. 7	268	238	177	149	121	191	212
	Aug. 14	204	214	160	160	193	190	186
	Aug. 21	176	125	153	152	157	160	219
	Aug. 28	176	155	124	98	152	230	192
	Sept. 4	159	161	...	...	...	...	...
Cow Creek	June 26	...	...	...	73	87	121	139
	July 3	97	110	102	122	87	102	85
	July 10	111	98	108	85	81	111	125
	July 17	98	94	98	70	79	83	104
	July 24	88	92	92	70	87	100	103
	July 31	83	90	91	64	100	117	115
	Aug. 7	92	100	81	88	99	104	117
	Aug. 14	111	87	94	93	89	69	90
	Aug. 21	90	68	89	78	70	87	98
	Aug. 28	76	75	79	66	52	95	92
	Sept. 4	84	90	...	...	...	...	...
Gold Hill	June 26	...	...	...	65	63	70	94
	July 3	63	68	72	72	59	77	68
	July 10	83	65	76	62	74	81	74
	July 17	69	52	74	74	83	69	67
	July 24	71	64	47	48	65	83	91
	July 31	65	72	79	60	74	78	80
	Aug. 7	89	103	88	105	78	69	109
	Aug. 14	87	92	91	111	70	81	90
	Aug. 21	78	75	81	93	95	62	87
	Aug. 28	63	65	75	64	73	68	76
	Sept. 4	91	78	...	...	...	...	...

[1] Data for Willow Creek Rest Area not available due to traffic recorder trouble.

traffic also had many other opportunities for stopping at private and public service facilities and another rest area and park before reaching this location.

Factors contributing to the comparatively low use of Cow Creek Rest Area were believed to be the nearness of the rest area to Canyon Creek and Packard Creek Rest Areas, located at distances of 11 and 9 mi north of the rest area, respectively. The volume of traffic entering Blue Star (Eugene) Rest Area though substantial in number was low as a percent of highway traffic (1.7 percent). Several elements contributed to the low rate of entry:

1. The rest area was in many instances crowded and operating above normal facility capacity.
2. The high summer traffic volume (10,300) on the adjacent highway included much local traffic that would not require rest area facilities.
3. The rest area was close to many alternate service and eating facilities in nearby cities.

The average daily number of vehicles and percent of highway traffic entering the study sites by day of week were obtained and summarized for each site. This is illustrated for Ladd Canyon Rest Area in the Appendix.

The daily rest area traffic volume counts obtained from automatic traffic recorders are given in Table 3. Saturday was typically the day with the largest amount of rest area traffic. Only at Cabin Creek Rest Area was there a slightly higher number of vehicles entering on Sunday than on Saturday. Small variations were noted in the average percent of daily highway traffic entering the rest areas by day of week. The largest variation occurred at John Day-Squally Hook Rest Area where the weekday average was 3.9 percent compared to 4.4 percent on Saturday and Sunday.

Comparative Traffic Data from Other States.—Limited data on rest area use are available from other States for comparative purposes. Rest area use data compiled by the Bureau of Public Roads from data submitted by 8 States in response to a 1959 questionnaire showed 1 to 12 percent of highway traffic on arterial routes using rest areas (*4*). The percentages of traffic using rest areas reported are given in Table 4.

TABLE 4

TRAFFIC USING REST AREAS IN VARIOUS STATES

State	Traffic Using Rest Areas (%)
South Dakota	1
North Dakota	1.2
California	2.4
Georgia	2.5
Oregon	2.9
North Carolina	3
Virginia	3.8
Tennessee	4
Ohio	12

These data are generally in line with the Oregon data reported in this study (average 2.9 percent), with the exception of that for Ohio. The North Carolina study figure of 10 percent for cars with trailer houses stopping in rest areas (*5*) compares closely with the 10.6 percent for Oregon light vehicles with trailer houses.

More recent surveys made by North Carolina in October 1960 at safety rest areas on the Interstate System show substantially higher usage (*6*). The North Carolina safety rest areas had drinking water, picnic tables, fireplaces, flush toilet restrooms, separate parking spaces for trucks and cars, and were located on 4-lane, divided Interstate Highway sections with full access control. The average percent of highway traffic stopping at the 5 North Carolina rest area survey sites was 5.5 percent for

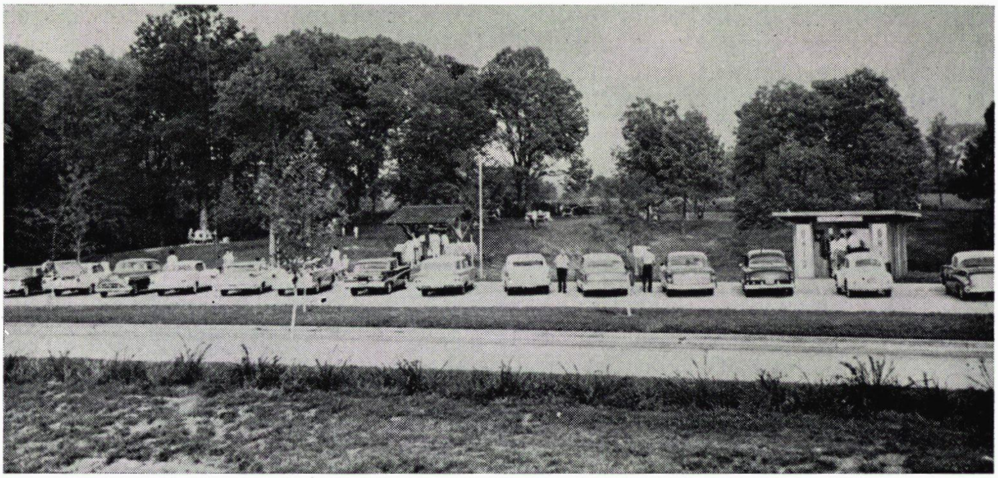

Figure 3. Heavy safety rest area use on Interstate 75, Ohio.

Figure 4. Safety rest area use at night on Interstate 75, Ohio.

cars and 7.6 percent for trucks between 10:30 AM and 4:30 PM. These percentages would have probably been higher during the summer months. Ohio's (*6*) traffic volume data collected in the summer of 1959-60 at modern safety rest areas on the Interstate System consistently showed substantially higher percentages of traffic entering rest areas (6.5 to 17.4) than the 1960 Oregon data.

Heavy use of safety rest areas observed in 1961 on Interstate 75 in Ohio is shown in Figures 3 and 4, with fully occupied parking area and line-up of persons using the well in Figure 3. Heavy usage of restrooms was also reported. Rest area use at night is shown in Figure 3, where the amount of nighttime highway traffic using the rest area increased from 8 to 12 percent when overhead lighting was turned on. About 75 percent of those interviewed indicated they would not have stopped had there been no lighting (7).

On the basis of these comparisons, use of the new Oregon safety rest areas on the Interstate System will be substantially greater than at the existing Oregon roadside rest areas surveyed in this study. This premise has been substantiated by data collected in July, 1961 at a newly opened Oregon safety rest area on Interstate 5 where the percentage of highway traffic entering was approximately 5 percent during its first month of operation, despite temporary facilities in comparison to 2 percent at rest areas on US 99, as shown by this study.

Rest Area Use

The rest area use data that follow are based on interview and observation data collected at each study site from 10 AM to 5 PM (Pacific Standard Time) on 7 summer days in 1960. The interview-observation periods included 5 weekdays, a Saturday, and a Sunday. The weather conditions during the study period were typical of Oregon summers (clear, dry, and warm) and the rest area use observed was not significantly affected by any unusual weather conditions.

Vehicles Entering.—For design of parking, the successful planning and operation of rest areas require knowledge of the number and types of entering vehicles that will be accumulated in the rest area. Relationships between peak vehicle accumulations and daily number of vehicles entering the rest area provide information that in addition to being of value in the analysis of existing rest area use may be used to indicate potential peak use for design of new rest area facilities.

TABLE 5
RELATIONSHIP OF VEHICLES ACCUMULATED TO THOSE ENTERING FOR AN AVERAGE SUMMER DAY

Rest Area	Route	Minimum Accumulation (%)[1]	Accumulation During Peak Moment (%)[1]	Avg. Accumulation During Peak Hour	
				(%)[1]	For Hour Beginning
John Day-Squally Hook	US 30	1.1	7.4	5.4	1,200
Willow Creek	US 30	1.8	7.3	4.7	1,145
Ladd Canyon	US 30	0.8	7.0	3.4	1,320
Avg.	US 30	1.2	7.1	4.5	—
Blue Star (Eugene)	US 99	2.5	10.1	8.4	1,215
Cabin Creek	US 99	1.2	6.3	4.5	1,155
Cow Creek	US 99	1.2	6.8	3.8	1,405
Gold Hill	US 99	0.9	10.3	6.3	1,205
Avg.	US 99	1.4	8.2	5.8	—
Avg. (all sites)		1.9	7.9	5.2	—

[1] Of daily traffic accumulated during 5-min intervals.

Average Vehicles Accumulated.—The relationship of vehicles accumulated in the rest areas to vehicles entering for an average summer day is shown in Table 5. The vehicle accumulations were computed for 5-min intervals from the entrance and exit times of vehicles stopping in the rest area from 10:00 AM to 5:00 PM. The vehicle accumulations tended to be least near the beginning and end of the 10:00 AM to 5:00 PM study period, and most around or during the noon hour. The minimum vehicle accumulations occurred predominantly in the late afternoon after 4:00 PM and averaged 1.9 percent of the daily rest area traffic. The percentage of daily rest area traffic accumulated during the peak hour ranged from a low 3.4 percent at Ladd Canyon to a high of 8.4 percent at the Blue Star (Eugene) Rest Area, with an all-sites average of 5.2

percent. The average number of vehicles accumulated during the peak hour ranged from a low of 4 at Cow Creek to a high of 15 at Blue Star (Eugene). The peak hour at 5 of the 7 locations occurred during hours from 11:45 AM to 12:15 PM. The other two locations (Cabin Creek and Ladd Canyon) did not have significant peaking of vehicle accumulations, but continuing moderate use from noon throughout the early afternoon.

The differences in the percent of vehicles accumulated at the various study sites reflect varying types of use and lengths of stay. For example, the much sharper peaks and higher percentages of vehicle accumulations observed at Blue Star (Eugene) and Gold Hill Rest Areas reflect the higher proportions of eating stops at these sites and longer durations of stay. Conversely, the lower accumulation percentage shown for Cow Creek reflects the higher proportion of restroom stops observed which were of shorter duration.

Daily Peak-Moment Accumulation.—Knowledge of rest area use under peak conditions is required to provide information for adequate planning of facilities as the amount of use varies considerably during the day and year. Table 5 gives the relationship of the number of vehicles accumulated in the study sites at the peak moment to the average daily number entering. The peak moment was defined as the 5-min interval during which the number of vehicles accumulated in the rest area was maximum. The peak-moment accumulation used in this analysis was the average of the peak moments for the 7 days studied at each rest area.

Nine vehicles, or 7.1 percent of the 24-hr traffic volume entering the rest areas, were accumulated during the peak moment at US 30 study sites and 11 vehicles, or 8.2 percent, at US 99 study sites. The peak-moment accumulation ranged from a low of 6 vehicles at Cow Creek Rest Area to a high of 19 vehicles at the Blue Star (Eugene) Rest Area. The all-sites average was 10 vehicles, or 7.9 percent of the daily rest area traffic. A factor limiting the peak-moment vehicle accumulations was that some rest areas were operating at or above the practical facility capacity during the peak hours.

As to the use of the average peak-moment data for design purposes, the peak-moment accumulation is not an extreme measure. Standard design practices normally preclude the use of extremes for design. An example is the standard highway design practice of designing roadway capacity for the 30th highest hourly volume (*8*). As the peak moment data shown are those for a 7-day period, they will probably be less than the peak moment of the summer because of the absence of holidays and size of the sample. (Statistical analysis of the variability of the daily peak-moment accumulations at the individual study sites indicated the sample means were within 12 to 23 percent of the true mean with a confidence level of 95 percent.)

Vehicle Type.—Nearly all the use of the rest areas was by light vehicles. The rest area traffic was composed of 85.9 percent single light vehicles, 8.0 percent light vehicle-trailer coach combinations, 4.0 percent light vehicle-other trailer combinations, and only 2.1 percent trucks and busses as shown by Table 6. This pattern of rest area use by vehicle type showed only minor variations between the individual study sites and there was no significant difference between the pattern of rest area use by vehicle type of sites grouped by highway. The predominant use by light vehicles was consistent with the design of the rest areas, all of which were of older design, planned primarily for passenger car use.

An analysis of the relationship of

TABLE 6
CLASSIFICATION OF TRAFFIC, ALL STUDY SITES

Vehicle Type	Vehicles Entering Rest Area (No.)	Vehicles Entering Rest Area (%)	Highway Traffic (No.)	Highway Traffic Entering Rest Area (%)
Light vehicle	2,865	85.9	103,955	2.8
Light vehicle and trailer coach	266	8.0	2,508	10.6
Light vehicle and other trailer	135	4.0	2,973	4.5
Bus or single-unit truck	60	1.8	4,867	1.2
Truck-trailer combination	10	0.3	6,756	0.1
All	3,336	100.0	121,059	2.8

the distribution of rest area traffic and the adjacent highway traffic by vehicle type indicated that the light vehicle was proportionately a larger user of rest areas than trucks and busses. Expressed as the percentage of each type of highway traffic entering rest areas during the 10:00 AM to 5:00 PM study period, light vehicle-trailer coaches were the heaviest rest area users (10.6 percent), light vehicle-other trailer next (4.5 percent) and single light vehicles (2.8 percent). Only 1.2 percent of the bus or single unit truck traffic and 0.1 percent of the truck-trailer traffic entered the rest areas.

A considerably larger amount of truck use at new Oregon safety rest areas is anticipated than shown previously, inasmuch as superior access, parking, and other facilities will allow as well as encourage use by trucks. For example, the percent of truck traffic stopping at 4 safety rest areas on North Carolina's Interstate Highways averaged 7.6 percent during 10:30 AM to 4:30 PM study period (*6*). Also a 1-day study of rest area use on the opening day of an Oregon safety rest area on a completed portion of Interstate 5 showed 5 percent of the truck traffic stopping.

Rest areas were especially attractive to light vehicle trailer-coach combinations as they travel at slower speeds, carry their own food supplies, and find parking difficult in cities. A number of comments were received during the survey relative to the need for parking facilities designed for light vehicle-trailer coaches. Trucks transporting large residence type trailers were classified as truck-trailer combinations and there was no use of rest areas observed for this type of vehicle combination.

Vehicle Occupancy.—Data collected during the rest area survey showed an average of 3.0 persons per vehicle entering the rest area study sites (Table 7). The persons per vehicle averages were remarkably stable among the rest area study sites, and ranged from a low of 2.8 persons per vehicle at Gold Hill, to a high of 3.1 persons reported for John Day-Squally Hook, Willow Creek, and Blue Star (Eugene) Rest Areas.

Comparison of these data with other sources indicates the number of persons per vehicle in rest areas is substantially larger than the average passenger car occupancy on Oregon

TABLE 7
VEHICLE OCCUPANCY

Rest Area	Route	No. of Vehicles Entering [1]	No. of Persons [1]	Persons per Vehicle
John Day-Squally Hook	US 30	71	221	3.1
Willow Creek	US 30	63	197	3.1
Ladd Canyon	US 30	56	171	3.0
Avg.	US 30	63	196	3.1
Blue Star (Eugene)	US 99	93	287	3.1
Cabin Creek	US 99	91	269	3.0
Cow Creek	US 99	46	136	3.0
Gold Hill	US 99	36	102	2.8
Avg.	US 99	66	198	3.0
Avg. (all sites)		65	198	3.0

[1] Excludes vehicles entering for which occupancy data were not obtained.

highways. Oregon vehicle occupancy data collected for a special study during July 1960 showed an average of 2.1 persons per passenger car on US 99 near Ashland and 2.4 persons per passenger car 10 mi east of Pendleton on US 30.

The rest area occupancy average of 3.0 persons per vehicle is more comparable to tourist-party vehicle occupancy averages. A comparable average of 3.1 persons per outbound tourist party was reported for Montana in 1958 (*9*). However, it is considerably lower than the size of vehicle-parties using Oregon recreation parks (5.0 persons) (*10*).

The higher persons per vehicle ratio reflects a large amount of family use of rest areas. Comments from rest area occupants verified the advantages of rest areas for family use, particularly for eating and rest stops. The opportunities for children to run and play with freedom during stops, and the fact that rest area stops for eating were easier on budgets were cited as major advantages over private facilities.

Vehicles Stopping.—Analysis of data on vehicles stopping in rest areas revealed that 91 percent of all vehicles entering the rest areas during the study stopped in the rest area and the other 9 percent drove through without stopping (Table 8). Although the percentages of stopping vehicles ranged from a low of 83 percent at Cabin Creek to a high of 96 percent at Gold Hill, there was no significant difference in the average proportions stopping in rest areas on US 30 and US 99.

Although occupants of "drive thru" vehicles were not interviewed, it was evident from observation and passing comments of drivers that there were several common factors contributing to the significant numbers of drivers entering the rest areas but failing to stop. The following are the more apparent and important reasons, based on the observations of interviewer-observers during the survey:

TABLE 8

RELATIONSHIP OF VEHICLES STOPPING TO THOSE ENTERING REST AREAS

Rest Area	Route	Daily Number of Vehicles		Percent Stopping in Rest Area [1]
		Entering	Stopping	
John Day-Squally Hook	US 30	76	68	89
Willow Creek	US 30	65	61	94
Ladd Canyon	US 30	57	52	91
Avg.	US 30	66	60	91
Blue Star (Eugene)	US 99	97	88	90
Cabin Creek	US 99	97	81	83
Cow Creek	US 99	47	43	91
Gold Hill	US 99	36	35	96
Avg.	US 99	69	62	90
Avg. (all sites)		68	61	91

[1] Based on unrounded data.

1. Certain facilities, such as drinking water, restrooms and overnight camp sites, were not available at the rest areas or could not be seen, and so the parties desiring to use these facilities did not stop.
2. At certain times of the day the rest area facilities were crowded or at capacity use, thus discouraging vehicle-parties from stopping.
3. The rest area access roads were used as turn-around points to reverse direction of travel.
4. Travelers were curious and/or apparently wish to observe Oregon rest area sites or were looking for recreational parks.

Major factors in the low percentage of entering vehicles stopping at Cabin Creek Rest Area (83 percent) were the dense foliage which obscured rest area facilities from view of entering vehicles, and the lack of drinking water. It is believed particularly significant that the three study sites with the highest percent-

TABLE 9

PERCENTAGE DISTRIBUTION OF VEHICLES STOPPING IN REST AREAS BY PLACE OF REGISTRATION

Place of Registration	Percent of Total
Oregon	39.8
Washington	15.5
California	21.9
Idaho	3.6
Montana	0.5
Utah	0.9
Nevada	0.1
British Columbia	2.1
Other States and countries	14.4
Out-of-state, unidentified	1.2
All places	100.0

age of entering vehicles stopping were those with drinking water. A minor cause of vehicles driving through the rest areas without stopping was believed to have been the lack of shaded parking on hot days.

State of Registration.—An analysis of the places of registration of vehicles stopping in the rest areas indicated that 39.8 percent of the summer use of Oregon rest areas on US 30 and US 99 (the traveled routes of the Interstate Highway System) was by Oregon residents and 60.2 percent by out-of-state users. Table 9 shows that the States other than Oregon having the largest proportions of rest area use were the adjacent States of California with 21.9 percent, Washington with 15.5 percent, and Idaho with 3.6 percent.

In relation to the volume of light vehicle traffic on the adjacent highway, out-of-state vehicles were proportionately greater users of rest areas than Oregon light vehicles (see Table 10). Out-of-state light vehicles constituted only 47.0 percent of the light vehicle highway traffic adjacent to the rest area, but were 61.1 percent of the light vehicle rest area stops. Conversely, Oregon light vehicles accounted for only 38.9 percent of the rest area stops, but 53.0 percent of the adjacent light vehicle highway traffic. Analysis by study site showed the percent of out-of-

TABLE 10

COMPARATIVE DISTRIBUTION OF LIGHT VEHICLE REST AREA STOPS AND HIGHWAY TRAFFIC BY PLACE OF REGISTRATION

Place of Registration	Rest Area Stops		Highway Traffic	
	No. of Light Vehicles	Percent of Total	No. of Light Vehicles	Percent of Total
State	1,141	38.9	57,955	53.0
Out-of-state	1,790	61.1	51,481	47.0
Total	2,931	100.0	109,436	100.0

state light vehicles using the rest areas was significantly higher at all study sites other than Willow Creek on US 30 and Cow Creek on US 99.

The lowest proportionate use by Oregon-registered light vehicles occurred at the Blue Star (Eugene) and Gold Hill Rest Areas where a large part of the highway traffic was of a local nature. It is apparent that the percent of highway traffic using safety rest areas will be inversely proportionate to the amount of local traffic.

Intervals Between Stops.—Figures 5 and 6 provide data on the time and distance traveled by rest area users since last stop before the rest area stop. The data are indicative of needed spacing of rest areas to satisfy the stopping habits of rest area users. The data shown for US 30 and US 99 are based on the composite data collected at the study sites on the respective routes.

A remarkable consistency was observed in the composite pattern of distances traveled since last stop by rest area users on the two highway routes as shown by Figure 5. This occurred despite the considerable differences in the type of country traversed, distances between cities, rest area locations, and frequency of alternate stopping places near rest areas on US 30 and US 99. At US 30 rest areas, 44, 62 and 76 percent of the vehicle-parties had traveled distances of 60, 45 and 30 mi or more, respectively, since last stop. The corresponding percentages of US 99

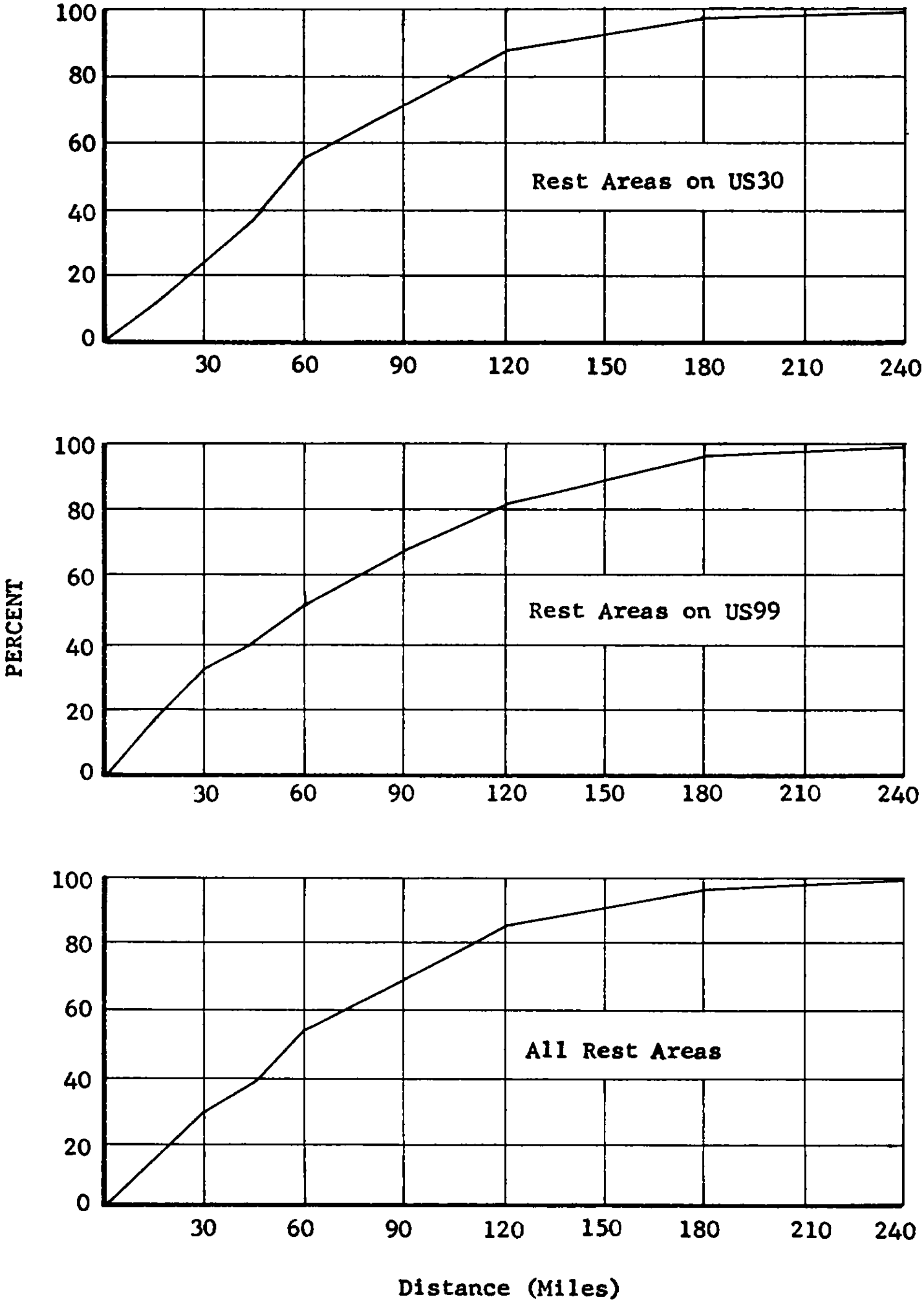

Figure 5. Cumulative percentage of vehicle-parties stopping in rest areas by distance traveled since last stop.

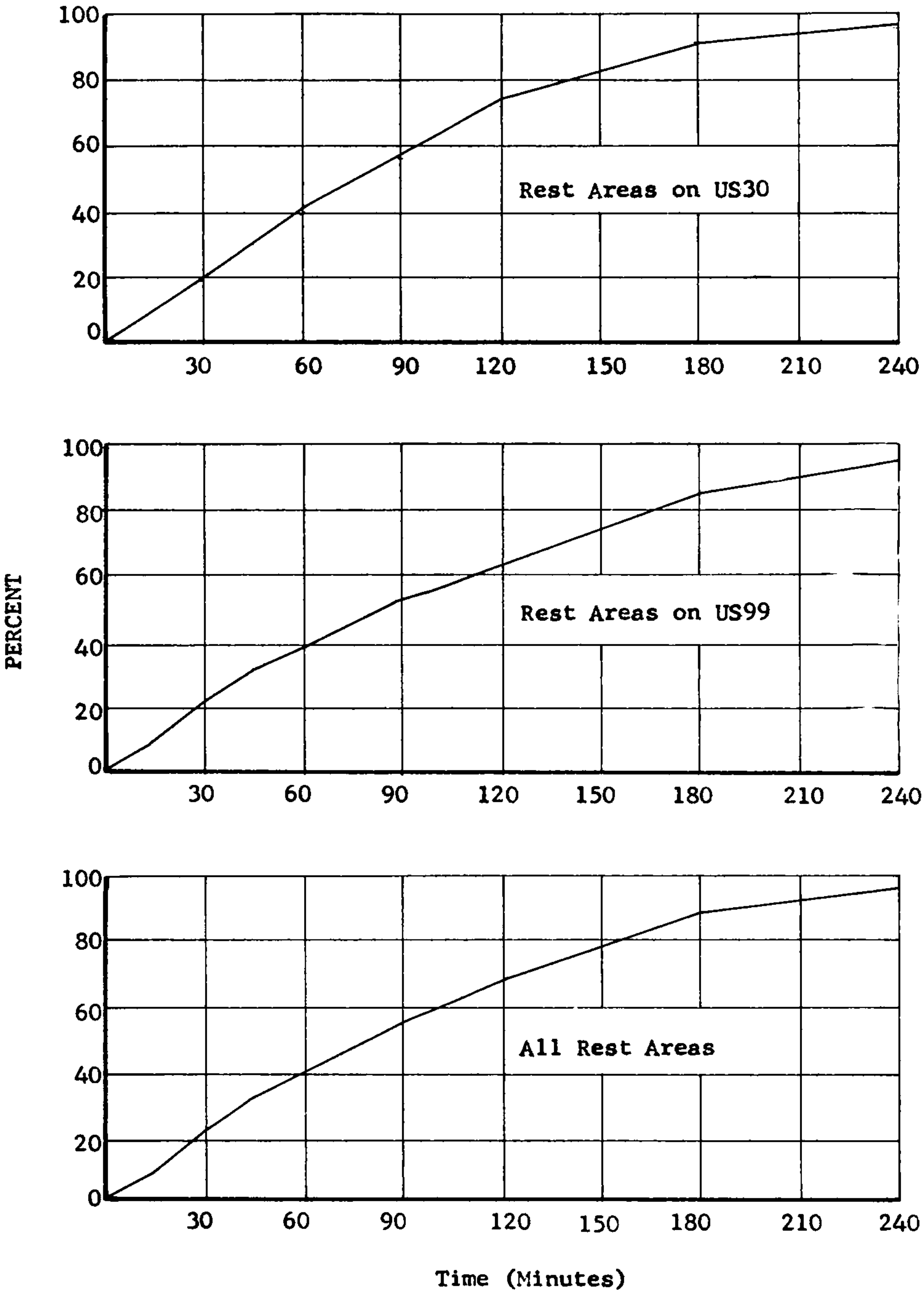

Figure 6. Cumulative percentage of vehicle-parties stopping in rest areas by time traveled since last stop.

rest area users traveling equivalent distances were 46, 60 and 66 percent.

The current policy of the Bureau of Public Roads states that safety rest areas on the Interstate System will be spaced at distances not closer than approximately 45 mi, when gas stations, hotels, restaurants, and other similar facilities are available on roads connecting on interchanges, and at minimum of 30 mi when such facilities are not available (*11*). Rest areas at these distances would make available rest area facilities to satisfy the present stopping practices of 60 and 70 percent of the rest area users, respectively, as shown by Figure 5.

In regard to the spacing of safety rest areas in respect to travel time, the AASHO policy recommends the location of safety rest areas be available approximately every one-half hour of driving time (*3*). As indicated by the time interval data since last stop in Figure 6 for all rest areas, 80 percent of the vehicle-parties have traveled 30 min or more since their last stop. Thus, spacing of rest areas according to the AASHO policy would have made available rest area facilities to accommodate the stopping practices of approximately 80 percent of the Oregon rest area users.

In the analysis of the distribution of rest area stops by mileage and time intervals since last stop, it is significant that the data were determined on the basis of travel on mostly non-access controlled highways. It is reasonable to assume that larger proportions of the highway traffic will stop at greater distance intervals on full access freeways than on non-access control sections due to higher vehicle speeds on freeways. Assuming average freeway speeds of 60 mph, spacing of rest areas at 30 and 45 mi would have made rest facilities available for 70 to 80 percent of the rest area users in terms of travel time between stops (Fig. 6).

Analysis of the individual study data showed the time and distance between stops were influnced significantly by the location of major cities on the highway route and the distance from them to the rest area, as the larger cities serve as major stopping points and trip origins. Small towns, parks, and other rest areas had much less influence on the intervals since last stop. In the combined data for all study sites the effects of the location of certain cities in respect to certain rest areas has been diminished and the composite data portray more accurately the typical intervals between stops of rest area users.

The average distance traveled since last stop was 66 mi on US 30 and 75 mi on US 99. The average time interval since last stop was 90 and 102 min on US 30 and US 99, respectively.

Length of Stop.—Information on the length of rest area stops is of value in the analysis of rest area use as the length of stay affects the accumulation of vehicles in the rest area. It is also indicative of the length of time that certain facilities are occupied, such as parking space and table-bench units. Figure 7 shows the cumulative percentage distribution of rest area stops by length of stop. The average length of stay was 43 min for all stops, 33 min for rest or nap stops, 38 min for eating stops, and 10 min for restroom stops.

The average stop length was influenced by a relatively few stops of long duration, mainly overnight stops. The permissive length of stop in an Oregon rest area was 18 hr. Approximately 49 percent of all stops were less than 20-min duration and 64 percent less than 30 min. Stops of less than 30-min duration accounted for 67 percent of the rest or nap stops, 40 percent of eating stops, and 96 percent of restroom stops.

Among the rest areas there were pronounced differences in the average length of stay. The average length of

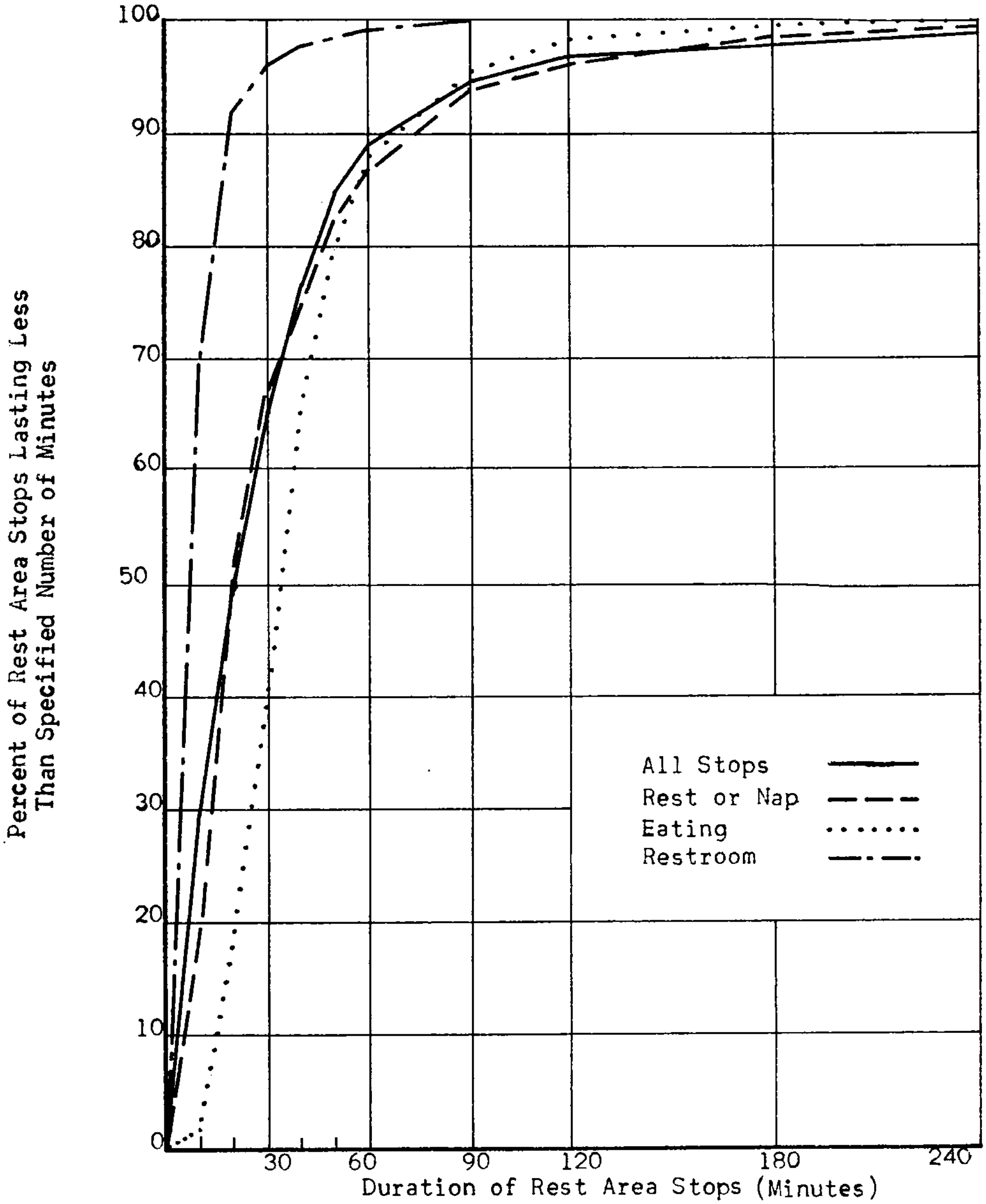

Figure 7. Length of stop, by purpose.

stay at the Gold Hill and Blue Star (Eugene) Rest Areas was significantly longer than at the remaining five rest areas due primarily to the higher proportion of overnight stops. The average stay was 78 and 68 min, respectively, at the Blue Star (Eugene) and Gold Hill Rest Areas. At the five other sites the average stay ranged from 27 to 33 min. Stops by seasonal workers accounted for much of the overnight use which lengthened the average stay. The Blue Star (Eugene) Rest Area average was also affected by the longer length of recreational stops. It was also apparent that the scenic setting of the Blue Star (Eugene) Rest Area had the effect of prolonging the length of stay for eating and rest stops.

Primary Purpose of Stop.—The percentage distribution of vehicle-parties stopping in the rest area study sites classified by primary purpose of stop is given in Table 11. The largest single purpose of stop was for eating, with an average of 35.5 and 39.9 percent of the vehicle-parties using rest areas on US 30 and US 99 classified in this category. Morning and afternoon lunch or "coffee break" stops were included in the eating stop purpose as well as the regular noontime lunch stops. The three major classes of stops (eating, rest or nap, and restroom) accounted for approximately 84 percent of the rest area stops.

The percentage of stops primarily for drinking water are shown at Ladd Canyon, Willow Creek, and Gold Hill rest areas only, as these were the only sites with drinking water installed. Reliable data on the percentage of vehicle parties stopping primarily for drinking water at other sites are not available. Drinking water was considerably more important to rest area users than indicated by the 8 percent average of drinking-water stops at the three sites with this facility. Heavy use and demand for drinking water was shown by facility use data (Table 12) and the comments of rest area users.

The category "other stops" included miscellaneous stops such as checking vehicle, reading historical markers located in rest areas, changing drivers, litter deposits, checking for campsites, and overnight stops. Overnight stops were permitted although signs in the rest areas limited occupancy to 18 hr.

The incidence of the use of rest areas for recreation was extremely low. Only 1.1 percent of all the rest area stops were classified in this category. Recreational use was defined as use of the rest area for the primary purpose of activities such as fishing, picnicking, swimming, or sunbathing. It did not include lunch

TABLE 11

PERCENTAGE DISTRIBUTION BY PRIMARY PURPOSE OF STOP

Rest Area	Route	Primary Purpose of Stop (%)						
		Rest or Nap	Eating	Rest-room	Drinking Water	Recreation	Other or Unknown	All Stops
John Day-Squally Hook	US 30	21.6	39.5	28.6	—[1]	—	10.3	100.0
Willow Creek	US 30	18.1	36.0	27.1	5.4	0.7	12.7	100.0
Ladd Canyon	US 30	14.8	31.0	27.7	11.0	1.3	14.2	100.0
Avg.	US 30	18.2	35.5	27.8	5.4	0.7	12.4	100.0
Blue Star (Eugene)	US 99	22.0	49.1	8.8	—[1]	4.5	15.6	100.0
Cabin Creek	US 99	22.7	37.4	31.7	—[1]	0.1	8.1	100.0
Cow Creek	US 99	25.6	30.2	34.6	—[1]	1.0	8.6	100.0
Gold Hill	US 99	24.1	40.8	15.1	8.6	—	11.4	100.0
Avg.	US 99	23.6	39.4	22.5	2.2	1.4	10.9	100.0
Avg. (all sites)		21.3	37.7	24.8	3.6	1.1	11.5	100.0

[1] Drinking water not available in rest area.

TABLE 12
PERCENTAGE DISTRIBUTION BY TYPE OF FACILITY USED

Rest Area	Route	Type of Use (%)[1]						
		Drinking Water	Restrooms	Tables and Benches	Shelters	None	Unknown	Total
John Day-Squally Hook	US 30	—[2]	68	25	—[2]	22	1	100
Willow Creek	US 30	51	67	33	19	14	*x*	100
Ladd Canyon	US 30	60	63	28	—[2]	16	*x*	100
Avg.	US 30	56	66	29	19	17	*x*	100
Blue Star (Eugene)	US 99	—[2]	53	45	—[2]	23	8	100
Cabin Creek	US 99	—[2]	60	29	—[2]	28	1	100
Cow Creek	US 99	—[2]	70	27	—[2]	18	—	100
Gold Hill	US 99	67	56	44	—[2]	7	—	100
Avg.	US 99	67	60	36	—	19	2	100
Avg. (all sites)		59	62	33	—	18	2	100

[1] Percent of vehicle-parties stopping that used designated facility. Percentages do not add to 100 percent as party may use more than one facility, percentages less than 0.5 percent indicated by *x*.
[2] Facility not available at rest area.

or refreshment stops made by vacationists en route to another destination. The Blue Star (Eugene) Rest Area had the only significant amount of recreational use (4.5 percent) due to its attractive location around a small lake bordered by trees which invited its use for swimming, fishing, and sunbathing. The attractiveness of the site also had an effect on the proportion of stops for eating (49 percent) which was significantly higher than average (38 percent). Also, the percentage of stops for the primary purpose of restroom use (9 percent) at this rest area was much lower than the average (25 percent). The low rates of recreational use at all study sites conclusively demonstrated that the use of roadside rest areas as parks by local population was extremely low.

Facility Use.—The use of installed rest area facilities related to the vehicle-parties stopping in the rest area is given in Table 12. Extensive use and need for restrooms, water, and table-bench units (in that order) are reflected in the high proportions of vehicle-parties using these facilities. The percentage of parties that used at least one of these three major facilities ranged from 72 to 93 percent at the seven rest areas. Restrooms were the most-used facility in terms of vehicle-parties (62 percent), with drinking water (59 percent) a close second.

Substantially more table use than the 33 percent shown would have occurred had there been more units available for peak-hour use. Numbers of potential users were forced to eat in cars or on the ground, or were turned away during peak hours. These observations are confirmed by Table 11 showing that 37.7 percent of the vehicle-parties stopped to eat.

The use of table-bench units is apparently related to the attractiveness of the rest area site. Gold Hill and the Blue Star (Eugene) Rest Areas, which appeared to be the most attractive rest area sites, had the highest percentages of table use. John Day-Squally Hook had the lowest rate of table-bench use. This rest area was probably the least attractive rest area site and at times there was considerable wind and dust which deterred use of tables for eating.

Despite the very hot, dry location of Willow Creek, the percent of parties using the drinking water facility was the lowest observed at the three sites with water. The inconspicuous location of the Willow Creek drinking water was believed responsible for this lower rate of use. Of particular concern to occupants of rest areas without drinking water was the lack of this facility.

Table Use.—The relationship of the number of vehicle-parties stopping to eat during the noon hour (the typical peak period of table use) to the total number of vehicle-parties stopping during the noon hour is given in Table 13. The percentage of parties stopping to eat during the noon hour did not differ appreciably for the rest area study sites on US 30 (58.9 percent) and US 99 (58.3 percent). There were, however, substantial differences between individual rest areas. Those parties stopping to eat represent potential users of table-bench units, although not all of them used the facilities due to lack of sufficient tables and use of trailer houses for eating. In addition to the use of tables for eating, parties stopping primarily to rest or to use the rest area for overnight stops also occupied tables when available, thus using the tables for a disproportionate period of time.

The average percentage of vehicle-parties stopping to eat during the noon hour may be used as a factor in connection with the "peak-moment" vehicle accumulation data to estimate the table-bench unit requirements.

Number of Persons Using Restrooms.—The number of persons observed using restrooms in relation to the number of vehicle-parties entering the rest area is given in Table 14. These data are based on an average 7-hr observation period. Restroom use of 1.5 persons per vehicle was observed at the three rest areas on US 30 and 1.2 persons per vehicle at two study sites on US 99. The average was 1.4 persons per vehicle for all sites. Factors contributing to the lower restroom use rate (1.3 persons per vehicle) observed at Ladd Canyon were the number of parties stopping only for drinking water or to read the historical marker. Factors attributing to the lower restroom use rate (1.1 persons per vehicle party) observed at Gold Hill were the stops for drinking water only and

TABLE 13

RELATIONSHIP OF VEHICLE-PARTIES STOPPING DURING NOON HOUR TO THOSE STOPPING TO EAT

Rest Area	Route	Vehicle-Parties: Number Stopping	Vehicle-Parties: Number Stopping To Eat	Percent of Vehicle-Parties Stopping During Noon Hour to Eat (Potential Peak Table Use)
John Day-Squally Hook	US 30	12	8	65.3
Willow Creek	US 30	8	4	55.7
Ladd Canyon	US 30	10	5	55.7
Avg.	US 30	10	6	58.9
Blue Star (Eugene)	US 99	20	15	74.1
Cabin Creek	US 99	13	8	60.2
Cow Creek	US 99	8	3	35.5
Gold Hill	US 99	7	5	63.5
Avg.	US 99	12	8	58.3
Avg. (all sites)		11	7	58.6

TABLE 14
RELATIONSHIP OF PERSONS USING REST ROOMS TO VEHICLE-PARTIES ENTERING

Rest Area	Route	Number of Vehicle Parties Entering [1]	Number of Persons Using Restrooms [1]	Restroom Use Persons per Vehicle-Party
John Day-Squally Hook	US 30	76	122	1.6
Willow Creek	US 30	65	103	1.6
Ladd Canyon	US 30	57	76	1.3
Avg.	US 30	66	100	1.5
Blue Star (Eugene) [2]	US 99	—	—	—
Cabin Creek [2]	US 99	—	—	—
Cow Creek	US 99	47	60	1.3
Gold Hill	US 99	36	42	1.1
Avg.	US 99	42	51	1.2
Avg. (all sites)		56	81	1.4

[1] Daily 7-hr average, 10:00 AM to 5:00 PM.
[2] Data on number of persons using restrooms not collected.

the lower number of persons per vehicle observed entering the rest area.

It was not possible to obtain accurate counts on the number of persons using restrooms at Cabin Creek as dense foliage obscured the restrooms from view of the observer. At the Blue Star (Eugene) Rest Area, the heavy volume of interviews and the location of restrooms did not permit accurate counts of users. Interviewers commented that restroom use was very heavy at the Cabin Creek and Blue Star (Eugene) Rest Areas.

A comparison of the vehicle occupancy ratio (3.0 persons per vehicle) with the restroom use persons per vehicle-party ratio (1.4) indicated that slightly less than one-half of the persons entering the rest areas used restrooms.

Means of Locating Rest Areas.— Answers to the interview question, "How did you learn of or locate this rest area?" provide an indication of the effectiveness of rest area signing, the use of maps, and other means in guiding the motorist to rest areas. The importance of proper rest area signing was shown by the answers. An average of 61.3 percent had located the rest areas by signs, 28.0 percent by previous visits, 1.4 percent by road maps, 1.8 percent by other means (mainly word-of-mouth from service station attendants, store clerks, or friends), with 7.5 percent unknown. More repeat use of the rest areas on US 30 was indicated by the larger percentage of location by previous visits (33. 7 percent) than on US 99 (23.8 percent) (Table 15).

Analysis of the distribution of vehicles stopping at the various rest area sites, classified by the means by which the rest areas were located, showed statistically significant differences between the rest areas. The proportion of vehicles locating the rest areas by signs was much lower than average at Ladd Canyon, US 30, with a corresponding increase in the proportion that stopped because of previous visits. Conversely, the rest areas at Cabin Creek and Cow Creek had substantially less use by vehicles through previous visits.

Analysis of the distribution of vehicle stops at each rest area by direction of travel showed no significant difference in the means by which rest areas were located by travel

TABLE 15

MEANS OF LOCATING REST AREA

Rest Area	Route	Means (%) Road Map	Road Signs	Previous Visits	Other	Un-known	All Stops (%)[1]
John Day-Squally Hook	US 30	1.9	55.9	31.9	1.5	8.8	100.0
Willow Creek	US 30	1.6	61.2	30.1	0.7	6.4	100.0
Ladd Canyon	US 30	0.5	46.6	39.2	3.8	9.9	100.0
Avg.	US 30	1.3	54.6	33.7	2.0	8.4	100.0
Blue Star (Eugene)	US 99	1.8	57.6	28.4	2.9	9.3	100.0
Cabin Creek	US 99	0.4	70.2	21.5	0.5	7.4	100.0
Cow Creek	US 99	2.7	72.1	16.9	1.7	6.6	100.0
Gold Hill	US 99	0.8	65.7	28.2	1.2	4.1	100.0
Avg.		1.4	66.4	23.8	1.6	6.8	100.0
Avg. (all sites)		1.4	61.3	28.0	1.8	7.5	100.0

[1] Percentage of vehicles stopping based on daylight (10:00 AM to 5:00 PM) summertime observations over 7-day period.

direction except at Ladd Canyon and Gold Hill Rest Areas. The absence of northbound signing at Gold Hill Rest Area was responsible for a lower proportion of vehicles locating this rest area by road signs in the northbound lane. As identical signing was used for both traffic lanes at Ladd Canyon, there was no ready explanation for the significant difference in the proportion of vehicles locating the rest area by signs by direction of travel.

The standard method of signing rest areas at the time of study was by the installation of two advance signs for each direction of travel: "Rest Area One Mile" (30 by 18 in.) and "Rest Area Ahead" (30 by 24 in.). In addition, one standard keystone-type sign "Roadside Rest Area" (30 by 30 in.) was usually installed off the shoulder of the highway, midway between the rest area entrances. Unfortunately, from a study viewpoint, there were only slight variations from this standard signing at the various study sites and no conclusions could be made as to most effective type of signing. It can be concluded that the request for more signs is directly attributable to the small size and inconspicuousness of existing signing.

Alternatives to Rest Area Stops.—The interview question, "Where would you have stopped had the rest area not been here?" provides an indication of the alternative locations of rest area stops and an indication of the amount of potential shoulder stops removed from the highway. Table 16 shows an average of 47.9 percent of the vehicle-parties stopping in the rest areas indicated they would have chosen to stop at the next rest area or park. The next largest category was next city or town with 15.0 percent indicating this alternate choice. Next service station was the choice of 9.6 percent and shoulder of highway the choice of 8.2 percent of the vehicle-parties. The 8.2 percent indicating shoulder of highway gives an estimate of the minimum number of potentially hazardous shoulder stops that were avoided by the development of rest areas, as undetermined percentages of vehicle-parties classified in the

TABLE 16

PERCENTAGE DISTRIBUTION BY ALTERNATE STOP LOCATION

Rest Area	Route	Alternate Location (%)							All Stops (%)
		Shoulder of Hwy.	Next Service Station	Next Rest Area or Park	Next City or Town	Would Not Stop	Other or Did Not Know	Data Not Obtained	
John Day-Squally Hook	US 30	10.0	10.7	53.6	12.8	1.7	1.5	9.7	100.0
Willow Creek	US 30	7.1	11.8	51.8	13.4	2.1	4.2	9.6	100.0
Ladd Canyon	US 30	9.6	6.8	32.3	25.4	4.1	12.0	9.8	100.0
Avg.	US 30	8.9	9.8	45.9	17.2	2.6	5.9	9.7	100.0
Blue Star (Eugene)	US 99	4.7	5.2	52.5	14.3	4.7	9.0	9.6	100.0
Cabin Creek	US 99	8.1	8.7	47.2	9.9	2.7	4.6	18.8	100.0
Cow Creek	US 99	10.3	16.6	46.2	12.3	1.4	6.6	6.6	100.0
Gold Hill	US 99	7.3	7.3	58.8	17.2	3.7	7.8	4.9	100.0
Avg.	US 99	7.6	9.5	49.4	13.4	3.1	7.0	10.0	100.0
Avg. (all sites)		8.2	9.6	47.9	15.0	2.9	6.5	9.9	100.0

do not know (6.5 percent) and data not obtained (9.9 percent) categories might also have stopped on the highway shoulders. Also, a substantial number of those desiring to stop in rest areas or parks would have stopped on highway shoulders if rest area facilities were not available.

Substantially the same patterns of stop location alternatives predominated at the various rest areas. The only rest area with any large difference was Ladd Canyon. Here the proportion indicating rest area or park as an alternate was only 32.3 percent, and next city or town was given as an alternate stop by 25.4 percent, apparently due to the nearby location of LaGrande. Despite the nearby location of the Blue Star (Eugene) Rest Area to the city of Eugene (8 mi), 52.5 percent of the parties stopping there indicated the next rest area or park as their alternate choice of stop.

Evaluation of Rest Area Occupant and Observer Comments.—Comments of rest area users provided an important source of information for evaluating the service offered by rest areas in the eyes of the motorists for the summer study periods. Observations of the operation of rest areas by the observer interviewers also afforded a reliable source of data for evaluating rest area service, even though these comments cannot be evaluated quantitatively.

User comments were predominantly favorable and indicated sincere appreciation of Oregon roadside rest areas and their facilities. Due to the large number of out-of-state tourists, many comparative statements expressing appreciation of Oregon roadside parks and rest areas, which were often regarded as superior in quality and quantity to those of home States, were received.

Critical comments usually expressed a desire for more rest areas and improved facilities. Particularly numerous were comments pertaining to the need for drinking water at those rest areas without this facility. The need for overnight camping places was often expressed. The lack of sufficient table-bench units to meet demands was a further source of criticism. Occasional comments indicated the need for better signing of rest areas.

The importance of good rest area maintenance was underscored according to the quality of rest area maintenance at the study sites. Many favorable comments pertaining to rest area maintenance and condition of facilities were made at the Ladd Canyon and Gold Hill Rest Areas, which were probably the best maintained of the seven study sites. Conversely, complaints were made by occupants of the Blue Star (Eugene) Rest Area regarding the condition of restrooms and grounds. Maintenance at this rest area appeared to be the poorest, apparently due to the large amount of use. It was evident from the complaints and observations that daily, or oftener, maintenance of rest areas with high usage is mandatory. The lack of weekend rest area maintenance, when rest area traffic is highest, presents a major problem. Although weekday maintenance is usually good, high weekend use without maintenance resulted in a poor image to the motorist during this period.

The observations and comments of interviewers confirmed the general appreciation and demand of the public for rest areas and their facilities. The interviewers confirmed that the lack of drinking water was the main public concern at rest areas without this facility, and that the lack of sufficient table-bench units caused many potential users to turn away, particularly at lunch hours.

The need for clean restrooms and adequate paper supplies, particularly over weekends, was a source of concern to interviewers as they received direct criticism and requests for supplies from occupants. The primitive type dry pit restrooms installed at all rest areas were occasionally a source of complaint, particularly by women. The need for additional restrooms at Blue Star (Eugene) Rest Area was noted by observers. If water were available then the presence of flush toilets would eliminate the unpleasantness of pit toilets and at the same time enhance the use of the entire rest area. This is especially true as would relate to the use of tables for eating in a more enjoyable atmosphere.

CONCLUSIONS

1. The average percent of highway traffic using Oregon roadside rest areas on US 30 (4.1 percent) and US 99 (2.0 percent) is substantially below that expected at new safety rest areas, planned or under construction on the Interstate System. With the superior types of access, signing, and facilities that will be provided with safety rest areas built to Interstate System standards on full-access controlled highways, the percentage of traffic ultimately using rest areas, based on experience of other States, will be approximately 10 percent.

2. The two major factors affecting the percent of highway traffic using rest areas of like design and facilities are the remoteness of the rest area from alternate facilities and the amount of total traffic that is local traffic. The percent of use is directly proportional to remoteness and inversely proportional to the amount of local traffic.

3. Rest area location in respect to distance from major cities does not appreciably affect the amount of use, although because of larger proportions of local traffic near cities, the percentage of highway traffic using rest areas at such locations will usually be lower.

4. The spacing of rest areas at distances of 30 to 45 mi would accommodate approximately 70 to 60 percent of rest area users, respectively. Spacing of rest areas at 30-min travel time intervals would serve approximately 80 percent of rest area users. However, the percentages would not be typical of full freeways.

5. The need for providing restroom, drinking water, and table and bench facilities (in that order) is indicated by the heavy demand for

these facilities and the use made of them.

6. The fact that only 1.4 percent of rest area stops were for recreational purposes is indicative that rest areas are being used for the purpose for which they are intended. That is, they are providing services to motorists en route to other destinations and are not being used to any significant extent for park or recreational use.

7. To serve heavy trucks, truck-trailer combinations, and light vehicle-trailer combinations adequately, rest area access and parking should be specifically designed to accommodate these vehicles.

8. The longer length of stay and the higher proportion of eating and rest stops observed at rest areas with scenic aspects are indicative of the need for providing larger than average parking space and additional table and bench units at such rest areas.

9. There is general public acceptance and praise of Oregon rest areas, particularly by out-of-state traffic. Public criticism of rest area operation is minor and mainly concerns the need for more rest areas and additional facilities such as drinking water, table-bench units, and more sanitary restrooms (flush toilets, etc.).

10. Adequate signing of rest areas is very important if rest areas are to provide maximum roadside service to highway traffic.

11. A minimum of daily rest area maintenance is important in rest area operation to keep the services offered up to standards of public approval and acceptance.

SUMMARY OF FINDINGS

1. Average daily summer highway traffic of 4.1 and 2.0 percent entered Oregon roadside rest areas on US 30 and US 99, respectively, in 1960. The average percent of highway traffic entering the 7 rest areas studied on non-access controlled routes ranged from 1.3 to 4.4 percent.

2. Saturday was typically the day of highest use in terms of the numbers of vehicles entering.

3. The peak hour of rest area use occurred between 12:00 NOON and 1:00 PM as shown by data on the average number of vehicles accumulated in the rest areas by hour of day. The average peak-hour vehicle accumulation was 4.5 percent and 5.8 percent of the vehicles entering from US 30 and US 99, respectively, and ranged from 3.4 to 8.4 percent for individual rest areas.

4. The average peak-moment vehicle accumulation was 7.1 and 8.2 percent of traffic entering the study sites on US 30 and US 99, respectively.

5. Light vehicles (passenger cars, panels, and pickups under 6,000 lb) were by far the heaviest rest area users, both as percent of rest area traffic (97.9 percent) and as proportion of the adjacent highway traffic by vehicle type. The proportions of each vehicle type that entered the rest areas were single light vehicle, 2.8 percent; light vehicle-trailer coach, 10.6 percent; light vehicle-other trailer, 4.5 percent; bus and single unit truck, 1.2 percent; and truck-trailers, 0.1 percent.

6. Vehicle occupancy rates of 3.1 and 3.0 persons per vehicle entering rest areas on US 30 and US 99, respectively, were observed. These averages are substantially higher than the occupancy of vehicles traveling on the adjacent highways (2.1 to 2.4 persons per passenger car).

7. The number of vehicles that entered the rest area and stopped was 91 percent.

8. The out-of-state proportion of light vehicles using rest areas (61 percent) was significantly higher than the proportion of out-of-state light vehicle traffic on the adjacent highways (47 percent).

9. The average distance from last stop was 66 mi at rest areas on US 30 and 75 mi on US 99.

10. The average time interval since last stop was 90 and 102 min at rest areas on US 30 and US 99, respectively.

11. The average length of stop was 43 min for all stops, with 49 percent of all stops, less than 20 min, and 64 percent less than 30 min. The average stay was 73 min at two rest areas with scenic attractions and ranged from 27 to 33 min at the five other sites.

12. Three primary purposes of rest area stops (rest or nap, eating, and restroom) accounted for 84 percent of the use. The average distribution of stops by primary purpose in rest areas on US 30 was rest or nap, 18 percent; eating, 36 percent; restroom, 28 percent. On US 99 the average distribution was rest or nap, 24 percent; eating, 39 percent; restrooms, 23 percent.

13. Sixty-two percent of the vehicle-parties stopping at the rest area percent table-bench units—18 percent used drinking water, and 33 percent table-bench units—18 per cent used no facilities.

14. The typical hour of peak table use occurred between 12 NOON and 1 PM during which 58.6 percent of the vehicle-parties stopping in the study sites stopped to eat. During periods of peak usage, available table-bench units did not always satisfy the demand.

15. The rate of restroom use was 1.5 persons per entering vehicle-party for three rest areas on US 30 and 1.2 persons per vehicle-party at two rest areas on US 99. Slightly less than one-half of the persons entering the rest area used the restrooms.

16. The primary means by which vehicle-parties located rest areas was by road signs (61.3 percent) with knowledge through previous visits the next most frequent means (28.0 percent). Location by road map and other sources, mainly word-of-mouth, was small (3.2 percent).

17. A large majority of user comments were favorable and expressed sincere appreciation for Oregon rest areas and facilities. Critical comments mainly expressed a desire for more rest areas and improved facilities. Of particular concern was the need for drinking water at those rest areas not having it. The need for additional tables at crowded rest areas and for weekend maintenance, particularly of restrooms not equipped with flush toilets, was apparent by critical comments of users and observations at rest areas with high usage.

ACKNOWLEDGMENT

William J. Byars, Statistical Supervisor, Planning Survey Section of the Oregon State Highway Department's Traffic Engineering Division was in direct charge of preparing study procedures, and collecting and analyzing the data contained in this paper.

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APPENDIX

A TYPICAL CASE HISTORY—LADD CANYON REST AREA

The Ladd Canyon data are typical of the data collected and analyzed at the seven Oregon study sites in the summer of 1960. Comparable data for each study site have been compiled and analyzed as the basis for this report on safety rest area use and are a matter of record in the files of the Oregon State Highway Department, Traffic Engineering Division.

DESCRIPTION

Location

Ladd Canyon Rest Area was located adjacent to the eastbound lane of a 2-lane section of US 30 (the traveled route of Interstate 80N) 7 mi southeast of LaGrande in northeast Oregon, at milepoint 268.3. Although the highway design was not up to ultimate Interstate design standards, the existing highway was adequate for present traffic. The highway at this location has two 14-ft asphaltic pavement traffic lanes and 8-ft paved shoulders. Highway shoulder parking was not restricted. A 2-lane highway extended from LaGrande, 36 mi southeast to Baker, US 30, with the exception of two 4-lane sections, each approximately 2 mi long, southeast of the rest area. There was partial control of access on this section from LaGrande to North Powder, and no access control from there on to Baker. West of LaGrande to Pendleton where parts of the Interstate Highway were under construction to ultimate design standards, there was a combination of 4- and 2-lane highways with full and partial access control.

The nearest city to Ladd Canyon Rest Area on US 30 was LaGrande (population 9,014). Southeast on US 30 were North Powder (population 399), Haines (population 331), and Baker (population 9,986), at distances of 16, 24, and 36 mi, respectively. These cities provided the only food, lodging, and auto service facilities in this sparsely settled area on US 30 other than a service station and cafe with showers and bunks that was located approximately 5 mi northwest of the rest area and catered to trucks. About 8 mi southeast on US 30 two large turnouts with drinking water fountains provided opportunity for stopping on both sides of the highway. The nearest roadside rest areas on US 30 were Rattlesnake Springs, 63 mi southeast, and Willow Creek, 100 mi northwest.

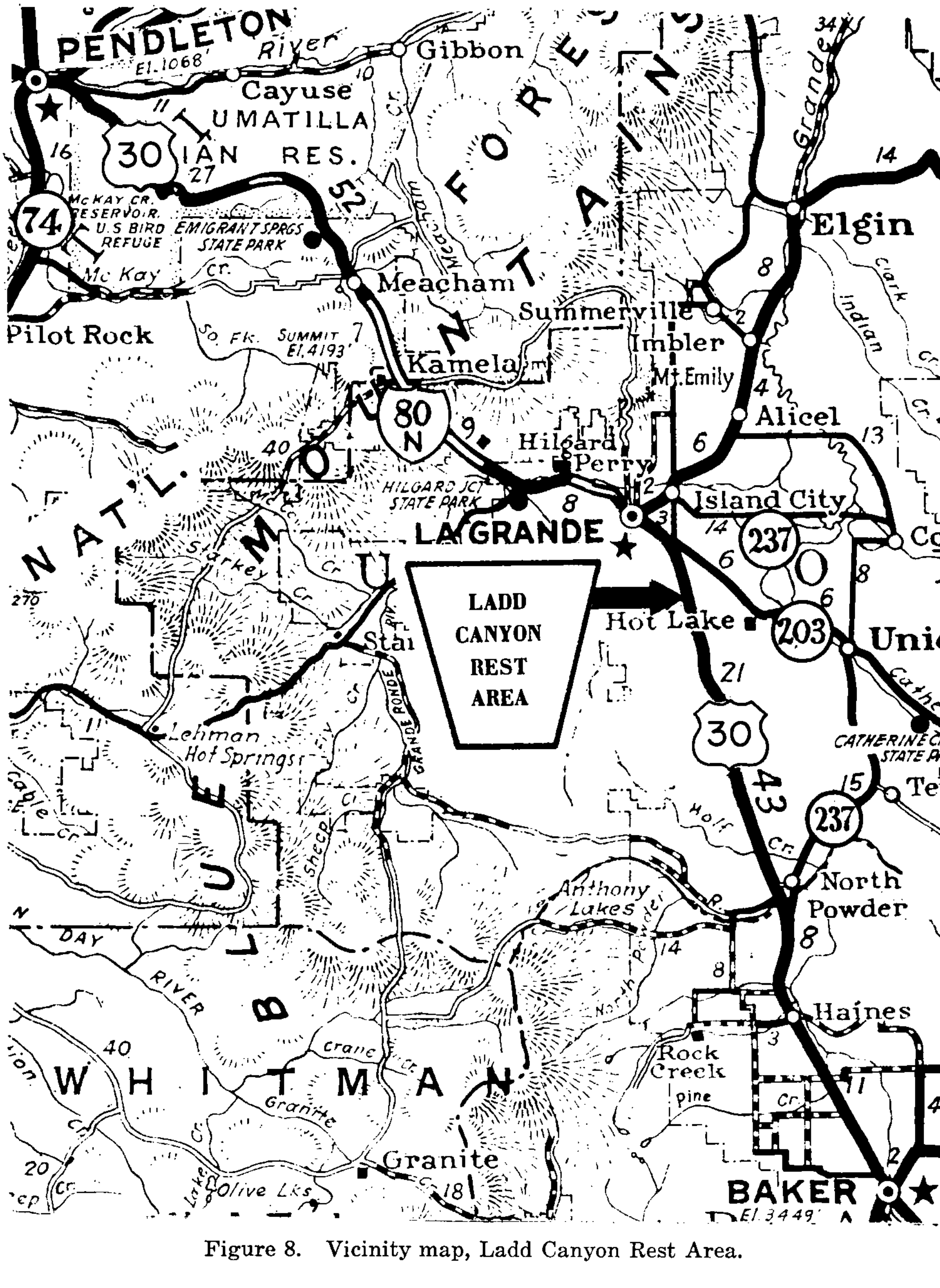

Figure 8. Vicinity map, Ladd Canyon Rest Area.

Hilgard and Emigrant Springs State Parks, approximately 16 and 37 mi northwest on US 30, respectively, in the Blue Mountains, furnished extensive facilities for rest and recreational stops as well as camping. The location of Ladd Canyon Rest Area in relation to the surrounding country and highway system is shown in Figure 8.

Figure 9. Ladd Canyon Rest Area, looking northwest from highway.

Physical Description

Ladd Canyon Rest Area was situated at the extreme southeast end of the Grand Ronde Valley where US 30 begins its climb up Ladd Canyon in the foothills of the Blue Mountains. A view of Ladd Canyon Rest Area looking northwest across the valley floor towards LaGrande is shown in Figure 9. The rest area was apparently the location of an old farm orchard with prune, apple, poplar, and willow trees, and a small creek forming an attractive rest area site. Ladd Creek, a small stream offering trout fishing, ran near the shoulder of the highway opposite the rest area. The area bordering US 30 southwest of Ladd Canyon Rest Area was mountainous, sparsely-settled land used mainly for grazing, whereas flat farm pasture and crop lands bordered the highway northwest into LaGrande.

The rest area was signed in each direction of travel by four signs: "Rest Area One Mile" (30 by 18 in.), "Deposit Litter Bags ¼ Mile" (24 by 18 in.), "Rest Area Ahead" (30 by 24 in.), "Historical Marker Ahead" (30 by 24 in.). In addition, one standard keystone-type roadside rest area sign (30 by 30 in.) was installed off the highway shoulder near the front center of the rest area. A large Oregon historical marker fronting the parking area attracted some rest area stops. The rest area was readily visible from the highway, with direct access to eastbound traffic. It was necessary for westbound traffic to cross the eastbound traffic lane in entering and leaving the rest area.

Eight tables with benches were placed among the trees in the rest area. Paths led to two dry pit restrooms located across a rivulet, apart from the picnic area. The restrooms were not readily observed from the highway. Drinking water was piped into a fountain and faucet in the center of the rest area next to the parking area. Rest area occupancy in excess of 18 hr and making of fires were prohibited by signs. No artificial lighting or cooking facilities were installed in the rest area. A few informal campfire spots had been made by picnic parties using creek bed rocks. Asphalt-surfaced parking space was provided by a parking area approximately 16 by 120 ft and supplemental parking space was provided by the gravel shoulders of the rest area access road. The parking space was not marked.

TRAFFIC VOLUMES

Traffic Counting Equipment

Daily 24-hr counts of the number of vehicles entering the rest area were obtained from traffic recorders from June 30 to September 5, 1960. A road tube type of hourly traffic recorder was installed on the rest area access road near the southeast entrance. The traffic recorder data from the North Powder permanent traffic recorder, located approximately 17 mi southeast on US 30, furnished data necessary to relate rest area traffic to highway traffic. The daily rest area access road traffic counts adjusted for axle overcounting are given in Table 3. Adjustment of the rest area traffic recorder data was necessary to compensate for overcounting due to multiple-axle vehicles such as passenger car-trailer coach combinations and multiple-axle trucks. The overcount correction factor developed from the study's traffic classification data was 0.9446. No correction factor was required for the US 30 highway traffic counts obtained from the permanent traffic recorder.

Rest Area Traffic

The average daily number of vehicles and percent of highway traffic entering the rest area by day of week are given in Table 17. Average weekday rest area traffic was considerably less than weekend traffic as a percent of highway traffic by day of week but was not appreciably different from the average day (4.4 percent).

TABLE 17
COMPARISON OF HIGHWAY AND REST AREA TRAFFIC AT LADD CANYON

Day	Average Daily Summer Traffic		
	Highway (No.)	Entering Rest Area (No.)	(%)
Weekday	3,049	132	4.3
Saturday	3,469	150	4.3
Sunday	3,165	144	4.5
Avg.	3,129	137	4.4

REST AREA USE

The data are based on interview and observation data collected from 10:00 AM to 5:00 PM on 7 days during the summer of 1960 (June 14-15, July 14-15, and August 26, 27, 28). The study period included five weekdays, one Saturday, and one Sunday.

Vehicles Entering

Average Vehicles Accumulated.—The average number of vehicles accumulated in Ladd Canyon Rest Area by time of day is shown in Figure 10. Although there was a large daily variation in the number of vehicle accumulations, there was a definite hourly trend with maximum rest area use usually occurring between 12:00 NOON and 2:00 PM. During the study period an average of five vehicles accumulated in the rest area during the peak hour. The irregular, random movements of the trend line partially reflect the transitory nature of rest area use. For example, during the course of an hour, the number of vehicles in the rest area may shift from a low of 1 or 2 to a maximum of 7 or 8 vehicles.

Daily Peak Moment Accumulation.—Data on the amount of summertime use are given in Table 18, in terms of the number of vehicles accumulated during the peak moment. The peak moment was defined as the 5-min interval of the day during which the number of vehicles accumulated in (occupying) the rest area was a maximum.

The daily peak-moment vehicle accumulations ranged from 5 to 10 vehicles during the 7 days of the study period, and averaged 8 vehicles. The average accumulation was 7.1 percent of the average daily 24-hr summertime rest area traffic. It

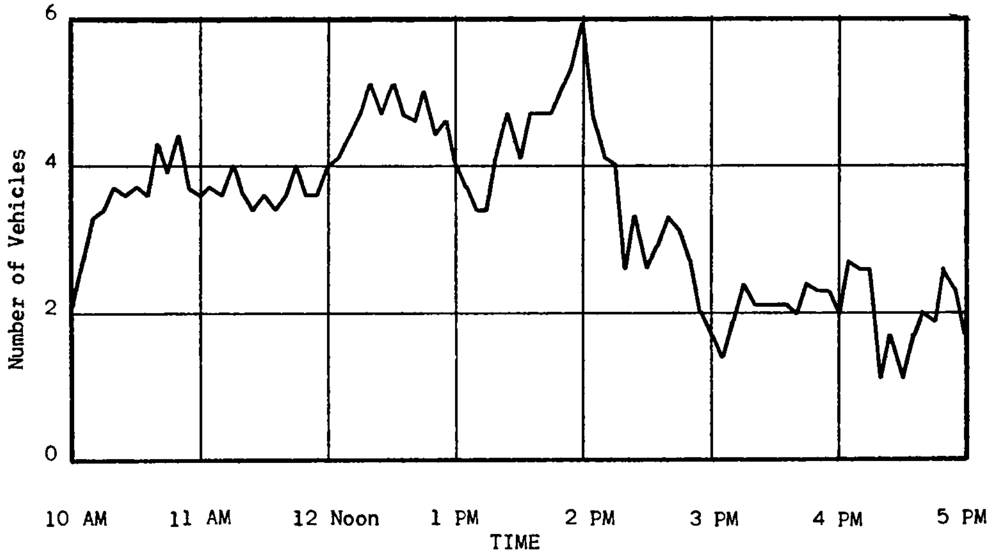

Figure 10. Average number of vehicles accumulated in Ladd Canyon Rest Area.

should be recognized that the peak moment data may be less than the maximum peak moment of the summer because of the absence of holidays from the study periods as well as the sample size. (An analysis of the peak-moment sample variability indicated that the true average daily peak-moment vehicle accumulation will lie between 5.8 and 9.3 vehicles 95 percent of the time.)

Vehicle Type.—Table 19 shows the classification of the rest area and adjacent highway traffic observed during the seven-day, 10:00 AM to 5:00 PM period. Data on the type of vehicles using the rest area are particularly useful in relation to access road and parking facility design. The use

TABLE 18

RELATIONSHIP OF VEHICLES ACCUMULATED DURING PEAK MOMENT TO THOSE ENTERING, LADD CANYON

Day	Number of Vehicles: Entering in 24 Hr	Number of Vehicles: Accumulated During Peak Moment	Percent Accumulated During Peak Moment [1]
Tuesday	100	10	10.0
Wednesday	88	5	5.7
Thursday	128	8	6.2
Friday	124	9	7.3
	94	8	8.5
Weekday avg.	107	8	7.5
Saturday	100	8	8.0
Sunday	127	5	3.9
Avg.	109	8	7.1

[1] Based on unrounded data.

TABLE 19

CLASSIFICATION OF TRAFFIC, LADD CANYON

Vehicle Type	Vehicles Entering Rest Area: No.	Vehicles Entering Rest Area: %	Highway Traffic: No. Observed	Highway Traffic: Percent Entering Rest Area
Light vehicle	353	88.2	7,469	4.7
Light vehicle and trailer coach	18	4.5	116	15.5
Light vehicle and other trailer	14	3.5	242	5.8
All light vehicles	385	96.2	7,827	4.9
Bus or single-unit trick	13	3.3	353	3.7
Truck-trailer combinations	2	0.5	569	0.4
All vehicles	400	100.0	8,749	4.6

of the rest area was almost entirely by single light vehicles (88.2 percent). The lack of appreciable heavy truck use of the rest area was anticipated as the rest area access and parking were of an older design, primarily for light vehicle use. The fact that a cafe and service station with showers and bunks which catered primarily to trucks was located only 5 mi away may also have had an effect in reducing truck use.

An analysis of the relationship of the rest area and highway traffic by vehicle type indicated that, although the adjacent highway traffic was predominantly light vehicles, the light vehicle was a larger proportion of the rest area users. A substantially higher percentage of light vehicle-trailer coach highway traffic (15.5 percent) entered the rest area than other light vehicle classifications (single light vehicles, 4.7 percent; light vehicle and other trailer, 5.8 percent). Only 0.4 percent of the truck-trailer highway traffic entered the rest area.

Vehicle Occupancy.—Data on the average number of persons occupying vehicles entering Ladd Canyon Rest Area are given in Table 20 by day of week. An average of 3.0 persons per vehicle was observed entering the rest area. Although the weekday average was 3.0 persons as compared to 2.9 and 3.3 persons for Saturday and Sunday, there appeared to be no significant differences in the persons per vehicle ratios by day of week as the weekday occupancy ratios ranged from 2.7 to 3.4 persons per vehicle. The number of vehicle occupants ranged from the single driver to 9 persons in family cars.

TABLE 20

VEHICLE OCCUPANCY, LADD CANYON

Day	No. of Vehicles Entering [1]	No. of Persons	Persons Per Vehicle
Weekday	58	174	3.0
Saturday	55	159	2.9
Sunday	50	166	3.3
Avg.	56	171	3.0

[1] Excludes vehicles entering for which occupancy data not obtained.

The rest area average of 3.0 persons per vehicle compares with the average of 2.4 persons per passenger car observed during another study in July 1960 on US 30, 10 mi east of Pendleton. The higher rest area vehicle occupancy average is representative of a large amount of family rest area use, particularly by tourist parties.

TABLE 21

RELATIONSHIP OF VEHICLES STOPPING TO THOSE ENTERING REST AREA, LADD CANYON

Day	Number of Vehicles		Percent Stopping in Rest Area
	Entering	Stopping	
Weekday Avg.	59	54	92
Saturday	55	47	85
Sunday	50	46	92
Avg.	57	52	91

Vehicles Stopping

The relationship of the average daily number of vehicles stopping in the rest area to the number of observed entering the rest area by day of week (10:00 AM to 5:00 PM) is given in Table 21. A daily average of 52, or 91 percent of the 57 vehicles entering, stopped in the rest area. The remaining 9 percent represented vehicles driving through the rest area without stopping.

Although occupants of vehicles driving through were not interviewed, it was evident from observation and passing comments of drivers, that the following were the more frequent reasons for the failure of entering drivers to stop:

1. The rest area road was used as a turn around point for traffic.

TABLE 22

VEHICLE REGISTRATION OF REST AREA USERS, LADD CANYON

Place of Registration	Number of Vehicles Stopping	Percent of Total
Oregon	121	33.1
Washington	88	24.1
California	20	5.5
Idaho	43	11.8
Montana	3	0.8
Utah	8	2.2
British Columbia	4	1.1
Other States and countries	73	20.0
Out-of-state, unidentified	5	1.4
All places	365	100.0

2. The drivers of vehicles failed to notice the restrooms and drove on.

3. Vehicle-parties were looking for parks or camp grounds (no fires were permitted in the rest area nor were camp sites available).

State of Registration.—The distribution of vehicles stopping by place of vehicle registration given in Table 22 provides a measure of the amount of resident and out-of-state use of the rest area. Approximately one-third were Oregon residents, and two-thirds were out-of-state residents. Of the out-of-state users, Washington residents accounted for the highest amount (24.1 percent) with Idaho next (11.8 percent) followed by California (5.5 percent). The remaining users (25.5 percent) were scattered among residents from various parts of the United States and Canada. The 1.4 percent classified as out-of-state, unidentified, mainly represented vehicles making short stops for which there was no direct interview made and which had unfamiliar out-of-state license plates. Most of the out-of-state users were tourists and vacationists.

Comparisons of the proportions of Oregon and out-of-state light vehicle traffic traveling the highway and stopping in the rest area indicate that a significantly higher proportion of out-of-state traffic used the rest area than Oregon (Table 23). Out-of-state registered light vehicles constituted 55 percent of the light vehicle traffic on the highway in comparison to 68 percent of the rest area light vehicle stops.

TABLE 23

COMPARATIVE DISTRIBUTION OF LIGHT VEHICLE REST AREA STOPS AND HIGHWAY TRAFFIC BY PLACE OF REGISTRATION, LADD CANYON

Place of Registration	Rest Area Stops		Highway Traffic	
	Number of Light Vehicles	Percent of Total	Number of Light Vehicles	Percent of Total
State	114	32	3,555	45
Out-of-state	240	68	4,272	55
Total	354	100	7,827	100

Interval Between Stops.—Figures 11 and 12 show the distance and travel time, respectively, since last stop for rest area occupants in the form of a cumulative percentage distribution. The stopping habits of rest area occupants were influenced by the distance of major stopping points and trip origins from the rest area. In Figure 5 the sharp increases in the percent of stops at 30 to 45 and 0 to 15 mi distant were due to the location of the cities of Baker and LaGrande at distances of 36 and 7 mi, respectively, from the rest area. In the time traveled curve, these sharp changes were smoothed by differences in driving habits and estimates of driving time. The average distance since last stop was 54 mi, and the average time was 75 min.

Length of Stop.—The percentage distribution of stops in the rest area by length of stay is given in Table 24. Approximately 40 percent of the stops were less than 10 min, and 21 percent from 10 to 19 min. Those of less than an hour accounted for 92 percent of all stops. The average length of stay was 32 min for rest or nap, 37 min for eating, 12 min for restroom use, and 48 min for other purposes.

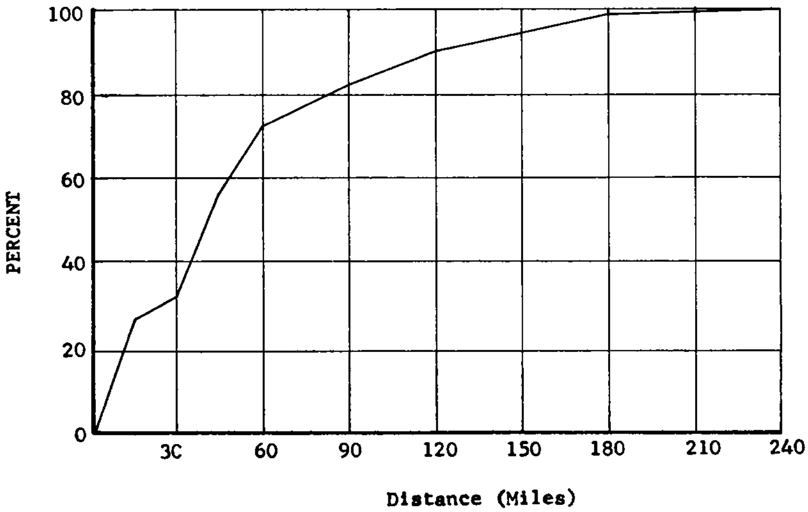

Figure 11. Cumulative percentage of vehicle-parties stopping in Ladd Canyon Rest Area by distance traveled since last stop.

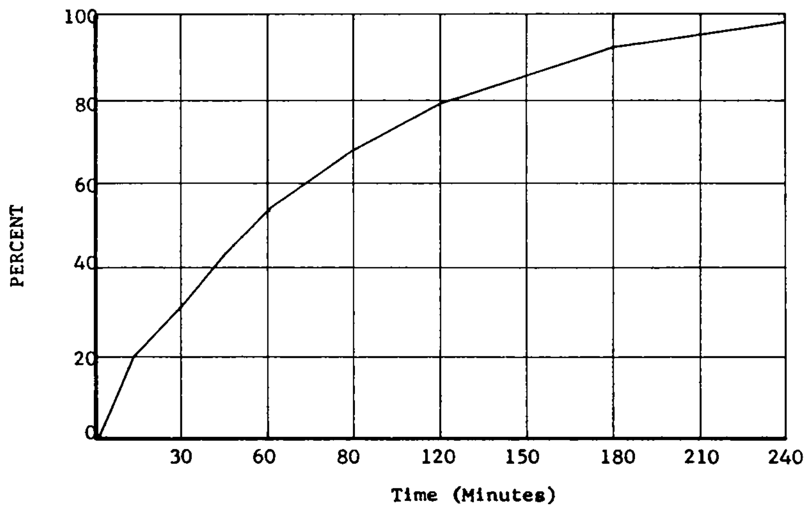

Figure 12. Cumulative percentage of vehicle-parties stopping in Ladd Canyon Rest Area by time traveled since last stop.

Purpose of Stop.—The percentage distribution of stops classified by primary purpose is given in Table 25. The data show that four categories (rest or nap, eating, restroom, and drinking water) account for the great bulk of rest area stops (approximately 84 percent). Although stops primarily for rest accounted for only 15 percent of the rest area stops, other stop purposes (such as eating, restroom, and drinking water) also incorporated many of the attributes of rest stops necessary for safety and relief from driving tension. The drinking water stops were

TABLE 24

PERCENTAGE DISTRIBUTION BY LENGTH OF STAY AND STOP PURPOSE, LADD CANYON

Length of Stay	Primary Purpose of Stop (%)				All Stops
	Rest or Nap	Eating	Rest-room	Other	
Less than 10 min	27.8	0.9	61.3	68.8	39.6
10-19 min	25.9	20.3	28.7	9.4	20.6
20-29 min	12.9	24.8	4.0	8.3	12.9
30-39 min	7.4	23.0	1.0	4.2	9.6
40-49 min	3.7	13.3	—	2.1	5.2
50-59 min	5.6	7.1	1.0	2.1	3.8
1-1½ hr	7.4	7.9	4.0	1.0	5.0
1½-2 hr	5.6	0.9	—	—	1.1
2-3 hr	—	0.9	—	1.0	0.6
3-4 hr	3.7	0.0	—	—	0.8
Over 4 hr	—	—	—	3.1	0.8
Total	100.0	100.0	100.0	100.0	100.0
Avg. (min)	33	37	12	48	32

TABLE 25

PERCENTAGE DISTRIBUTION BY PRIMARY PURPOSE OF STOP, LADD CANYON

Day	Primary Purpose of Stop (%)								Total
	Rest or Nap	Change Drivers	Eating	Recrea-tion	Rest-room	Drink of Water	Litter Deposit	Other or Unknown	
Weekday	14.0	1.1	32.4	1.1	24.3	13.2	2.2	11.7	100.0
Saturday	19.1	4.3	29.8	—	31.9	—	—	14.9	100.0
Sunday	15.2	—	23.9	4.3	43.6	8.7	—	4.3	100.0
Avg.	14.8	1.4	31.0	1.3	27.7	11.0	1.6	11.2	100.0

primarily short stops in which the use of drinking water facility was the major reason for stopping. The use of the rest area for litterbag or refuse deposits only was very low, with an average of only one stop a day reported despite the advance signing of the rest area by the signs "Deposit Litter Bags ¼ Mile."

The more frequent types of stops included in the other stop classification were those for reading the Oregon historical marker, checking vehicle, asking for information, or checking for park or overnight camping facilities and overnight stops. Recreational use of the rest area was very low (less than 1 stop per day) and consisted of a few local parties coming out to the rest area to picnic or to fish in Ladd Creek.

Facility Use

The use of the vehicle-party as a measure of facility use is most significant for planning purposes in respect to table-bench units, cooking facilities, or shelters as a vehicle-party will usually occupy a single unit regardless of the number of persons in the party.

The use of facilities in terms of the percentage of vehicle-parties using specified facilities during the study period are given in Table 26. Restrooms and drinking water were the facilities with the highest proportions of vehicle-party use (62.8 and 60.1 percent, respectively). On the basis of the study period, it appears there was proportionally greater use by vehicle-parties of restrooms on weekends than on weekdays (77 vs

TABLE 26

PERCENTAGE DISTRIBUTION BY TYPE OF FACILITY USED, LADD CANYON

Day	Type of Use (%)						Total [2]
	Drinking Water	Rest-rooms	Table and Bench Units	Shel-ters [1]	None	Unknown	
Weekday	59.6	57.7	28.3	—	17.3	0	100.0
Saturday	61.7	80.9	34.0	—	14.9	2.1	100.0
Sunday	60.9	73.9	19.6	—	10.9	—	100.0
Avg.	60.1	62.8	28.0	—	16.1	0.2	100.0

[1] Facility not available.

[2] Percentage of vehicle-parties using specified facilities does not add to total as party may use more than one facility.

58 percent). On the average day, 28.0 percent of the vehicle-parties reported table-bench unit use. The percent of vehicle-parties using the tables (28.0 percent) was slightly lower than the use of the rest area for the primary purpose of eating (31.0 percent) as some parties ate in their cars and trailer coaches.

Only 16 percent of the parties stopping did not use any of the specified facilities. Among the more common types of stops not involving use of facilities were those for checking vehicle, reading the historical marker, rest stops, and obtaining information.

Table Use.—The data on the potential peak use of table-bench units for eating purposes are an important consideration in the design of rest area table facilities. Table 27 shows the potential use of tables in terms of the percentage of vehicle-parties stopping to eat during the noon hour (the typical peak hour of table use). At Ladd Canyon, an average of 5 vehicle-parties (56.7 percent) stopping in the rest area during the noon hour stopped to eat. The percentage stopping to eat during the noon hour on weekdays was substantially higher (60.7 percent) than on the Saturday (37.5 percent) and Sunday (25.0 percent). However, the lower Saturday and Sunday rates were primarily because these days (August 27 and 28) were near the end of the summer season and the weather was cool. The proportion of rest area vehicle-parties eating during the noon hour (55.7 percent) was just double the 10:00 AM to 5:00 PM average daily proportion of vehicle-party use of tables (28 percent).

TABLE 27

RELATIONSHIP OF VEHICLE-PARTIES STOPPING DURING NOON HOUR TO THOSE STOPPING TO EAT, LADD CANYON

Day	Vehicle-Parties		Percent of Vehicle-Parties Stopping During Noon Hour to Eat (Potential Users of Tables) [1]
	Number Stopping	Number Stopping to Eat	
Weekday	11	7	60.7
Saturday	8	3	37.5
Sunday	4	1	25.0
Avg.	10	5	55.7

[1] Based on unrounded data.

In addition to the potential table use given in Table 27, other tables were used when available by parties resting or using the rest area for overnight stops. Observation indicated that the 8 table-bench units were ordinarily sufficient to satisfy demand.

Restroom Use.—The number of persons using restrooms is useful in determining restroom facility requirements for design purposes. Table 28 gives the average daily use of the two restrooms at Ladd Canyon during the observation periods in

TABLE 28

RELATIONSHIP OF PERSONS USING REST-ROOMS TO VEHICLE-PARTIES ENTERING, LADD CANYON

Day	Number of Vehicle-Parties Entering Rest Area	Number of Persons Using Rest Room	Restroom Use (Person per Vehicle-Party)
Weekday	59	76	1.3
Saturday	55	62	1.1
Sunday	50	90	1.8
Avg.	57	76	1.3

terms of the daily number of persons observed using the restrooms, and as a ratio of persons per vehicle-party entering the rest area. An average of 76 persons, or 1.3 persons per vehicle-party, used the restrooms during the study period.

Observer comments indicated the two restrooms were ordinarily adequate to serve rest area occupants, although there was some waiting at times. The number of persons per vehicle-party using restrooms (1.3) compares with the rest area vehicle occupancy average of 3.0 persons, indicating that approximately 43 percent of the persons entering the rest area used the restroom facilities.

Means of Locating Rest Areas

The distribution of vehicle-party answers to the question, "How did you learn of or locate this rest area?" is given in Table 29. This question was asked as a means of determining the effectiveness of signing and other means in routing highway traffic to the rest area. It is evident that the road signs were the predominant means of locating the rest area, with a 46.6 percent of the parties using signs. There was a substantial amount of repeated use of the rest area as indicated by the large proportion (39.2 percent) of parties who located the rest area by previous visits. Road maps were little used, with less than 1 percent locating the rest area by this means. Other means consisted mainly of information relayed by word-of-mouth—principally by friends, or service station attendants and store clerks in nearby towns. The undetermined classification mainly represented vehicle-parties that were not interviewed due to the lack of sufficient interview time during peak periods of use or short duration stops.

TABLE 29

MEANS OF LOCATING REST AREA, LADD CANYON

Means	Number of Vehicles Stopping	Percent of Total
Road map	2	0.5
Road signs	170	46.6
Previous visits	143	39.2
Other	14	3.8
Undetermined	36	9.9
Total	365	100.0

Alternate Stop Location

The distribution of answers to the question, "Where would you have stopped had this rest area not been here?" is given in Table 30. This question was asked to determine probable alternative locations of rest area stops and provide an indication of the proportion of potential shoulder stops that are removed from the highway by rest areas. The predominant alternate stop location was the

TABLE 30

ALTERNATE LOCATION OF STOP IF REST AREA DID NOT EXIST, LADD CANYON

Alternate Location	Number of Vehicles Stopping	Percent of Total
Shoulder of highway	35	9.6
Next service station	25	6.8
Next rest area or park	118	32.3
Next city or town	93	25.4
Would not have stopped	15	4.1
Did not know, and other	44	12.0
Data not obtained	36	9.8
Total	366	100.0

next rest area or roadside park, with 32.3 percent of the vehicle-parties classified in this category. The next largest categories were next city or town, 25.4 percent; shoulder of highway, 9.6 percent; and next service station, 6.8 percent, while 4.1 percent indicated they would not have stopped. Additional undetermined proportions of those parties classified as did not know or from which data were not obtainable would, of course, in actual circumstances have fallen within the above alternate stop locations. The did not know category typically included parties that planned to stop at the rest area and had considered no alternative stop locations.

General Comments

Voluntary comments of Ladd Canyon Rest Area users during the survey period furnish a useful means of evaluating the service provided by the rest area in the eyes of the motorist. Observations of the rest area operation during the study by the observer-interviewers furnished additional information for appraising the rest area service.

Comments from users were usually very favorable and indicated the motorist's appreciation of rest areas. Typical favorable comments were as follows:

"Like to stop in rest areas—believe they are necessary for safety."
"Especially good for families."
"Wish more States had rest areas."
"Makes trip cheaper."
"Wonderful."
"We enjoy these rest areas."

The comments classified as critical expressed a desire for more and better rest area facilities. Some of these comments were as follows:

"Need more information on where rest areas are located—need one about every 20 to 30 miles."
"Need signs to show on which side of road rest area is located."
"Build more."
"Not enough road signs for parks."

Interviewers indicated that the public was very much in favor of rest areas, particularly in hot, dry regions. The drinking water and shade were especially important to rest area users. The maintenance at this rest area was very good and the restrooms were generally clean with adequate supplies. The interviewers also indicated that although the restrooms were well placed at a distance away from the eating and resting area, a restroom sign would have been helpful to indicate the existence and location of the restroom facilities to entering traffic. There was a tendency for fast-moving vehicles to overrun the first entrance and use the second rest area entrance. It was also pointed out to interviewers that the rest area was used extensively during the winter as a place to change tire chains.

Lateral Vehicle Placement as Affected by Shoulder Design on Rural Idaho Highways

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During 1957, 1958, and 1959, lateral placement observations were made by the Idaho Department of Highways on bituminous-paved 2- and 4-lane rural highways having different shoulder designs. Placement data were recorded for 7,777 free-moving passenger and commercial vehicles at eight locations throughout Idaho. The study was made to evaluate the influence of shoulder design on vehicle placement. Before-and-after data were recorded to measure the effect of shoulder striping and contrasting shoulders. Effects from other factors have been kept to a minimum by rational selection of study locations. Lateral placement on the roadway was recorded from visual observations of the vehicle in relation to 1-ft markings on the pavement.

• MANY RURAL HIGHWAYS in Idaho have been constructed with full base and pavement thickness carried out to the outside edge of the shoulders. No pavement contrast, difference in surface texture, or increase in shoulder slope is provided to differentiate between the shoulders and the travel way. Two-lane rural highways, having standard total cross-section widths of 28, 34, and 40 ft, are normally constructed in this manner, as well as some 4-lane, divided sections of Interstate highways in Idaho.

There is a tendency for traffic on these sections to encroach on the shoulders, and a wide difference of opinion exists as to the desirability of this type of operation. (The term *shoulder* refers to the portion of the roadway beyond 12 ft from roadway centerline, even in the case without shoulder designation.)

Proponents of this design state that the sections without defined shoulders promote less restricted traffic operation with greater opportunity for passing.

The contrary arguments are that traffic should be restricted to the travel lanes to keep the shoulders reserved for emergency use. Furthermore, as traffic volumes increase and become more complex, additional controls such as shoulder definition are necessary to obtain orderly and safe flow characteristics.

It was felt that a more solid foundation of facts was needed before this type of design could be analyzed further. For this reason, the Traffic Section of the Idaho Department of Highways decided to make a study of lateral vehicle placement.

Previously, extensive lateral placement studies were made by the Bureau of Public Roads and the State highway departments of Arizona, California, Colorado, Idaho, New Mexico, Oregon, Texas, Utah, and Washington in 1955. The data were summarized and reported by Taragin (HRB Bull. 170, pp. 54-76; 1958).

This report contains valuable information and has been of help in planning the work of the Highway Department. However, because information was desired pertaining to specific sections of Idaho highways, it was decided to undertake additional studies.

In planning this study, it was decided that placement information should be obtained on the controversial sections. This would then give data on shoulder encroachment also. Further, it was important to measure the effect on lateral placement of different shoulder defining means, such as shoulder striping and shoulder contrast. It was felt that this could best be done by collecting before-and-after data. Observations were made prior to installation of shoulder stripes and again after they had been installed. This before-and-after procedure was used whenever possible.

Because no information was available regarding travel path pattern on 4-lane Interstate sections, such sections were also studied. Altogether 7,777 single observations were made at eight rural locations.

Table 1 gives the number and type of observations at each study location.

CLASSIFICATION OF DATA

For the purpose of this study the term *free-moving vehicles* applies to vehicles that passed the study site when there was no opposing traffic in the adjacent lane or no vehicles following closely behind. These vehicles were considered uninfluenced by other traffic as to their choice of lateral position on the roadway. After a briefing of the observers to assure consistency in their judgment, it was left to them to estimate when a free-moving condition existed.

To compare the lateral placement patterns for different vehicle types, the vehicles observed were separated into two main groups. At some study locations, these groups were divided into two secondary groups.

1. *Passenger vehicles*. This classification included passenger cars, pickups, panels, and other automobiles having comparable size and operating characteristics.
   a. Local passenger vehicles. This group included all passenger vehicles carrying Idaho license plates.
   b. Foreign passenger vehicles. This group included all passenger vehicles with license plates from other States or countries.
2. *Commercial vehicles*. This classification included trucks, buses, and all vehicles with dual wheels.
   a. Single-unit trucks.
   b. Semi-unit trucks.

RECORDING PROCEDURES

Visual observations of the lateral position of the vehicles in relation to

TABLE 1
NUMBER AND TYPE OF OBSERVATIONS

Study Location No.	No. of Lanes	Passenger Vehicles		Commercial Vehicles		Total Vehicles
		Before	After	Before	After	
1	2	985	446	213	96	1,740
2	2	365	—	87	—	452
3	2	409	174	75	62	720
4	2	439	146	82	31	698
5	2	350	107	75	22	554
6	4	819	—	260	—	1,079
7	4	—	1,328	—	369	1,697
8	4	641	—	196	—	837
Total						7,777

markings on the pavement were recorded for this study. The markings consisted of 1-ft long and 1-in. wide pieces of tape nailed to the pavement at 1-ft intervals measured from the roadway centerline. Every 5-ft mark was indicated with a contrasting color. By watching the center of the right front wheel of the approaching vehicles, the observers were then able to determine the vehicle placement. The observers were located outside the shoulder approximately 120 ft beyond the markings, depending on field conditions at the study location. The observer was concealed as much as possible from the vehicle operators normal line of vision, so as not to disturb the normal lateral placement pattern.

CHARACTERISTICS OF STUDY LOCATIONS

Previous studies have indicated that lateral placement is affected by interaction of many factors. Some of these factors (such as operating speeds, vehicle dimensions, and traffic volumes) can be related to the driver and his vehicle. Other factors (such as grade and curvature, shoulder treatment and width, striping, curb and gutter characteristics, roadside conditions, and illumination) are related to the condition of the highway facilities.

Because the purpose of this study was to examine the influence of some of these factors on vehicle lateral placement, effort was made to eliminate the effect of unwanted factors through rational selection of study locations. The study sites were located as far as practical on straight and level sections away from intersections, bridges, and other lateral restrictions. All sections studied were located in rural areas having no influence from roadside development. No pedestrian traffic existed at or near the study sites. All observations were made during daylight hours. Posted speeds at all study locations were 60 mph during daylight hours. A summary of study location characteristics is given in Tables 2 and 3.

Study Location 1

Observations at this location were made on US 20-26 (Mile Post 46.6) between Star Junction and Eagle Junction. This is a 2-lane section having 12-ft travel lanes with 8-ft shoulders. The roadway surface consisted of a bituminous plant mix pavement continuous across the travel lanes and the shoulders. Placement data were initially recorded August 27, 1957 (Fig. 1) Further data were taken on July 14, 1958, after the installation of a solid, white, reflectorized, 2-in. wide shoulder stripe 17.0 ft from the roadway centerline (Fig. 2). The 1958 ADT for this section was 4,200 vehicles.

Study Location 2

This 2-lane section having 12-ft travel lanes with 8-ft shoulders is on US 20-26 (Mile Post 41.8) between Star Junction and Caldwell. Some surface color and texture contrast between the travel way and the shoulders existed at this location. The travel way was surfaced with a bituminous plant mix pavement, and a bituminous surface treatment had been applied to the shoulders. There was no shoulder stripe installed. Observations were made September 12, 1957 (Fig. 3). The recorded 1958 ADT for this section was 4,200 vehicles.

Study Location 3

These data were recorded on Idaho 15 (Mile Post 64.5) south of Cascade. This is a 2-lane section consisting of 12-ft travel lanes with 5-ft shoulders. A bituminous plant mix pavement continuous across the travel lanes and

TABLE 2

CLASSIFICATION OF TWO-LANE STUDY LOCATIONS

Study Location No.	Fig. No.	Study Location	ADT 1958 (vehicles)	Date of Study		Traffic Lanes		Shoulders			Shoulder Stripe Location	
				Before	After	Width (ft)	Surface	Width (ft)	Surface	Contrast	Before	After
1	1, 2	US 20-26 M.P. 46.6	4,200	8-27-57	7-14-58	12	Bit. plant mix	8	Bit. plant mix	None	None	17 ft from centerline
2	3	US 20-26 M.P. 41.8	4,200	9-12-57		12	Bit. plant mix	8	Bit. surface treatment	Some surface and texture contrast	None	None
3	4	SH 15 M.P. 64.5	850	8-29-57 9-5-57	10-10-57	12	Bit. plant mix	5	Bit. plant mix	None	None	15 ft from centerline
4	5	SH 72 M.P. 41.5	1,850	9-13-57	7-15-58	12	Bit. plant mix	4	2-ft bit. plant mix and 2 ft of gravel	Contrast between gravel and plant mix	None	13.5 ft from centerline
5	6, 7	US 30 M.P. 28.3	3,000	9-4-57	7-14-58	12	Bit. plant mix	3	gravel	Contrast	None	11.5 ft from centerline

TABLE 3

CLASSIFICATION OF FOUR-LANE INTERSTATE STUDY LOCATIONS

Study Location No.	Fig. No.	Study Location	ADT 1959 (vehicles)	Date of Study	Traffic Lanes		Shoulders				Median (ft depressed)	Other Characteristics
					Width (ft)	Surface	Width (ft) Right	Width (ft) Left	Surface	Contrast		
6	8	I-80N Station: 1855+00	4,200	9-17-59 9-23-59	12	Light chip seal	10	4	Bit. plant mix	Color and surface texture contrast	78	
7	9, 10	I-80N Station: 1809+00	4,200	10-30-59 11-18-59 12-14-59	12	Light chip seal	10	4	Bit. plant mix	Color and surface texture contrast	78	After installation of sign: Do Not Travel on Paved Shoulder
8	11	I-15W Station: 1033+00	3,600	10-7-59 10-8-59	12	⅝-in. chip seal	10	4	⅝-in. chip seal	None	78	

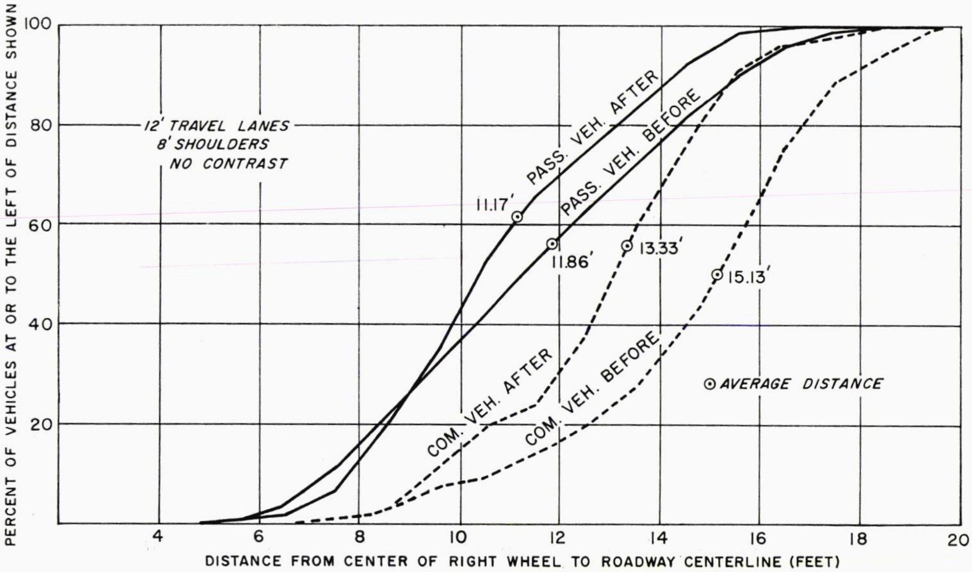

Figure 1. Position of free-moving vehicles before and after installation of shoulder stripes at Study Location 1.

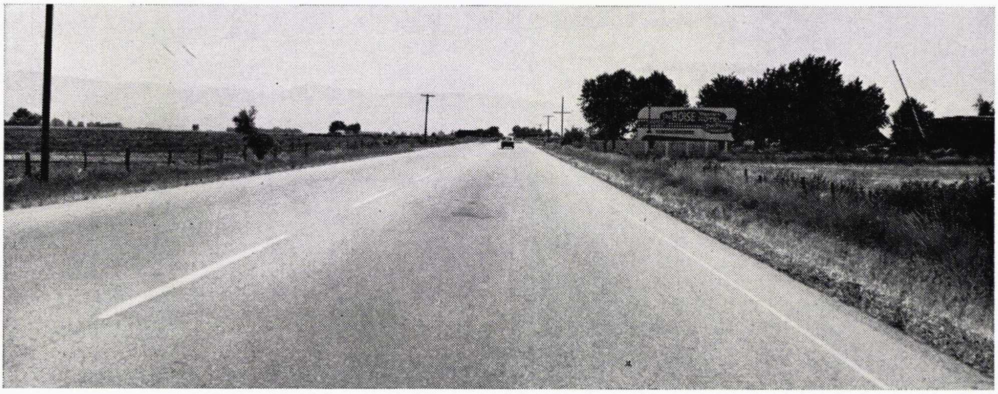

Figure 2. Study Location 1.

the shoulders provided no shoulder contrast. Before data were recorded August 29 and September 5, 1957 (Fig. 4). After data were recorded October 10, 1957 after the installation of a 2-in., solid, reflectorized, white shoulder stripe located 15.0 ft from the roadway centerline. The recorded 1958 ADT for this section was 850 vehicles.

Study Location 4

This study site was located on Idaho 72 (Mile Post 41.5) between Karcher Junction and Marsing. This is a 2-lane section having 12-ft lanes and 4-ft shoulders. The pavement on the travel lanes was a bituminous plant mix type. The 4.0-ft wide shoulder consisted of 2.0 ft of bituminous plant

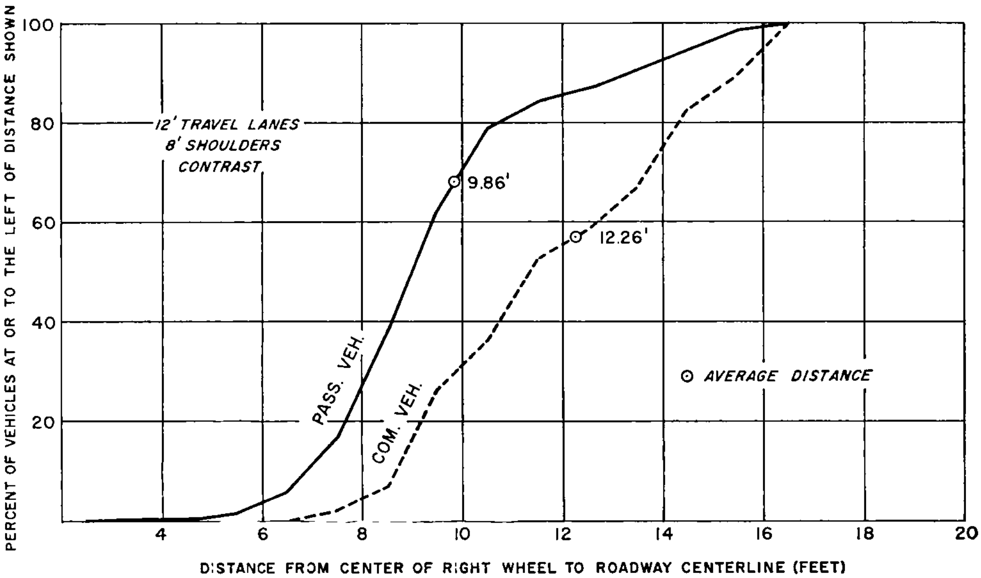

Figure 3. Position of free-moving vehicles at Study Location 2.

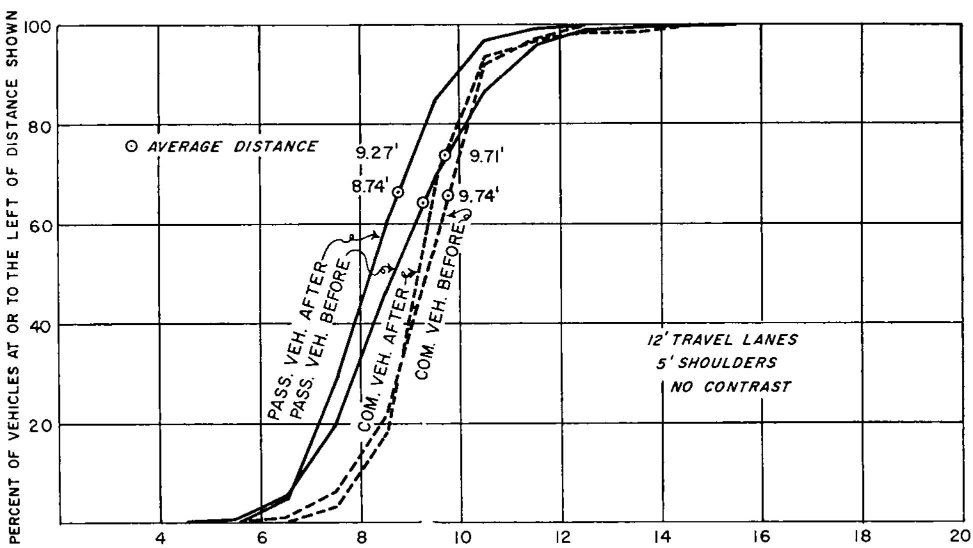

Figure 4. Position of free-moving vehicles before and after installation of shoulder stripes at Study Location 3.

mix and 2.0 ft of gravel, thereby providing a texture and color contrast between the outer 2.0 ft of the shoulder and the rest of the cross-section. Before observations were taken September 13, 1957 (Fig. 5). Later a 2-in., solid, reflectorized, white shoulder stripe was installed 13.5 ft from the roadway centerline, and after data recorded July 15, 1958. The recorded

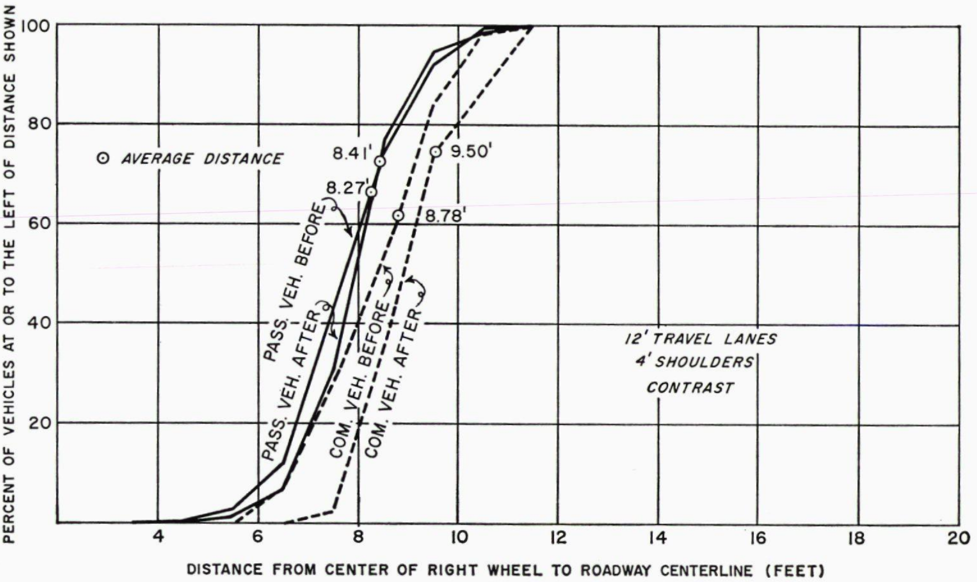

Figure 5. Position of free-moving vehicles before and after installation of shoulder stripes at Study Location 4.

1958 ADT for this section was 1,850 vehicles.

Study Location 5

Observations were also made on US 30 (Mile Post 28.3) between the Idaho 44 Junction and Sand Hollow Creek. This is a 2-lane section with 12-ft lanes and 3-ft shoulders. Shoulder contrast through the use of bituminous plant mix travel lanes and gravel shoulders was very noticeable (Fig. 6). Initial observations were made September 4, 1957 (Fig. 7). Data were also recorded July 15, 1958, after the application of a 2-in., solid, reflectorized, white shoulder stripe

Figure 6. Study Location 5.

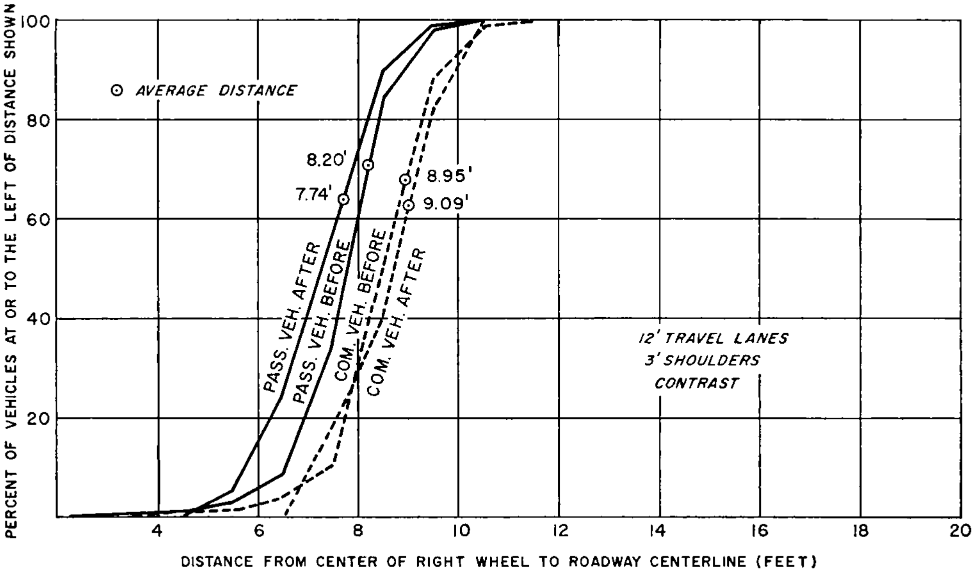

Figure 7. Position of free-moving vehicles before and after installation of shoulder stripes at Study Location 5.

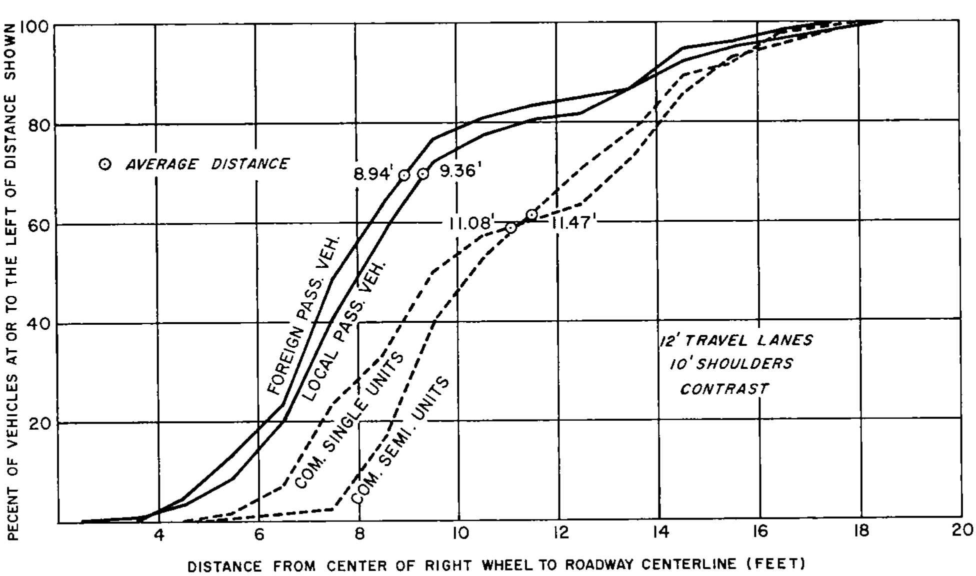

Figure 8. Position of free-moving vehicles before installation of signs at Study Location 6.

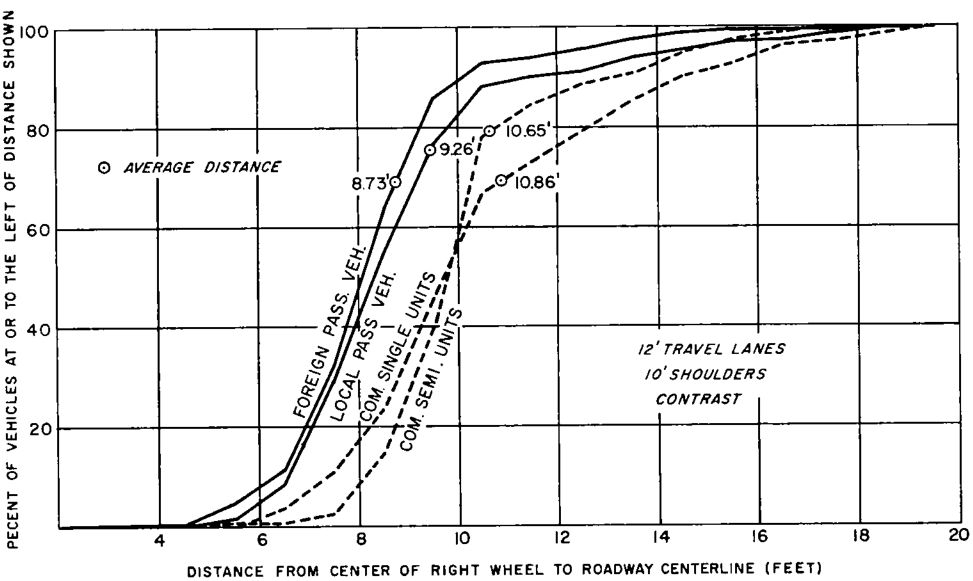

Figure 9. Position of free-moving vehicles two days after installation of sign at Study Location 7.

located 11.5 ft from the roadway centerline. The recorded 1958 ADT was 3,000 vehicles for this section.

Study Location 6

Observations were also made on Interstate 80N (Station 1855+00) west of Mountain Home. This is a 4-lane section having a 78-ft depressed median, 12-ft travel lanes, 10-ft outside shoulders, and 4-ft inside shoulders. The travel lanes were surfaced with a ⅝-in. chip seal and the shoulders were of bituminous plant mix, thereby providing some color and texture contrast between the shoulders and travel lanes. The two travel lanes serving one direction of traffic were separated by a 4-in., white, reflectorized, broken lane stripe (Fig. 8). No shoulder stripes were applied. The recorded 1959 ADT was 4,200 vehicles.

Study Location 7

This study site was also on Interstate 80N at approximately the same location (Mile Post 1809+00) as Study Location 6. This location, therefore, has the same characteristics as Study Location 6. Observations were made at it October 30, 1959, two days after installation of signs reading "Do Not Travel On Paved Shoulder" (Fig. 9). The signs, measuring 3.0 by 4.0 ft were installed at approximate 2-mi intervals, 8.0 ft outside the right-hand shoulder.

Data were also recorded at this location November 18, 1959, three weeks after the installation of the signs and on December 14, 1959, seven weeks after installation of the signs (Fig. 10). Because no significant difference was noticed between these recordings, the observations are shown combined in Figure 9.

Study Location 8

Observations were also made on Interstate 15W (Station 1033+00) located adjacent to the community of American Falls. This is also a 4-lane Interstate section having a 78-ft de-

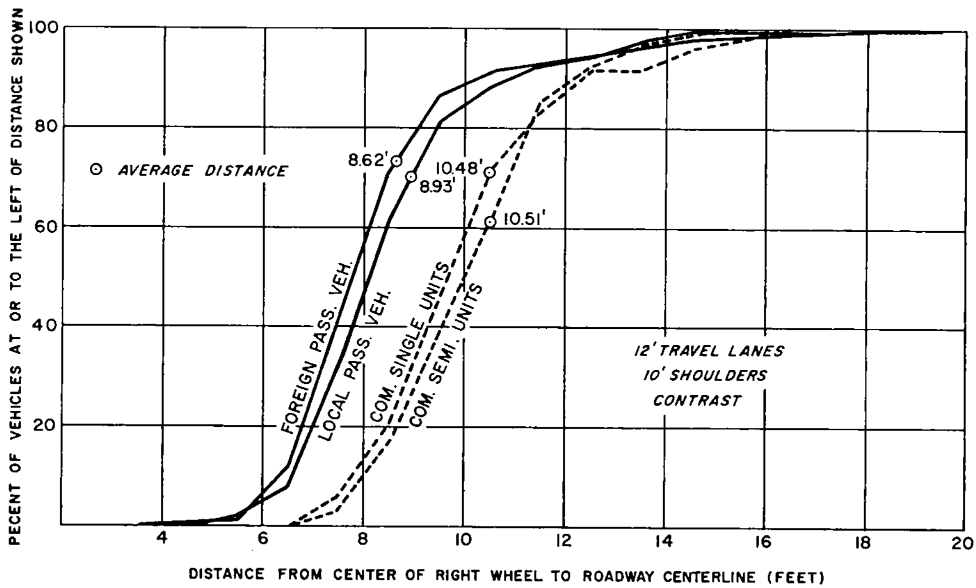

Figure 10. Position of free-moving vehicles three to seven weeks after installation of signs at Study Location 7.

pressed median, 12-ft travel lanes, 10-ft outside shoulders, and 4-ft inside shoulders. A ⅝-in. chip seal had been placed covering both travel lanes and shoulders leaving no shoulder contrast. The two travel lanes serving one direction of traffic were separated with a 4-in., white, reflectorized, broken lane line. There were no lines to mark the shoulders. The observations were made October 7 and 8, 1959 (Fig. 11). The recorded 1959 ADT was 3,600 vehicles.

LATERAL PLACEMENT

Shoulder Width

Comparison of traffic on a section having 8-ft shoulders with no shoulder contrast or striping (Study Location 1) with traffic on a section having 5-ft shoulders also with no contrast or striping (Study Location 3) shows a difference in lateral vehicle placement. The average passenger vehicle placement distance from the centerline was 11.86 ft on the section having 8-ft shoulders and 9.27 ft on the section having 5-ft shoulders. This comparison shows an average placement shift towards the center of the roadway of 2.59 ft for a difference in shoulder width of 3.00 ft. At the same time the average commercial vehicle placement shifted from 15.13 ft on the section having 8-ft shoulders to 9.74 ft on the section having 5-ft shoulders, a distance of 5.39 ft.

A comparison of average vehicle placement on the section having 4-ft shoulders (Study Location 4) to the section with 5-ft shoulders (Study Location 3) shows that the average passenger vehicle lateral placement moved in towards the center of the roadway from 9.27 to 8.27 ft, a distance of 1.00 ft. For commercial traffic, the lateral shift was 0.96 ft from 9.74 to 8.78 ft. The lateral shift detected between a 5-ft shoulder and a 4-ft shoulder, however, may have also been affected by some shoulder contrast existing for the 4-ft shoulder

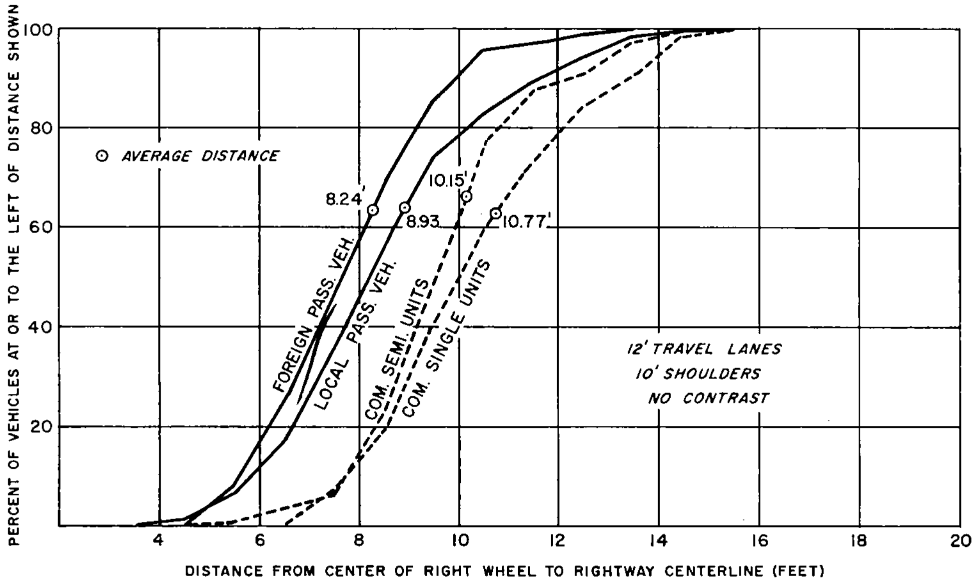

Figure 11. Position of free-moving vehicles at Study Location 8.

while no contrast existed for the 5-ft shoulder.

A further reduction in shoulder width of 1.00 ft from a 4-ft shoulder (Study Location 4) to a 3-ft shoulder (Study Location 5) shows no significant change in lateral placement either for passenger or commercial traffic.

Edge Striping

One of the reasons for making this study was to investigate the effect of shoulder striping on the vehicle placement. Data recorded before and after installation of a 2-in., solid, white reflectorized, shoulder stripe on 2-lane sections having different shoulder widths, show a change in the average travel path both for passenger vehicles and commercial vehicles.

Observations after installation of a shoulder stripe 17 ft from the roadway centerline (Study Location 1) show a move of 0.69 ft for the average passenger vehicle and 1.80 ft for the average commercial vehicles towards the roadway centerline. In this case, there was no contrast between the shoulder surface and the travel lane surface. An analysis of the curves in Figure 1 shows that this change was not caused by a change in travel of the portion of traffic traveling closest to the roadway centerline, but rather by a change in travel of the traffic that before was traveling close to the shoulder or even encroaching on the shoulder. Comparing passenger and commercial traffic, a two to three-times greater lateral shift was found for commercial traffic.

The same trends were observed on a section with 12-ft lanes and 5-ft shoulders before and after installation of a shoulder stripe 15 ft from the roadway centerline (Study Location 3). However, the differences between the before and the after averages are less. The lateral shift toward the roadway centerline was 0.53 ft for the average passenger vehicle and 0.03 ft for the average commercial vehicle. On the section with 12-ft lanes and

4-ft shoulders having some color and texture contrast (Study Location 4), no significant difference in placement was found for passenger vehicles after installing a shoulder line 13.5 ft from the roadway centerline. The average travel path for commercial traffic, however, was closer to the shoulder after installation of the shoulder stripe. This is contrary to what is found at other study locations and may possibly be explained by the probable insufficient sample size at this location. Observations for the narrowest shoulder studied (Study Location 5) showed a move of the average travel path for passenger vehicles towards the roadway centerline after installation of a shoulder stripe 11.5 ft from the centerline. No significant difference was observed for commercial traffic.

A comparison of the observations for the four different shoulder widths studied indicates that shoulder striping has greater effect on the lateral placement of both passenger and commercial vehicles on the wider sections. On the section with 8-ft shoulders, there was a 0.69-ft lateral shift towards the roadway centerline for the average passenger vehicle after installation of shoulder stripes. On the 5-ft shoulder the lateral shift was 0.53 ft and even less on the sections with 4- and 3-ft shoulders.

Passenger Traffic vs Commercial Traffic

Observations from all eight study locations show that the average passenger vehicle travels with its right front wheel closer to the roadway centerline than does the average commercial vehicle. The greatest difference between the average position of the right front wheel for passenger and commercial vehicles was found on sections with wide shoulders such as on the 4-lane Interstate sections having 10-ft wide right-hand shoulders (Figs. 8, 9, and 11) and on the 2-lane section with 8-ft wide shoulders (Figs. 1 and 3). Less difference was found on other 2-lane sections having 5-, 4-, and 3-ft shoulders (Figs. 4, 5, and 7).

A comparison of passenger and commercial traffic with respect to travel path is not complete without considering the difference in front wheel gauge (center-to-center of tires) for these two vehicle groups. Using 5.5 ft as an average value for front wheel center-to-center distance for passenger vehicles and 7.0 ft for commercial vehicles, a conversion from position of the right wheel to position of vehicle center can be made. These conversions are shown by the curves in Figure 12. The center of the average observed passenger vehicle on a 4-lane Interstate section having 10-ft outside shoulders (Study Location 6) was 6.69 ft from the roadway centerline. The center of the average observed commercial vehicle was 7.80 ft from the roadway centerline or 1.11 ft closer to the shoulder than the average passenger vehicle. On a 2-lane section having 8-ft shoulders (Study Location 2), the corresponding figures were 7.11 ft for passenger vehicles and 8.78 ft for commercial vehicles, the difference being 1.67 ft. Observations on other 2-lane sections with 5-, 4-, and 3-ft shoulders show the same tendency for passenger vehicles to travel closer to the centerline than commercial vehicles.

Foreign vs Local Traffic

For all observations made on the 4-lane Interstate locations (Study Locations 6 through 8), the data for passenger vehicles were grouped in foreign and local traffic.

Recordings for all three study groups show that foreign passenger vehicles traveled closer to the roadway centerline than did local passenger vehicles. The differences between local and foreign passenger traffic in average position of the right

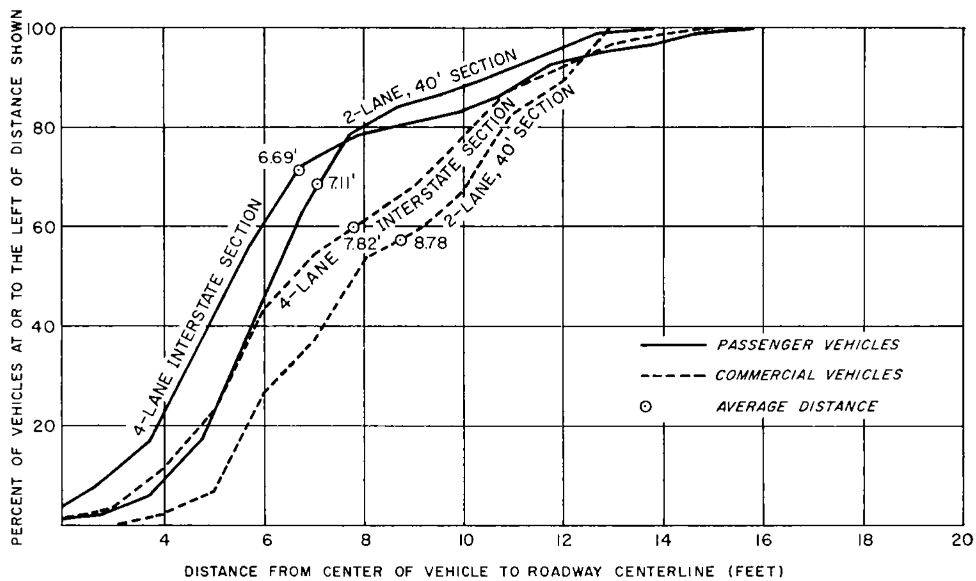

Figure 12. Comparison of vehicle placement for passenger and commercial vehicles on 2- and 4-lane sections, Study Locations 2 and 6.

front wheel range from a high of 0.69 ft to a low of 0.31 ft for the different study locations. Unfortunately, no reason for this tendency can be detected from the collected data.

Signing

Traffic was observed at Study Location 7 before and after installation of signs with the legend "Do Not Travel On Paved Shoulder." These 3- by 4-ft signs, placed 8 ft outside the right shoulder, were spaced 2 mi apart. Observations were made before, 2 days after, and 3 to 7 weeks after installation of the signs. Graphs have been prepared from the recorded data (Fig. 13).

The curves indicate the signing was somewhat effective. The average position of the right front wheel for passenger vehicle was 9.44 ft from the roadway centerline before signing. The average position was recorded as 9.10 ft two days after signing and 8.89 ft three to seven weeks after signing. The corresponding averages for commercial vehicles were 11.30 ft before, 10.73 ft 2 days after, and 10.50 ft three to seven weeks after signing.

Shoulder Contrast

Traffic observations on 40-ft sections with 12-ft travel lanes and 8-ft shoulders were made of Study Locations 1 and 2. No contrast between travel lanes and shoulders existed at Study Location 1; however, both surface texture and color contrast existed at Study Location 2. Comparing the data for Study Location 1 before installation of shoulder stripe with Study Location 2 (Figs. 1 and 3), traffic traveled closer to the roadway centerline at Study Location 2 having shoulder contrast.

Without shoulder contrast (Study Location 1) the average position of the right front wheel was 11.86 ft from the roadway centerline for passenger vehicles and 15.13 ft from the centerline for commercial vehicles. With shoulder contrast (Study Loca-

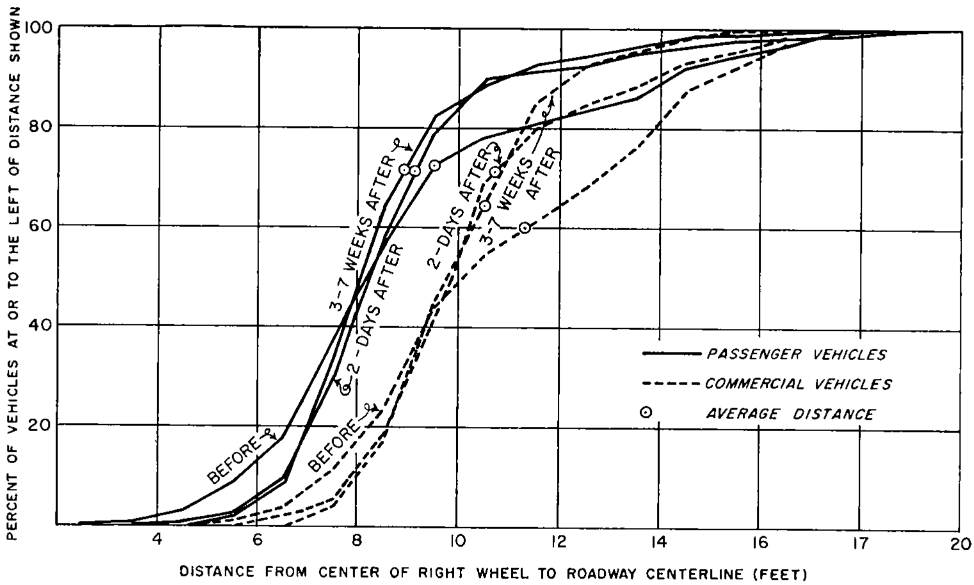

Figure 13. Effect of signing on vehicle placement (Figs. 8, 9, and 10 combined).

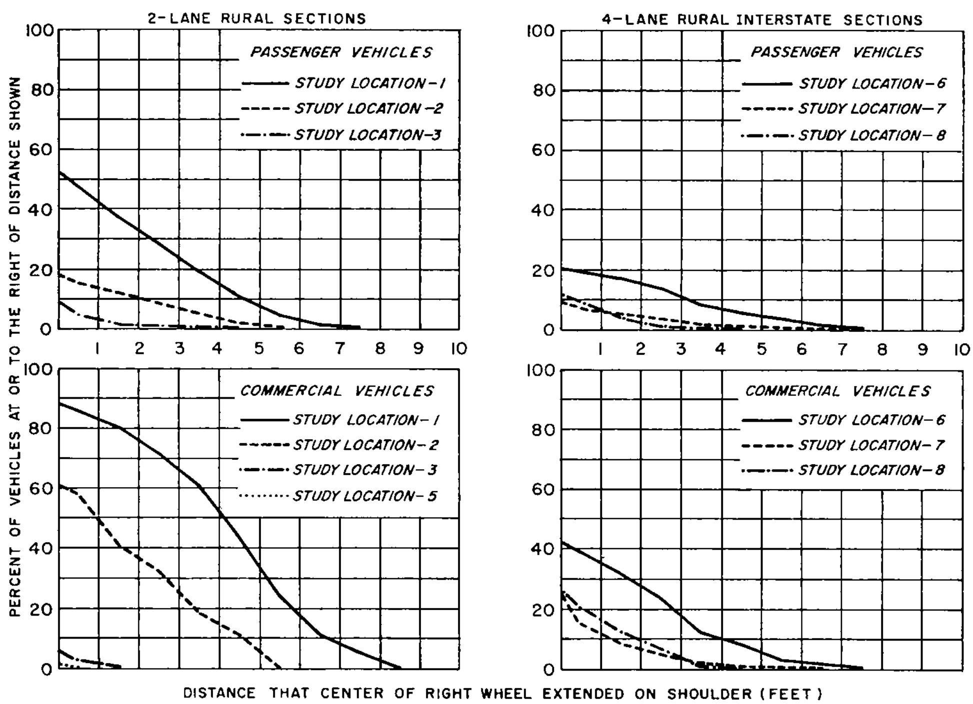

Figure 14. Shoulder encroachment on 2- and 4-lane sections with no shoulder stripes.

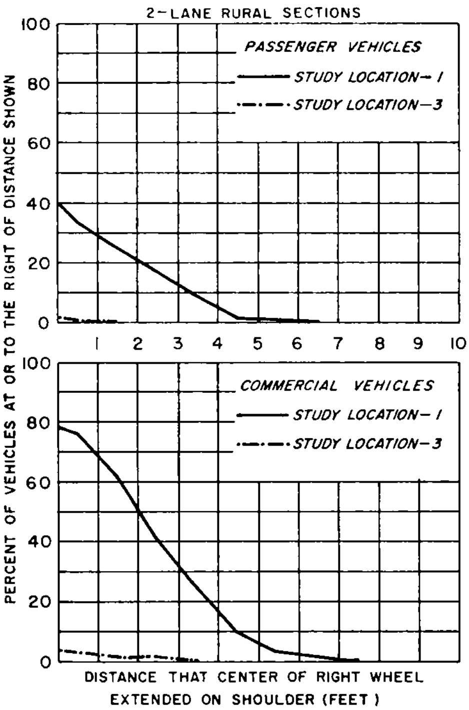

Figure 15. Shoulder encroachment on 2-lane sections with shoulder stripes.

tion 2) these average values decreased to 9.86 ft for passenger vehicles and 12.26 ft for commercial vehicles. This shows that the average path of the observed vehicles was approximately 2 ft closer to the roadway centerline when some surface texture and color contrast existed between the travel lane and shoulder.

Recordings at Study Locations 6 and 8 contain similar data for 4 Interstate sections. Study Location 6 has 10-ft outside shoulders having both color and surface texture contrast created by a light chip seal on the travel lanes and a bituminous plant mix on the shoulders. Study Location 8 has 10-ft outside shoulders having no color or surface texture contrast differentiating the shoulders from the travel lanes.

Traffic observations at these locations indicate that the average position of both passenger and commercial vehicles is approximately ½ ft closer to the roadway centerline on the section having shoulder contrast (Study Location 6).

From the shape of the ogives, the lateral shift towards the centerline is not so much from a change in travel for the portion of the vehicles traveling closest to the centerline, but rather by a lateral shift in position of the vehicles that before were traveling close to or even encroaching on the shoulders.

SHOULDER ENCROACHMENT

Any factor causing a change in the amount of shoulder encroachment on a section of a highway will also affect the average lateral placement of traveling vehicles on the highway. However, it cannot be reasoned that a factor found to affect the average lateral placement will necessarily change the amount of shoulder encroachment.

Figures 14 and 15 show observed shoulder encroachment on 2- and 4-lane sections with and without shoulder striping. It is found that commercial vehicles generally encroach on the shoulders more than do passenger vehicles on the wider road sections. Approximately 50 percent of the passenger vehicles and about 90 percent of the commercial vehicles (Fig. 14) traveled with the right wheels on the shoulder on a 40-ft section without shoulder contrast and without shoulder striping (Study Location 1). These percentages were reduced to 40 percent for passenger traffic and 80 percent for commercial traffic after the installation of a 2-in., solid, white, reflectorized, shoulder stripe (Fig. 14). Observations on narrow roadways show the same trend. On the 34-ft section (Study Location 3), approximately 5 percent of the passenger traffic and 10 percent of the commercial traffic encroached on shoulders having no contrast or striping. Application of a shoulder stripe reduced the encroachment to about 2 percent for passenger traffic and eliminated encroachment of commercial vehicles.

Generally, more shoulder encroachment was observed on the wider sections. Though over 50 percent of the passenger vehicles encroached on the 8-ft shoulders of the 40-ft section at Study Location 1, only 10 percent encroachment was found under similar conditions on the 5-ft shoulder of the 34-ft section at Study Location 3, and no encroachment at all for the 30-ft section at Study Location 5. Also, more encroachment by commercial traffic was observed on the wide sections than on the narrow sections. After installation of the 2-in., solid, white, reflectorized, shoulder stripe, it was found that, although 40 percent of the passenger traffic encroached on the shoulder on the 40-ft section (Study Location 1), less than 2 percent encroachment was found on the 34-ft section (Study Location 3) and no encroachment observed on the 30-ft section (Study Location 5). The same trends were detected for commercial traffic after installation of shoulder stripes.

Shoulder encroachment was less on 2-lane sections where some color or texture contrast between the shoulder and the travel lanes existed. Over 50 percent encroachment was observed for passenger traffic on the 2-lane, 40-ft section having no shoulder contrast (Study Location 1), and less than 20 percent encroachment was found on a similar section with shoulder contrast (Study Location 2). For commercial traffic the corresponding percentages are approximately 90 and 60 percent.

An opposite trend, however, was observed on the 4-lane Interstate sections. At Study Location 6, having surface and color contrast, a passenger vehicle encroachment of 20 percent was observed, while at Study Location 9 having no shoulder contrast, a 10 percent encroachment was observed. This same pattern was noticed for commercial traffic on the same sections. Unfortunately, the observations do not give any indication as to the reason for this trend.

TENDENCY TO TRAVEL IN THE SAME TRACKS

The average slope of the ogives plotted from the observations indicate the closeness with which the data group around the average value; in other words, the tendency of the drivers to travel in the same wheel tracks.

A measure for the slope of the curves is obtained by examining the distance between the upper and the lower quintile. These values are given in Table 4.

Data in Table 4 indicate that on the section with an 8-ft shoulder (Study Location 1), the observed passenger traffic between the quintiles traveled with the right front wheel on

TABLE 4
OBSERVED QUINTILES ON SECTIONS WITH DIFFERENT SHOULDER WIDTHS

Study Location No.	Figure No.	Shoulder Width (ft)	Lower Quintile	Upper Quintile	Distance Between Upper and Lower Quintile (ft)
1	1	8	8.4	14.4	6.0
3	4	5	7.5	10.1	2.6
4	5	4	6.8	8.8	2.0
5	7	3	6.9	8.4	1.5

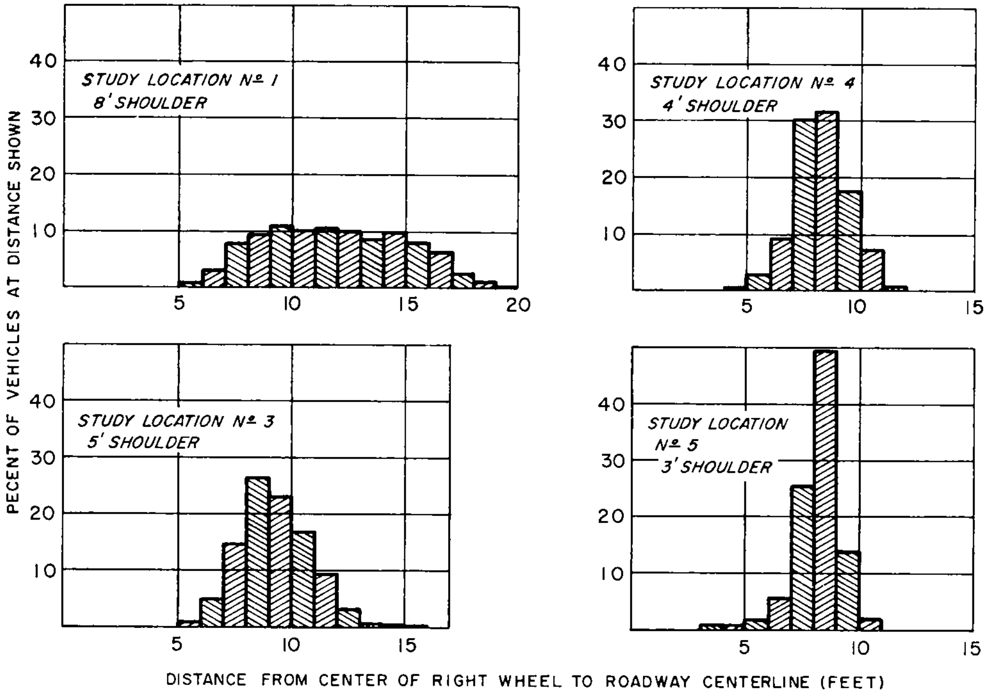

Figure 16. Distribution of wheel tracks for passenger vehicles on 2-lane sections.

a 6.0-ft wide area located between 8.4 and 14.4 ft from the roadway centerline. On the section with a 5-ft shoulder (Study Location 3) a 2.6-ft wide area was chosen located between 7.5 and 10.1 ft from the centerline. On the sections with narrower shoulders (Study Locations 4 and 5) even more restricted areas were chosen for the right front wheel. A study of the observations for commercial vehicles shows the same trend.

Figure 16 shows the distribution of the wheel tracks for the right front wheel of passenger vehicles. At Study Location 1 these wheel tracks of the vehicles between the quintiles covered 58 percent of the total roadway width including the shoulders. On the section with 5-ft shoulders (Study Location 3) the same wheel tracks covered 31 percent of the total roadway width. With a 4-ft shoulder (Study Location 4) 25 percent was covered and with 3-ft shoulder the percentage was 20 percent.

SUMMARY OF FINDINGS

1. The width of the shoulder influenced the lateral placement of vehicles. Both passenger and commercial vehicles traveled closer to the roadway centerline on sections with narrow shoulders than on sections with wide shoulders.
2. Both passenger and commercial vehicles traveled closer to the roadway centerline after the installation of 2-in., white, solid, reflectorized shoulder stripes. The greatest lateral shift was observed on commercial vehicles on sections having the widest shoulders.
3. Passenger vehicles traveled with the center of the vehicle closer to the roadway centerline than did commercial vehicles.

4. Vehicles with out-of-state licenses traveled closer to the roadway centerline than did vehicles with Idaho licenses.

5. The installation of signs with the legend "Do Not Travel On Paved Shoulder" tended to shift the average lateral placement towards the roadway centerline.

6. On sections with contrasting shoulders, the average travel path was located closer to the roadway centerline than on sections with no shoulder contrast. This effect held for both passenger and commercial traffic on both 2- and 4-lane study locations. Affected the most by the use of contrasting shoulders were those vehicles that with no contrast would travel closest to the shoulder edge.

7. More shoulder encroachment was observed from commercial than from passenger vehicles, and more encroachment was found on the sections with wide shoulders. The use of shoulder striping reduced the amount of encroachment. Less shoulder encroachment was observed on 2-lane sections with contrasting shoulders than on sections without shoulder contrast.

8. The narrower the roadway, the greater was the tendency for drivers of passenger vehicles to travel in the same wheel tracks.

Median Barriers: One Year's Experience and Further Controlled Full-Scale Tests

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Development of two types of barriers for use in the medians of California freeways was reported at the 39th Annual Meeting. As outlined in that report, it was planned to continue studying these two barriers under actual operating conditions.

This report covers one year of operation and additional full-scale collision tests of cable-chain link barriers. The before-and-after operational studies indicate that (a) the barriers were successful but need some improvement; (b) the total accidents increased when the barriers were installed, but the head-on fatalities were virtually eliminated; and (c) the maintenance cost of the cable-chain link barrier was more than the metal beam barrier, but this was offset by the higher first cost of the metal beam barrier. Controlled collision tests resulted in an improved design of the cable-chain link barrier.

• IN THE SUMMER of 1959, two types of median barriers were developed and tested for use on California freeways. These were the cable-chain link barrier, hereafter referred to as "cable," and the blocked-out metal beam barrier, hereafter referred to as "beam." Details of the barriers, and tests leading to their adoption, were reported in HRB Bulletin 266 (*1*).

Since the latter part of 1959, several miles of median barriers conforming to these developed designs have been placed on California freeways. The status of California's barrier construction program as of September 1961 is given in Table 1.

To compare the performance of the two types of barrier, the first contracts were split, providing some of each type in each contract. These are referred to as test sections. One test section was on the Santa Ana Freeway in Los Angeles where 3.17 mi of cable barrier were erected end-to-end with 2.57 mi of beam barrier; the other test section was on the Nimitz Freeway in Oakland where 3.87 mi of cable barrier were erected end-to-end with 2.87 mi of beam barrier.

TABLE 1

Barrier	Net Miles of Barrier-Divided Highway		
	Cable	Beam	Total
Constructed	31.5	16.8	48.3
Under construction	47.0	15.1	62.1
Budgeted (prelim. rept. received)	38.5	11.6	50.1
Total	117.0	43.5	160.5

Before-and-after accident records on these test sections have been examined. In addition, a complete operational study, including both construction and maintenance problems, has been made.

These studies indicated that (a) improvements in design details of the cable barrier were desirable and (b) more information was needed concerning the effect of the cross-section and profile of the highway surface on the trajectory of a fast-moving automobile.

The barrier deficiencies were analyzed and certain changes made. The corrected designs then were tested by a new series of controlled full-scale collisions. Exhibit 1 in the Appendix shows an over-all view of the collision test site. The results of both the operational study and the controlled tests are given in this report. Plates outlining the details of each test are included in the Appendix.

SUMMARY

1. Head-on accidents were virtually eliminated by the barriers. On the Santa Ana and Nimitz test sections, there were 49 cross-median accidents in the before period, including 8 fatal accidents, and there were two cross-median accidents in the after period, one of which was fatal.

2. Total accidents and injury-accidents increased in the locations where barriers were installed.

3. The freeway test sections with the cable barrier experienced a smaller increase in the over-all accident rate than did those with the beam barrier. There was no proof that the accidents involving the cable barrier were less severe. However, the findings of the controlled impact tests indicated that high speed collisions with the cable barrier would result in much less severe injury to vehicle occupants, and it is believed that in general the accidents involving the cable barrier are less severe.

4. The maintenance cost of the cable barrier is considerably higher than that of the beam barrier. First cost of the beam barrier is much greater than the cable barrier. It would require some 19.5 yr for the total expenditure to balance.

5. More accidents are evident involving the cable barrier. The proportion of single-vehicle accidents is much higher with the cable barrier than with the beam barrier. There is no indication that drivers are more reluctant to swerve into the beam barrier, but there are indications that there may be more hit-and-drive-away accidents involving the cable.

6. There was little difference in the cable barrier accident rate between the sections with 12- and 22-ft medians, and the maintenance cost per mile was essentially the same. There was no evidence to indicate that the deflection of the cables led to collisions by permitting momentary encroachment in the opposing lanes.

7. In installations other than the test sections, two vehicles climbed up and over the cable barrier and there were indications that others made partial climbs up the barrier.

Subsequent controlled collision tests indicated this tendency could be minimized by removing the lower cable from the original design. The revised design is shown in Exhibit 2, (Appendix).

8. In addition to the two vehicles that climbed the cable barrier two vehicles jumped barriers. One on the Santa Ana test section was apparently due to the car striking a curb in front of the barrier. The other was not on the test sections and was judged to result from the barrier being too low in relation to the plane of the roadway super-elevated surface.

Subsequent controlled collision

tests indicate that a 30-in. high barrier should be placed at or before the point of intersection of the shoulder slope and ditch slope. If it is necessary to place the barrier down the ditch slope, then it should be placed no further down the slope than will result in the top of the barrier being at least 27 in. above a horizontal projection from the point of intersection of the slopes.

9. Analysis of controlled collision test results indicate that the cable in a cable-chain link barrier should be placed no higher than 33 in. above and no lower than 27 in. above the ground line (or surface of control elevation).

10. Details of design of the cable barrier should be such that no fixed restraints exist insofar as the cable clamps or chain link fabric are concerned. A design incorporating these features as well as improvements for maintenance purposes is shown in Exhibit 2.

11. Expanded metal for a more effective headlight screen substituted in place of the chain link fabric makes little change in the cable barrier performance.

OPERATIONAL EXPERIENCE

Effectiveness of Barriers in Preventing Accidents

Both types of barrier have proven effective in accomplishing the purpose for which they were designed. Including installations in addition to the "test sections," they have been struck hundreds of times, and only two head-on accidents have occurred at locations where they are in place. The two head-on accidents were the result of vehicles that climbed or jumped clear over the barrier. Only one of these head-on accidents occurred within an experimental section. The other happened on the Ventura Freeway.

In addition to these two crossovers, which resulted in head-on collisions, two other crossovers took place. One was a small sports car that passed through the cable barrier under the top cable; the other resulted from a car traveling at high speed up the superelevation of a curve and jumping high enough to clear the top cable. The barrier in this latter case was located in the bottom of the ditch of a typical "saw-tooth" cross-section. Only the first of these accidents was within an experimental section.

Three partial crossovers occurred during the year. Two were within an experimental section, both of which involved truck-trailer combinations with the beam barrier on the Nimitz Freeway. In each case the barrier failed, but the trucks were stopped short of serious encroachment in the opposing lane. The third took place on the Ventura Freeway and involved a cable barrier. The automobile in this case came to rest on top of the barrier, half on one side and half on the other, but entirely within the median.

Effect of Barriers on Over-all Accident Record

As described previously, test sections of both types of barrier were erected on the Santa Ana Freeway and the Nimitz Freeway for the purpose of comparing the effectiveness of the two types of barrier.

Although there is no way of being sure that the differences between sections are attributable solely to the difference in type of barrier, it was thought that as many extraneous factors as possible would be eliminated by an end-to-end comparison on the same freeway where traffic volume remains approximately uniform, and, in fact, the very same vehicles pass by first one type of barrier and then the other.

TABLE 2

BEFORE-AND-AFTER RECORD, SANTA ANA AND NIMITZ FREEWAY TEST SECTIONS

	Barrier		All Reported Accidents						Injury Accidents [1]					
			Before		After		Change		Before		After		Change	
Test Section	Type	Length (mi)	No.	Rate Per MVM [2]	No.	Rate Per MVM [2]	In Rate	%	No.	Rate Per MVM [2]	No.	Rate Per MVM [2]	In Rate	%
Santa Ana	Cable	3.17	120	1.08	153	1.30	+0.22	+20	55	0.49	56	0.48	—0.01	— 2
	Beam	2.57	74	0.80	107	1.12	+0.32	+40	26	0.28	41	0.43	+0.15	+53
Nimitz	Cable	3.87	185	1.51	250	1.90	+0.39	+26	71	0.58	105	0.80	+0.22	+38
	Beam	2.87	146	1.55	216	2.13	+0.58	+37	—[3]	—[3]	95	0.94	—[3]	—[3]

[1] Including fatalities.

[2] Number per million vehicle-miles. Injury-accident rate of 0.49 per MVM shows over 2,000,000 car-miles (total life of about 30 cars) of travel accumulated for each accident serious enough to cause minor injuries to occupants.

[3] Where beam barrier installed, there was a change in enforcement agency jurisdiction between before and after periods. Because of change in reporting methods and definition of injury, rate in before period not known.

TABLE 3
BEFORE-AND-AFTER RECORD, HOLLYWOOD FREEWAY
BETWEEN HARBOR FREEWAY AND BENTON WAY[1]

Accidents	Before		After		Change	
	No.	Rate	No.	Rate	Rate	%
All	242	2.10	266	2.21	+0.11	+5
Injury	158	1.37	155	1.29	−0.08	−6

[1] Beam Barrier, 1.68 mi; ADT = 190,000.

Comparisons between cable barrier on one freeway and beam barrier on another should be interpreted very cautiously, because there are so many other potential variables that could affect accident rates that the difference owing to type of barrier can be smothered in irrelevancies.

The Santa Ana test sections were between the Long Beach Freeway and Buhman Avenue, and the Nimitz Freeway test sections were between High Street and Washington Avenue. These are both 6-lane freeways with 12-ft medians. The average daily traffic was between 90,000 and 100,000 on all sections. Grades are practically level and alignment is excellent.

Before-and-after statistics, using one year prior to construction as the before period and one year after completion as the after period (omitting the period during construction), are given in Table 2 for the Santa Ana and the Nimitz test sections. In addition, Table 3 gives the statistics for a section of the Hollywood Freeway.

The following points are made about the Santa Ana test sections:

1. Before barriers were erected on either section, the section where beam barrier was later erected had a much lower accident rate than the section where the cable barrier was erected.
2. The total accident rate increased significantly after erection of the barriers, on both sections.
3. The percentage increase on the section with the beam barrier was much greater than the percentage increase on the section with the cable barrier.
4. Although total reported accidents increased where the cable barrier was installed, accidents severe enough to cause injuries did not increase. Where the beam barrier was installed, the injury accident rate increased by 53 percent. This increase cannot be directly related to cars that crashed into the barrier, however.

The following points are made about the Nimitz test sections:

1. The over-all accident rates were about equal on both sections before the barriers were erected on either section.
2. The accident rates increased significantly after erection of the barriers, on both sections.
3. The increase in accident rate was somewhat greater on the beam section than on the cable section (37 percent against 26 percent).
4. During the after period, 42 percent of the reported accidents on the cable section resulted in injuries, and 44 percent of the reported accidents on the beam section resulted in injuries. This is about the normal ratio for all freeways.

Although not included in the test sections, the Hollywood Freeway installation is listed to show the effect of extremely congested traffic. The following points are made regarding this beam barrier installation:

1. The rates were high before and after. This is probably characteristic of extremely congested freeways. (Although these rates are considered high for urban freeways, they are still only about one-third the rate on urban arterials other than freeways.)
2. The barrier did not affect the rates, either over-all or injury.
3. The ratio of injury accidents to total accidents (60 percent) is very high. It is possible that many non-injury accidents are being overlooked on this section.

The barriers have generally resulted in an increase in over-all accidents, except on the Hollywood Freeway where the volume is 190,000. An earlier study (*2*) had indicated that barriers would increase accidents on roads where the volume is less than 130,000.

The percentage increase in both the all-accident rate and injury-accident rate was greater where the beam barrier was placed than where the cable barrier was placed, although the sample is so small and other unaccounted-for differences in rates are so large that these differences could be due to reasons other than difference in barrier types.

It may be significant that the rise in accidents on the cable barrier section of the Santa Ana was not accompanied by a rise in the injury-accident rate. However, the rise in injury accidents on the Nimitz cable section was just as great as the rise in all-accidents on this section.

The ratio of all-accidents to injury-accidents lies in the expected range of 2.2 to 2.8 in the before and after samples for both types of barrier in the test sections. This is significant because it shows that the increase in reported accidents is not comprised of mere fender-benders or fence-scrapers.

Accidents Involving the Median

Although head-on accidents were virtually eliminated by both types of barrier, in general there was a rise in accident rates where the barriers were installed on freeways having traffic volume less than 130,000 vehicles per day. One explanation would be that without a barrier many vehicles are able to encroach on the median without suffering a reportable accident, whereas after the barriers are installed, they strike a barrier. Table 4 gives the relation between the number of cars hitting the barrier and the rise in accidents when barriers are installed.

TABLE 4
PROPORTION OF INCREASE IN ACCIDENT RATES ACCOUNTED FOR BY ENCROACHMENT IN MEDIAN

Test Section	Barrier	Over-All Increase in Accidents (rate per MVM)	Accidents Involving Median			Proportion of Increase [a] (%)
			Before (rate per MVM)	After (rate per MVM)	Increase (rate per MVM)	
Santa Ana	Cable	0.22	0.17	0.33	0.16	73
	Beam	0.32	0.23	0.15	(0.08) [b]	0
Nimitz	Cable	0.39	0.22	0.53	0.31	80
	Beam	0.58	0.19	0.52	0.33	57

[a] Accounted for by accidents involving the median.
[b] Decrease.

In Table 4 a considerable proportion (57 to 80 percent) of the increase can be accounted for by collisions with the barrier except in the case of the Santa Ana beam section. Before reaching any conclusions, the over-all increase in the accident rate on the Santa Ana beam section was 50 percent greater than the increase on the Santa Ana cable section. The decrease in rate of accidents involving the median on the Santa Ana beam section is one of the inexplicable things frequently encountered when making a statistical study involving small numbers.

In events associated with barrier collisions, as Table 4 shows, a lot more drivers are getting involved with the median than before the barriers were erected. What the table does not show is the number of times the median was violated in the before period with no resulting accident.

There has been speculation that people are deliberately driving into the cable barrier on the theory that it is softer than the car ahead. In an effort to explore this possibility, Table 5 was prepared, classifying the accidents involving the barrier according to events preceding the collision. There is also a subjective classification in the right-hand two columns as to whether the vehicle was deliberately or involuntarily driven into the barrier. This classification represents the analyzer's judgment, based on reporting officer's opinion, statements by the drivers, and statements of witnesses, as well as on the events.

In addition to data on the test section of cable-chain link barrier, information is also included concerning a cable-chain link installation on the Ventura Freeway so as to point out certain differences probably influenced by different median design features.

From Table 5, the following points may be seen:

1. On the test sections, 58 percent of accidents involving the beam barrier were two-or-more-car accidents, whereas only 39 percent of accidents involving the cable barrier were two-or-more-car accidents. On the Ventura Freeway, only 20 percent involved more than one vehicle.

2. About one-fifth of the median barrier collisions were deliberate, a sort of "fielder's choice," in which the driver thought he was choosing the less severe consequences. This ratio was the same for the cable barriers as for the beam barriers, although on the Ventura Freeway only two "deliberate" swerves resulted in reported accidents. This freeway has 8-ft paved shoulders in the median, whereas the test sections on the Santa Ana and Nimitz Freeways have curbs and only a 6-ft half-width.

3. On the test sections, 86 percent of collisions with the beam barrier and 55 percent of collisions with the cable barrier were associated with maneuvers such as rear-end and sideswipe collisions or near-collisions.

4. On the test sections, 22 percent of the cable barrier collisions and 4 percent of the beam barrier collisions were due to erratic driving, drifting, and unknown reasons. Erratic driving refers to cars observed by witnesses to be driving erratically for some time before colliding with the barrier.

5. Unknown, miscellaneous, drifting, and sleep accidents (nearly all involving only one car) accounted for 19 of the 26 collisions with the cable barrier on the Ventura Freeway. This relatively high proportion is owing more to a lack of other kinds of accidents than to an excessive number of these kinds. The fact that there were only 7 accidents associated with rear-end and sideswipe maneuvers is probably attributable to the shoulders and absence of curbs. It was also determined that 16 of the 26 on the Ventura Freeway were at night.

TABLE

MEDIAN BARRIER ACCIDENTS CLASSIFIED BY ASSOCIATED EVENTS ONE

Barrier Type	Freeway	Single Vehicle		Multi-Vehicle		Avoid Rear-End Accidents: Deliberate Action		Avoid Rear-End Accidents: Lost Control		Unsafe Lane Change: Avoiding		Unsafe Lane Change: Making		Knocked into Barrier By: Sideswipe		Knocked into Barrier By: Rear-End	
		No.	%	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%
Chain	Santa Ana	23		15		8		2		2		1		3		1	
	Nimitz	37		23		5		12		8		0		2		5	
	Total	60	61	38	39	13	13	14	15	10	10	1	1	5	5	6	6
	Ventura	21		5		2		0		1		0		2		2	
Beam	Santa Ana	5		7		1		5		2		1		1		0	
	Nimitz	16		22		7		11		5		0		2		5	
	Total	21	42	29	58	8	16	16	32	7	14	1	2	3	6	5	10

[1] Because of sleep, drink, inattention, etc.

Repeated crash tests demonstrated conclusively that when a car collides with the cable barrier there is far less shock and that there should be far fewer injuries for a given number of barrier collisions. The first year's experience on the test sections is given in Table 6.

Contrary to expectation, experience of one year does not show conclusively that collisions with the cable barrier are less severe than with the beam. Observations and actual measurements of test crashes showed that deceleration rates, which are closely related to injury potential, are significantly less with the cable. There were so few serious injuries involving collisions with either type that it is believed the measured evidence of physical tests outweighs the statistical evidence, in which chance plays a major part.

Maintenance records show that the number of repairs of the cable barrier greatly exceeds the number of reported accidents involving the barrier. On the other hand, there have been reported accidents involving the beam barrier that did not require repairs.

Table 7 shows that collisions were much more likely to damage the cable barrier, and that for a given number of reportable accidents there is more disruption to traffic caused by barrier repairs, as well as additional maintenance cost. It does not necessarily show that there were more drive-away or hit-and-run collisions with either type, but it does show definitely that about one-third of the collisions with the cable barrier were so minor that the vehicles were able to drive away.

Construction and Maintenance Costs

Initial Cost.—By the end of the 1960-61 fiscal year, approximately 49 mi of barrier had been installed. Average unit prices for the barriers are given in Table 8. The unit price of beam barrier was 2.6 times that of the cable barrier. In later contracts, the unit price of cable barriers has declined.

Maintenance Costs.—Maintenance costs of the two types of barrier during the 1-yr period after construction are given in Table 9. The average yearly cost of repair is given in

5
YEAR AFTER CONSTRUCTION, TEST SECTIONS AND VENTURA FREEWAY

Ran into Barrier After: Side swipe No.	%	Rear-End No.	%	Erratic Driver No.	%	Drifted into Barrier [1] No.	%	Misc. No.	%	Unknown No.	%	Deliberate Action No.	%	Involuntary Action No.	%	Total Accidents No.	%
1		3		2		2		8		5		11		27		38	
0		1		8		3		14		2		11		49		60	
1	1	4	4	10	10	5	5	22	23	7	7	22	22	76	78	98	100
0		0		1		5		4		9		2		24		26	
0		1		0		0		0		1		2		10		12	
0		2		0		1		5		0		10		28		38	
0	—	3	6	0	—	1	2	5	10	1	2	12	24	38	76	50	100

TABLE 6
SEVERITY OF REPORTED ACCIDENTS INVOLVING MEDIAN BARRIERS

Test Section	Barrier	No. of Vehicles Colliding with Barrier	No. of Collisions Involving Injuries: Serious	Minor Wounds, Contusions	Total	Fatal Accidents: No.	Type
Santa Ana	Beam	12	0	3	3	0	
	Cable	38	3	9	12	1	Cross-median, head-on
Nimitz	Beam	38	7	11	18	2	1 suicide, 1 truck driver ejected when truck hit barrier
	Cable	60	8	19	27	2	1 motorcycle, 1 spinning car, occupants ejected

TABLE 7
COMPARISON OF REPORTED ACCIDENTS WITH BARRIER REPAIRS

Test Section	Barrier	No. of Accidents Reported	No. of Repairs
Santa Ana	Beam	12	25
	Cable	38	60
Nimitz	Beam	38	37
	Cable	60	91

TABLE 8

Barrier	Cost of Barrier ($): Per Lin Ft	Per Mi
Single metal beam	5.84	30,700
Double metal beam	8.31	43,800
Double metal beam on steel posts (structures)	14.53	—
Cable-chain link	3.25	17,100

terms of cost per mile, cost per accident, and cost per million vehicle-miles of travel or exposure. The unusually large cost per mile for the beam barrier on the Nimitz Freeway was due to the two accidents involving truck-trailer combinations.

The annual cost per mile of $2,078

TABLE 9

COST OF BARRIER REPAIRS FOR ONE YEAR AS REPORTED BY MAINTENANCE DEPARTMENT

Barrier	Freeway	Length (ft)	Million Vehicle-Miles	No. of Repairs	Cost ($)			
					Total for One Year	Per Repair	Per Mile-Year	Per MVM
Chain	Nimitz	3.87	131.65	91	6,879.53	75.60	1,777.66	52.26
	Santa Ana	3.17	117.21	60	7,848.16	130.80	2,475.76	66.96
	Subtotal	7.04	248.86	151	14,727.69	97.53	2,092.00	59.18
	Ventura	2.35	78.06	43	4,782.00	111.21	2,034.89	61.26
	Total	9.39	326.92	194	19,509.69 [1]	100.57	2,077.71	59.68
Beam	Nimitz	2.87	101.30	37	3,658.41	98.88	1,274.71	36.11
	Santa Ana	3.29	127.66	21	1,205.26	57.45	366.20	9.50
	Subtotal	6.16	228.96	58	4,863.67	83.90	780.00	21.25
	Bayshore	1.43	53.42	4	599.88	149.97	419.50	11.23
	Total	7.59	282.38	62	5,463.55 [1]	88.10	720.00	19.35

[1] Approximately 60 percent of this recovered from vehicle owners whose cars damaged barrier.

for the cable barrier was 2.9 times the $720 per mi cost of the beam barrier. With a $1,358 per mi difference in the annual cost of barrier repairs, it requires 19½ yr for the damage cost of the fence barrier to equal the difference in construction cost between the two barriers. However, approximately 60 percent of the damage costs have been recovered, hence the actual difference in the maintenance costs to the State was $540 per mi.

At $540 per mi per yr, it would require 49½ yr to make up the difference of $26,700 per mi in initial cost.

More important than cost is the hazard to both maintenance workers and the traveling public of continual maintenance in the median. There is also a certain amount of congestion caused by such operations. In this regard, comparison of the two types should include the bulkiness of equipment and size of crew required, the time per job, as well as the number of repairs required. The width of median is also important in this respect.

This report covers a limited amount of experience acquired during the year following initiation of the barrier construction program. Although there are indications regarding the effectiveness of the barriers, both in preventing cross-median head-on collisions and in increasing over-all accident rates, the experience so far should be interpreted with caution and only tentative conclusions should be made at this time. Additional data are being accumulated covering more extensive sections of barriers over a greater period of time. It is planned to continue the investigation. In the meantime, barriers are being installed on all 8-lane freeways and on freeways where the average daily traffic exceeds 60,000 per day. It has been shown in the 1959 study and confirmed by 1960 experience that four-fifths of all the cross-median head-on fatal accidents occur on these high-volume freeways.

OPERATIONAL FAILURES

Special detail studies were made from time to time of all accidents, as well as of each accident where vehicles passed over, went through, or climbed the barriers. These observations were made of accidents with all installations of the new designs of median barriers rather than only on the test sections.

Over all, during the past year three vehicles passed over new designs of barriers, three went through, and one came to rest on top of a barrier. The three crossovers all involved cable barrier. Two of these were the result of jumps due to causes unrelated to the cable barrier and therefore could have occurred over any 30-in. high barrier. One was the result of the vehicle hitting a curb and jumping high enough to clear the barrier cable. The second was a high-speed vehicle that jumped over the barrier after leaving the road on the outside of a curve. The barrier in this case was placed in a low ditch section where the roadway had been rotated to provide for superelevation. This provided the car with an inclined ramp from which to jump.

The third crossover and also the case of the vehicle coming to rest on top of the barrier also occurred on cable-chain link designs. The cause was the same in both cases. The original cable barrier design called for a tension cable attached 9 in. above the ground, and that the chain link fabric be firmly clamped between the lower cable and the post. In both of these accidents and in many others resulting in only partial climbing, it was found that the vehicles approached at a low angle (less than 15 deg) and high speed. Under these conditions of impact, a post and the firmly secured chain-link fabric, combined with the lower cable, served as a ramp for the front colliding wheel to get started in an upward direction. Such a start often elevated the automobile before the car body had an opportunity to penetrate the barrier far enough to provide for restraint by the top cable. Thus the car tended to ride the barrier down.

Two of the penetration-type accidents involved trucks colliding with the blocked-out metal beam barrier and resulted in a complete failure of the system. The third penetration involved a small sports car hitting the cable barrier between posts at a high angle of collision and passing between the lower and upper cables. This car had a front end clearance of 29 in. and an over-all height of 33 in., exclusive of the windshield.

A careful analysis of the above barrier crossovers indicates that they could be divided into two categories: one group that probably could be precluded by improvements in design and another that reasonably could not be prevented by a physical barrier. For instance, in the case where the vehicle hit the curb, the car apparently jumped higher than the 30 in. necessary to clear the barrier cable. Because cars have been reported, as a result of accidents, to have jumped as high as 8 to 10 ft and in other cases to have cartwheeled, it would not be reasonable to build a barrier high enough to contain every chance accident that might occur.

Because of the required strength involved, it is also not considered practical to design a barrier that will effectively and completely resist the collision of the heaviest trucks. In the two failures on beam barriers that occurred during this past year, the barrier was completely destroyed within the collision area. However, in neither case did the truck penetrate more than a few feet onto the opposing roadway. In other words, even in failure the barriers provided sufficient resistance so as reasonably to contain the vehicles.

The third penetration was an accident unique to the cable barrier in that very small sports cars can penetrate below the top barrier cable under conditions where the angle of collision is relatively high (over 30 deg). At smaller angles of collision it is probable that the fence post combined with the cable would still function as a positive barrier against penetration of this type of car.

Analysis of the above failures indicated that the beam barrier was functioning about as well as could be expected; however, it appeared that further development work should be done on the cable barrier. Studies indicated that it should be possible to make improvements to prevent the tendency of cars to climb the barrier and also that it would be worthwhile to investigate the possible prevention of penetration by sports cars.

The accident in which the car jumped the barrier after leaving the outside of a curve showed that further information should be gathered concerning the effect of differences of grade and elevation on the trajectory of a moving vehicle. Such information could be used to determine the placement of a barrier.

Maintenance of the cable barrier showed certain improvements of details to be desirable. Most of the effort during maintenance was expended in replacing the posts and concrete footings, so this detail was worthwhile of redesign. With one exception, no problems were encountered in maintaining the cables. In one case of collision with the cable barrier, it was necessary to cut the cable so as to remove the vehicle (in this case the vehicle was a truck-trailer combination). This break was repaired by a cable splice using cable clamps and presented no real problem.

CONTROLLED COLLISION TESTS

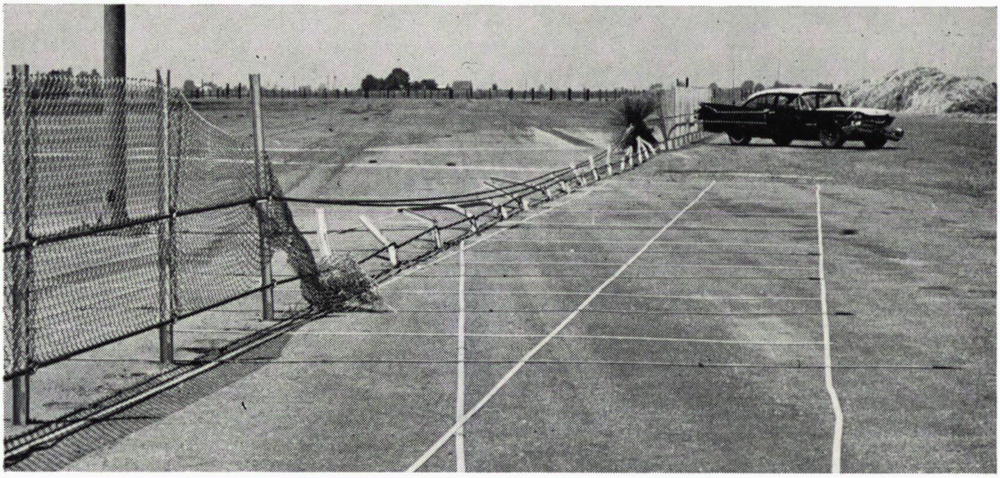
To develop details to correct the discussed failures, the nine tests were performed. In addition to testing corrective details, certain substitute details were also tested: (a) alternate post footings, (b) highway guardrail-type cable, (c) alternate cable turnbuckles, (d) cable splices, and (e) expanded metal light screen. Exhibit 3 (Appendix) shows the different footings tested and Exhibit 1 shows the over-all test site layout. Exhibits 4 through 12 give the pertinent facts concerning each test.

ANALYSIS OF CONTROLLED TESTS

A crossover type of accident considered intolerable is one where the vehicle climbs the side of a cable barrier and knocks it down as the vehicle passes on over. This type of accident is unique to the cable barrier. As stated previously, analysis of this type of crossover indicated that it was the result of a deficiency in the details of design rather than in the basic flexible barrier concept. Controlled collision tests for the purpose of analyzing these deficiencies were made at flat angles and high speed; first on the original design (Test 1), altered only by moving the chain link fabric outside the lower cables, and then by elimination of the lower cable entirely as in all tests following Test 1, except Test 7.

Elimination of the chain link fabric from the lower cable clamps resulted in an improvement in the action. However, high speed moving pictures revealed that the lower cable alone gave the left front end of the car an upward impetus as the front colliding wheel passed over the junction of the cable and the post. Thus, under certain circumstances it would be possible for the car to continue on upward. Removal of the lower cable resulted in penetration of the barrier by the vehicle with no tendency toward upward movement and no loss in barrier action. Post collision investigation of details of the damaged test barriers indicated that the elimination of the lower cable resulted in no loss of barrier effectiveness but did cause a slight loss in stiffness of the system behind the collision. However, any barrier damage due to this loss of rigidity was insignificant.

One of the original design considerations in placing the lower cable in the system was that it would serve



TEST NO. 1

Fabric: Chain link on impact side of barrier. Fabric under top cable but not contained under bottom cable.

Cables: 3 each 3/4 inch - 6 x 19 IWRC - 1 @ 9 inches and 2 @ 30 inches above pavement.

Post Footing: Type "A" 8 inch x 30 inch PCC (See Exhibit 3)

Purpose: To test current design for correlation with previous test series (1959). This test was also an attempt to duplicate the climbing that has occurred on the Ventura Freeway.

Performance: See Exhibit 4. Left front wheel raised 14 inches off pavement while climbing over lower cable. A slight yawing occurred near end of run with a violent 180° "spin out" approximately 100 feet from impact.

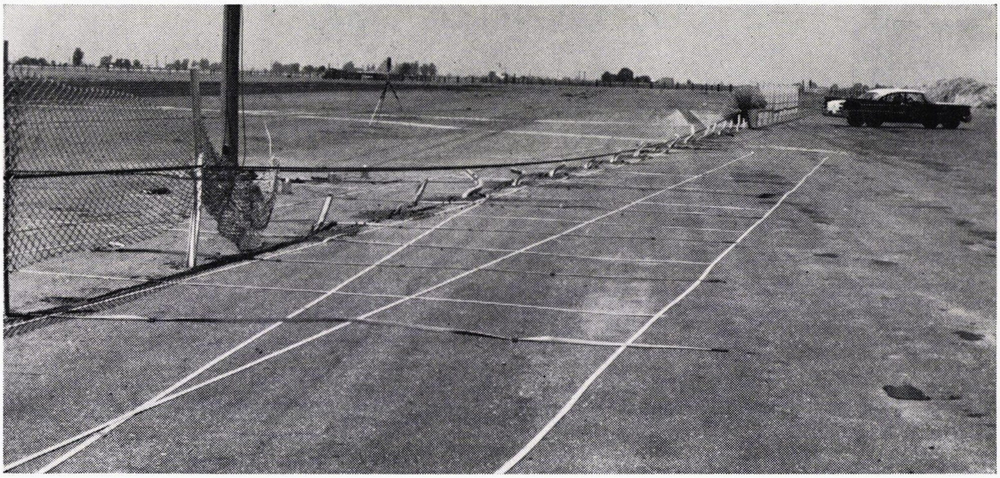
Maximum encroachment on traveled side: 21 feet.

Maximum encroachment on opposing side: 5½ feet.

Opposing side 4 foot or more encroachment for 6/10 seconds.

Opposing side 5 foot or more encroachment for 3/10 seconds.

Barrier Damage: Approximately 130 feet of mesh gathered up between top cables at point of spin-out. No cable fitting damage or failures. Damage was typical of that recorded during 1959 test series. Slight cracking of post footings was result of "green" concrete. No posts pulled out of footings. There was no appreciable movement of the post footings.



TEST NO. 2

Fabric: Chain link on impact side of barrier. "U" of cable clamp on impact side. Fabric contained under cable.

Cable: 2 each 3/4 inch - 6 x 19 IWRC @ 30 inches above pavement.

Post Footing: Type "A" 8 inch x 30 inch PCC (See Exhibit 3).

Purpose: To test current design with deletion of bottom cable. All other parameters same as Test No. 1.

Performance: See Exhibit 5. All wheels remained on pavement throughout run. A slight yawing occurred 30 feet before a violent 280° "spin out" approximately 110 feet from impact.

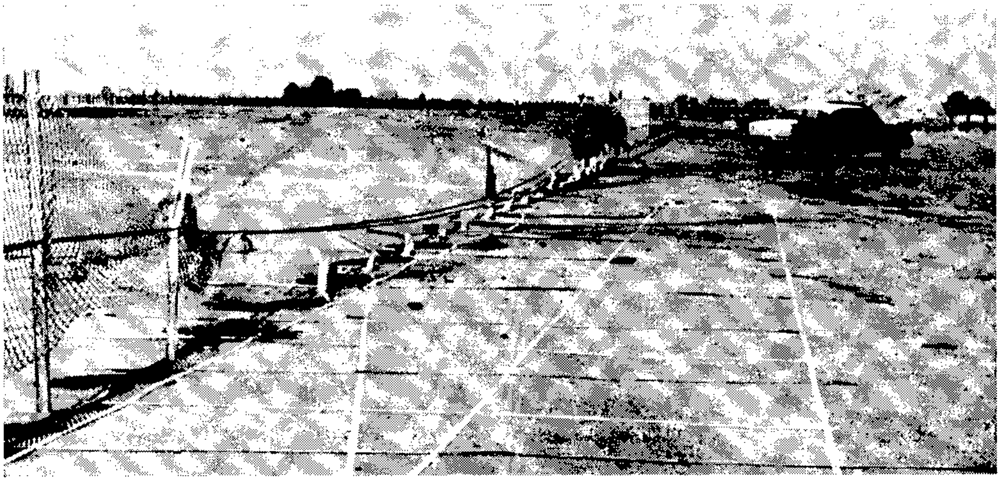
Maximum encroachment on traveled side: 29 feet.

Maximum encroachment on opposing side: 5½ feet.

Opposing side 4 foot or more encroachment for 6/10 seconds.

Opposing side 5 foot or more encroachment for 5/10 seconds.

Barrier Damage: Approximately 130 feet of mesh gathered up between cables at point of "spin out". Fabric and post damage was very similar to that of Test No. 1. Slight cracking of post footings was result of "green" concrete. No posts pulled out of footings. There was no appreciable movement of the post footings.



TEST NO. 3

Fabric: Chain link on impact side of barrier. "U" of cable clamp on impact side. Fabric contained under cable.

Cable: 2 each 3/4 inch - 6 x 19 IWRC @ 30 inches above pavement.

Post Footing: Type "D" Sheet Metal socket in PCC with 12 inch long wood wedge (See Exhibit 3).

Purpose: To test "2 cable" design with socket type post footings. All other parameters same as Test No. 2

Performance: See Exhibit 6. All wheels remained on pavement throughout run. No appreciable yawing. Violent 300° "spin-out" occurred approximately 125 feet from impact. Second post ahead of impact was pulled out of socket and carried down cables to point of "spin-out". Cable clamp was not completely stripped from post.

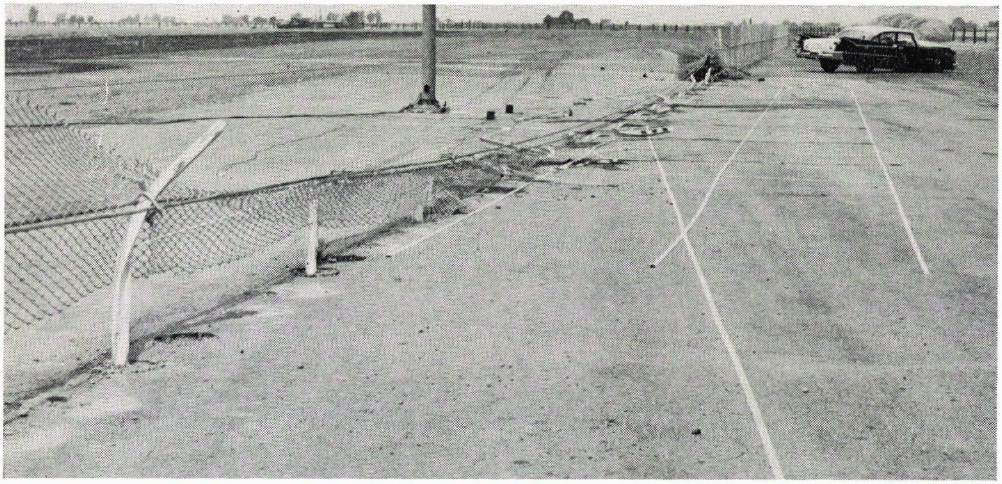
Maximum encroachment on traveled side: 23 feet.

Maximum encroachment on opposing side: 6 feet.

Opposing side 4 foot or more encroachment for 9/10 seconds.

Opposing side 5 foot encroachment for 6/10 seconds.

Barrier Damage: Approximately 125 feet of mesh gathered up between cables at point of spin-out. Fabric and post damage was very similar to that of Tests No. 1 and 2. One post pulled out of socket footing. Twenty posts pulled between 1/2 inch and 2 inches out of sockets. Slight cracking of footings was not severe enough to prevent re-use on Test No. 5.



TEST NO. 4

Fabric: Chain link and "U" of cable clamps on opposite side from impact. Fabric contained under cable.

Cable: 2 each 3/4 inch - 7 x 7 Highway Guard Cable @ 30 inches above pavement.

Post Footing: Type "C" 8 inch x 12 inch PCC (see Exhibit 3).

Cable Fittings: 2 each Type II Pipe Turnbuckles with swaged pulls located 100 feet ahead of point of impact.

Purpose: To compare with Tests No. 1, 2, and 3, the effect of collision with fabric fastened on opposite side from impact. To compare retention efficiency of 12 inch deep post footing with that of the 30 inch post footing. To compare susceptibility to jamming of cable clamps on pipe type turnbuckle with previous (1959) tests on Type I drop forged turnbuckles (see Exhibit 2).

Performance: See Exhibit 7. All wheels remained on pavement until spin-out. Fabric and clamps jammed at turnbuckle, tearing entire front fender off vehicle. Violent 270° "spin out" at 115 feet from impact. During first part of spin, left side of car raised 18 inches. Right front of car tore next to last post out of footing and stripped it from the cables. No yawing occurred during run.

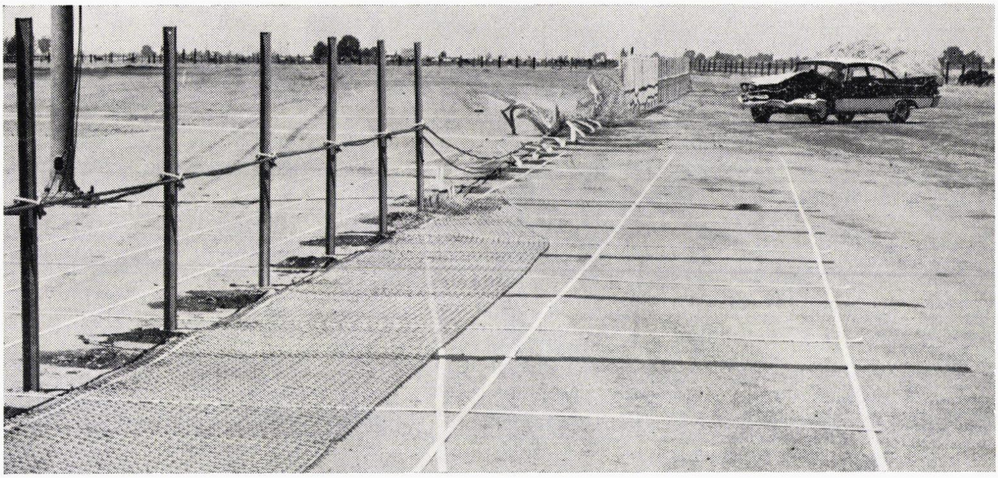
Maximum encroachment on traveled side: 19 feet.

Maximum encroachment on opposing side: 6 feet.

Opposing side 4 foot or more encroachment for 5/10 seconds.

Opposing side 6 foot or more encroachment for 1/10 seconds.

Barrier Damage: Approximately 90 feet of fabric gathered up between cables at point of "spin out". There was no failure of the turnbuckles; however, the cable was badly kinked adjacent to the cable pull on the front turnbuckle (impact side) and also 10 feet ahead of point of spin-out where post had pulled out of footing. Footing failure and cracked footings were result of "green" concrete.



TEST NO. 5

Fabric: Chain link on impact side of barrier. "U" of cable clamps on opposite side. Fabric not contained by cable.

Cable: 2 each 3/4 inch - 7 x 7 Highway Guard Cable @ 30 inches above pavement.

Post Footing: Type "D" Sheet metal socket with 3/4 inch x 2 inch x 24 inch wood wedges. (See Exhibit 3).

Cable Fittings: 2 each Type II Pipe Turnbuckles located 50 feet (impact side) and 58 feet (opposite side) ahead of point of impact. Preformed dead-end on each cable 100 feet behind point of impact.

Purpose: Repeat test on Type II Pipe Turnbuckles with fabric outside of cable not contained by cable or clamps. Also a repeat test on the socket type post footing with a longer wood wedge in an attempt to retain posts in sockets.

Performance: See Exhibit 8. Car started "spin-out" approximately 70 feet from point of impact due to posts #2 and #3 pulling out of sockets and jamming at the second turnbuckle located 58 feet ahead of impact. Posts, clamps and fabric passed over first turnbuckle located 50 feet ahead of impact and jammed on second turnbuckle. The vehicle carried entire bundle 10 feet further as dead-end located 100 feet behind point of impact failed. Fabric fastened with 12 gage steel tie wires was torn from posts for 70 feet behind point of impact and fell to pavement.

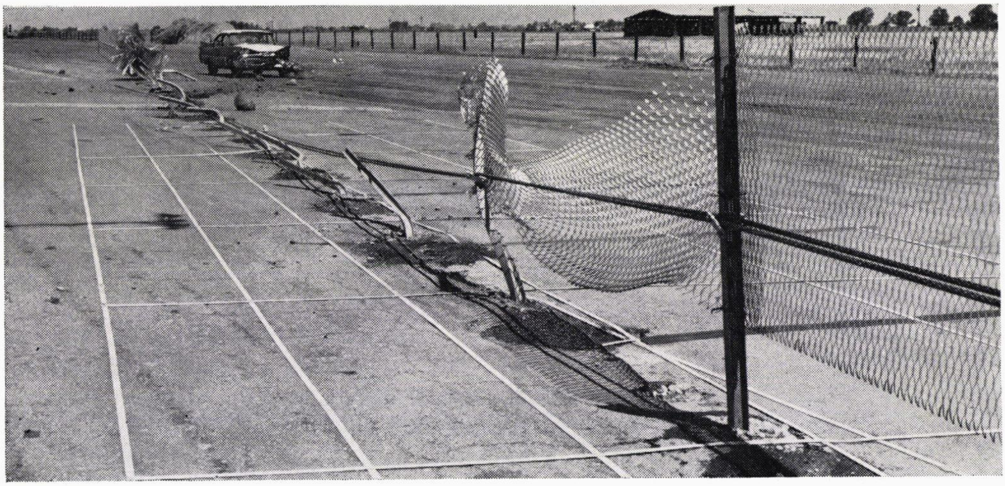
Maximum encroachment on traveled side: 24 feet.

Maximum encroachment on opposing side: 7 feet.

Opposing side 4 foot or more encroachment for 5/10 seconds.

Opposing side 6 foot or more encroachment for 2/10 seconds.

Barrier Damage: Preformed dead-end failed under extreme loading caused by posts and clamps jamming on turnbuckle. The dead-end had been removed from a cable used on a previous installation. During removal, a critical amount of aluminum oxide coating was stripped from the inside of the weave resulting in insufficient friction for the assembly to retain the cable under normal collision loads. Socket type post footings from Test No. 3 were re-used for this test. No additional cracking was noted.



TEST NO. 6

Fabric: Expanded steel mesh on impact side of barrier. "U" of cable clamp on impact side. Fabric not contained by cables. Fabric 18 gage galvanized steel, 1.33 inch x 3 inch diamond, 8 foot 4 inch x 42 inch panels.

Cable: 2 each 3/4 inch - 6 x 19 IWRC @ 30 inches above pavement.

Post Footing: Type "C" 8 inch x 12 inch PCC (see Exhibit 3).

Purpose: To test effectiveness during collision of expanded metal fabric compared to previous tests on chain link fabric. Also a re-test on the 8 inch x 12 inch concrete collar type post footing.

Performance: See Exhibit 9. All wheels remained on pavement throughout run with a very slight yawing of vehicle. Post No. 11 pulled out of footing; however, there was no measurable change in vehicle reaction when compared to Test No. 2 on 8 inch x 30 inch PCC footings. The expanded metal fabric reacted very similarly to chain link fabric under identical collision conditions. Very smooth deceleration to 160° spin-out at approximately 80 feet from impact.

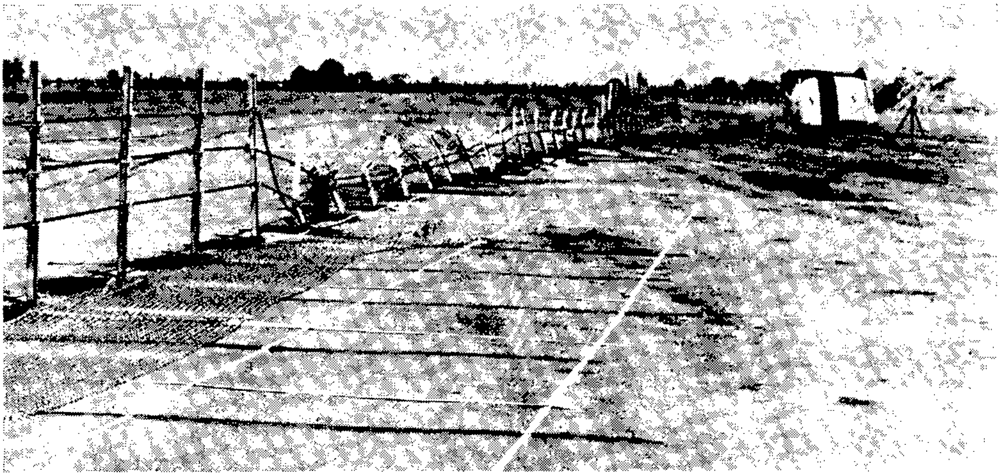
Maximum encroachment on traveled side: 18 feet.

Maximum encroachment on opposing side: 6 feet.

Opposing side 4 foot encroachment for 4/10 seconds.

Opposing side 5 foot encroachment for 3/10 seconds.

Barrier Damage: Fourteen panels (112 feet) of expanded metal gathered up at point of spin out. The severe cracking of all post footings in the collision zone and complete failure of one was due to "green" concrete.



TEST NO. 7

Fabric: Chain link on impact side of barrier. Fabric not contained by cable. "U" of clamp on impact side.

Cable: 3 each 3/4 inch - 6 x 19 IWRC @ 20 inches, 32 inches, and 44 inches above pavement.

Post Footing: Type "D" sheet metal socket in PCC with 3/4 inch x 2 inch x 24 inch wooden wedges. Voids filled with 200-300 pen. asphalt. (See Exhibit 3)

Cable Fittings: 3 each preformed cable splices 125 feet ahead of point of impact.

Purpose: To compare the efficiency of 3 cables at different heights with that of the preceeding tests on 2 cables at the same height. To test the retention of the posts in sockets by the addition of paving asphalt.

Performance: See Exhibit 10. The vehicle crossed the two bottom cables approximately 10 feet from point of impact and was retained by the top cable for 130 feet. All wheels were clear of the pavement for 80 feet. At a point 100 feet from impact, the vehicle was nose down on the left front wheel, rolling to an angle of 45 degrees and yawing to the left at a 30° angle. High speed data films show evidence that the compressed tire and spring of the front suspension, added to the lateral energy stored in the deflected cable, was impetus for the final roll of the vehicle in a direction opposite to that attained at the point of spin out.

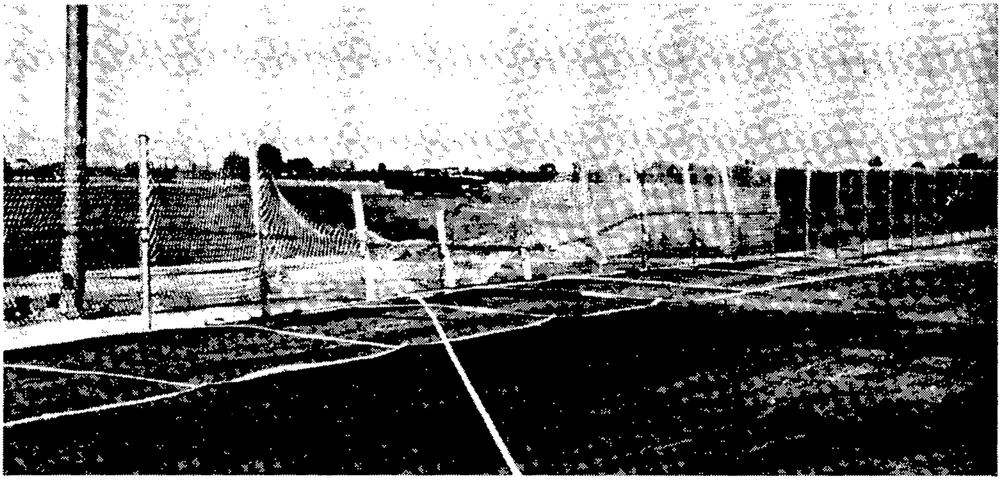
Maximum encroachment on traveled side: 18 feet.

Maximum encroachment on opposing side: 5½ feet.

Opposing side 4 foot encroachment for 1-1/10 seconds.

Opposing side 5 foot encroachment for 1/10 seconds.

Barrier Damage: The preformed cable splice located 125 feet ahead of impact failed as the windshield post sliding along the cable passed the frayed ends of the splice. At this location, the failure had no appreciable effect on the vehicle reaction; however, had the splice been installed 20 feet closer to the point of impact, the vehicle would have rolled over the barrier as the splice released.



TEST NO. 8

Fabric: Chain link on impact side of barrier. "U" of cable clamps on impact side. Fabric not contained by cables.

Cable: 2 each 3/4 inch - 6 x 19 IWRC @ 30 inches above slope grade, 18 inches above crown of ramp.

Post Footing: Type "B" sheet metal socket with 200-300 pen. asphalt. (See Exhibit 3)

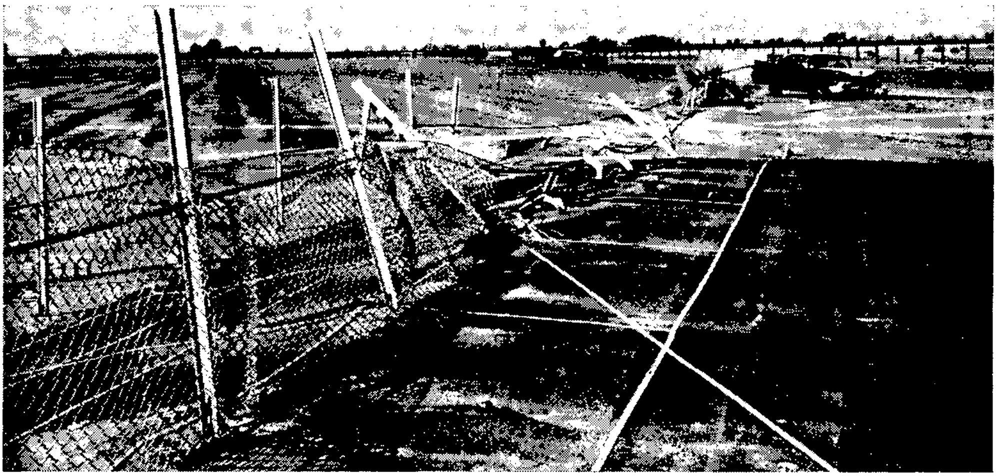
Cable Anchor: 3 foot diameter x 2 foot deep PCC.

Top Tension Wire: 7 gage spring steel wire 57 inches above slope grade.

Purpose: To test for retention of vehicle on a simulated 8 degree superelevated curve with 6:1 side slope in the median.

Performance: See Exhibit 11. The vehicle left the crown of the ramp and traveled airborne 17 feet to point of impact. Data films show that the vehicle had dropped only 2 inches along the trajectory from the crown of the ramp to the barrier. At the point of impact, the cables were below the center of the bumper at an effective height of 20 inches. The bumper contacted the post forcing it back and down and the cable was carried down with the post. The left front tire contacted the barrier at an intersection of post and cable further forcing the cable down and rolling over with no tendency to snag. As the vehicle progressed across the barrier each wheel was successively forced into its wheel well. There was no tendency for any part of the vehicle to snag on the cables. The 7 gage tension wire cracked the top of the windshield before failing at the 3/8 inch turnbuckle located 150 feet behind the point of impact.

Barrier Damage: Post collision height of cables at point of impact was approximately 8 inches above the crown elevation of the simulated super. No post footings were cracked.



TEST NO. 9

Fabric: Chain link on impact side of barrier. "U" of cable clamps facing impact. Fabric not contained by cable.

Cable: One 3/4 inch - 6 x 19 IWRC @ 30 inches above crown of superelevated ramp. Cable on impact side of barrier.

Post Footing: Type "B" sheet metal socket filled with 200-300 pen. asphalt (see Exhibit 3)

Cable Anchor: 3 foot diameter x 2 foot deep PCC.

Tension Wire: 7 gage spring steel @ 58 inches above crown of ramp.

Purpose:

1. To retest vehicle retention on an 8% superelevated curve by moving the barrier constructed for Test No. 8 to within 1 foot of the crown and placing the cable at 30 inches above a horizontal projection of the superelevation crown.
2. To determine the encroachment on the opposing side when barrier is constructed with a single cable.
3. To test the efficiency of the single cable envelope barrier design as a possible alternate method of construction on superelevated curves having a sloped center drainage ditch.

Maximum encroachment on traveled side: 16 feet

Maximum encroachment on opposing side: 12 feet

Opposing side 10 foot encroachment for 4/10 seconds: Measured from centerline of front cable.

Opposing side 2 foot encroachment for 2/10 seconds: Measured from centerline of rear cable.

Performance: At point of impact the cable made contact with the vehicle between the headlight and bumper and was contained in the fender over the front left wheel as it progressed through collision. The vehicle continued level and airborne as it left the ramp in a trajectory similar to that of Test No. 8 for approximately 20 feet. The snubbing action of the cable forced the front of the car down just prior to contact with the rear cable in the envelope design. The vehicle continued through collision snubbed nose-down by the front and rear cables through the transition of single cables to double cables with a very smooth deceleration to 180° spin-out at approximately 130 feet from point of impact.

Barrier Damage: 125 feet of fabric was gathered up at point of spin out. Seven post footings were cracked due to "green" PCC. None of the posts were moved from the asphalt filled sockets.

to trap the car in the median area as it attempted to return to the on-side roadway at the end of the collision path. Operation experience showed that at a flat angle of collision, whether or not the lower cable was in position, the car tended to spin at the end of the collision path back into the traveled lane. This was verified by test collisions. A review of the accident reports from both the Nimitz and the Santa Ana test sections indicated that this vehicle reaction was typical of a majority of the collisions that occurred on the freeway and that in no case had a secondary collision resulted from this spin-out.

Two other details of construction were tested and adopted as a result of these studies. The original design called for a standard turnbuckle every 500 ft along the cable. Because the smoothness of deceleration of the colliding vehicle with the cable-chain link barrier depends primarily on the friction brake effect of the cable clamps stripping from the posts, it is important that this action proceed unhindered if possible. Test 4 showed that when the test collision vehicle progressed along the cable through a turnbuckle, the clamps and the contained mesh jammed at the turnbuckle. This resulted in an abrupt deceleration and violent spin-out of the colliding vehicle. This defect was also illustrated by the General Motors (*3*) tests of 1960.

Tests 4 and 5 were made to judge the effect of repositioning chain link fabric outside the cables. This design eliminated binding, and at the same time the removal of the chain link to the outside of the cable had no appreciable effect on the rate of deceleration of the car.

As was originally anticipated, the cable-chain link barrier on the Nimitz and Santa Ana test sections were subjected to a great deal of collision damage. The Maintenance Department found the most costly single item was the removal and replacement of the steel posts and their concrete footings. In addition to replacing the concrete and post, it was necessary for the posts to set in the new footings for at least 24 hr before the cable and chain link could be rehung on the post. This required two trips under heavy traffic conditions. It was decided that, if economically feasible, a post socket design or an otherwise modified footing could solve this problem.

Two designs were developed and successively tested. One was a concrete collar around the upper 1 ft of the footing and the second a socket in a full depth footing. In the first, the principle was that the earth below the collar would furnish support for the barrier while the collar was curing. Thus the barrier fabric and cable could be re-erected immediately.

In two test collisions, the concrete collar-type footing was used and in five the socket type.

The collar-type footing proved adequate. However, several of the footings broke during collision. It was therefore necessary to remove the concrete piecemeal before backfilling the hole and redrilling for the new footing. Though this design proved adequate, it is considered practical only for locations where the soil is fairly tight and free of rocks.

Several methods of holding the posts in the sockets were considered. Among these were the use of steel wedges, bolts, set screws, wood wedges, sand (plain and also topped with sulfur), sulfur, and asphalt. All were discarded in favor of asphalt. However, it was considered necessary to determine the minimum restraint needed to keep the post in place during collision. Therefore, tests were made using an oversize socket with the posts held in place only with wooden wedges (Tests 3 and 5). In each of these two tests at least one post pulled out.

Analysis of the pictures indicates

that during the early part of a collision, the posts are subjected to a substantial vertical force. Sufficient resistance must be offered to prevent uplift of the posts during this period of vertical loading.

These two tests proved that wooden wedges alone provided insufficient resistance. Therefore, because sand has little or no internal resistance, it also was discarded. Steel wedges, throughbolt or set screws could be made to work but were discarded because of cost and possibility of jamming. Sulfur would also work but was discarded because of potential corrosion in addition to the difficulty of cleanout or reheating of the sulfur during repair.

After completing tests using wooden wedges alone, the sockets were filled with asphalt which in Tests 7, 8, and 9 proved to be adequate. Tests 8 and 9 used sockets that fit the posts, but the socket for Test 7 was oversized with the space taken up by a wooden wedge. Grade 200-300 paving asphalt was chosen. Asphalt proved able to resist the shock loading with no movement. At the same time, the damaged post could be removed by a slow pull and a new post placed by slow pressure. In a controlled laboratory test, it was found that a pull of 700 lb was necessary to remove a post from an asphalt-filled socket when tested at 0 F. It took 1 min to complete the removal.

Of particular interest are Tests 8 and 9. Here a cross-section of highway found on many California freeways was simulated in which an 8 percent superelevation intersected a 6:1 sloped center drainage ditch. Actual barrier installations have been placed in the center of the ditch which is coincident with the centerline of the freeway median area, with the thought that the cars on a collision course would follow the 6:1 side slope down to the barrier. Test 8 showed that when the barrier was 6 ft away from the edge of the shoulder at the bottom of the simulated ditch, the car traveling at a 20-deg angle of collision would pass on over the barrier. After a study of the car's trajectory this was remedied in Test 9 by duplicating the previous test conditions but moving the barrier up the slope of the ditch to within 1 ft of the edge of the simulated shoulder, thus giving the car an opportunity to penetrate the barrier and become engaged under the cable.

Analysis of the trajectory of the car in the pictures of Test 8 indicates that a barrier to be effective should be placed no lower than 27 in. above a horizontal projection from the top edge of an approaching 8 percent grade. This is about the maximum superelevation that will be encountered in roads that justify the use of median barriers. The best solution for this condition is to place the barrier at the top or before the top of the superelevation. If it is necessary to place the barrier on the ditch side of the cross-slope, then the barrier cable should be no lower than 27 in. above the crown nor higher than 33 in. above the ground surface.

Test 6 used expanded steel mesh instead of chain-link fabric. No difference in barrier action was noted. However, first cost and maintenance costs of the expanded fabric will be markedly higher than for the chain link-fabric. This is due primarily to the higher first cost of the expanded metal, but it will also be affected by the fact that this material is furnished at present only in short panels.

In Tests 4 and 5 the use of 7 by 7 highway guard cable was substituted for the 6 by 19 IWRC cable usually used. This 7 by 7 cable was more difficult to handle during repairs and placement. In addition its cross-section did not lend itself to proper adjustment during the tightening of the cable to post clamps. In general these tests seemed to indicate that the

TABLE 10

Camera No.	Type	Rate (frames/sec)	Lens	Location
1	Fastax	1,200	12.5-mm	100 ft behind barrier
2	Photosonics	400	12.5-mm	Tower covering preimpact
3	Photosonics	400	12.5-mm	Tower covering impact
4	Photosonics	400	12.5-mm	Tower covering post impact
5	Photosonics	400	4-in.	Rear ground mount
6	Photosonics	400	4-in.	Front ground mount
7	Photosonics	400	12.5-mm	In crash vehicle
8	Photosonics	400	1-in.	Rear platform

cross-section and relative stiffness of the 7 by 7 cable to the 6 by 19 cable make the latter cable more desirable for use as a flexible barrier member.

TEST PROCEDURE AND INSTRUMENTATION OF TEST COLLISIONS

With the exception of the type of the cars, and speed and angle of approach, this series of tests was conducted in the same manner as the full-scale tests reported in 1960 (*1*).

So as to simulate more nearly the type of accident that seemed to cause problems with the cable barrier, heavier cars (over 4,000 lb) driven at higher speeds (over 80 mph) and colliding at flatter angles (10 deg or less) were used.

Because this series of tests was designed to test refinements of design rather than the over-all effectiveness of the barriers, the instrumentation was not as complex as that previously used. Decelerations were determined from an analysis of the high-speed data films rather than from decelerometers mounted in the vehicle and the dummy.

The anthropometric dummy was unrestrained, and his movements through collision were observed by a high-speed data camera mounted inside the vehicle.

The photographic instrumentation was approximately as used previously, except that the cameras differed from those listed in the previous test (*1*). The 16-mm data cameras gave 100 percent reliability rather than the 25 to 50 percent reliability obtained in the past (Table 10).

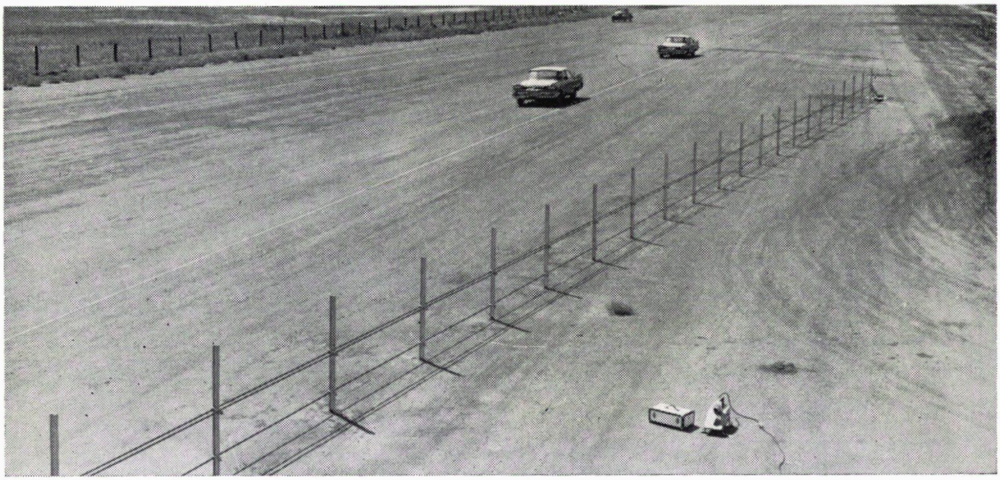
In addition to the 16-mm cameras, one Bolex and one Bell and Howell 24 frames per sec camera were placed at various locations for documentary coverage. All sequence pictures shown in the exhibits were recorded with a 70-mm Hulcher Mod. 20 camera at a rate of 20 frames per sec.

REFERENCES

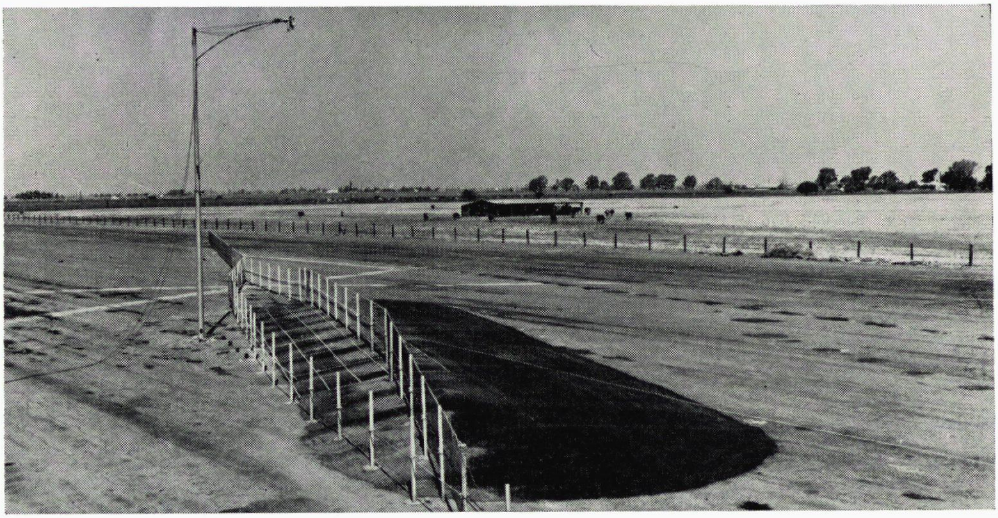
1. BEATON, J. L., and FIELD, R. N., "Dynamic Full-Scale Tests of Median Barriers." HRB Bull. 266, 78-125 (1960).
2. MOSKOWITZ, K., and SCHAEFER, W. E., "California Median Study: 1958." HRB Bull, 266, 34-62 (1960).
3. CICHOWSKI, W. G., "Automobile to Chain Link-Cable Barrier Collision." General Motors Report PG-12387.

APPENDIX

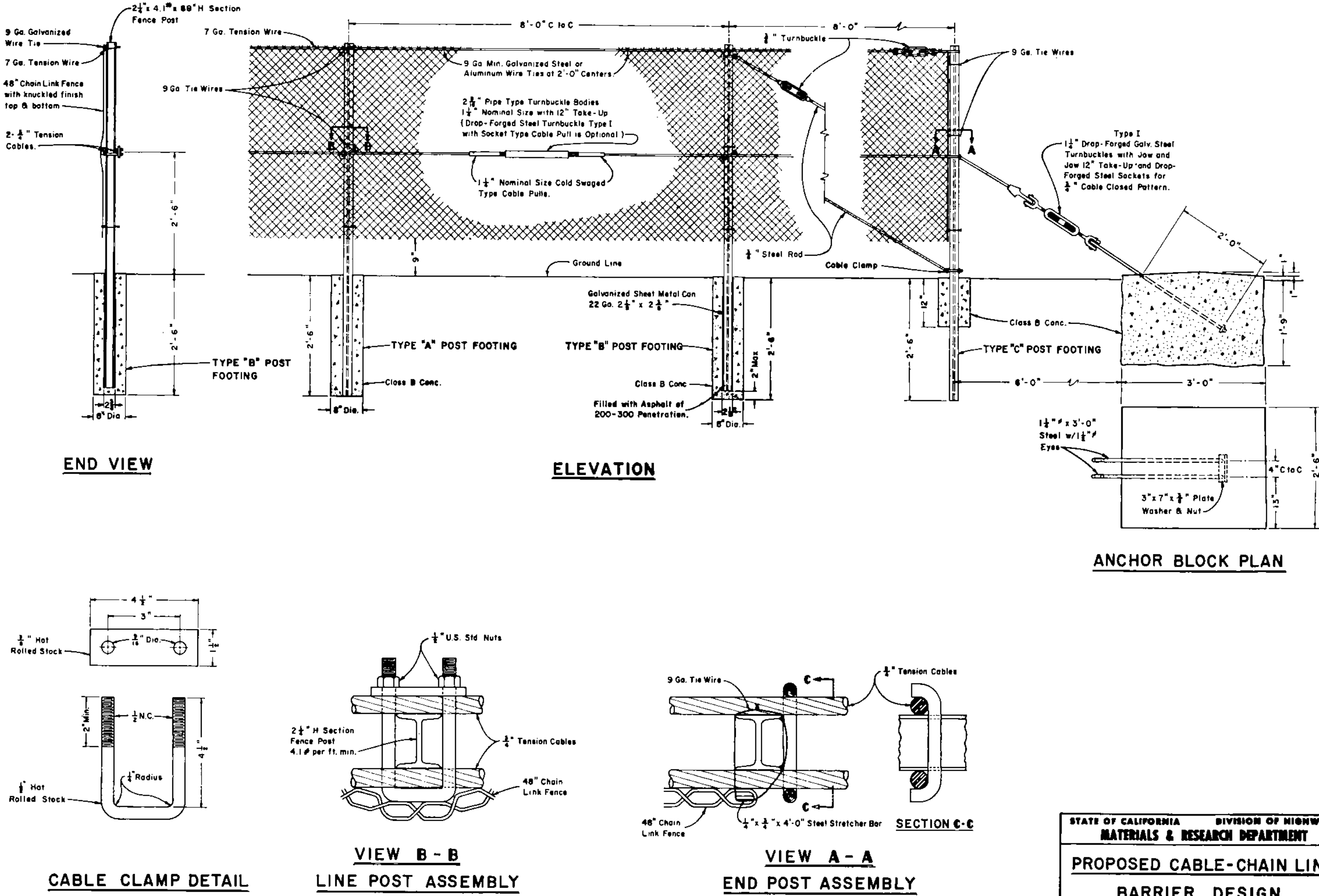
EXHIBIT 1



Crash vehicle on 7° collision course, followed by control car. Chain link fabric was deleted from first 200 ft on 600-ft installations. Ground-mounted data camera in right foreground.



Typical photographic instrumentation installation, showing data camera tower, ground grid, and guide tape intersecting point of impact at 20°.

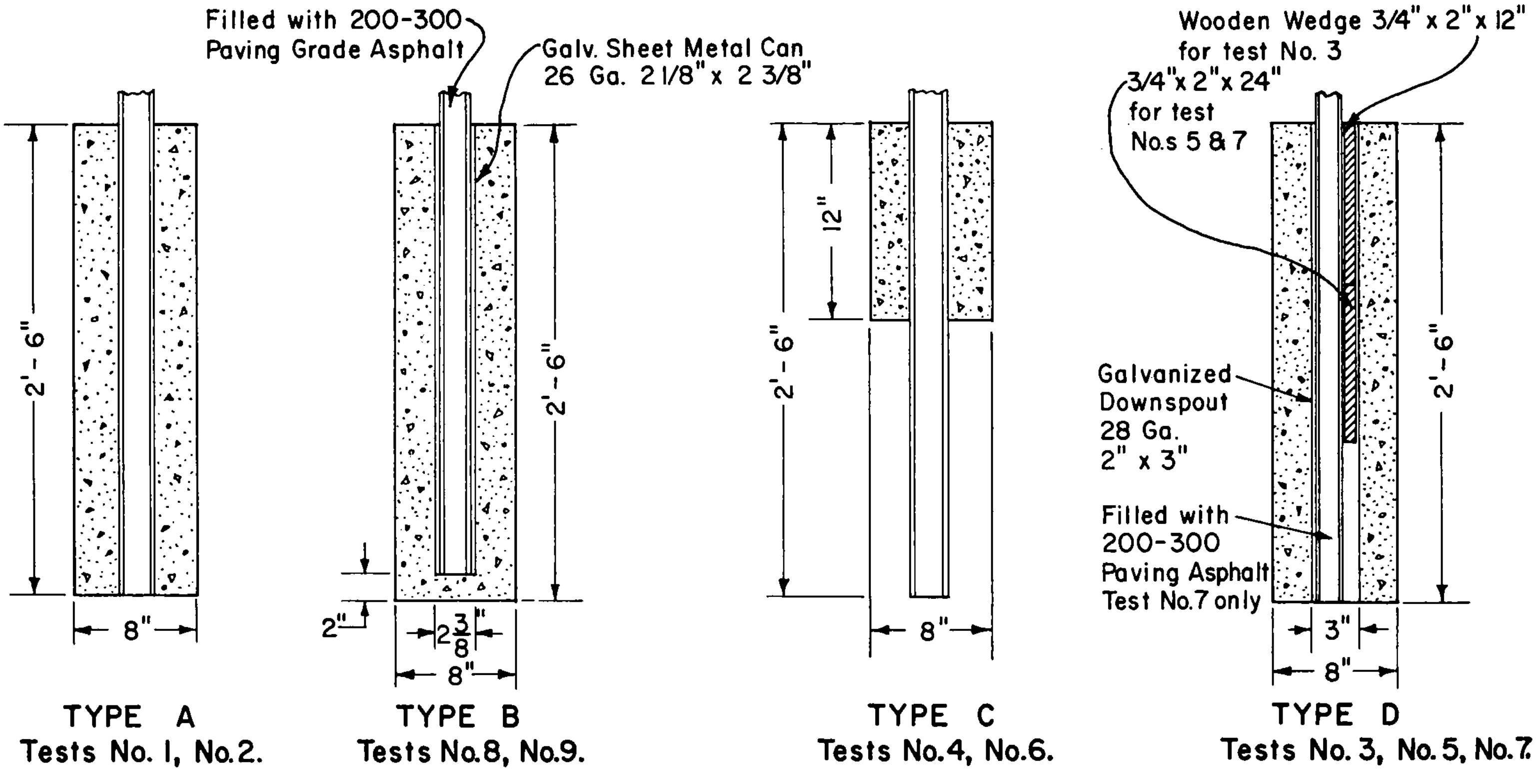


STATE OF CALIFORNIA DIVISION OF HIGHWAYS
MATERIALS & RESEARCH DEPARTMENT

PROPOSED CABLE-CHAIN LINK BARRIER DESIGN

DATE 12-1-61 | DRAWN BY R.P. | CHECKED BY R.N.F.
SUBMITTED BY Robert N. Field
APPROVED BY John L. Beaton
C-466
SHEET NO. 1 OF 1 SHEETS

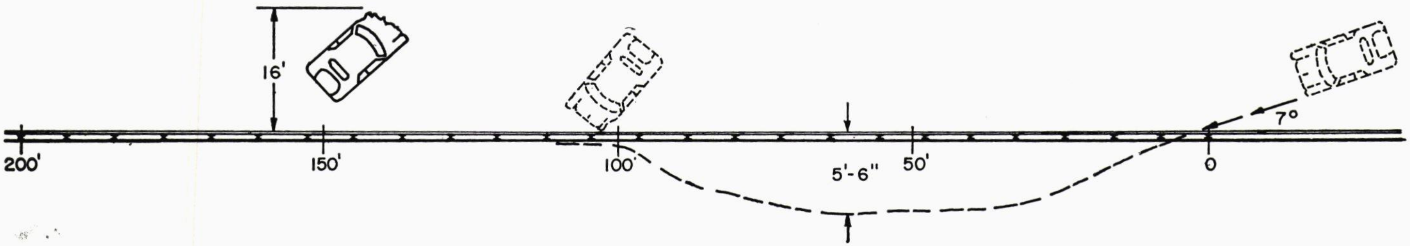
EXHIBIT 2



NOTE: CLASS B CONCRETE USED FOR ALL TYPES

POST FOOTING DETAILS

EXHIBIT 3



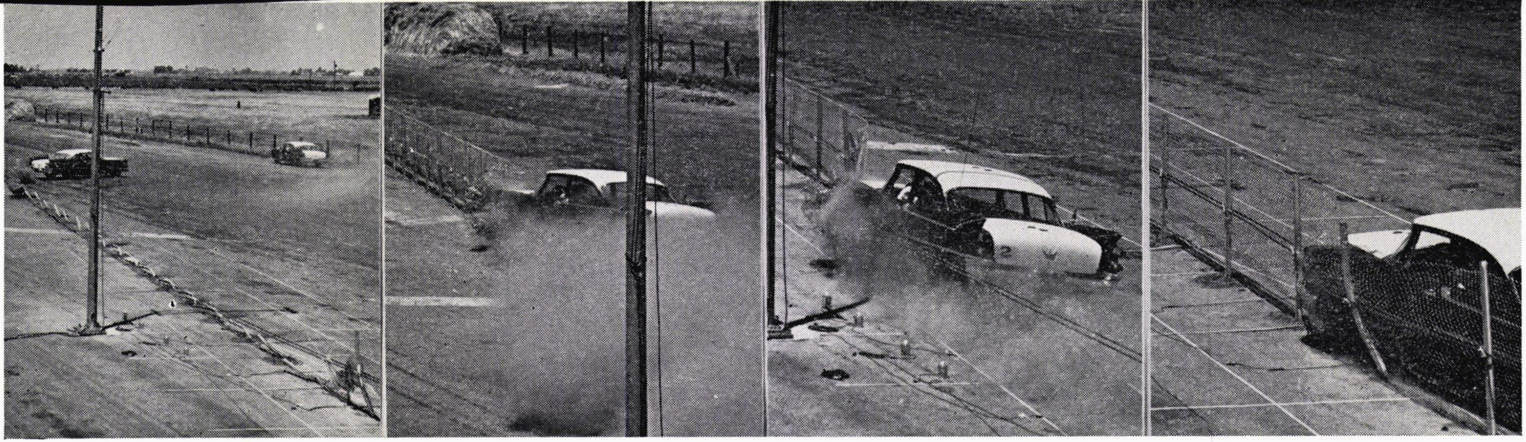
FENCE	48" chain link at 9" above pavement.
CABLE	3/4" 6x19 IWRC 1 at 9" and 2 at 30" above pavement.
POST FOOTING	Design A
POST SPACING	8' O.C.
LENGTH OF INSTALLATION	600'
GROUND CONDITION	Dry

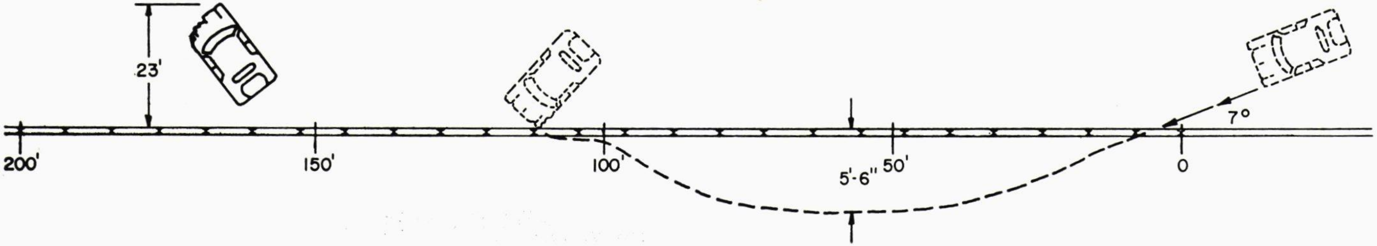
FENCE DAMAGE	152' damaged
CABLE FITTING DAMAGE	none
CABLE DAMAGE	none
POST DAMAGE	19 damaged
POST FOOTING DAMAGE	7 cracked
MAX DYNAMIC DEFLECTION OF CABLES	5 1/2
VEHICLE DECELERATION (PEAK LONG)	6.5g's
VEHICLE DAMAGE	$350

TEST NO.	1
DATE	6-8-61
VEHICLE	Dodge 59 Sedan
SPEED	78 MPH
IMPACT ANGLE	7°
VEHICLE WEIGHT	4300 lbs. (W/DUMMY & INSTRUMENTATION)

EXHIBIT 4

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS — MATERIALS & RESEARCH DEPT





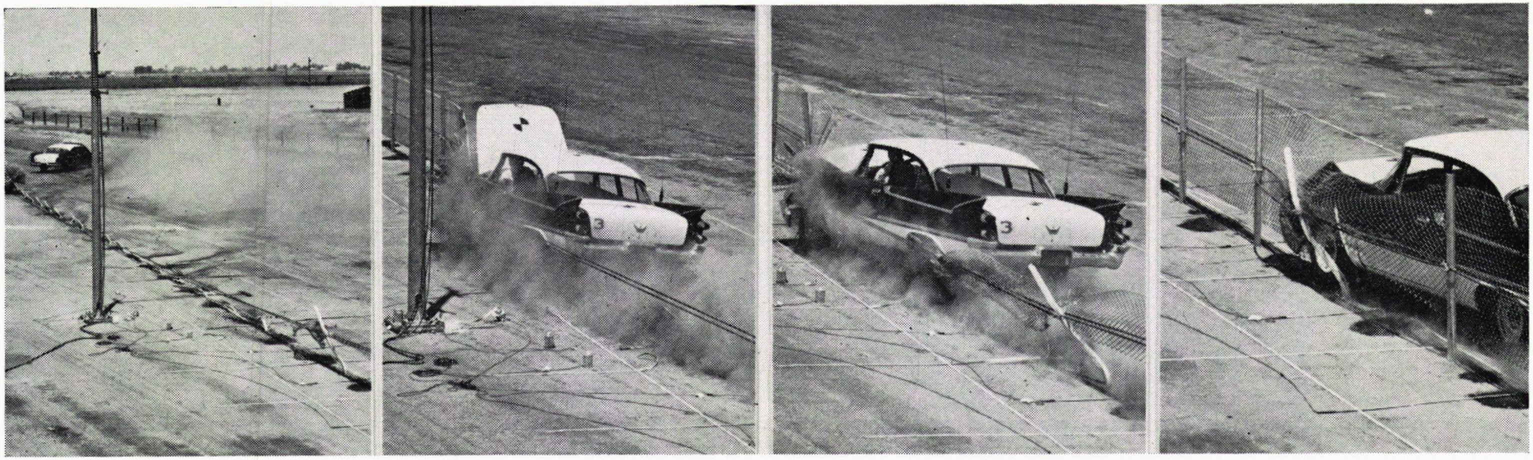
FENCE .... 48" chain link at 9" above pavement.
CABLE .... 3/4" 6 x 19 IWRC 2 at 30" above pavement.
POST FOOTING .... Design A
POST SPACING .... 8' O.C.
LENGTH OF INSTALLATION .. 600'
GROUND CONDITION ...... Dry

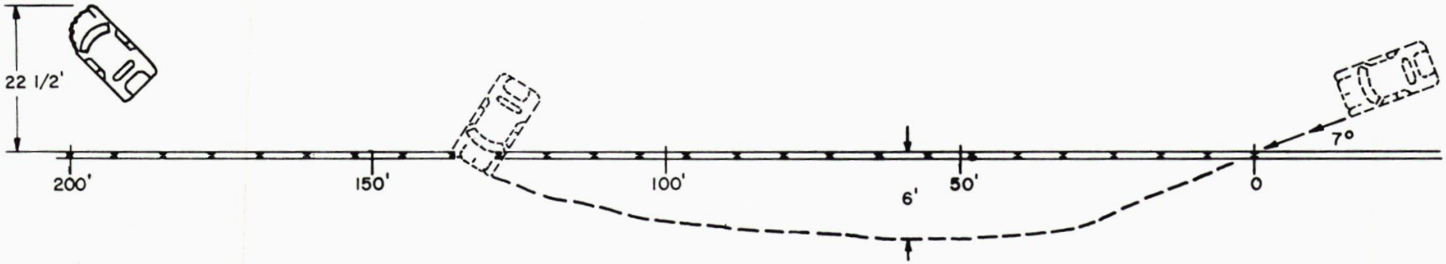
FENCE DAMAGE .... 140' damaged
CABLE FITTING DAMAGE .... none
CABLE DAMAGE .... slight fraying
POST DAMAGE .... 19 damaged
POST FOOTING DAMAGE .... 12 cracked
MAX. DYNAMIC DEFLECTION OF CABLES .. 6'
VEHICLE DECELERATION (PEAK LONG.) .. 2.4 g's
VEHICLE DAMAGE .... $ 350

TEST NO. 2
DATE 6-13-61
VEHICLE .... Dodge 59 Sedan
SPEED .... 84 MPH
IMPACT ANGLE ... 7°
VEHICLE WEIGHT ... 4300 lbs.
(W/DUMMY & INSTRUMENTATION)

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS — MATERIALS & RESEARCH DEPT.

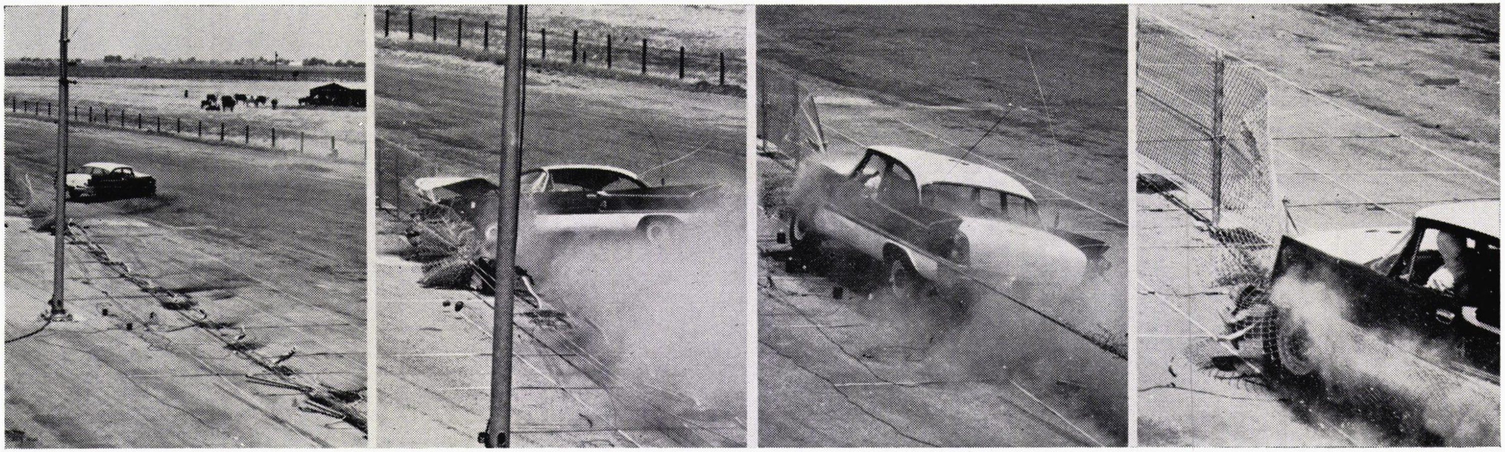
EXHIBIT 5

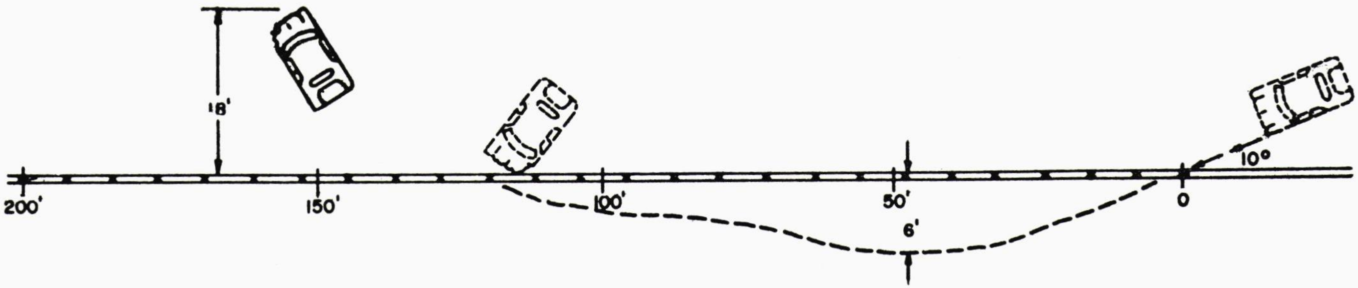




FENCE	48" Chain link at 9" above pvmt.	FENCE DAMAGE	152' Damaged	TEST NO.	3
CABLE	3/4" 6 x 19 IWRC 2 at 30" above pavement.	CABLE FITTING DAMAGE	None	DATE	6-19-61
		CABLE DAMAGE	None	VEHICLE	Dodge59 Sedan
		POST DAMAGE	20 Damaged - 1 Pulled Out	SPEED	86 M.P.H.
POST FOOTING	Design D	POST FOOTING DAMAGE	5 Cracked	IMPACT ANGLE	7°
POST SPACING	8' O.C.	MAX. DYNAMIC DEFLECTION OF CABLES	6'	VEHICLE WEIGHT	4300 lbs. (W/DUMMY & INSTRUMENTATION)
LENGTH OF INSTALLATION	600'	VEHICLE DECELERATION (PEAK LONG).	4 G's		
GROUND CONDITION	Dry	VEHICLE DAMAGE	$ 350		

EXHIBIT 6





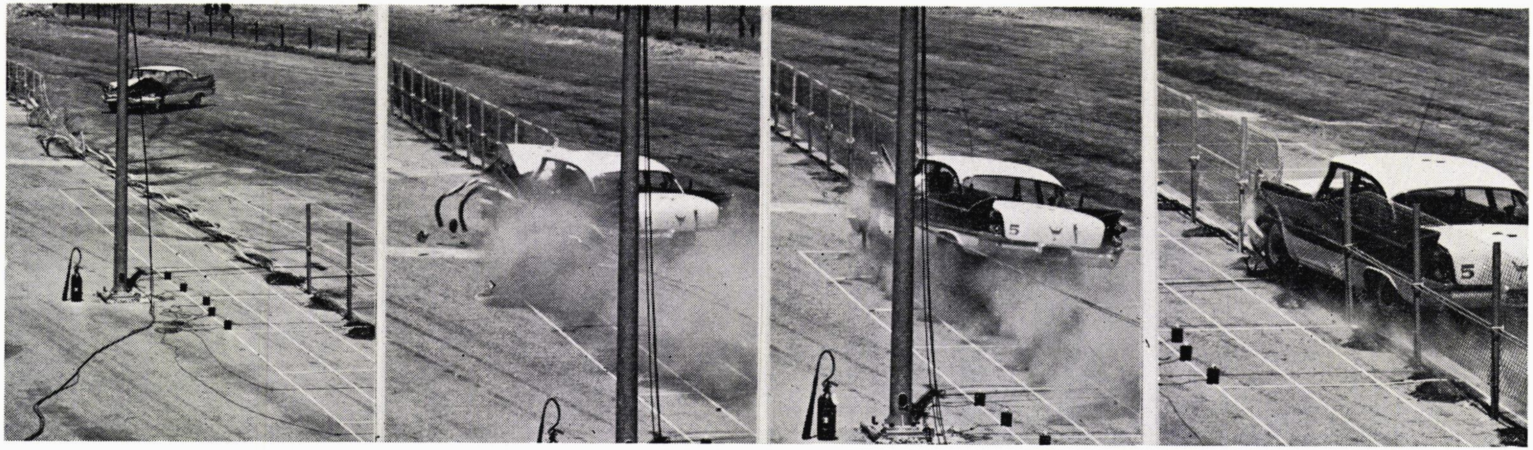
FENCE............48" chain link at 9" above pavement.
CABLE..........3/4" 7x7 2 at 30" above pavement.
POST FOOTING.........Design C
POST SPACING..........8' O.C.
LENGTH OF INSTALLATION..600'
GROUND CONDITION......Dry

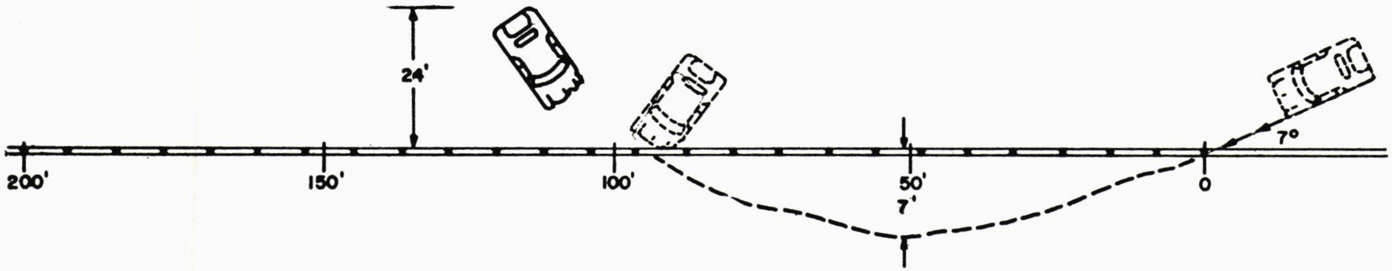
FENCE DAMAGE....................136' damaged
CABLE FITTING DAMAGE..........1 turnbuckle slightly bent
CABLE DAMAGE................none
POST DAMAGE................17 damaged—1 pulled out
POST FOOTING DAMAGE...........16 cracked—1 failed
MAX DYNAMIC DEFLECTION OF CABLES..6'
VEHICLE DECELERATION (PEAK LONG)...3.6 g's
VEHICLE DAMAGE...............$700

TEST NO..........4
DATE............6-26-61
VEHICLE.........Dodge 59 Sedan
SPEED..........76 MPH
IMPACT ANGLE....10°
VEHICLE WEIGHT... 4300 lbs. (W/DUMMY & INSTRUMENTATION)

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS — MATERIALS & RESEARCH DEPT.

EXHIBIT 7





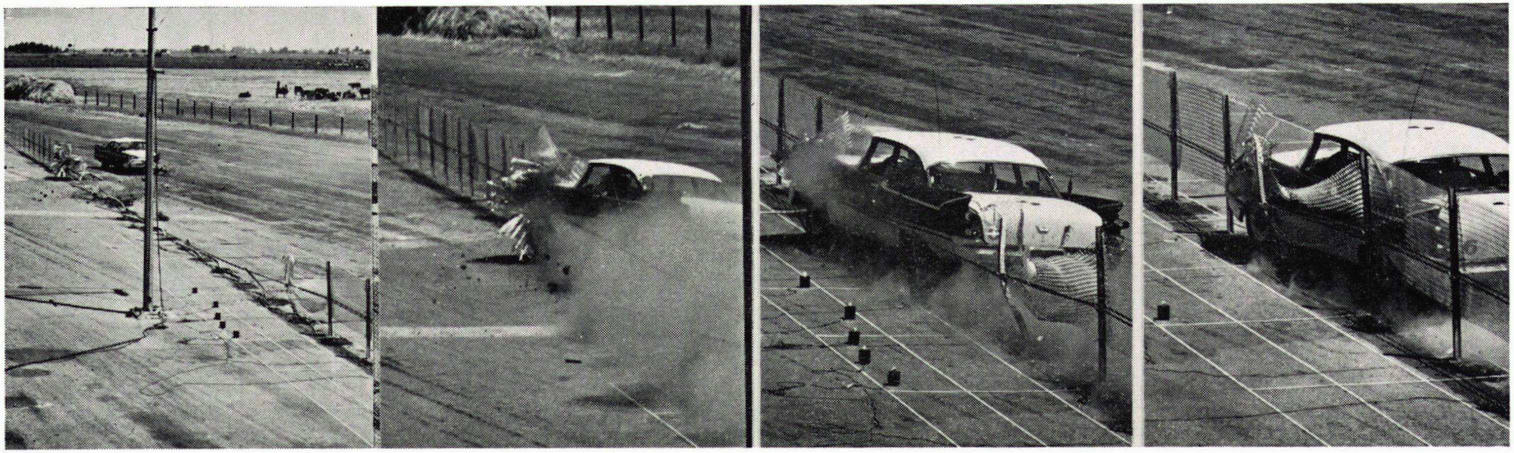
FENCE............48" chain link at 9" above pavement.
CABLE.......... 3/4" 7 x 7.2 at 30" above pavement.
POST FOOTING.......... Design D
POST SPACING.......... 8' O.C.
LENGTH OF INSTALLATION.. 600'
GROUND CONDITION...... Dry

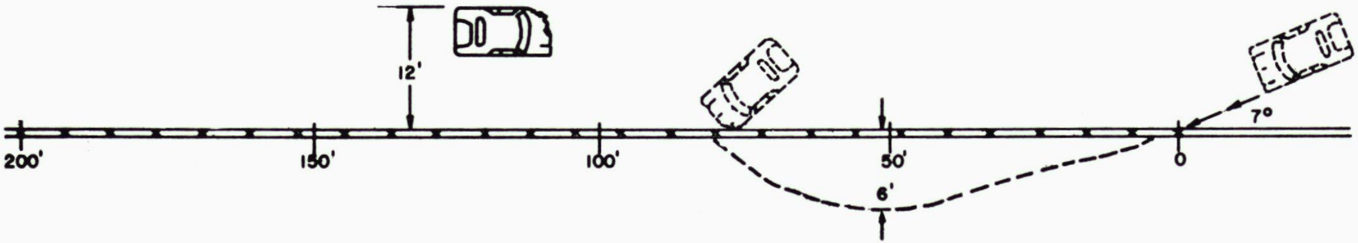
FENCE DAMAGE.........................112' damaged
CABLE FITTING DAMAGE...........1 deadend failed
CABLE DAMAGE...............none
POST DAMAGE................15 damaged-2 pulled out
POST FOOTING DAMAGE 10 cracked
MAX. DYNAMIC DEFLECTION OF CABLES.. 7'
VEHICLE DECELERATION (PEAK LONG).. 4.8 g's
VEHICLE DAMAGE..............$ 350

TEST NO........ 5
DATE............ 6-30-61
VEHICLE .... Dodge 59 Sedan
SPEED............84 MPH
IMPACT ANGLE.... 7°
VEHICLE WEIGHT... 4300 lbs.
(W/DUMMY & INSTRUMENTATION)

EXHIBIT 8

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS — MATERIALS & RESEARCH DEPT.





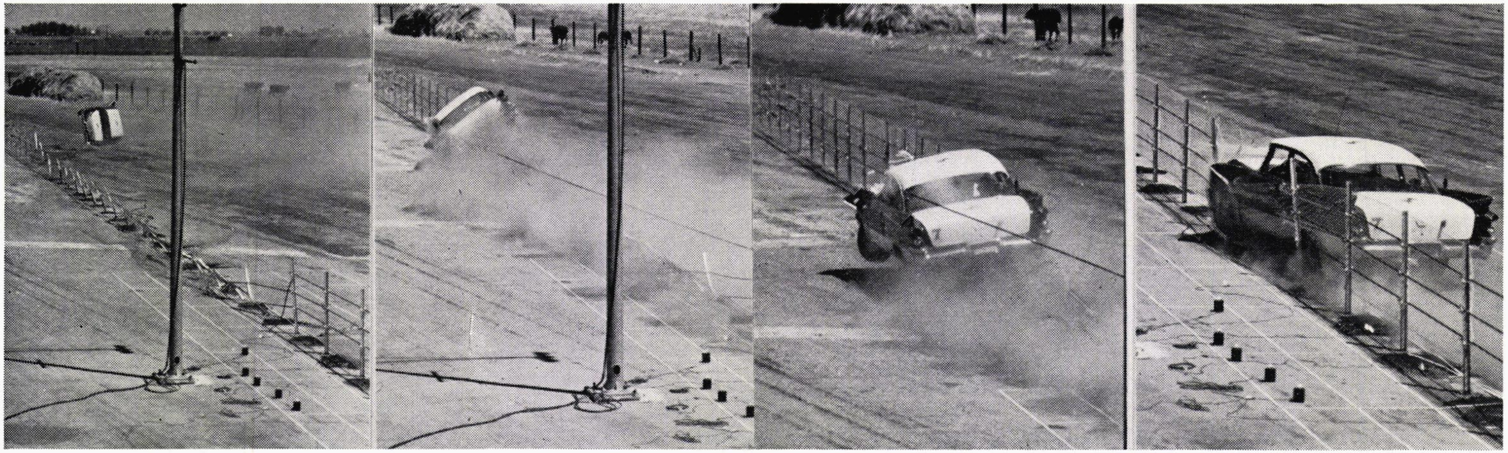
FENCE . . . 42". 18 ga. steel expanded metal
1.35"x 3" diamond at 15" above pavt.
CABLE . . . . 3/4". 6x19 IWRC 2
at 30" above pavement.
POST FOOTING . . . . Design C
POST SPACING . . . . 8' O.C.
LENGTH OF INSTALLATION . 600'
GROUND CONDITION . . . . . Dry

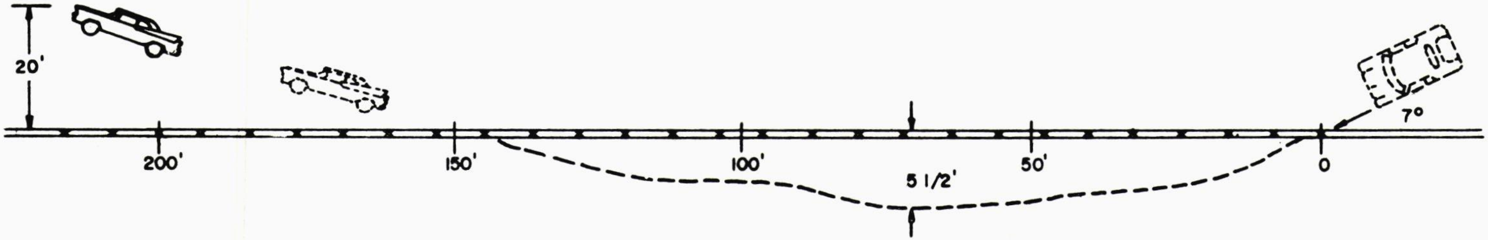
FENCE DAMAGE . . . . 136' expanded metal damaged
CABLE FITTING DAMAGE . . . . none
CABLE DAMAGE . . . . none
POST DAMAGE . . . . 16 damaged, 1 pulled out
POST FOOTING DAMAGE . . . . 15 cracked, 1 failed
MAX. DYNAMIC DEFLECTION OF CABLES . . 6'
VEHICLE DECELERATION (PEAK LONG) . . 3 g's
VEHICLE DAMAGE $350

TEST NO. 6
DATE 7-10-61
VEHICLE . . . . Dodge 59 Sedan
SPEED . . . . 75 MPH
IMPACT ANGLE . . . . 7°
VEHICLE WEIGHT . . . 4300 lbs.
(W/DUMMY & INSTRUMENTATION)

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS — MATERIALS & RESEARCH DEPT.

EXHIBIT 9





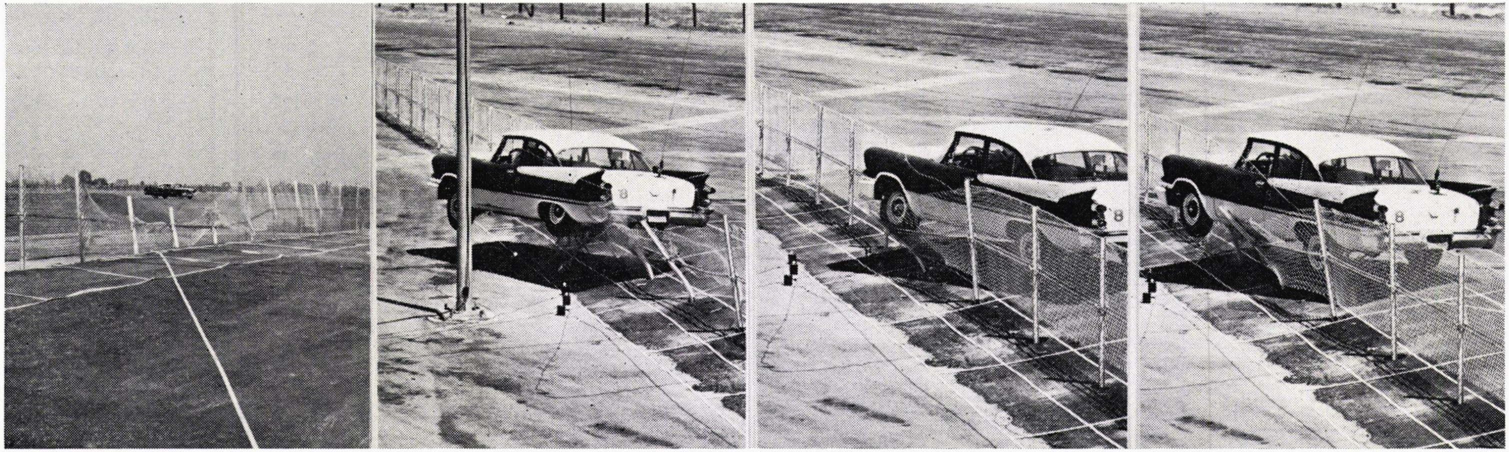
FENCE .... 48" chain link at 9" above pavement.
CABLE .... 3/4" 6 x 19 IWRC 3 at 20" 32" and 44" above pavement
POST FOOTING .... Design D
POST SPACING .... 8' O.C.
LENGTH OF INSTALLATION . 600'
GROUND CONDITION ..... Dry

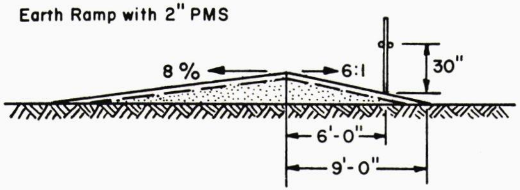
FENCE DAMAGE .... 184' damaged
CABLE FITTING DAMAGE .... preformed splice failed
CABLE DAMAGE .... none
POST DAMAGE .... 19 damaged
POST FOOTING DAMAGE .... 12 cracked
MAX. DYNAMIC DEFLECTION OF CABLES . 5 1/2'
VEHICLE DECELERATION (PEAK LONG) .. 6.8 g's
VEHICLE DAMAGE .... Total loss

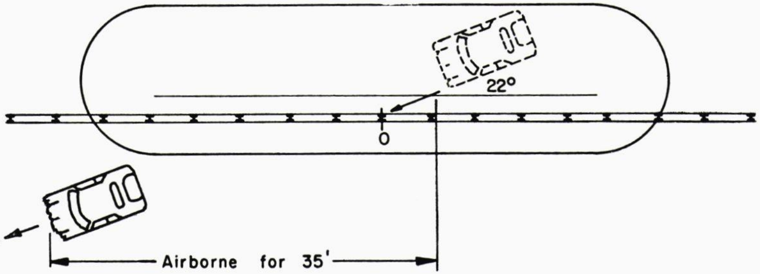
TEST NO. 7
DATE 7-14-61
VEHICLE .... Dodge 59 Sedan
SPEED .... 77 MPH
IMPACT ANGLE ... 7°
VEHICLE WEIGHT ... 4300 lbs.
(W/DUMMY & INSTRUMENTATION)

EXHIBIT 10

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS — MATERIALS & RESEARCH DEPT.







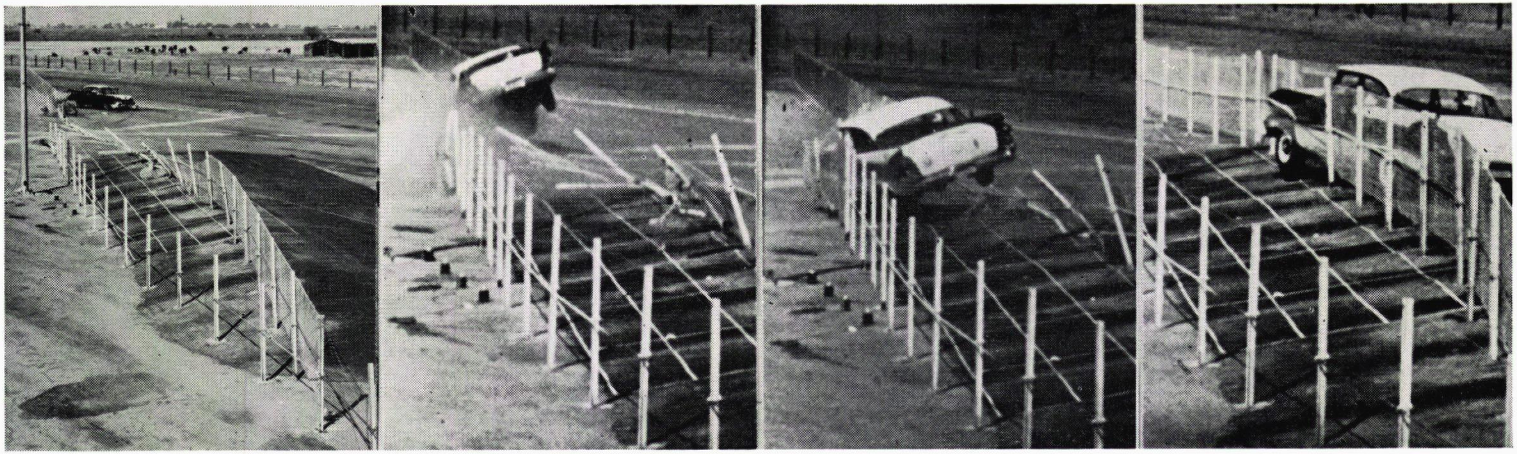
FENCE .... 48" Chain link at 9" above pavement.
CABLE .... 3/4" 6 x 19 IWRC 2 at 30" above pavement.
POST FOOTING .... Design B
POST SPACING .... 8' O.C.
LENGTH OF INSTALLATION .. 400'
GROUND CONDITION ...... Dry

FENCE DAMAGE .... 35' damaged
CABLE FITTING DAMAGE .... none
CABLE DAMAGE .... none
POST DAMAGE .... 7 damaged
POST FOOTING DAMAGE .... none
MAX. DYNAMIC DEFLECTION OF CABLES ... Vehicle jumped barrier
VEHICLE DECELERATION (PEAK LONG.) ... 1.7 G's (at end of jump)
VEHICLE DAMAGE .... $ 150

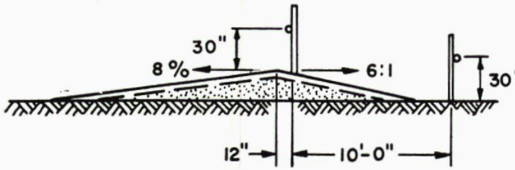
TEST NO. 8
DATE 8-18-61
VEHICLE .... Dodge 59 Sedan
SPEED .... 74 MPH
IMPACT ANGLE ..... 22°
VEHICLE WEIGHT ... 4300 lbs.
(W/DUMMY & INSTRUMENTATION)

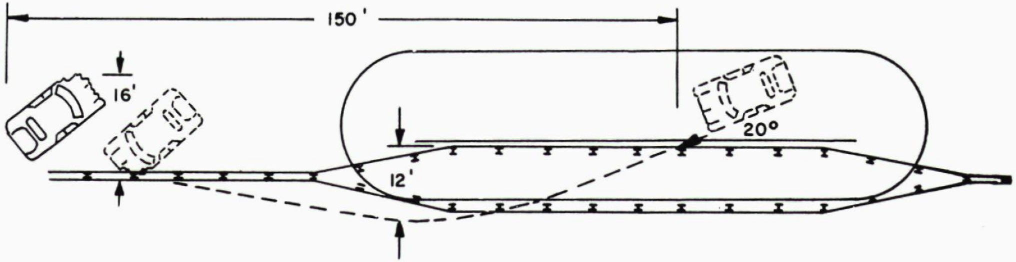
EXHIBIT 11

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS — MATERIALS & RESEARCH DEPT.



Earth Ramp with 2" PMS





FENCE . . . . 48" chain link at 9" above pavement.

CABLE . . . . 3/4" 6 x 19 IWRC 2 at 30" above pavement.

POST FOOTING . . . . Design B

POST SPACING . . . . 8' O.C.

LENGTH OF INSTALLATION . . 400'

GROUND CONDITION . . . . . Dry

FENCE DAMAGE . . . . 136' damaged

CABLE FITTING DAMAGE . . . . none

CABLE DAMAGE . . . . none

POST DAMAGE . . . . 22 damaged - 11 slightly bent

POST FOOTING DAMAGE . . . . 7 anchors cracked

MAX. DYNAMIC DEFLECTION OF CABLES . . 12'

VEHICLE DAMAGE . . . . $ 750

TEST NO 9

DATE 9-29-61

VEHICLE . . . . Dodge 59 Sedan

SPEED . . . . 82 MPH

IMPACT ANGLE . . . . . 20°

VEHICLE WEIGHT . . . . 4300 lbs. (W/DUMMY & INSTRUMENTATION)

EXHIBIT

Effect of Rumble Strips on Traffic Control and Driver Behavior

MARK L. KERMIT, *Traffic Engineer, Public Works Department, Contra Costa County, California;* and
T. C. HEIN, *Research Engineer, California Research Corp., Richmond*

Rumble strips were installed at four different locations in Contra Costa County, Calif. Accident rates were greatly reduced, stop sign violations were significantly reduced, and vehicle speeds and deceleration rates before a sharp curve were reduced. Before-and-after motion pictures of vehicle paths showed marked changes in driver behavior. The rumble strips consisted of a series of spaced overlays placed on the road surface using ¾-in. stones and seal coat techniques. In three locations durability of the strips was increased by using synthetic resin formulations to hold the stones in place.

Explanations for reductions in accident rates and changes in driver behavior are based on the added visual, audible, and tactile stimuli produced by the rumble strips. The noise level in a vehicle with closed windows was raised from 92 decibels background to 102 decibels on the strips. These strong signals help alert the driver to changing road conditions. Driver reaction times are faster in response to audible and tactile stimuli than to visual stimuli. In addition, roads where drivers are preconditioned to expect high speeds and few hazards, or where driver boredom may result in visual hallucinations, are especially in need of the strong stimuli provided by the rumble strips to produce desired behavior. Economic justification for the strips is analyzed in terms of accident cost reduction. Other strip designs are shown for a variety of road conditions.

• TRAFFIC ENGINEERS are frequently disconcerted when low-volume intersections with good alignment and visibility have unexpectedly high accident rates or many violations of existing traffic control devices. Normal corrective measures, such as signals or grade separations, are often too costly or not warranted because of the low traffic volumes involved. Contra Costa County, Calif., has had four examples of this problem in the past three years. In each location, a specially designed series of rumble strips was installed in order to alert drivers and slow down fast-moving cars.

The rumble strips consist of a series of 25-ft long areas of rough textured aggregate placed on the appropriate lanes at 50- to 100-ft intervals. Texture of these surfaces is shown in Figure 1. Motorists traversing these rumble strips receive strong audible, tactile, and visual stimuli from this coarse texture as it alternates with the smooth texture of the road. Without any other signs for

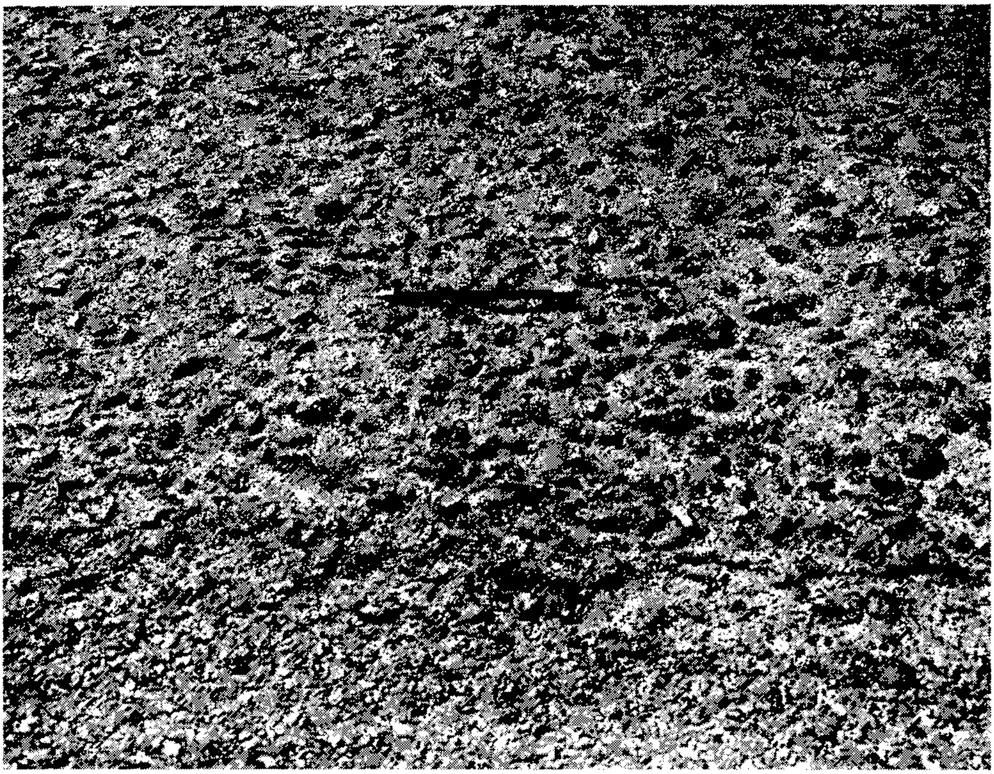


Figure 1. Rumble strip surface texture after one year, Case II, Third Street.

explanation, these stimuli are sufficient to evoke a greater awareness of road and traffic conditions on the part of the driver. He subconsciously tends to slow down (*1*, p. 20).

To determine the effect of the rumble strip installations, accident records were carefully evaluated, vehicle speeds measured, and motion picture studies made to determine vehicle paths followed around a sharp turn. In each study, significant changes occurred that provide economic justification for the use of rumble strips, as well as an explanation for the accident reductions observed.

RUMBLE STRIP INSTALLATIONS

The first problem (Case I) occurred when a new, controlled-access expressway was temporarily terminated at a T-intersection. The latest standard signs, street lights, and pavement markings were used to warn motorists approaching the stop at the intersection. In many cases, the advance signs either were not observed or not believed until it was too late to slow down. Figure 2 is a plan of this rural intersection of Taylor Boulevard with Pleasant Hill Road showing the location of the signs, lights, pavement markings, and rumble strips after installation. These strips were constructed by using conventional seal coat asphalts to hold ¾-in. chips on the road. After a year of service, approximately 60 percent of the stones had been whipped out by the high speed traffic; and rumble noises, although distinct, were con-

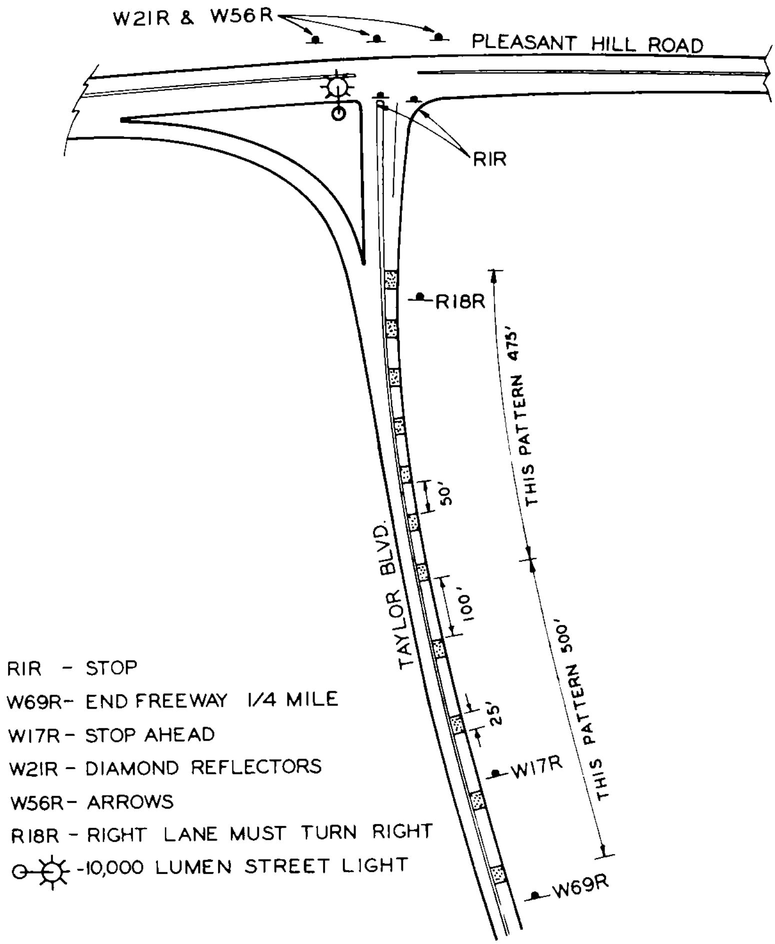


Figure 2. Layout of rumble strips, Case I.

siderably diminished. This intersection was subsequently changed to a four-way when Taylor Boulevard was extended and the stop eliminated.

The second installation (Case II) was made at Third Street and Parr Boulevard. This is a T-intersection with a long history of overruns, but where stop sign control would have severely penalized the major movement and was not warranted. Analysis of the accidents at this location indicated a large proportion of night occurrences. Safety lighting was installed, which reduced the night accidents, but failed to reduce the daytime occurrences. A plan of this intersection is shown in Figure 3. Rumble strips were installed at this location in July 1960 in a layout similar to the previous case. One-inch chips were bedded in asphalt and held in place with polyester resin (*2*). No loss of chips is evident one year later in any of the strips, including the strip placed on the top of the concrete bridge deck. This is a specially designed method which is the subject of a pending patent application. Measurements were made of

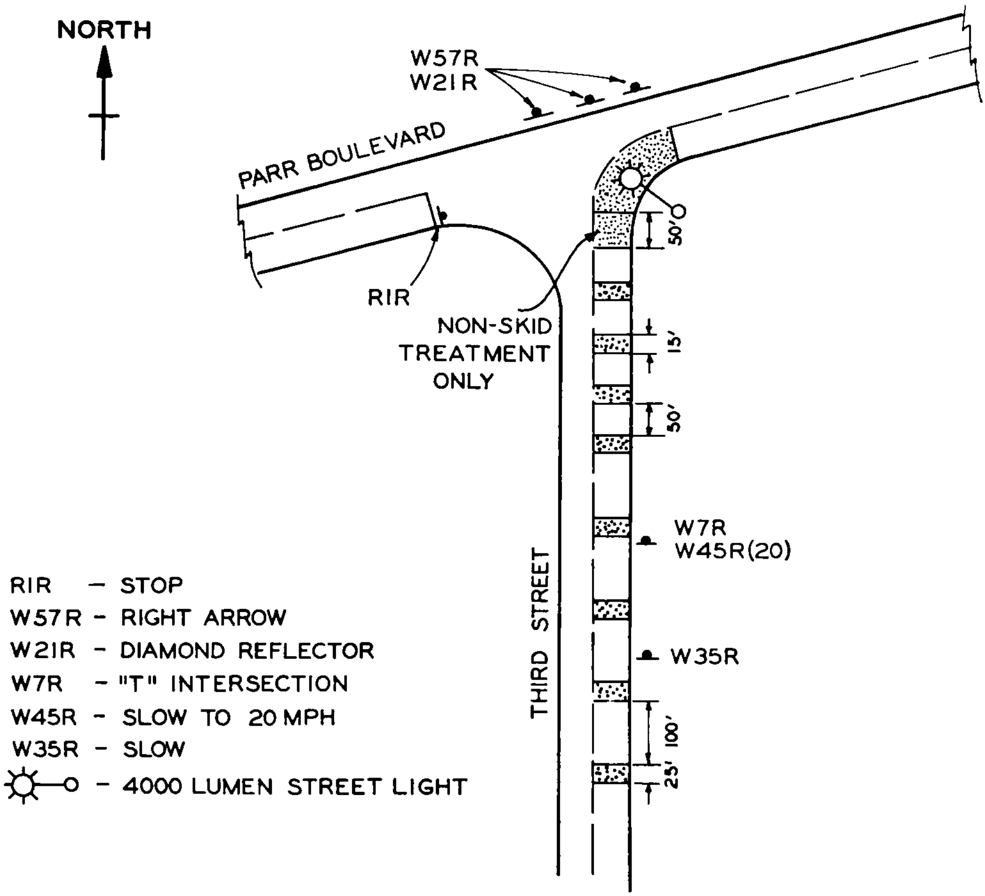


Figure 3. Plan of rumble strips, Case II.

changes in sound amplitude next to the driver's ear when traversing these strips. Figure 4 shows the oscilloscope traces of the noise level pulse and spacing for several different speeds. Both the frequency and intensity of the sound increased as speed increased. Background noise level averaged 92 decibels, and noise level crossing the strips averaged 102 decibels at 35 mph. Changes in vibration of the steering wheel were observed, although none were so severe that control of the vehicle was affected.

The third set of strips (Case III) was installed on old US 40 at a Y-intersection of a county road with this former State highway, each 4 lanes wide. The county road now serves as access to a new freeway, and the superseded State highway is controlled by a stop sign which was installed when the freeway was opened. Accidents increased despite the marked decline in traffic volume. At this intersection, a complex series of strips using different asphalt ratios, chip sizes, amounts of polyester resin, etc., (*2*) was used to determine optimum construction practices. Chip loss was observed only in those rumble strips where no resin was applied to hold the stones in place. A plan of this

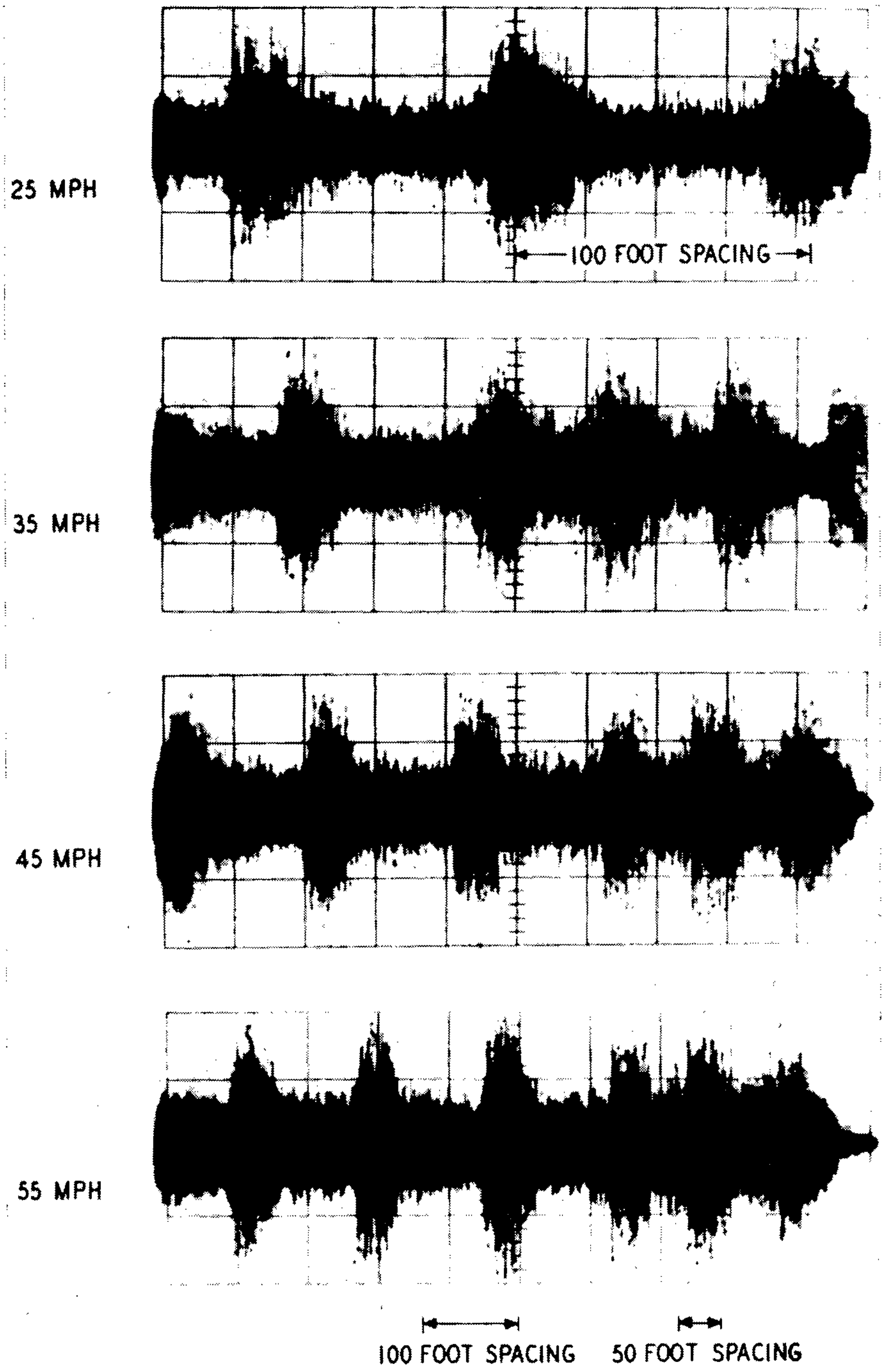


Figure 4. Oscilloscope traces of changes in sound amplitude when rumble strips are crossed. Microphone inside car next to driver's right ear with all windows closed.

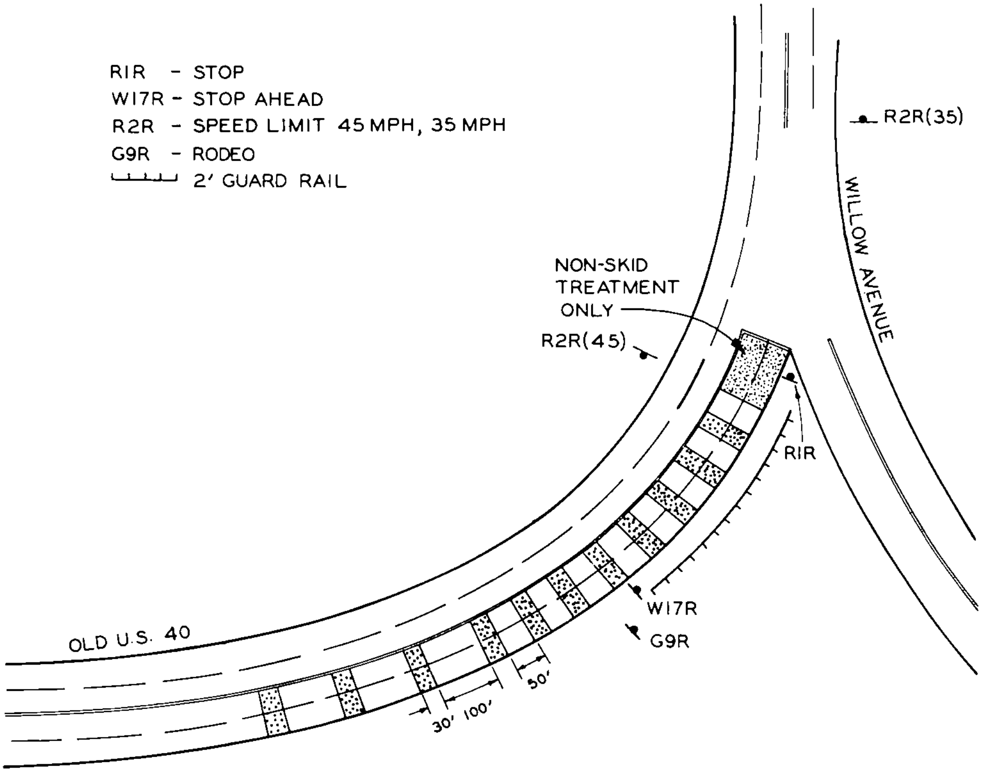


Figure 5. Plan of rumble strips, Case III.

intersection of old US 40 and Willow Avenue is shown in Figure 5. Appearance of the rumble strips is shown in Figure 6.

In addition to the intersections described, a set of rumble strips (Case IV) was installed on Bailey Road at a four-way intersection where accident rates were low but where one extremely severe accident had taken place. An epoxy resin binder was used in this experiment. Plans of the rumble strip layout for this intersection were similar to those shown for the first case.

ACCIDENT REDUCTIONS OBSERVED

Accident reports for each of these intersections were reviewed. Extraneous accidents were eliminated so that a valid determination of the effect of the rumble strips could be made. At each intersection, the number of accidents per year was, at the very least, cut in half and accident severity reduced, as shown in Table 1. This occurred despite the fact that total accidents on County roads in Contra Costa have been increasing over the time period involved in these studies. In Case I, at Taylor Boulevard, the average number of accidents per year was reduced from 2.5 to 0.4 during the 4½-yr interval studied. In Case II, at Third Street, the number of accidents per year has diminished from 4.9 to 2.0 since the rumble strips were installed. For Case III, at the old US 40 and Willow Avenue intersection, the average

Figure 6. Appearance of rumble strip experiments on old US 40, Case III.

number of accidents per year was reduced from 4.2 to 1.0. In Case IV, at Bailey Road, no accidents have yet occurred since the strips were installed.

STUDIES OF DRIVER BEHAVIOR

A more precise conception of how the rumble strips affected driver behavior was obtained by a series of radar speed studies and motion pic-

TABLE 1

ACCIDENT DATA BEFORE AND AFTER RUMBLE STRIP INSTALLATION

Case	Intersection	Refer. to Fig.	Before Rumble Strips: No. of Months Records Include	No. of Accidents [1] PD	PI	F	Avg. Annual Accident Rate	After Rumble Strips: No. of Months Records Include	No. of Accidents [1] PD	PI	F	Avg. Annual Accident Rate
1	Taylor Blvd. and Pleasant Hill Rd.	2	24	3	2	0	2.5	31	1	0	0	0.4
2	Third St. and Parr Blvd.	3	32	7	6	0	4.9	12	2	0	0	2.0
3	Old US 40 and Willow Ave.	5	20	4	2	1	4.2	12	0	1	0	1.0
4	Bailey Rd. and Concord Blvd.	—	22	2	0	1[2]	1.7	10	0	0	0	0

[1] Property damage (PD), personal injury (PI), and fatal (F).
[2] Three deaths considered as one fatal accident.

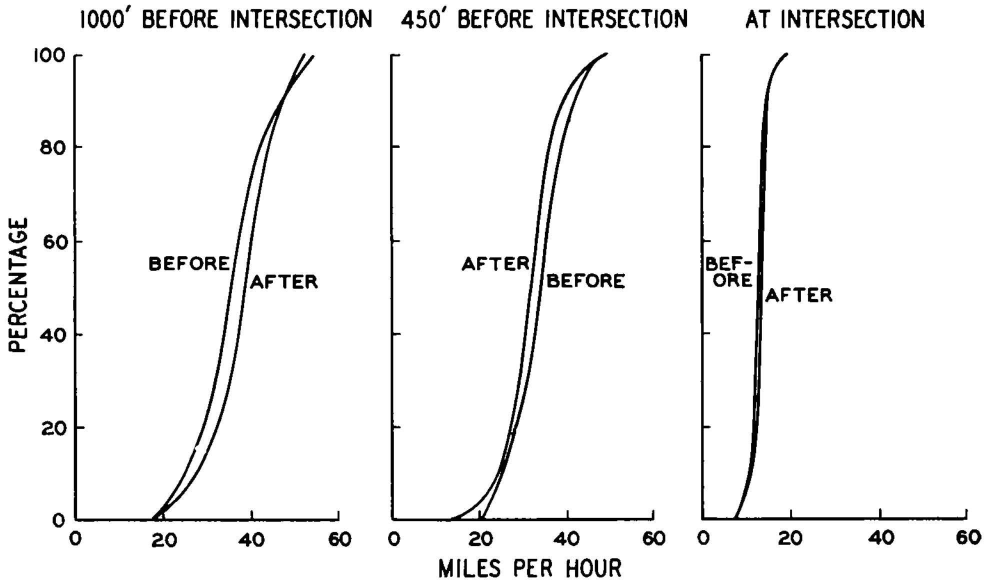


Figure 7. Cumulative speed curves, Case II, Third Street.

tures made from a distance at the Third Street location in Case II. Special precautions were taken to insure that all instruments and cameras were concealed and would have no effect on the behavior observed. Samples were obtained at the same time of day and on the same day of the week and by the same operator and equipment. Figure 7 summarizes the results of the speed studies made one week before and two months after the strips were installed and the effects of novelty had worn off. The curves show the cumulative distribution of speeds for the entire sample. Speed measurements were made at three points. The first measurement was made 1,000 ft from the intersection before traffic encountered the first rumble strip; next, at a distance of 450 ft from the intersection after traffic had crossed three rumble strips; and finally, at the intersection itself.

The cumulative curves (Fig. 7) show that both average speed and 85th percentile speed of vehicles in Case II, before they reach the rumble strips, have increased slightly since the rumble strips were installed. Despite this increase in approach speed, the curves show that significant reductions in average and 85th percentile speed were obtained after the first three rumble strips were crossed.

Deceleration rates were calculated for the 85th percentile speeds and the distances between the points where speed measurements were made. The 85th percentile speeds and deceleration rates are given in Table 2. Before the rumble strips were installed, most of the deceleration occurred immediately before the intersection, as shown by the 3.46 ft per sec^2 deceleration rate for the last 450 ft of road. However, after the rumble strips were installed, deceleration took place over a greater distance and was consequently much more gradual. Deceleration rates were reduced to 2.70 ft per sec^2 in the last 450 ft of road.

Motion picture records of vehicle paths after turning the corner at the

TABLE 2
SPEEDS AND DECELERATION RATES ON THIRD STREET AT THE 85TH PERCENTILE LEVEL

Location of Measurement	Before Rumble Strips Installed	After Rumble Strips Installed
A. Speed 1,000 ft from intersection (mph) before first rumble strip	44.0	46.0
Average deceleration rate (ft/sec^2) from Location A to B	0.57	1.43
B. Speed 450 ft from intersection (mph) after three rumble strips are crossed	41.0	37.0
Average deceleration rate (ft/sec^2) from Location B to C	3.46	2.70
C. Speed at intersection (mph)	14.8	15.1

TABLE 3
EFFECT OF RUMBLE STRIPS ON CENTERLINE VIOLATIONS AFTER RIGHT TURN FROM THIRD STREET

	Before Rumble Strips	After Rumble Strips
Total number right turns counted	78	100
Centerline violations (%):		
No opposing traffic	17	12
Oncoming vehicles	9	3
Total violations (%)	26	15

Case II location show significant differences in vehicle placement after the rumble strips were installed. The camera was concealed 500 ft west of the intersection shown in Figure 3, and movements of northbound vehicles were recorded. The percentage of northbound drivers who violated the centerline after making a right turn was reduced from 26 to 15 percent as a result of the rumble strip warning (Table 3).

Other studies of stop sign observance have been made in Cook County, Ill. (*3*). In these studies, a nonintermittent rumble area was installed for a distance of 300 ft from the intersection. Observations of more than 1,000 vehicles at one intersection are summarized in Table 4. The percentage of drivers making a full stop was increased from 46 to 76 percent after the rumble areas were installed. This large suburban county has installed approximately 212 rumble areas in front of stop signs in order to increase driver observance of potentially hazardous, low-volume intersections.

Cost and Benefit Analysis of Rumble Strips

Accidents each year after rumble strips were installed have shown a significant decrease, as previously described. It is possible to estimate the savings resulting from this reduction in accidents by using the most recent estimates of the National Safety Council (*4*) for accident costs. Savings to motorists include reduced costs of property damage, medical treatment, wage loss, and insurance overhead as follows (in dollars per accident): property damage, 300; non-fatal injury, 1,600; and fatal injury, 30,000. These accident cost estimates were applied to annual rates for the Contra Costa intersections previously given in Table 1. Table 5 gives the annual cost of accidents before the rumble strips were installed, after the rumble strips

TABLE 4
STOP SIGN OBSERVANCE[1]

	Before Rumble Strip Installed (%)	After Rumble Strip Installed (%)
Full stop	46	76
Rolling stop	50	24
No stop	4	0
Total	100	100

[1] Adapted from data reported by Cook County, Ill. (*3*).

TABLE 5

ESTIMATED COST OF ACCIDENTS AT RUMBLE STRIP INTERSECTIONS

Intersection	Annual Accident Cost (dollars)		
	Before	After	Annual Savings
Taylor Blvd. and Pleasant Hill Rd.	2,050	115	1,935
Third St. and Parr Blvd.	4,400	600	3,800
Old US 40 and Willow Avenue	20,400	1,600	18,800
Bailey Rd. and Concord Blvd.	16,700[1]	0	16,700

[1] Calculated on basis of a single fatality although three deaths occurred.

were installed, and the estimated savings obtained as a result of the rumble strips. Even in Case I, the intersection with the lowest initial annual accident cost, savings are estimated at nearly $2,000 a year as a result of accident reductions. Greater savings are shown for the other cases where accident rate or severity was higher.

Offsetting these annual savings, the cost of installing a set of rumble strips made with polyester resins is estimated at $2 per sq yd (*5*). For typical installations on both sides of a four-way intersection, the total cost is approximately $1,000. Indications are that, because of the durability of strips installed with these resins, a useful life of at least five years can be expected without any loose stones to add to the traffic hazard. On this basis, the annual cost of the strips is so small compared with the benefits involved that the use of rumble strips may be justified at intersections with as few as one or two property damage accidents a year.

Concepts to Explain Results Observed

Four general circumstances may be described in which the strong stimulus of a rumble strip is required to prevent drivers from making serious mistakes: (a) where there are other distractions competing for his attention, such as red neon advertising signs in the vicinity of a traffic signal (*6, 7, 8*); (b) where he may become bored, fatigued, or drowsy (*9, 10, 11*) from driving on long, monotonous stretches of rural roads; (c) where he is preconditioned to easy, rural driving conditions, then suddenly enters an urban community (*1*, p. 24); and, (d) where his previous experience may lead him to ignore information or warnings because he feels capable of judging the situation himself (*12*); for example, where motorists disregard stop signs at low-volume intersections with good sight distances. There are numerous other examples of similar situations familiar to every traffic engineer.

To obtain a rational solution of these problems, an examination was made of some of the psychological

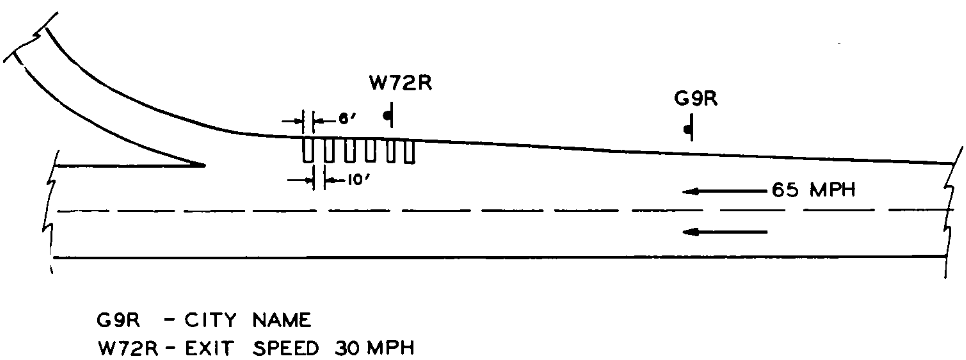


Figure 8. Deceleration lane preceding curve on freeway off-ramp.

principles involved. For example, it is well known that the duration and degree of a person's attention depend on the intensity of the stimulus and the contrast between it and surrounding stimuli (*13, 14*). Rumble strips are particularly effective because the driver's visual, auditory, and tactile senses simultaneously warn him of an unexpected situation and he consequently tends to slow down. Contrast with background impressions, as shown by the noise intensity (Fig. 4), is especially strong. In addition, the amount and duration of his attention depend on the absence of counterattractions. The fewer the number of conflicting stimuli, the greater the chance that any desired single stimulus will attract his attention. Thus, rumble strips appear in an area of the driver's visual field where conflicting signs do not exist.

It has been established that decreased variation in one's sensory environment produced disturbances in visual perception, childish emotional responses, and impaired individual thinking (*15*). When this condition occurs on long, monotonous trips, it is necessary to have a strong stimulus to produce an arousal reaction in the brain. Rumble strips are thus effective in alerting the occasional woolgathering or drowsing driver.

Rumble strips also have a useful effect in producing faster reactions as a result of the audible and tactile stimulus given. Matson, Smith, and Hurd (*1*) refer to research that showed brake reaction times were faster when an audible signal was used than when a visual signal was used for a variety of different conditions of vehicle movement and foot position. Work in a related field (*16*) has also shown that loud sound intensity produced faster speed of movement and more forceful muscular contractions than low sound intensity. Thus, the increased rumble noise level at higher speeds is beneficial in slowing down a fast driver.

Application of Rumble Strip Concepts to Other Situations

The preceding discussion has shown why rumble areas are effective in obtaining control of driver behavior where normal practice has not been entirely satisfactory. On this basis, it is possible to suggest several other situations suitable for further experimentation. One possibility is on the deceleration lanes preceding a sharp curve on freeway off-ramps, as shown in Figure 8. Standardized spacing of the strips, as suggested in this diagram, will also enable drivers to judge their speed by the time interval between sound impulses. Thus, a glance at the speedometer may be avoided at the critical point of the curve when eyes are needed on the road.

Other situations in which the need for warnings exists (*17*) are shown in Figures 9, 10, and 11 where obstructions, constrictions, or curvature in the normal roadway can be indicated and drivers alerted by the rumble strip stimulus. The advantage of a rumble strip in these instances is important because driver control over the vehicle is easily maintained at all times.

Another important area in which use of rumble strips might be given serious consideration is on road shoulders. Various means have been employed to differentiate the road shoulder from the traveled lanes. A change in the texture of the surface, such as a seal coat, is generally used. In recent years, however, a painted shoulder line has been tried in many places. Both of these means of delineation have drawbacks in that the small stones of the seal coat are whipped off by traffic, and the painted line becomes invisible in wet weather. Use of rumble strips as a means of marking the road shoulder would overcome the weaknesses of the conventional methods mentioned above, as well as give the motorist an audible

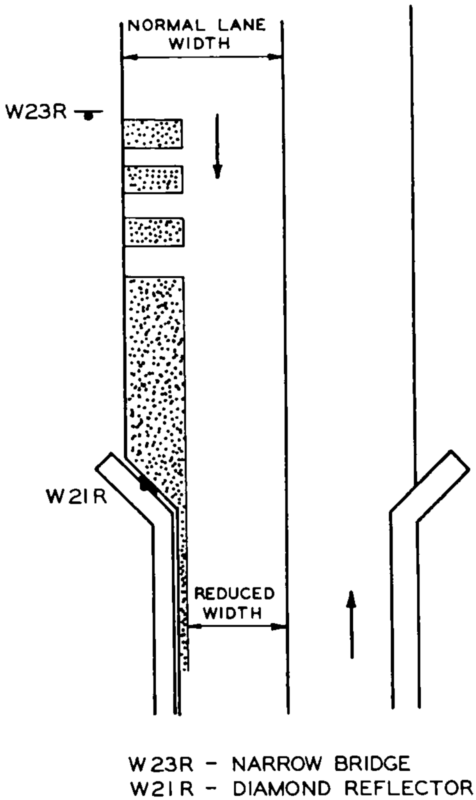


Figure 9. Reduced lane width at bridge abutment.

warning when he leaves the traveled lane of the roadway.

SUMMARY AND CONCLUSIONS

The need to alert motorists to changes in road conditions is clearly indicated by accident records at locations where normal practice is not completely effective and where more expensive solutions are not warranted. Experimental installations have shown that rumble strips can produce substantial savings in accident costs. They are readily justified where there are as few as one or two accidents a year attributable to failure to observe existing warnings. Maintenance and annual cost of the strips may be reduced by a spray coating of polyester resin to hold the stones in place.

The strong stimulus to slow down, produced by rumble strips, is especially important where other distractions are present, where boredom and fatigue exist, after long stretches of easy driving, and where drivers are tempted to use their own judgment rather than observe the existing warnings. These principles make it possible to suggest a series of other locations where rumble strip installations should be used with a view toward developing nationally standardized geometric designs.

ACKNOWLEDGMENTS

The authors acknowledge the support of the American Bitumuls and Asphalt Company and Public Works Department, Contra Costa County, Calif., who sponsored this work. The polyester resins, as well as technical assistance, were provided by Oronite Division of California Chemical Company. Radar speed studies were conducted by the Institute of Transportation and Traffic Engineering, University of California.

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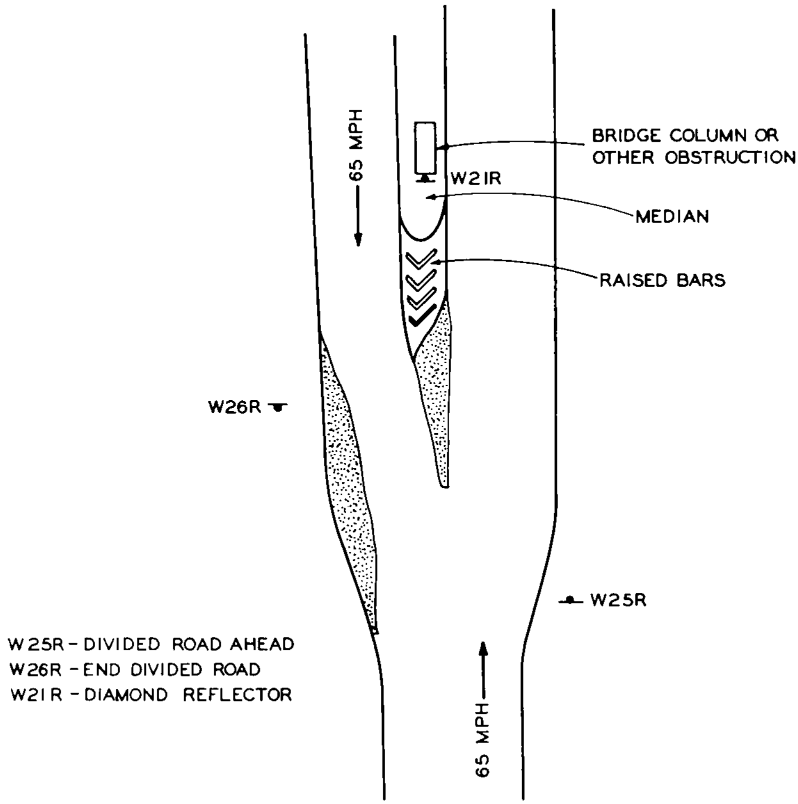

Figure 10. Transition at start and end of median.

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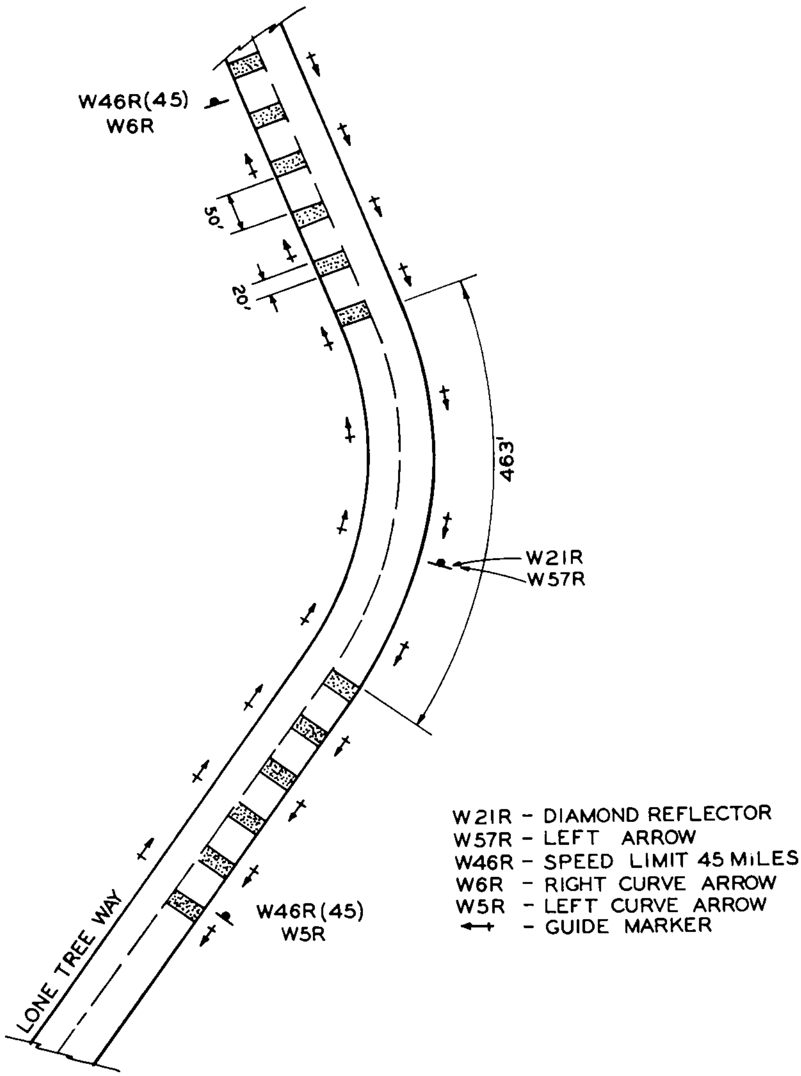

Figure 11. Plan of rumble strips installed on rural curve with history of overruns.

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Advance Route Turn Markers on City Streets

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Although much attention has been given to directional signing for rural areas, particularly on expressways, the problems of route signing on city streets remain. For cities such as Washington, D. C., where tourists abound, the problem is a vital one.

To evaluate the effectiveness of route turn directional markers and advance turn markers, test drivers negotiated a fictitious route through downtown Washington. The 12 intersections at which turns were to be made were chosen so as to be as nearly alike as possible. The test variables, on which an analysis of variance was made, were (a) the number of advance turn markers, (b) the direction of turn, and (c) the distance between successive turns.

The results indicate that it is desirable to provide at least one advance turn marker in advance of a turn. In addition, inferences are drawn relating to route marker design and placement.

• MANY CITIES without facilities for bypassing through traffic or carrying it on freeway-type facilities must continue to rely on the urban street system to carry marked routes. Even where major through routes in cities are carried on expressway-type facilities or on bypasses, secondary marked routes will probably still be carried through smaller towns and suburban communities. It has come to be recognized that more and more attention needs to be given to the adequate advance warning and guidance of traffic, but most of the emphasis has been on high-speed facilities. In urbanized areas, where there are numerous intersections, marginal conflicts, complex traffic patterns, and many competing signs and confusing backgrounds, the need for adequate advance signing is equally important.

As an example, there is a certain turn marked by a single turn directional marker at an intersection. A certain approaching driver is aware of the impending turn in the route, perhaps because he has driven it before and is familiar with it. He may therefore drive along in confidence at his desired speed, get into the proper lane in advance of the turn, signal his intentions, and generally proceed along the route just as though it were not marked at all.

The driver who has driven the route before, but who is not familiar with it, perhaps, may remember that he is in the vicinity of a turn, but may not be sure of its exact location. Consequently, he may tend to drive more slowly in order to avoid overshooting the turn and he may pay less attention to traffic conditions because he is searching for the turn marker.

A driver traveling the route for the first time, however, may have to rely completely on route markers. He has to divide his attention continually between driving and searching for markers. If he arrives at the turn and sees the directional marker, he may not have sufficient time or maneuvering distance to make the turn safely.

To assist the driver who needs to follow the route markers, a turn marker is placed in advance of the intersection warning him that the route is going to turn. He is informed of the impending turn and still has time to safely decelerate and prepare for the maneuver. However, the many distracting backgrounds and competing signs in urban areas may pose a problem. What if he misses the advance marker? His attention may be taken up by traffic, or the marker may be obscured by large vehicles. Perhaps another such marker should be placed in advance of the first one for insurance. How much justification is there for the second advance turn marker, or for the first one, for that matter?

To answer this question, a controlled study was made of the effect of the presence and number of advance turn markers on the ability of drivers to negotiate a route in an urban area.

DESCRIPTION OF STUDY

The route involved was a fictitious one, with many turns, through the downtown area of Washington, D. C. The number of advance turn markers was varied at each intersection. Test

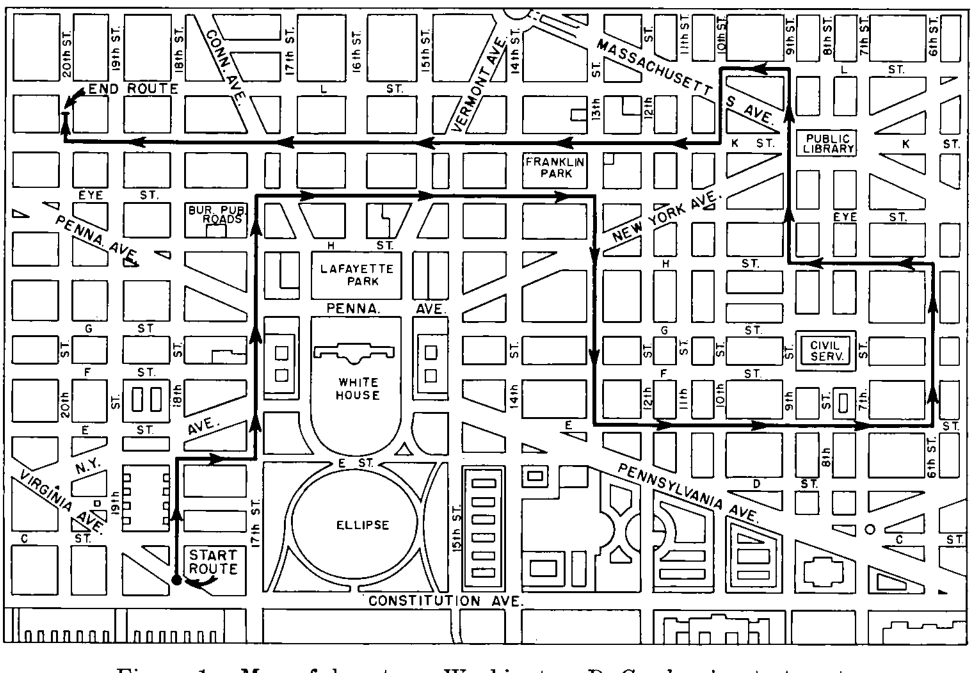

Figure 1. Map of downtown Washington, D. C., showing test route.

subjects driving over the route were observed for: (a) the number of turns they missed, (b) the distance from the intersection at which the test subject entered the proper lane from which to turn, and (c) the distance from the intersection at which the test subject gave the proper turn signal.

The test route (Fig. 1) was approximately 4 mi long and had 12 turns in it. The selection of intersections for turns was based on the following criteria: (a) a minimum block length of 400 ft in advance of the intersection, (b) signal control, (c) absence of islands or channelized movements, (d) four legs at right angles, (e) at least two moving lanes on the intersection approach. Of the twelve turns, six were to the right, and six were to the left. For each direction of turn, three turns were a short distance (1 to 3 blocks) from the preceding turn, and three were a long distance (over 3 blocks). It would have been more desirable to have had all intersections unsignalized; however, this was impossible to attain in the downtown area. To have them as much alike as possible, therefore, all chosen intersections had signal control.

A directional assembly with a horizontal arrow was mounted on the far right corner of each intersection where the route turned. The number of advance turn markers (none, one, or two) was varied at each intersection on the three test days. A balanced design was selected so that each subject saw each marker combination twice for each direction of turn. The number of long and short turns was, unfortunately, not balanced with marker combinations for each subject.

The design of the route marker was chosen to conform to the standards in the Manual on Uniform Traffic Control Devices for Streets and Highways (1961 edition). The route marker consisted of a 16-in. white circle on an 18-in. black square (Fig. 2). The designation 00 in 9-in. black series C numerals was centered in the circle. Appropriate marker assemblies were made up using this route marker and an appropriate arrow. The size of the arrow plates was identical with the 13- by 10-in. arrow plate which was the standard size in the previous edition of the Manual. The material used was white, nonreflectorized cardboard.

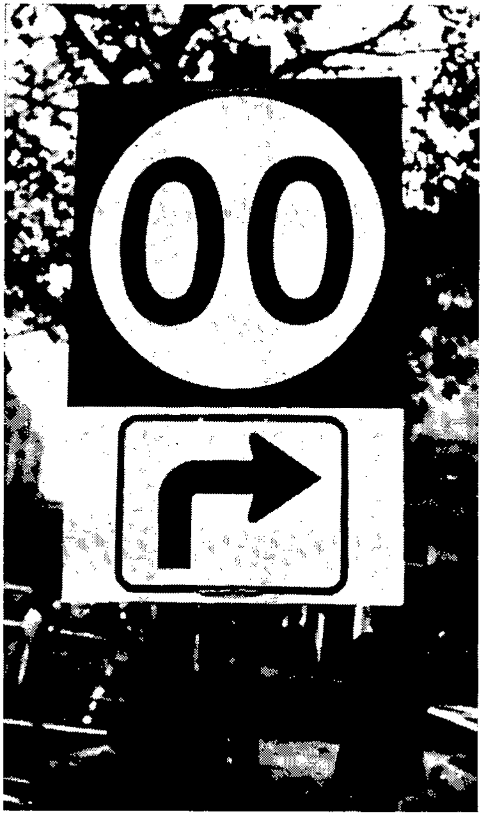

Figure 2. Close-up of advance turn marker assembly.

All markers were placed on the right-hand side of the street, and an approximate spacing of 150 ft was maintained between all markers at any one intersection. For ease of mounting the markers were placed on light poles, signal poles, existing sign posts, trees, or portable stanchions. The height of the signs varied from 7 to 11 ft from the sidewalk. The higher limit was sometimes necessary to avoid other signs, signals, parked and moving vehicles, and other obstructions.

PROCEDURE

Subjects were told only that the aim of the study was to observe driver characteristics in a downtown environment and that to have a consistent basis of comparison they were all going to drive over the same course. It was explained that the observer who sat in the car would not give directions; instead, the subjects were to follow the course that was marked out. It was hoped this would somewhat satisfy their curiosity and at the same time give them a plausible reason for following the route, though major emphasis was not laid on the route or markers themselves being the object of the study.

The subjects were further instructed to drive normally and to obey all traffic laws, such as signaling for turns, turning from the proper lane, and obeying speed limits.

The 53 subjects were divided into two groups of 18 and one of 17; one group per day was tested. Runs were made between noon and 3:00 PM. Markers were put up in the morning and taken down in the afternoon. Drivers started out at 5-min intervals to prevent them from overtaking each other. Each subject drove the course once.

An observer seated in the subject car recorded the following data: the distance, estimated in car lengths, from each turn at which the driver entered the proper turning lane; the distance at which the proper turn signal was given; and any errors made. Any distances estimated as longer than 15 car lengths (approximately 300 ft) were recorded as 15 car lengths. When a subject missed a turn he was returned to the route at the point where he would have been had he made the proper turn. Additional remarks were recorded by the observer when necessary.

RESULTS

Errors

The desired situation is that where the driver makes the correct turn and does so without creating a hazard. For the purposes of this study, missed turns and turns that were correct but which created hazardous conditions were considered errors.

The condition where only the directional marker at the intersection was present with no advance turn markers resulted in 19 errors, 9 percent of the possible 212 turns for this condition (Table 1). For all practical purposes, no errors occurred where there were advance turn markers. One other error occurred where there were two advance turn markers, but this was attributed to extraneous factors. A χ^2 test performed on the frequencies of errors for the different test conditions indicated that there was less than one chance in a thousand that such a disproportionate

TABLE 1

ERRORS IN FOLLOWING ROUTE

No. of Advance Turn Markers	Possible No. of Turns	Errors (No.)	Errors (%)
0	212	19	9.0
1	212	0	0.0
2	212	1	0.4

number of errors could have occurred by chance.

Fourteen of the 19 errors occurred at left turns and 9 of these occurred at one particular intersection where a combination of topography and traffic density was deemed responsible for the high error occurrence. As a total of 18 drivers encountered this intersection under the condition where no advance turn markers were present, these 9 errors represent a 50 percent error occurrence. (This intersection will be discussed later.)

TABLE 2
AVERAGE DISTANCE FROM INTERSECTION AT WHICH SUBJECT WAS IN PROPER LANE FOR TURN

Type of Turn	Average Distance (car lengths) 0 Markers	1 Marker	2 Markers
Short left	5.6	7.4	9.1
Short right	7.0	9.0	10.0
Long left	6.6	11.9	11.7
Long right	6.5	8.6	8.4
All left	6.1	9.6	10.4
All right	6.8	8.8	9.1
All short	6.3	8.2	9.5
All long	6.6	10.2	10.0
Average	6.4	9.1	9.7

Proper Lane and Use of Turn Signals

Another measure of the effectiveness of advance turn markers was felt to be the distance from the turn at which the driver entered the proper lane to make the turn. A third measure of effectiveness and of the driver's awareness of the presence and direction of the turn was assumed to be the distance from the turn at which he displayed the proper turn signal. These distances were estimated by the observer to the nearest car length. All subsequent data analysis was also carried out in terms of car lengths.

For convenience in the field, distances greater than 15 car lengths were recorded as 15. For distances in the proper lane, values of 15 averaged 11 percent with no advance markers, 25 percent with one advance marker, and 33 percent with two. For turn signals, the respective figures were 1, 8, and 9 percent. Because many of the distances recorded as 15 may actually represent greater distances, and because they represent a substantial percentage of the field entries for the lane data, the average distances and the variability of the distribution of lane distances are probably higher than were computed.

Table 2 gives the average distance, in car lengths, from the intersection at which the driver was in the proper lane for the turn, by number of advance turn markers. The data have been analyzed with respect to whether the turn was to the right or to the left and also as to whether the distance from the preceding turn was long or short. Figure 3 shows the same analysis for the average distance from the intersection at which the proper turn signal was given.

In almost every case, there appears to be some benefit from the use of an advance turn marker. When distance in the proper lane is the criterion (Table 2) for both right and left turns, some additional benefit may be derived from two advance markers when the turn is a short distance from the preceding turn, whereas one advance turn marker seems sufficient for long turns. When turn signal distances are analyzed (Fig. 3), however, the situation is just the opposite. For both right and left turns, no additional benefit is seen for short turns, whereas there appeared to be some for long turns.

To further isolate the factors influencing the effectiveness of the route marker installations, an analysis of variance was performed on

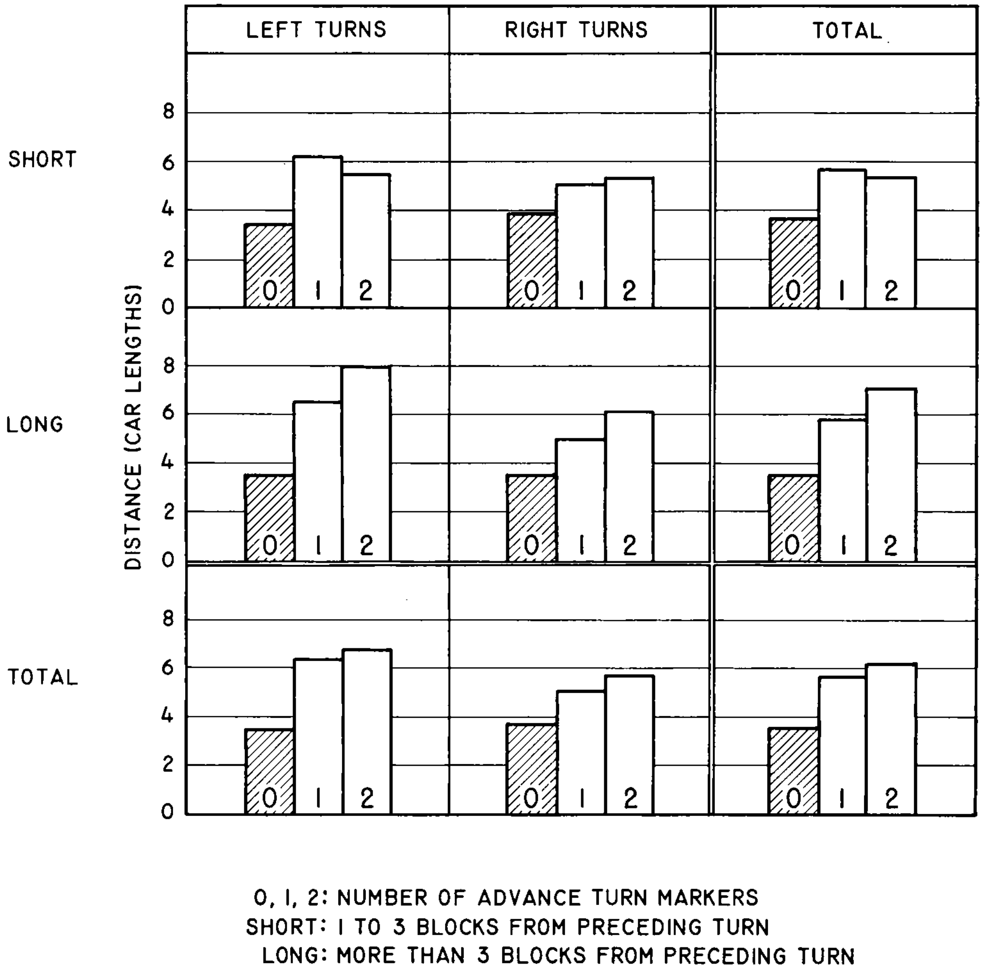

Figure 3. Effect of number of advance turn markers on use of turn signals.

each major dependent variable (Table 3). For the variable of distance in proper lane, the number of advance turn markers was a significant factor; however, the direction of turn did not seem to have an appreciable effect. There was also a high variability in magnitude of response among subjects. However, all the interaction terms involving subjects were not statistically significant; therefore, it is concluded that the relative response between conditions with different numbers of advance markers was not significantly different from one subject to another. It is believed that the possible higher actual variability due to the 15-car length cutoff would not have affected these results.

For the variable of proper turn signal, the number of advance turn markers is again a significant factor in difference of response. In this case, however, the direction of turn seems to make a difference, possibly because

TABLE 3
SUMMARY ANALYSIS OF VARIANCE FOR VARIABLES INDICATED

Source	Degrees of Freedom	Distance in Proper Lane			Distance at Which Turn Signal Given		
		Sum of Squares	Mean Square	F	Sum of Squares	Mean Square	F
Number of advance turn markers	2	1,302	651	32.6[a]	833	417	37.9[a]
Direction of turn	1	32	32	1.6	78	78	7.1[a]
Subjects	52	2,449	47	2.4[a]	3,963	76	6.9[a]
Number of advance turn markers:							
By direction of turn	2	106	53	2.7	75	38	3.5[b]
By subjects	104	1,666	16	0.8	1,073	10	0.9
Direction of turn by subjects	52	925	18	0.9	406	8	0.7
Number of advance turn markers by direction of turn by subjects	104	1,742	17	0.9	790	8	0.7
Error	318	6,331	20	—	3,466	11	—
Total	635	14,553	—	—	10,684	—	—

[a] Significant at 0.01 level. [b] Significant at 0.05 level.

drivers are more likely to signal for a left turn, which involves a greater probability of conflict, than for a right turn. This is reflected in Figure 3. In addition, because the interaction of number of advance markers with direction of turn leads to statistical significance, it is concluded that certain combinations of number of advance markers and the direction of turn have a different effect than others, which can also be seen in Figure 3. As in the case of the lane data, there was a high variability in magnitude of response among subjects. Again, because all the interaction terms involving subjects were not statistically significant, it is concluded that the relative response to different numbers of advance turn markers was not significantly different from one subject to another.

Combining the nonsignificant interaction terms with the error term and recomputing the ratios of the mean squares did not affect these results.

ANALYSIS

It might be argued that the test subjects were not typical of the average driver. All were graduate engineers who had worked for the Bureau of Public Roads from six months to three years. Their experience in the field of highway and traffic engineering might have tended to result in somewhat higher performance in the study. However, this study was concerned with the relative effects of various numbers of advance turn markers, and such relative effects would be reflected in the performance of this group. Perhaps the observed effects would have been more pronounced with more typical subjects.

The number of errors that occurred was lower than had been expected. The reasons for this small number of errors are subject to conjecture but there are three factors believed to be responsible: (a) the group of subjects were all graduate engineers and possibly more alert and more aware than average drivers; (b) the target value of the sign was important, which is discussed later, and (c) the directional markers at the intersections were always mounted on the far right corner which is also a location for

Figure 4. Advance turn marker installation largely hidden from view of approaching driver by truck on extreme right.

traffic signals in the District of Columbia. For convenience, these turn markers were often mounted on the signal poles, usually very close to the signal. It is presumed that most, if not all, drivers were looking for the signal and therefore had more of a chance to see the marker. Those stopped for the traffic signal also had much more time to see it. However, the chances for being stopped for a traffic signal were the same regardless of the number of advance markers. Therefore, the fact that almost all the errors occurred when there were no advance turn markers indicates that the presence of an advance turn marker was beneficial in reducing errors in following the route.

Because any distance over 15 car lengths was reported as 15, the average distances for the condition where one or two advance turn markers were present were very likely higher than those calculated since many of them are recorded as 15 car lengths. Other factors that may have affected the data are that the subjects were not specifically told the purpose of the study, and did not receive specific instructions relating the proper lane and the use of the turn signal to their knowledge of the impending turn. They may have been in the proper lane due to chance. Conversely, they may have turned on their turn signals when close to the intersection even though the markers had been seen further back. However, it is felt that the net result of these factors tended to minimize the observed differences.

ADDITIONAL OBSERVATIONS

The field crew, consisting of junior engineers of the Bureau of Public Roads, made additional observations pertinent to this report.

At several intersections topography, alignment, or physical obstructions (such as transit buses or double parked trucks) obscured to some extent the driver's view of the turn

Figure 5. Intersection where route turns left is just beyond stanchion in center of street. When no advance turn markers were present, 50 percent of subjects missed this turn.

markers. Figure 4 shows a type of situation in which two advance turn markers might prove of value.

Errors

A large percentage of the missed turns came from one such intersection in the heart of the downtown area, shown in Figure 4. The block on the approach to the intersection was on a downgrade, whereas the intersection itself and the preceding one were level. Neither the intersection nor the directional marker could be seen until the driver had passed through the preceding intersection. While he was on the downgrade, the directional marker, mounted as it was on the far side of the level intersection, was not in his direct line of sight. Because he was not aware of the impending left turn until he was close to the intersection, the heavy traffic often prevented him from getting into the proper turn lane.

Where no advance turn markers were posted, drivers repeatedly had near misses due to being positioned in the wrong lane (Fig. 5). The driver often held up traffic near or at the intersection in an attempt to position himself in the correct turning lane. This situation was considered by the observer to be an error if the turn created a hazardous condition.

When the density of traffic was low on one-way streets, there was frequently no realization on the part of the driver that the street was one-way. Consequently, on left turns the driver would often signal and turn from the wrong lane. For purposes of analysis this was not treated as an error because no actual hazard was created due to the light density of traffic; however, these subjects were not credited with being in the proper lane.

Figure 6. White route marker on black square shows up well against both dark and light background.

Design of Markers and Arrows

One possible reason for the low number of errors is the high target value of the design used for the route marker. Observation in the field showed that the white circle on black square design, based on the new standards in the Manual on Uniform Traffic Control Devices, was visible at a much greater distance than was expected, and long before the numerals were legible. It could be easily picked out from the array of other signs visible along a street (Fig. 6).

With respect to the arrows that were used, it was observed that for the directional marker at the intersection it was difficult to determine which direction the horizontal arrow was pointing except at relatively short distances away from it (Fig. 7). The direction in which the advance turn arrow was pointing could be determined at a somewhat greater distance because the position of the vertical part of the shaft gave a clue to its orientation. It would seem, therefore, that the old 13- by 10-in. arrows are insufficient even for low-speed urban usage.

Some of the test drivers were not aware of any difference in meaning between the bent advance turn arrow and the horizontal directional arrow. This was particularly evident in situations where two advance turn markers were used and a short block length

Figure 7. Difference in legibility of marker numerals and arrow apparent when photo is viewed at various distances.

preceding the turn made it necessary to place the first marker seen by the driver close to the preceding intersection. Some drivers, unaware of the difference in arrows, became confused and almost turned a block too soon.

CONCLUSIONS

There is a need to have at least one advance turn marker placed in advance of an urban intersection where a route turns. The lone directional marker at the intersection gives the driver too little time for response and, in doing so, may create confusion, congestion, and possible hazard.

There seems to be little evidence from this study that a second advance turn marker is of much value except possibly in cases where alignment or large vehicles may obstruct the driver's view, or where heavy traffic or other distractions may cause him to miss seeing the one advance marker. There is also some slight evidence from the turn signal data to show that two advance turn markers may be advantageous where there is a long distance between successive turns.

ACKNOWLEDGMENTS

The cooperation of the District of Columbia Department of Highways which prepared the route markers and arrows used in this study is gratefully acknowledged.

The following engineers, during their period of training as junior engineers in the Washington office of the Bureau of Public Roads, conducted the field work, performed the preliminary data analysis, and submitted a preliminary report: David M. Ham, Thomas E. Knisely, Gardner M. Rice, George A. Rodes, Albert E. Stone, and John R. Webster.

Evaluating Effectiveness of Lane-Use Control Devices at Intersections

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Field studies were made at several high-volume signalized intersections in the Chicago area to evaluate the effectiveness of different designs of roadside and overhead traffic signs for controlling multiple turns (double-left turns or double-right turns). The comparative effectiveness of different roadside double-turn signs was also studied in the laboratory by measuring response times and accuracy of responses to questions asked of groups of subjects concerning their interpretations of test signs, when shown in relation to simulated driving situations. Slides were used to display the driving situations and the test signs.

It was found that effectiveness of multi-turn controls could be evaluated in the field by studying changes in the use of the second, or optional, lane for turns, coupled with observations of violations of lane-use controls. In the laboratory the questions on accuracy of response appeared to provide the most useful information for evaluation of effectiveness of double-turn control signs.

It was concluded that the regulatory lane-use control signs in the 1961 edition of Manual on Uniform Traffic Control Devices should be satisfactory for control of multiple turns, until refinements are developed from further research.

• AT MANY URBAN intersections, turning volumes in one direction during peak hours are sufficiently heavy so that more than one traffic lane is required to accommodate the turns. This condition applies especially at intersections of one-way streets, and on routes leading to and from freeways.

State motor vehicle codes have generally required that turns at an intersection must be made only from the left-hand lane (for left turns) or from the right-hand lane (for right turns) unless special turning controls are established and posted. However, experience in and usage of special turn controls had been too limited to warrant inclusion of any standards on design and application of special turn control devices in the 1948 edition of the Manual on Uniform Traffic Control Devices, or the 1954 revisions.

The purpose of this study was to evaluate the effectiveness of different designs of signs and markings for controlling turns from more than one lane. The study was begun in 1958; preliminary results have been made

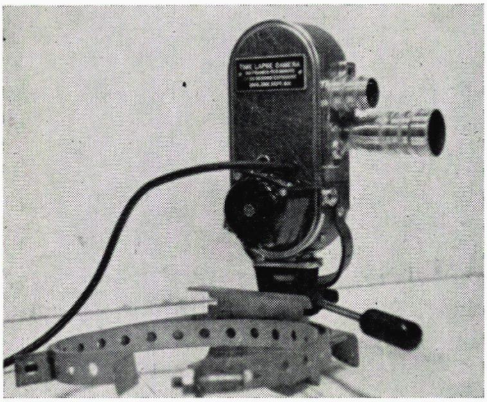


Figure 1. Time-lapse camera used in field studies.

available to the National Joint Committee on Uniform Traffic Control Devices to aid it in preparing standards for special turn controls for the 1961 Manual (*1*).

The study was conducted in three parts:

1. Field studies to develop criteria for evaluating effectiveness of different designs of regulatory double-turn signs.
2. Field tests at several right-angle, skew, and T-intersections, before and after different types of multi-turn control devices were installed.
3. A laboratory simulation study for evaluating minor differences in designs of roadside signs.

Most of the data for the field studies were gathered by time-lapse cameras, with pictures taken at 1-sec intervals (Fig. 1). Lane volumes, turning movements, lane changes, lengths of queues, and patterns of exiting movements were taken from the films with the aid of a modified Kodak film analyzer, shown in Figure 2. Hand-tally methods were used to collect turning volume data for some of the later studies.

FIELD STUDY TO ESTABLISH EFFECTIVENESS CRITERIA

Preliminary studies were conducted at Lincoln and Peterson Avenues, Chicago, to aid in establishing criteria for evaluating effectiveness of double-turn control signs. This intersection, shown in Figure 3, was controlled by a three-phase, fixed-time signal. Traffic entering from the northwest approach was studied. A separate phase for the northwest approach provided 17 sec of green and 3 sec of yellow in the 65-sec cycle.

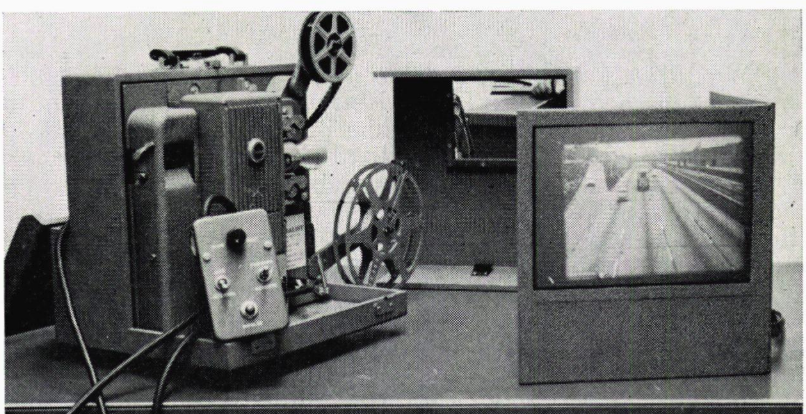


Figure 2. Projector used in film analysis.

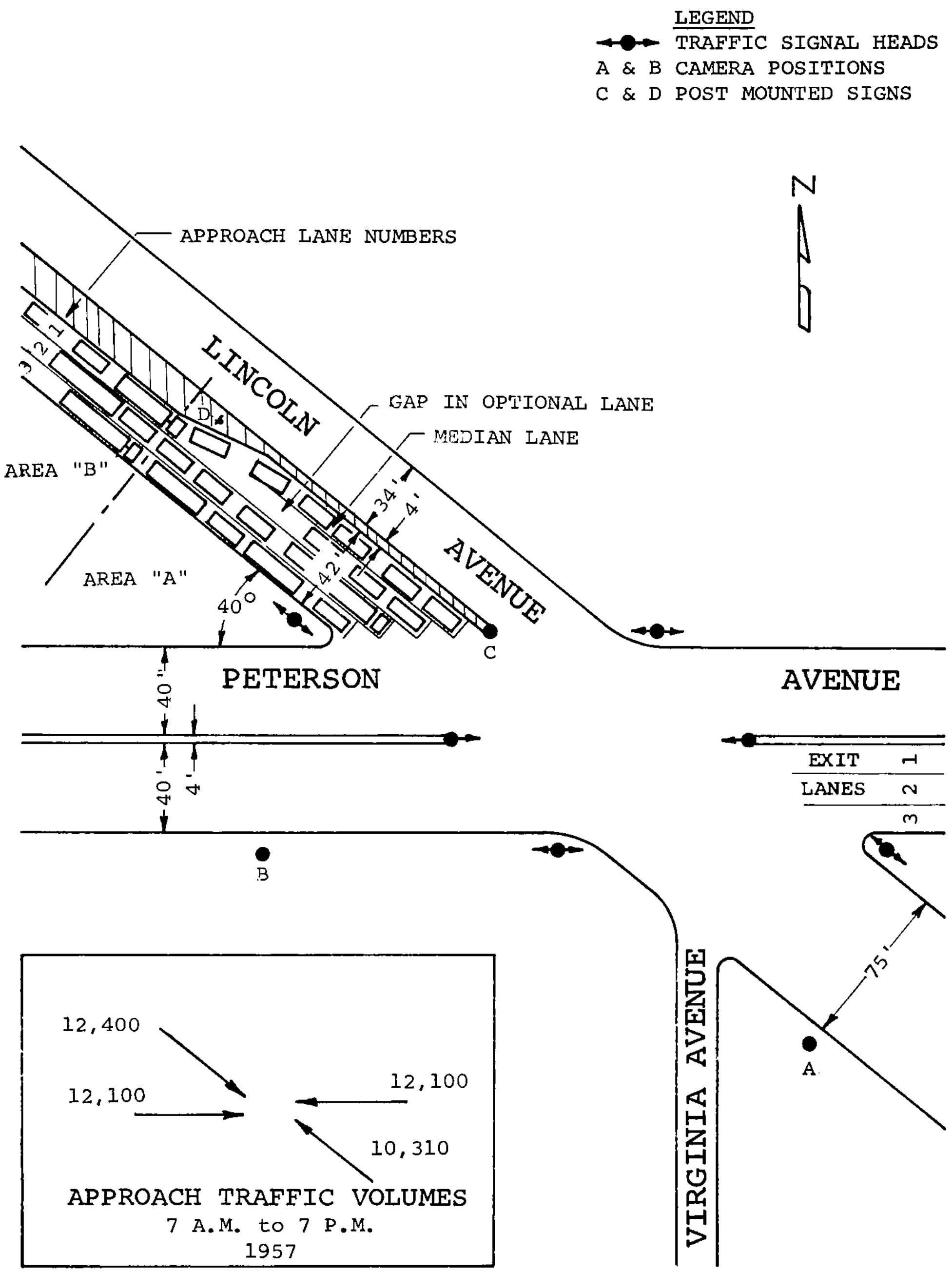


Figure 3. Intersection layout at Lincoln and Peterson Avenues, Chicago, showing "before" conditions and terms used in evaluation of lane changes and vehicle gaps.

TABLE 1

TRAFFIC VOLUME CONDITIONS FOR SIX STUDY PERIODS
(NORTHWEST APPROACH TO INTERSECTION OF LINCOLN AND PETERSON AVENUES, CHICAGO, 1959)

Study Period		Code Designation		Average Traffic Volume	
Day	Time	Before	After	Total Approach Volume Per Cycle	Left Turns (%)
Weekday	7:30- 8:30 AM	A	a	32.0	38.1
Weekday	5:00- 6:00 PM	B	b	27.2	29.9
Weekday	2:15- 3:15 PM	C	c	14.8	34.0
Sat.	10:45-11:45 AM	D	d	17.6	35.8
Sun.	4:00- 5:00 PM	E	e	21.5	34.4
Sun.	5:00- 6:00 PM	F	f	23.2	35.6

In these preliminary studies, data were gathered for two sets of lane-use controls:

1. Before. Double-left turn authorization had been provided by arrows and word messages painted on the pavement; these were worn off at the time of the Before study.
2. After. Two double-left turn signs (Fig. 4-a) were mounted on the median.

To evaluate possible effects of different double-turn control signs over a wide range of traffic volume conditions, before and after data were gathered for six study periods, as summarized in Table 1. Data were summarized for all six study periods combined (Periods a through f). Summaries were also prepared combining the three off-peak periods (Periods c, d, and e).

Criteria

It was hypothesized that criteria for the evaluation of the effectiveness of lane-use controls should be based on driver reactions to the controls before and after changes were made in the controls. The most effective control should produce the highest level of performance in one or more of the following criteria:

1. A high level of use of the optional lane for turns (the optional lane is the lane adjacent to the lane normally used for turns).
2. More balance in the use of the approach lanes, as measured by length of the queues, gaps in the queues, and/or delays.
3. Lane-changing—with adequate advance notice of double-turn authorization, drivers should change lanes farther in advance of the intersection.
4. Turning and exiting—drivers turning from each lane should follow more definite paths into the appropriate exit lanes.
5. Violations should be held to a minimum, especially those in which median-lane vehicles cross the intersection instead of turning.

Use of the Optional Lane

The preliminary studies at Lincoln and Peterson Avenues revealed that the comparative percent of left turns made from the optional lane offered real promise as a sound criterion of effectiveness of double-left turn controls, because of the ease of measurement of optional lane turns and their relationships to total left turns per cycle and to median-lane volumes.

Figure 5 shows the relationship between optional lane left turns and total left-turning volumes, for both Before and After conditions for the six study periods given in Table 1. There was a sharp increase in optional lane turns for the AM peak-hour conditions as compared with the

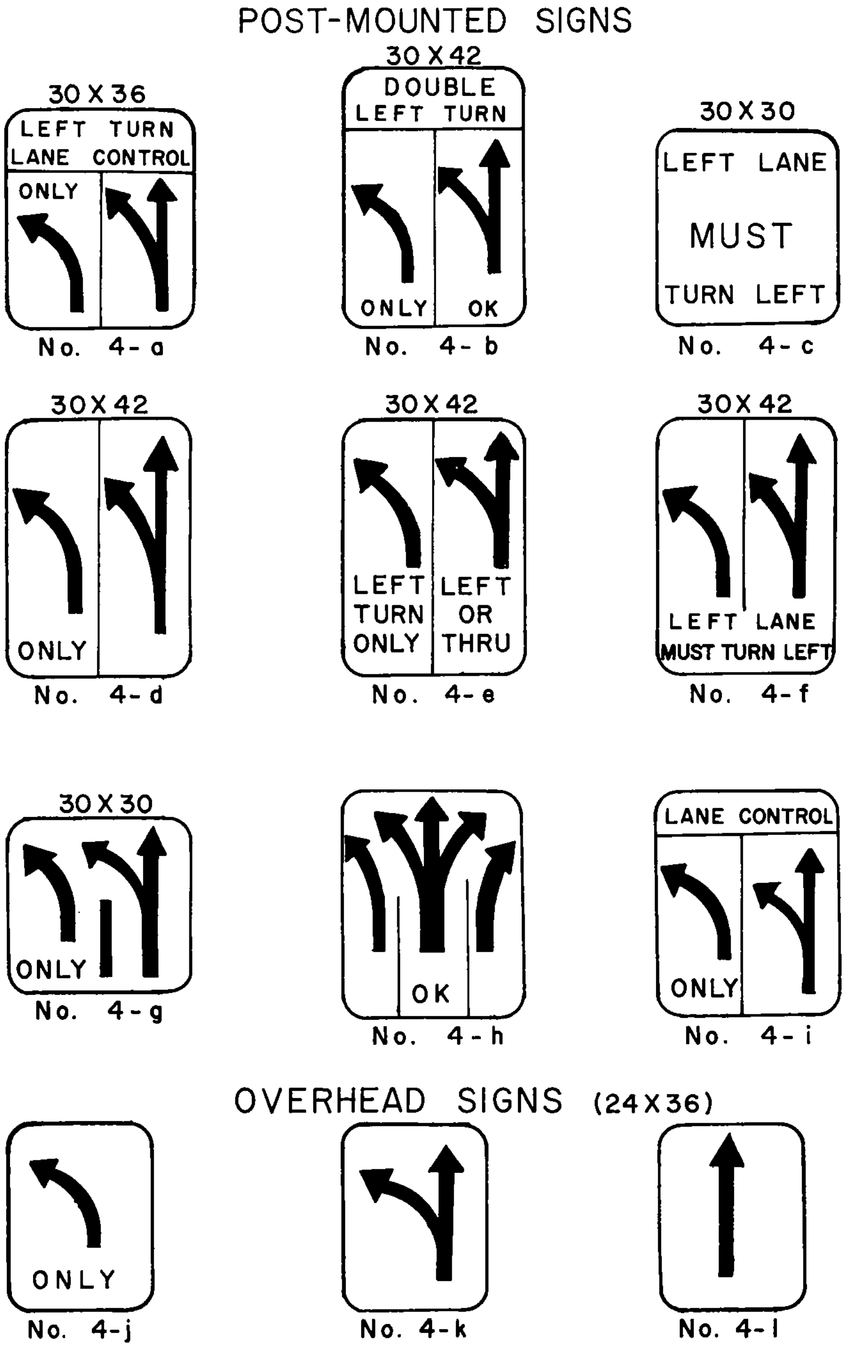


Figure 4. Post-mounted and overhead lane-use control signs tested in field and laboratory.

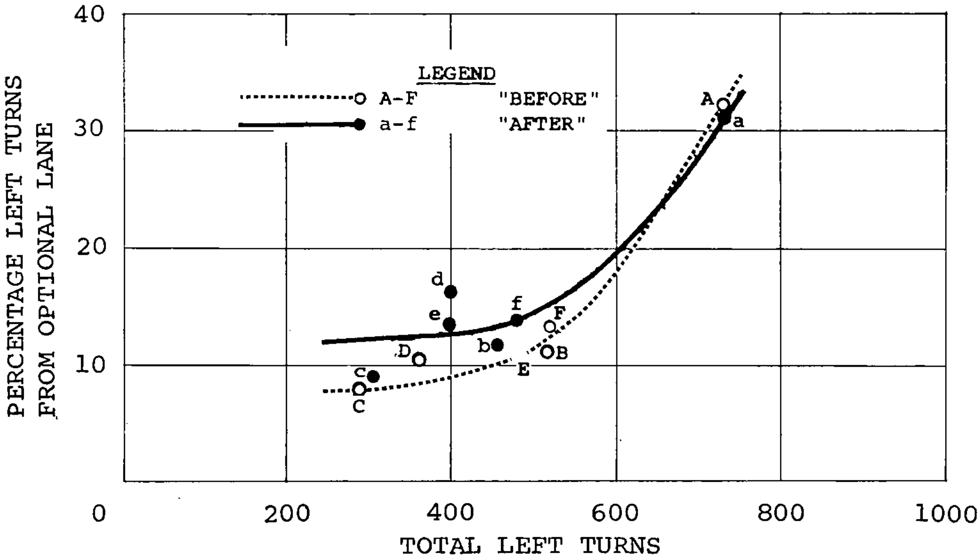


Figure 5. Use of optional lane for left turns before and after installation of double-left turn signs at Lincoln and Peterson Avenues, Chicago, 1959.

other study conditions. This occurred because the left-turning volumes of 740 vph in the AM peak hour could not possibly be accommodated in the median lane, which had a possible capacity of about 500 vph.

Tests of statistical significance of differences between Before and After data obtained in these preliminary tests are covered later.

TABLE 2

LANE LOADINGS AS PERCENT OF VEHICLES STILL WAITING TO ENTER AT END OF GREEN (LINCOLN AND PETERSON AVENUES, CHICAGO, 1959)

Condition (6 hr each)	Percent of Vehicles Delayed			
	Lane 3	Lane 2	Lane 1	Median Lane
Before	44.4	55.5	72.8	29.4
After	31.8	39.2	51.8	18.9
Difference	28.4	29.2	29.0	35.7

Balanced Use of Approach Lanes

Balance in the use of approach lanes was studied in two parts: (a) loadings of lanes at the end of the green phase and (b) gaps in the queue of vehicles in the optional lane, resulting from the lane being blocked by vehicles waiting to enter the median lane.

Table 2 gives results of lane loadings at the end of the green, for the sum of the six study periods combined, for the before and after conditions. Each lane loading value is, in effect, that percent of total lane volume waiting in line at the end of the green. The median lane loadings were low, compared with the optional lane, reflecting the effect of the short median lane. Vehicles waiting to enter the median lane actually queued up in the optional lane. Thus, with a short median lane, this criterion is not particularly satisfactory.

Gaps frequently occurred in the queue of vehicles waiting in the optional lane, because this lane was blocked by the queue of vehicles waiting to enter the median lane (Fig. 3). Statistical analyses were made of data on these gaps. It was concluded that gap data are too closely related to dimensional effects

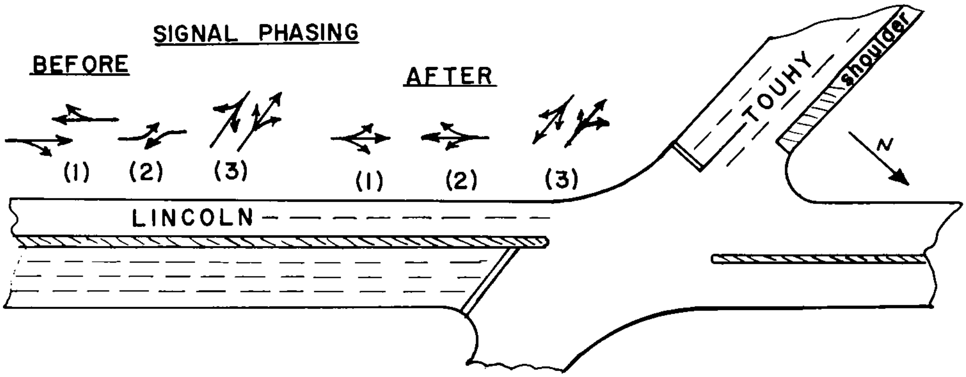


Figure 6. Intersection layout at Lincoln and Touhy Avenues, Lincolnwood, Ill., with changes in signal phasing.

of length of median lane to be generally useful for a criterion of effectiveness of lane-use control devices. They could be useful, however, in comparing before and after data at one intersection.

Lane loadings would be particularly useful as a criterion at an intersection such as Lincoln and Touhy Avenues (Fig. 6), where the median lane is sufficiently long to accommodate the queue of vehicles waiting to turn. In studies at that intersection, prior to changes in signal phasing, the median lane was frequently loaded much heavier than the other lanes.

Lane Changing

Studies were made of lane changing occurring in each of two zones: a zone within 150 ft of the intersection (Zone A), and the zone in the next 150 ft (Zone B). It was reasoned that when lane-use controls provide adequate advance information, a decrease in lane changes may be expected in the immediate vicinity of the intersection, and possibly an increase in the zone farther away.

Figure 7 shows that lane changing did decrease somewhat in Zone A after installation of post-mounted double-turn control signs at Lincoln and Peterson, with an increase in lane changing in Zone B, farther from the intersection. This differential was found only for lower volume conditions. The study also revealed however that most of the lane changes in Zone A involved changes from Lane 2 to Lane 1, which were not changes associated with optional lane turn controls.

Because of the lane changes not related to turns, and the dimensional effects of the median left-turn lane, and the time required to take lane-change data off the films, the lane-change criterion was not considered suitable for general application in this study.

Exiting

The turning paths of vehicles negotiating double turns can furnish clues as to the value of double-turn controls. When drivers turning in the optional lane are confident that the median-lane drivers will always turn, the optional-lane drivers are less likely to swing wide on the turn.

Analyses of the before and after exit-lane data for morning peak-hour conditions at Lincoln and Peterson Avenues indicated that there was a significant shift toward use of exit lanes corresponding more closely to

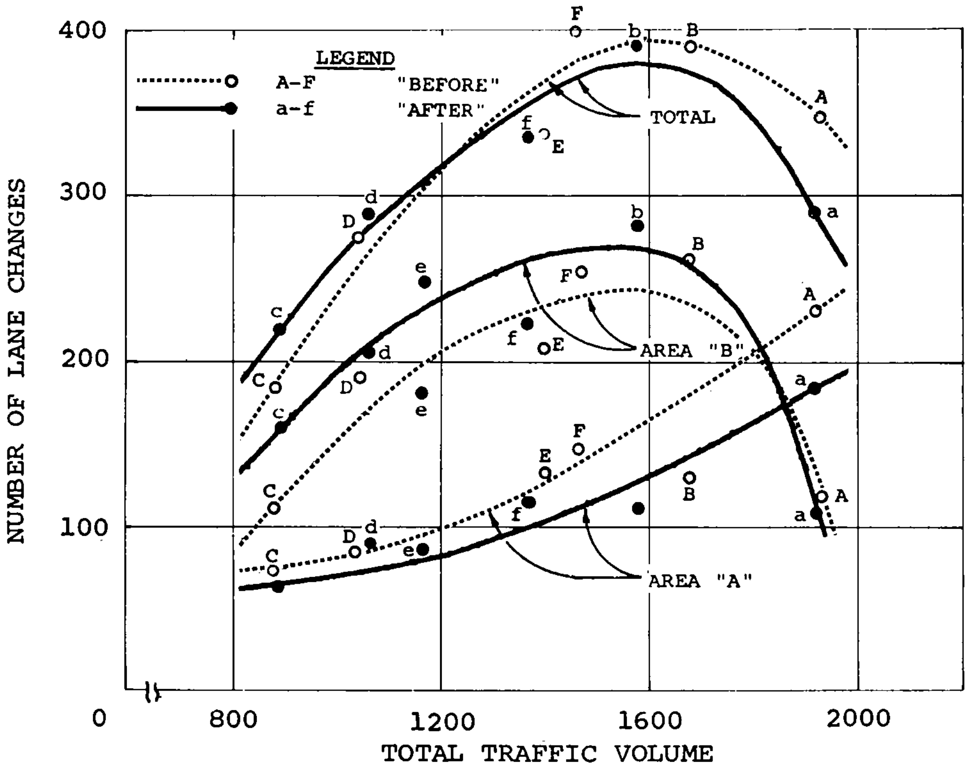


Figure 7. Lane changes by vehicles in Zones A and B, on Lincoln Avenue in approach to Peterson Avenue intersection before and after installation of two double-left turn signs, 1959.

TABLE 3
LANE USE IN EXIT ROADWAY
(LINCOLN AND PETERSON AVENUES, CHICAGO, 1959, AM PEAK HOURS)

Alignment in Exit Roadway	Entry from Median Lane		Entry from Optional Lane	
	Before	After	Before	After
In Median Lane (Lane 1)	166	215	0	0
Straddling Lanes 1 and 2	258	262	1	7
In Lane 2	74	8	15	62
Straddling Lanes 2 and 3	7	0	68	129
In Lane 3	2	0	128	26
Total	507	485	212	224

entrance lanes. Results are given in Table 3. This criterion was not adopted for use at other intersections, however, because it is useful primarily when there are 3 exit lanes for a 2-lane turn.

Violations

Frequency of violations is a possible criterion because violations are indications of potential accident hazard. Accordingly, counts were made

TABLE 4

FIELD TESTS WITH ROADSIDE AND OVERHEAD LEFT-TURN LANE-USE CONTROL SIGNS
(PHASES I AND II, PEAK AND OFF-PEAK CONDITIONS, LINCOLN AND PETERSON AVENUES, CHICAGO, 1959)

Study Period	Type of Double-Turn Control	Date Installed	Date of Field Tests		Left Turns			
			Off-Peak	Total, Incl.	Median Lane (no.)	Optional Lane (no.)	Optional Lane (%)	Total (no.)
Before	Pave. markings authorizing double turns worn out	—	4/22; 4/27; 5/10		1,027	113	9.9	1,140
				3/25: 4/10, 15	2,429	460	16.0	2,889
After I	Roadside sign, "Left Turn Lane Control," "Only"	4/22/59	6/9; 6/20; 6/21		974	136	12.2	1,110
				6/9, 21, 22	2,315	478	17.2	2,793
After II	Signs over each of 4 lanes, Nos.	7/9/59	10/3; 10/11, 13		911	183	16.8	1,094
				9/28; 10/11, 13	2,148	574	21.1	2,722

of the number of median-lane drivers who crossed the intersection instead of making the mandatory turn. The number of such violations generally did not exceed 5 for 4 hr of observation; thus, the numbers were too small to provide differences of statistical significance. The violations criterion was included, however, to supplement the criterion on use of the optional lane for turns.

FIELD STUDY OF SIGN DESIGNS

Roadside vs *Overhead Signs*

After selecting the two criteria for field evaluation of the effectiveness of lane-use controls, additional studies were made at Lincoln and Peterson Avenues, using both roadside and overhead double-turn control signs. The six 1-hr study periods used in the Before and the After I field studies were also used for the overhead sign study, After II study. The schedule of the tests is given in Table 4. Signs used in the After II study are shown as signs 4-j, 4-k, 4-l (Fig. 4).

Data on the use of the optional lane for left turns for each of the three test conditions were classified according to total left turns per cycle. Summary tabulations were prepared for the six study periods combined (upper portion of Table 5) and for three off-peak periods only (lower portion of table). Mean number of optional-lane left turns per cycle and standard deviations were computed for each study condition, omitting those cycles with less than four left turns per cycle.

Because the frequency distributions of total left turns per cycle differed somewhat for the three study conditions, a uniform frequency distribution was computed (Table 6) so that, for each study condition, each class of total left turns per cycle would contain the same number of cycles. Mean values of optional-lane left turns per cycle were computed for this uniform frequency distribution for each study condition, and these means were used in comparing effectiveness of the double turn controls in increasing optional-lane left turns.

The one-side t' statistic was used to compare the mean number of left turns per cycle from the optional lane, as determined for the three study conditions (*2*). The results in Table 7 show changes in numbers of optional-lane left turns per cycle from Before to After I conditions, and to After II conditions. Computed values of the t' statistic are also given. Results indicate that there was a statistically significant increase in optional-lane use at the 95 percent level of confidence after installation

TABLE 5

LEFT TURNS FROM OPTIONAL LANE AT DIFFERENT TOTAL LEFT TURNS PER CYCLE, FOR AFTER I AND II STUDY

(LINCOLN AND PETERSON AVENUES, CHICAGO, 1959)

Total No. of Left Turns per Cycle	Before[1] No. of Cycles	Before[1] Left Turns from Optional Lane: Total	Before[1] Left Turns from Optional Lane: Per Cycle	After I[2] No. of Cycles	After I[2] Left Turns from Optional Lane: Total	After I[2] Left Turns from Optional Lane: Per Cycle	After II[3] No. of Cycles	After II[3] Left Turns from Optional Lane: Total	After II[3] Left Turns from Optional Lane: Per Cycle
(a) ALL SIX STUDY PERIODS									
0-1	2	0	0.00	3	0	0.00	8	0	0.00
2-3	25	3	0.12	31	5	0.16	32	7	0.22
4-5	44	14	0.32	56	23	0.41	71	26	0.37
6-7	86	39	0.45	92	64	0.70	82	80	0.97
8-9	101	85	0.84	86	85	0.99	66	101	1.53
10-11	53	110	2.07	51	111	2.17	49	110	2.25
12-13	32	109	3.40	21	85	4.02	28	107	3.82
14-15	12	65	5.42	18	94	5.22	17	91	5.32
16-17	5	35	7.00	2	11	5.55	7	52	7.41
Total	360	460	1.28	360	478	1.33	360	574	1.59
(b) THREE OFF-PEAK STUDY PERIODS ONLY									
0-1	2	0	0.00	3	0	0.00	6	0	0.00
2-3	24	2	0.08	25	4	0.16	24	4	0.17
4-5	38	14	0.37	42	12	0.28	48	18	0.38
6-7	59	26	0.44	60	45	0.75	56	53	0.95
8-9	41	37	0.90	35	37	1.06	27	43	1.59
10-11	13	26	2.00	13	29	2.23	14	36	2.57
12-13	3	8	2.67	1	4	4.00	2	9	4.50
14-15	0	0	—	1	5	5.00	3	20	6.67
Total	180	113	0.63	180	136	0.76	180	183	1.02

[1] Worn-out turn control markings.
[2] Roadside double-turn control sign.
[3] Overhead turn-control signs.

of the overhead lane-use control signs, especially for the off-peak conditions, as compared with the Before conditions. The increases from the After I to the After II conditions were also statistically significant for the one-sided t' test.

The effects of length of exposure to lane-use control signs were not evaluated in this series of studies. It is possible that part of the increased use of the optional lane might have occurred if the roadside signs had remained, and motorists had the additional three months of exposure to the roadside signs, instead of to the overhead signs.

After installation of roadside signs (After I), the change in optional lane use was not significant, as compared with optional-lane use under the Before condition (with worn-out double-turn pavement markings). This undoubtedly reflected the high level of optional-lane use, especially during the peak hours, at this intersection under the Before conditions. Motorists had been making use of the optional lane for turns because of the lack of median-lane capacity for turns, and a signal phasing that was favorable for multi-lane left turns, even though no authorizing markings were present at the time.

After III Roadside Signs Study

Five different designs of roadside lane-use control signs were tested during four off-peak hours (Saturday, 2:30-3:30 PM; Sunday, 1:00-2:00 PM; Monday, 1:00-2:00 PM; Tuesday, 2:30-3:30 PM) as the next phase of the Lincoln-Peterson study.

TABLE 6

COMPUTED LEFT TURNS FOR OPTIONAL LANE FOR UNIFORM FREQUENCY OF LEFT TURNS PER CYCLE, FOR AFTER I AND II STUDY

(LINCOLN AND PETERSON AVENUES, CHICAGO, 1959)

Total Left Turns per Cycle	Uniform Frequency Used as Comparison	Left Turns from Optional Lane for Uniform Cycle Frequency		
		Before	After I	After II
(a) ALL SIX STUDY PERIODS				
0-1	—	—	—	—
2-3	—	—	—	—
4-5	57	18.3	23.4	21.2
6-7	87	39.3	61.0	84.5
8-9	84	70.5	83.0	129.0
10-11	51	105.5	111.0	115.0
12-13	27	91.8	108.5	101.5
14-15	16	86.7	83.7	85.1
16-17	5	35.0	27.6	37.1
Total	327	447.1	498.2	573.4
Avg. per cycle		1.37	1.52	1.76
(b) THREE OFF-PEAK STUDY PERIODS ONLY				
0-1	—	—	—	—
2-3	—	—	—	—
4-5	43	15.9	12.1	16.4
6-7	58	25.6	43.5	55.0
8-9	34	30.5	36.0	54.0
10-11	13	26.0	29.0	33.4
12-13	2	5.3	8.0	9.0
14-15	—	—	—	—
Total	150	103.3	128.6	167.8
Avg. per cycle		0.69	0.86	1.12

TABLE 7

SIGNIFICANCE[1] OF CHANGES IN NUMBER OF OPTIONAL-LANE LEFT TURNS PER CYCLE

(AFTER I AND II STUDIES, LINCOLN AND PETERSON AVENUES, CHICAGO, 1959)

Study	After I[2]						After II[3]					
	All Study Periods			Off-Peak Only			All Study Periods			Off-Peak Only		
	Change[4]	Significance	t'	Change[4]	Significance	t'	Change[4]	Significance	t'	Change[4]	Significance	t'
Before[5]	1.37-1.52	None	1.26	0.69-0.86	None	1.63	1.37-1.76	Incr.	2.89	0.69-1.12	Incr.	3.26
After I[2]							1.52-1.76	Incr.	1.94	0.86-1.12	Incr.	1.88

[1] A t' of 1.65 is needed for statistical significance at the 95 percent level, using a one-sided test.
[2] Roadside signs.
[3] Overhead signs.
[4] In numbers of optional-lane left turns per cycle.
[5] Worn-out lane-use pavement markings.

The purpose of this study was to evaluate minor differences in sign design. The study schedule, the numbers of turns made from the median lane and the optional lane during each of the six study periods, and the sign design tested in each study period are given in Table 8. The five different sign designs are shown in Figure 4 as signs 4-b, 4-d, 4-e, 4-i and 4-f. Sign 4-d was used for two of the six study periods (After IIIb and After IIIf). The percentage of total left turns made from the optional lanes varied from 19.0 to 21.3 percent for the six study periods.

TABLE 8

SCHEDULE FOR FIELD TESTS OF EFFECT OF CHANGES IN POST-MOUNTED SIGNS ON USE OF OPTIONAL LANE

(PHASE III, OFF-PEAK CONDITIONS, LINCOLN AND PETERSON AVENUES, CHICAGO)

Study Period	Sign	Date Installed	Dates of Tests	Left Turns			
				Median Lane (no.)	Optional Lane (no.)	Optional Lane %	Total (no.)
After IIIa	Fig. 4-b	11/25/59	5/23, 24/60; 6/4, 5/60	1,112	275	19.9	1,387
After IIIb	Fig. 4-d	6/8/60	6/18-20, 22/60	1,105	282	20.3	1,387
After IIIc	Fig. 4-e	7/6/60	7/16-19/60	1,056	287	21.3	1,343
After IIId	Fig. 4-i	7/20/60	7/30, 31/60; 8/1, 2/60	1,109	294	20.9	1,403
After IIIe	Fig. 4-f	8/3/60	8/13-16/60	1,087	261	19.4	1,348
After IIIf	Fig. 4-d	8/17/60	8/27-30/60	1,012	237	19.0	1,249

The numbers of left turns made from the optional lane were classified according to the number of total left turns per cycle (Table 9). Average values and standard deviations were computed for each of the study conditions, omitting cycles with less than four total left turns per cycle. A uniform frequency distribution of cycles was computed, and the mean number of left turns from the optional lane was computed for each study condition. The results are given in Table 10.

Statistical tests then were made to determine whether the differences in the average number of left turns per cycle (as given in Table 10) were significant at the 95 percent level. None of the differences were found to be significant.

Table 10 also gives the number of violations of the requirement that vehicles in the left lane must turn left. When sign 4-e for condition After IIIc (with messages, "Left Turn Only" and "Left or Thru") was in place, the greatest number of violations was observed.

Optional-lane use for peak-hour conditions only was also studied (summarized in Table 11). Statistical tests did not reveal any significant changes in optional-lane left turns per cycle, after installing roadside signs or overhead signs, for peak-hour conditions.

Effect of Signal Phasing

A before-and-after study was made at the intersection of Lincoln and Touhy Avenues, Lincolnwood, to determine the effect of a change in signal phasing on use of a second lane for left turns. This intersection (shown in Fig. 6) formerly had a three-phase vehicle-actuated traffic signal with a separate left-turn phase. Because the traffic in the lane adjacent to the left-turn lane moved only on the signal phase allocated to through traffic, it was difficult for drivers to use the second lane for left turns, even though the heavy left-turn volumes resulted in high delay at many times.

The signal cycle was changed (as shown in Fig. 6) to permit all vehicles from the southwest approach to enter on the same signal phase. Before and after studies were made for six different hours of observation. Results (given in Table 12) indicated that for four comparable periods, the average percentage of left turns from the optional lane increased from 7.0 to 17.2 percent. This indi-

TABLE 9

LEFT TURNS FROM OPTIONAL LANE AT DIFFERENT TOTAL LEFT TURNS PER CYCLE FOR SIX STUDY CONDITIONS OF TABLE 8

(PHASE III, OFF-PEAK PERIODS, LINCOLN AND PETERSON AVENUES, CHICAGO, 1960)

Left Turns per Cycle from Both Lanes	After IIIa			After IIIb			After IIIc			After IIId			After IIIe			After IIIf		
	No. of Cycles	Left Turns from Opt. Lane		No. of Cycles	Left Turns from Opt. Lane		No. of Cycles	Left Turns from Opt. lane		No. of Cycles	Left Turns from Opt. Lane		No. of Cycles	Left Turns from Opt. Lane		No. of Cycles	Left Turns from Opt. Lane	
		Total	Per Cycle		Total	Per Cycle		Total	Per Cycle		Total	Per Cycle		Total	Per Cycle		Total	Per Cycle
0-1	9	1	0.11	6	0	0.00	8	0	0.00	7	0	0.00	14	0	0.00	8	0	0.00
2-3	33	10	0.30	36	8	0.22	50	21	0.42	44	12	0.27	34	9	0.26	46	12	0.26
4-5	81	53	0.66	70	34	0.49	60	42	0.70	63	37	0.59	73	50	0.68	77	46	0.60
6-7	51	59	1.16	73	95	1.30	66	86	1.30	60	77	1.29	69	77	1.13	72	100	1.39
8-9	47	93	1.97	35	74	2.12	41	81	1.98	41	86	2.10	33	66	2.00	30	58	1.93
10-11	19	59	3.11	16	51	3.18	11	38	3.45	20	51	2.55	13	43	3.31	6	16	2.67
12-13	—	—	—	3	15	5.00	4	19	4.75	3	13	4.33	4	16	4.00	1	5	5.00
14-15	—	—	—	1	6	6.00	—	—	—	2	12	6.00	—	—	—	—	—	—
Total	240	275	1.15	240	283	1.18	240	287	1.19	240	294	1.22	240	261	1.09	240	237	0.99

cates that where large demands for left turns exist and where signal phasing permits, motorists will use an optional lane for left turns even though there are no appropriate controls that permit them to do so. In this case, the hazard of making left turns from the second lane was reduced by a "Left Turn Only" message painted in the median lane.

One-Way Street Study

A before-and-after study was made on two Saturday mornings at Chestnut and Main Streets, Rockford, to determine whether new designs of post-mounted double-turn signs might be more effective than the turn controls already in use. The Before controls consisted of double-left turn pavement markings, plus curb-mounted signs reading "Curb Lane Left Turn Only," as shown in Figure 8. In the After study, the old signs were replaced by two signs of the type like Figure 4-b and one like Figure 4-c.

Double-left-turn controls had been in effect at this intersection for more than a year under the Before condition. There was no significant change in use of the optional lane after replacing the old signs with the new curb-mounted signs. The percentage of left turns made from the optional lane dropped from 18.6 percent before, to 17.5 percent after the change in signs.

Truck-Turn Location

At the intersection of State and Chicago Streets in Elgin, State highway traffic entering from the north approach on State Street turned left into a one-way street (Chicago Street), as shown in Figure 9. Traffic entering from State Street moved on a separate signal phase.

A Before study was made with the double-turn controls that already were in effect. These controls included signs installed in the median with messages, "Inside Lane Left

TABLE 10

LEFT TURNS FROM OPTIONAL LANE PER CYCLE FOR UNIFORM FREQUENCY OF TOTAL LEFT TURNS PER CYCLE

(PHASE III, OFF-PEAK STUDY OF SIX SIGN INSTALLATIONS, LINCOLN AND PETERSON AVENUES, CHICAGO, 1960)

Left Turns	Uniform Frequency Used as Comparison	Left Turns from Optional Lane per Cycle					
		After IIIa	After IIIb	After IIIc	After IIId	After IIIe	After IIIf
Total per cycle:							
0-1	0	—	—	—	—	—	—
2-3	0	—	—	—	—	—	—
4-5	70	0.66	0.49	0.70	0.59	0.68	0.60
6-7	65	1.16	1.30	1.30	1.29	1.13	1.39
8-9	38	1.97	2.12	1.98	2.10	2.00	1.93
10-11	14	3.11	3.18	3.45	2.55	3.31	2.67
12-13	0	—	—	—	—	—	—
14-15	0	—	—	—	—	—	—
Avg. per cycle		1.29	1.30	1.37	1.29	1.30	1.30
Std. Dev., s		1.11	1.25		1.22		
Violation (thru from median lane)		3	1	5	2	1	0

TABLE 11

PEAK-HOUR OPERATION WITH DIFFERENT DOUBLE-LEFT-TURN LANE CONTROLS

(8:30-9:30 AM, LINCOLN AND PETERSON AVENUES, CHICAGO)

Study [1]	Cycles per Study	Date	Thru Volumes (No.)		Left Turn Volumes			
			Median Lane	Optional Lane	Median Lane (No.)	Optional Lane		
						No.	%	Per Cycle
Before	60	Wed, 3/25/59	3	228	493	234	30.8	3.71
		Thur, 4/30/59	—	216	507	212		
After I	60	Wed, 6/10/59	3	264	480	189		
		Thur, 6/11/59	—	251	485	224	30.4	3.56
		Mon, 6/22/59	5	212	505	229		
After II	60	Wed, 7/22/59	0	232	478	206	31.4	3.75
		Mon, 9/28/59	0	210	511	245		
After IIIa	51	Fri, 4/29/60	1	173	402	208	34.3	4.00

[1] See Tables 4 and 8 for descriptions of the signs used.

Turn Only," and pavement arrows and word messages, as shown in Figure 9. The intersection did not appear to be fully loaded in peak hours. The double-turn control had been provided primarily to aid trucks in turning and to provide a more even balance in lane use.

Two sets of After I studies were made at 1-week intervals after replacing the signs with post-mounted double-left-turn signs, as shown in Figure 4-a. The After II studies were made using overhead signs as shown in Figure 4-j, k, and l. Studies were made both on weekday peak hours and on Saturday mornings.

Results (given in Tables 13 and 14) revealed no significant change in optional-lane use after any of the changes in the double-turn control signs. The prior use of lane-use controls had produced a pattern of optional-lane use in which 37 percent

TABLE 12
LANE USE BEFORE AND AFTER REPHASING TRAFFIC SIGNALS
(LINCOLN AND TOUHY AVENUES, LINCOLNWOOD, 1959)

Time of Study [1]	Total Approach Volume			Left Turns	
	No. of Vehicles	Optional Lane (%)	Median Lane (%)	Total Volume (%)	From Optional Lane (%)
(a) BEFORE REPHASING SIGNALS					
6/26 4:45-5:50	1,437	12.7	50.2	54.0	7.1
6/30 4:15-5:20	1,441	14.2	46.1	50.9	8.9
6/30 5:30-6:35	1,435	12.5	49.0	52.1	6.2
7/20 4:30-5:35	1,377	10.7	49.4	52.5	5.8
Total	5,690	12.4	49.5	52.4	7.0
(b) AFTER REPHASING SIGNALS					
8/28 5:00-6:05	1,438	15.6	47.0	56.0	16.3
10/6 4:30-5:35	1,387	16.5	43.6	52.7	16.9
10/6 5:45-6:20	762	15.6	47.4	56.0	15.0
9/10 4:30-5:35	1,459	17.5	43.8	53.6	19.4
Total	5,046	16.4	44.8	54.1	17.2

[1] Afternoon, 1959.

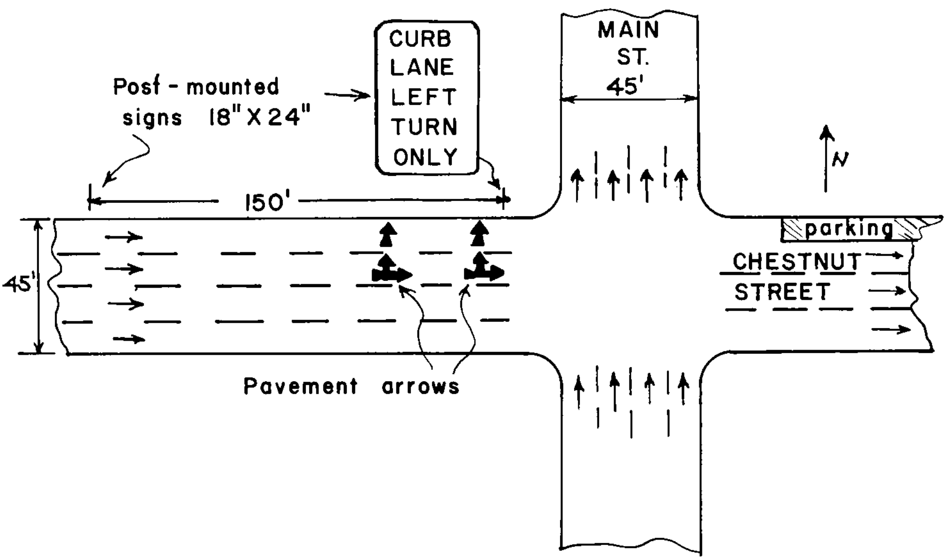


Figure 8. Intersection plan and lane-use controls in "before" study at Main and Chestnut Streets, Rockford, Ill., 1959.

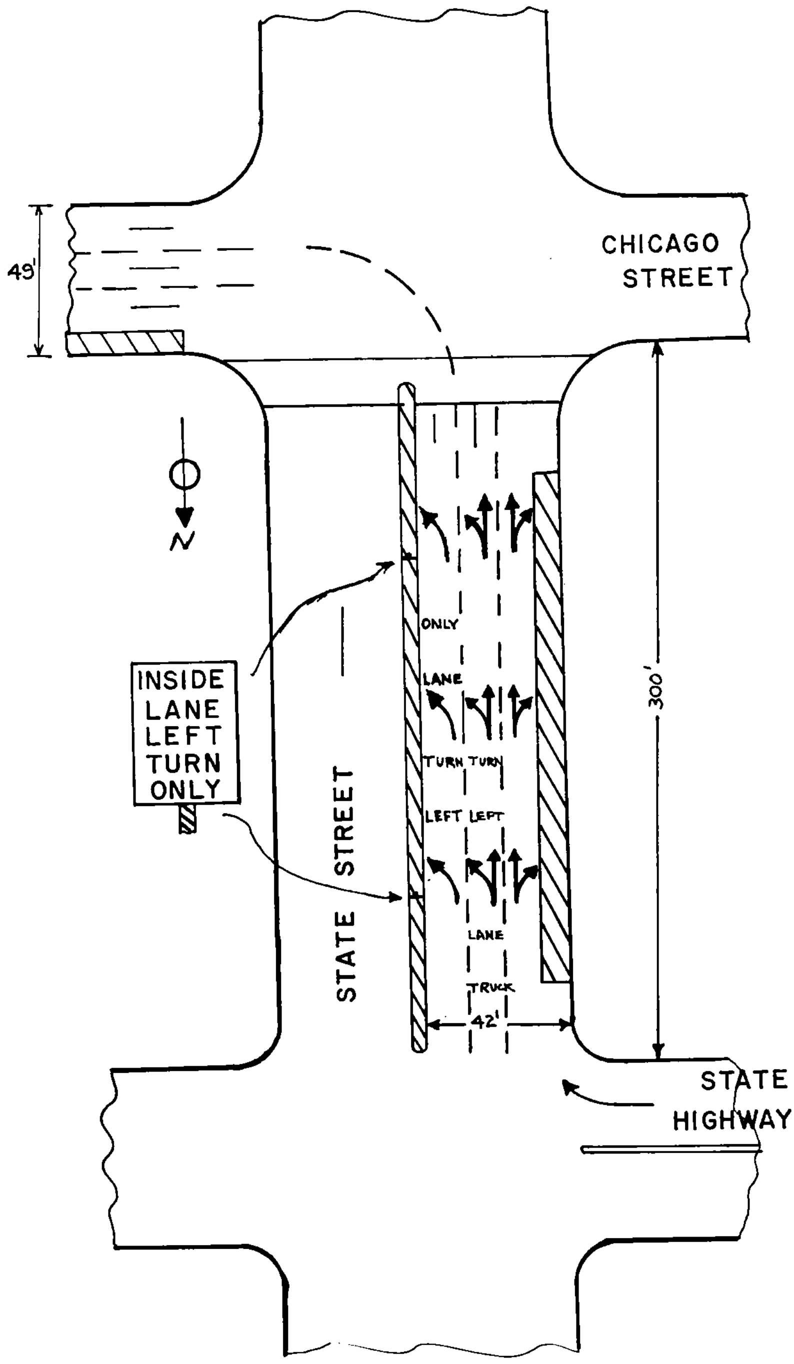


Figure 9. Intersection plan and lane-use controls in "before" study at State and Chicago Streets, Elgin, Ill., 1959.

TABLE 13

LANE USE WITH DIFFERENT LANE-USE CONTROLS

(TRAFFIC ENTERING ON STATE STREET AT CHICAGO STREET, ELGIN, 1959)

Condition	Total Volume [1]		Percent Turning from Optional Lane	Violations Thru from Median Lane
	No.	Percent Left Turns		
Before, with lane use markings and "Inside Lane Left Turn Only" sign	2,805	48.8	37.0	1
After Ia, one week later, with double turn sign (Fig. 4-a)	3,012	48.6	40.0	1
After Ib, another week later	3,075	51.9	41.0	3
After II, with overhead signs (Fig. 4j, k, l)	2,898	49.9	38.4	5

[1] In 3 hr.

of the left turns were already being made from the optional lane. The optional lane was used for 38 to 41 percent of the left turns after installation of new double-turn control signs. The number of violations of the "Left Turn Only" control for the median lane was low for each set of conditions.

The T-Intersection Study

A T-type intersection was also included as part of the field study program, because the effects of different controls on optional-lane turning might be more readily apparent at a T-intersection than at a four-way intersection.

The T-intersection selected for study was 95th Street and Cottage Grove Avenue, in Chicago, as shown in Figure 10. The approach studied was the 36-ft approach from the north on Cottage Grove Avenue.

At the beginning of the study, there were no markings or signs authorizing turns from the center lane of the 36-ft approach. Preliminary volume counts indicated that approximately one-half of the traffic approaching on Cottage Grove turned right, and the other half turned left. Very little use was made of the center lane, even though the curb lane and the median lane were loaded for much of the peak hour from 5 to 6 PM. A separate signal phase provided 20 sec of green and 3 sec of yellow for the Cottage Grove approach in a 65-sec pre-timed cycle. Commercial vehicles comprise 5 to 6 percent of the volume, with bus lines turning both to the right and to the left onto 95th Street.

The design of the experiment called for making observations two or three times a month so as to permit evaluation of the effects of the length of time drivers were exposed to lane-use controls, or "learning" on the use of the center lane. The plan also called for tests with pavement markings only, as well as installations of roadside lane-use control signs at a later stage in the study.

Table 15 gives the description of each lane-use control, the dates of installation, and the dates of collection of data. Conditions applying during the After IV study are shown in Figure 10. Authorization of center-lane use was given only for conditions labeled as After Ia, After III, and After IV.

A summary of data collected for the 16 different days of field study is given in Table 16. This summary includes, for each of the 2 hr of observation taken each day, total approach volumes per cycle, center-lane turns per cycle, and standard devia-

TABLE 14

LANE USE WITH DIFFERENT LANE-USE CONTROLS

(STATE AND CHICAGO STREETS, ELGIN, 1959)

Study	Date		Approach Lane Traffic Volume (no.)										Percent Left Turns	
			Right Turn from			Thru from			Left Turn from					
	Day	Time	Right Lane	Opt. Lane	Left Lane	Right Lane	Opt. Lane	Left Lane	Right Lane	Opt. Lane	Left Lane	Total	Of Total	From Opt. Lane
Before [1]	Wed 7/8	4:00- 5:00	87	0	0	315	85	0	0	186	302	975	50.3	38.0
		5:15- 6:15	86	2	0	247	85	1	2	130	224	777	45.6	36.7
	Thu 7/16	4:00- 5:00	91	1	0	321	112	0	1	192	335	1,053	50.0	36.5
	Sat 7/25	11:00-12:00	54	1	0	262	81	5	1	163	297	864	53.2	35.5
After Ia [2]	Wed 7/29	4:00- 5:00	69	0	0	346	95	1	0	197	302	1,010	49.9	39.4
		5:00- 6:00	84	0	0	290	119	0	1	150	215	859	42.4	41.0
	Thu 7/30	4:00- 5:00	97	1	0	327	118	0	1	240	358	1,143	52.2	40.0
After Ib [3]	Wed 8/5	4:00- 5:00	68	0	0	354	103	1	1	218	321	1,066	50.5	40.5
		5:00- 6:00	99	0	0	247	73	1	1	170	252	843	50.2	40.4
	Thu 8/6	4:00- 5:00	85	0	0	346	98	1	2	265	369	1,166	54.6	41.9
	Sat 8/8	11:00-12:00	78	0	0	254	105	1	1	159	328	926	52.7	32.6
After II [4]	Wed 8/19	4:00- 5:00	79	0	0	317	141	2	0	212	344	1,095	50.7	38.3
		5:00- 6:00	80	0	0	226	94	3	0	130	244	777	48.2	35.8
	Thu 8/20	4:00- 5:00	66	0	0	320	123	0	0	209	308	1,026	50.4	40.5
	Sat 8/15	11:00-12:00	81	0	0	292	97	0	0	228	302	1,000	53.0	43.0
	Sat 8/29	11:00-12:00	74	0	0	250	86	2	1	174	293	880	53.1	37.2

[1] Left-turn signs and lane-use markings authorizing double-left turns.
[2] New lane-use control signs—one week; Saturday missed on account of rain.
[3] New lane-use control signs—two weeks.
[4] Overhead lane-use control signs.

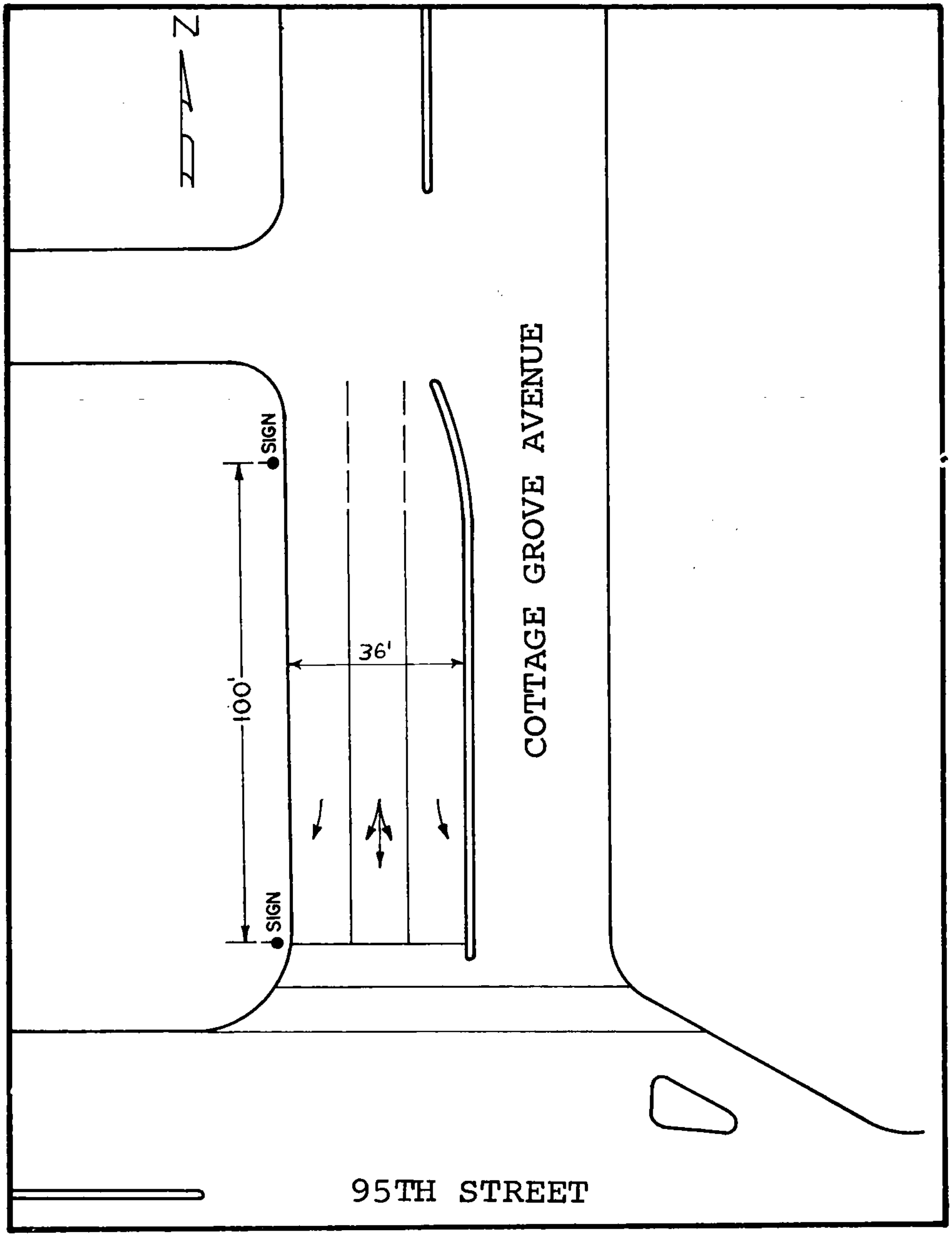


Figure 10. Intersection plan and lane-use controls during the After IV condition at Cottage Grove Avenue and 95th Street, Chicago, 1961.

TABLE 15

SCHEDULE OF LANE-USE CONTROL INSTALLATIONS AND OBSERVATIONS
(COTTAGE GROVE AVENUE, SOUTHBOUND LANES, CHICAGO, 1960-61)

Study	Date	Lane-Use Controls at Intersections	Date of Installation
Before	10/13/60	None	—
After I	10/20; 12/1, 6/60	Pavement markings, 4 lanes, word messages: "Left Turn Only," "Right Turn Only," Arrows in each lane	10/16/60
After Ia	2/9, 23/61	Pavement markings worn off	
After II	3/9, 23, 30; 4/6/61	Pavement markings, 3 lanes painted, no words or arrows	2/28/61
After III	4/13, 27/61	Pavement markings, 3 lanes painted, with turn control arrows added, no words	4/7/61
After IV	5/4, 18; 6/4, 22/61	Post-mounted signs, pavement markings (of After III condition, Fig. 10)	4/28/61

TABLE 16

USE OF CENTER LANE ON THREE-LANE APPROACH AT A T-INTERSECTION WITH VARIOUS LANE-USE CONTROLS
(COTTAGE GROVE AT 95TH STREET, CHICAGO, 1960-1961)

Study	Control	Date of Study	Traffic Volume Per Signal Cycle (54 Cycles): 4:00-5:00 PM Data: Total, All Lanes	4:00-5:00 PM Data: Center Lane Turns $\bar{x}_{4-5}$	4:00-5:00 PM Data: Center Lane Turns σ_{4-5}	5:00-6:00 PM Data: Total All Lanes	5:00-6:00 PM Data: Center Lane Turns $\bar{x}_{5-6}$	5:00-6:00 PM Data: Center Lane Turns σ_{5-6}
Before	None	10/13	12.2 [1]	0.854	0.76	12.3	0.815	1.28
After Ia	4 lanes, with arrows, words	10/20	11.2	1.63	1.34	13.0	1.93	1.37
		12/1	11.8	1.89	1.48	12.0	2.32	1.49
		12/6	13.0 [2]	1.82	1.61	13.0	2.37	2.03
After Ib	Markings gone	2/9	11.2	0.815	0.87	12.3	1.67	1.62
		2/23	11.2	1.11	1.15	11.7	1.72	1.44
After II	3 lanes, no words or arrows	3/9	—	NO DATA	—	11.9	1.35	1.35
		3/23	11.2	1.03	1.39	11.6	0.908	1.12
		3/30	11.5	1.24	1.27	12.4	1.50	1.32
		4/6	11.2	1.13	1.12	11.8	1.39	1.31
After III	3 lanes with arrows	4/13	11.8	1.39	1.75	13.1	1.44	1.72
		4/27	12.1	1.50	1.38	—	NO DATA	—
After IV	3 lanes, arrows, and signs	5/4	12.4	1.83	1.78	12.4	2.22	2.01
		5/18	12.3	1.84	1.80	13.9	2.43	1.53
		6/1	12.7	2.15	1.88	13.6	2.42	1.88
		6/22	12.5	2.30	2.03	12.1	2.28	1.87

[1] 41 cycles.
[2] 45 cycles.

tions of total center-lane turns per cycle.

After Ia: Pavement Turn Arrows, Words and Lane Lines (4 lanes)

The first After I study was made on October 20, four days after pavement markings were painted authorizing turns from two 9-ft center lanes. Turns per cycle from the center lanes more than doubled for both the 4:00-5:00 and 5:00-6:00 PM periods, as compared with the Before study (Table 16); the increase was statistically significant at a 95 percent level of confidence (Table 17). The center-lane use continued to increase with time, but field tests 6 weeks later did not show a significant

TABLE 17
SIGNIFICANCE[1] OF CHANGES IN NUMBERS OF CENTER-LANE TURNS PER CYCLE AT T-INTERSECTION
(COTTAGE GROVE AT 95TH STREET, CHICAGO, 1960-1961)

Conditions Compared	Time of Test (PM)	Change in Center-Lane Turns per Cycle	Change Significant at 95% Level	t'
(a) WITH CHANGE IN CONTROLS				
Before (Oct 13)	4-5	0.854-1.63	Yes	3.62
vs After Ia (Oct 20)	5-6	0.815-1.93	Yes	5.46
After Ia (Dec 6)	4-5	1.82 -0.815	Yes	3.90
vs After Ib (Feb 9)	5-6	2.37 -1.67	Yes	2.00
After Ib (Feb 23)	4-5	1.11 -1.13	No	
vs After II (Apr 6)	5-6	1.72 -1.39	No	1.24
After II (Apr 6)	4-5	1.13 -1.50	No	1.54
vs After III (Apr 27)	5-6	1.39 -1.44	No	
After III (Apr 13) vs After IV (May 4)	5-6	1.44 -2.22	Yes	2.40
After III (Apr 27) vs After IV (May 18)	4-5	1.50 -1.84	No	
(June 1)	4-5	1.50 -2.15	Yes	2.05
(b) NO DIFFERENCE IN CONTROLS				
After Ia	4-5	1.63 -1.82	No	
(Oct 21) vs (Dec 6)	5-6	1.93 -2.37	No	1.28
Before (Oct 13)	4-5	0.854-1.11	No	1.29
vs After Ib (Feb 23)	5-6	0.815-1.72	Yes	4.60
After IV May 4 vs May 18	4-5	1.83 -1.84	No	
	5-6	2.22 -2.43	No	
May 4 vs June 1	4-5	1.83 -2.15	No	
	5-6	2.22 -2.42	No	
May 4 vs June 22	4-5	1.83 -2.30	No	1.26
	5-6	2.22 -2.28	No	

[1] t' of 1.96 needed for significance.

increase over the October 20 values for either period.

The After Ib studies were made after the pavement markings of the After Ia study were almost completely erased by wear and weather. A significant decrease in center-lane use took place during these 10 weeks. However, the use of the center lane in February was still considerably greater than for the original condition on October 13, even though the approach was physically the same in both conditions. This indicated that the drivers had learned of the legality of the center-lane turns and had retained this knowledge even though the controls permitting this maneuver were no longer evident.

Lane lines providing a 3-lane approach were painted on Cottage Grove Avenue on February 28, 1961. No arrows or word messages were painted to authorize turns from the center lane.

Observations made on four days in March and April generally indicated decreases in the use of the center lane, as compared with the After Ib condition. These decreases were not large enough to be significant at the 95 percent level (Table 17). The remarking of lane lines, without arrows or messages to authorize center-lane turns, undoubtedly were interpreted by some regular peak-hour drivers to mean that center-lane turns were now prohibited.

Pavement arrows authorizing turns from the center lane were painted next to create the After III condition. Studies made one to three weeks

TABLE 18
PERCENT OF TOTAL LEFT TURNS AND TOTAL RIGHT TURNS MADE FROM CENTER LANES FOR VARIOUS STUDY CONDITIONS
(FROM COTTAGE GROVE AVENUE TO 95TH STREET, CHICAGO, 1960)

Study	Date	Percent of Total Left Turns from Center Lane		Percent of Total Right Turns from Center Lane	
		4:00-5:00 PM	5:00-6:00 PM	4:00-5:00 PM	5:00-6:00 PM
Before	Oct 13	11.4	8.8	3.3	4.2
After Ia	Oct 20	15.9	14.7	15.0	15.8
After Ib	Feb 9	10.6	17.5	4.7	9.8
After II	Apr 6	7.9	11.4	9.8	12.2
After III	Apr 13	15.4	11.1	8.7	11.0
After IV	May 4	16.2	12.3	12.6	22.4
	May 18	17.6	13.8	13.8	19.6
	June 1	17.4	16.3	20.0	19.4
	June 22	18.9	16.0	22.7	15.9

later showed slight increases in center-lane use, but statistical tests indicated that the increases might have been due to chance.

Next, post-mounted lane-use control signs, as shown by sign 4-i (Fig. 4), were installed at the curb for the After IV condition. Center-lane use in the After IV condition was generally significantly greater than under the After III conditions (see Table 17 for details).

An analysis was made of the percentages of turns to the right and to the left from the center lanes, for each study condition. During the After III condition, with turning arrows on the pavement, 11.0 percent of the right-turning traffic used the center lane. After the post-mounted signs were placed on the right side of the roadway (the After IV condition), the center lane carried an average of 20.4 percent of the right turns (Table 18). In contrast, left turns from the center lane increased only from 11.1 to 14.4 percent. The larger increase in right turns from the center lane may be explained by the belief that the new post-mounted signs were more readily visible to drivers intending to turn right than to left-turning drivers.

Most of the major changes in center-lane use at the T-intersection have been explained as being the result of changes in lane-use controls. However, the effects of continuing exposure to a new control may be identified in some portions of Table 16. Center-lane use after installation of the After Ia controls increased with time, and then appeared to level off. Similarly, there were continuing increases (not statistically significant) in center-lane use for the 4:00-5:00 PM data of the After IV series, as shown by four studies made a week apart.

The percentages of vehicles using the center lane for turns were plotted for each of eight ranges of total vehicles per cycle, for each group of data, and for the average of each group of observations. Results in Figure 11 show an increasing percent of use of the center lane with an increase of per-cycle volume. The curve for the After IV condition reveals higher percentages in use of the center lane at all volume ranges, as compared with the Before conditions, and with the After II condition.

Figure 12 shows a composite summary of data on center-lane use for each of the five conditions, for the T-intersection study, indicating that center-lane use was very responsive to different types of lane-use controls.

LABORATORY STUDY

The laboratory study was conducted in an attempt to identify differences in driver reactions to alternate designs of double-left-turn signs, under controlled conditions.

Past laboratory studies were reviewed before designing the experiment for this study. Adaptations of some of the methods used in a laboratory study of reversible lane marking for the Mackinac Bridge in Michigan were used in this study (*3*).

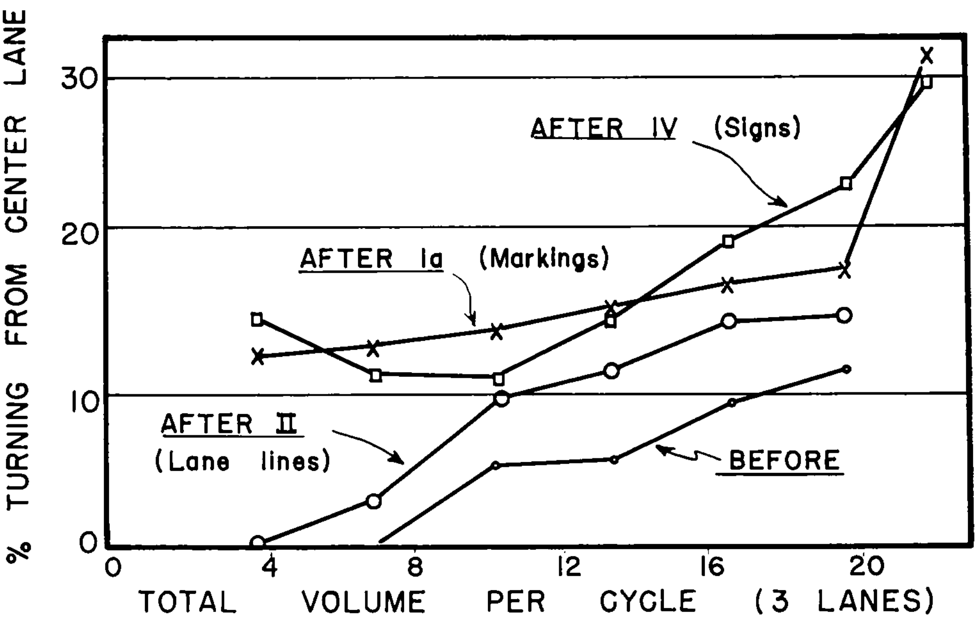


Figure 11. Turns from center lane at T-intersection as percent of total volume per cycle.

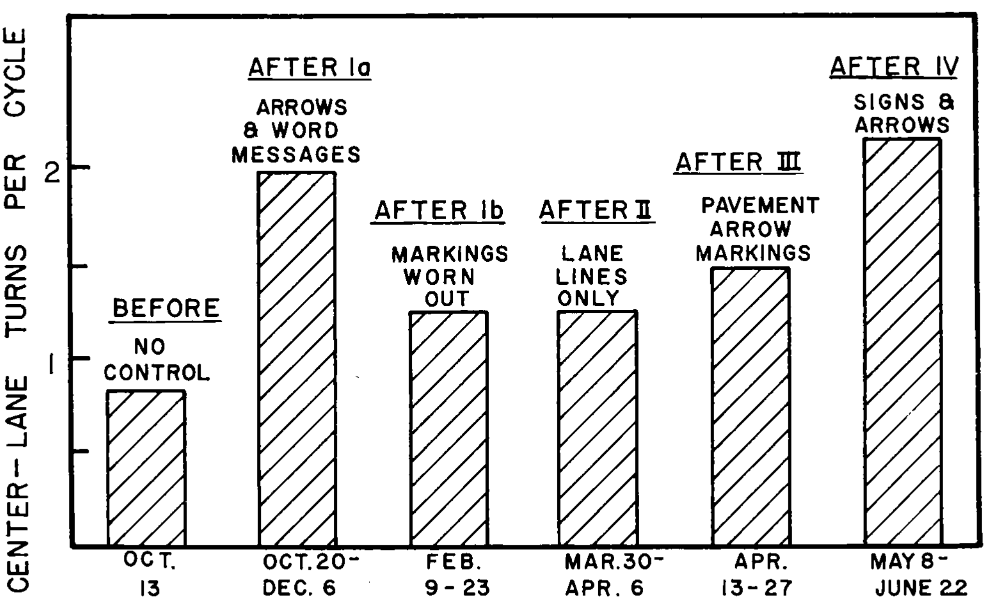


Figure 12. Center lane turns per cycle with different controls at T-intersection.

Experiment Design

The following characteristics of subject responses were studied in the laboratory evaluation of the effectiveness of alternate designs of double-turn signs:

1. *Accuracy of response*—the ability of a sign to convey accurately its intended messages: (a) vehicles in median lane must turn left and (b) vehicles in the optional lane may turn left.
2. *Time of response*—the time elapsing after first seeing the sign until the observer indicates that the message is understood.
3. *Learning*—the relation between the exposure or number of viewings of a sign, and the changes in accuracy and time of response.
4. *Personal opinions*—the statements of observers as to which design would be best as a uniform standard.

Most of the variables (such as sign location, traffic and visibility conditions, type of intersection, and speed of approach to the test sign) can be held constant in a laboratory study. Thus, the major variable studied would be the alternate designs of the turn control signs.

Several simulation procedures were considered for possible use in the laboratory. Each procedure was investigated, using the following criteria as the basis for comparison:

1. The test must simulate field conditions to the extent necessary to provide responses comparable to field responses.
2. The subjects must be clearly informed as to (a) what is being asked, (b) what driving situations the sign and questions apply, and (c) what sign governs his answer.
3. The test must present all of the information at once and keep the information in view so that the subjects do not forget part of this information before answering the questions.
4. It is necessary to disguise the purpose of the test; that is, the subjects should not be able to identify easily which signs are being tested. If the subjects were able to do this, they might tend to think about the signs during moments of lull, to concentrate too much on the test signs, and to anticipate those signs.
5. The test must be easy to repeat exactly for different groups so that the conditions of the test remain the same for all groups of subjects.
6. The time required for the test must be kept as short as is compatible with the amount of data needed.
7. Manpower and equipment requirements must be kept as low as possible.

One method investigated involved the use of a series of 16-mm film strips taken through the windshield of an automobile moving toward a field-mounted sign. It was hoped that by this method a reasonable simulation of an actual driving situation would be obtained.

This method had shortcomings, however. It was difficult to stabilize the line of sight of the camera when taking the pictures. Secondly, it was difficult to control all of the variables such as the amount of light and traffic conditions. The requirement of police officers to control the traffic conditions when taking the pictures was also a drawback. This method also introduced problems of variable legibility and of photographing both the sign and the driving situation simultaneously.

The laboratory method finally adopted used slides of intersection situations that also displayed questions on the legality of certain driving maneuvers. Also shown were slides of various traffic signs that governed the movements at the intersection. As these slides were projected, the time and accuracy of the response of each individual in each of the groups of subjects were recorded. A sequence of 24 such intersection-question-sign sets was shown to the group. Each sequence contained six sets which included one design of a double-left-turn sign. These sets were distributed among the other 18 sets. An intersection-question-sign set included two slides; one, a scene with a question, and the other, the sign controlling the legality of the movement called for in the question.

Four alternate designs of double-left-turn control signs were used in the laboratory experiment. (Test signs 1, 2, 3, and 4 correspond to signs 4-d, 4-e, 4-f, and 4-g of Figure 4.) Because each such sign carried two messages, it was necessary to determine the observers' understanding of each of these messages. The situations presented to the subjects also had to differentiate between at least two approach lanes.

The number of times an observer has seen a double-left-turn control sign affects his time and accuracy of response. Thus, it was decided that each group should see the same test sign six times during the sequence, so as to evaluate the effects of learning. Each of four groups of subjects saw a different test sign. Groups were compared with one another also by comparing their reactions to a European "No Left Turn" sign, which was shown four times to each group of subjects.

In a study of this type, too much emphasis should not be placed on the absolute values of the response times determined under laboratory conditions. Due to the artificiality of the laboratory conditions, it was not possible to equate these times to response times in real driving situations. Of interest, however, are relative differences among response times to traffic signs of alternate design but conveying the same message. These differences can be reliable, at least qualitatively, if the same level of artificiality is maintained for all of these signs.

With this method, emphasis was placed on measuring driver comprehension of the test signs and not on legibility. The legibility of the symbols and word messages was not a variable, because all signs were shown as though seen from a distance of only a few feet.

Procedure

For the intersection situations 35-mm color slides were used, and for the signs either color or black-and-white slides were used.

The slides of the intersection were taken from the middle of the lane in which the subject's vehicle was assumed to be traveling. All of the intersection slides were taken at northwest approach of Lincoln Avenue about 125 ft from its intersection with Peterson Avenue in Chicago. Hence, the intersection was not a variable. This location was chosen because it had enough lanes (4) to make a double-turn movement seem to be a realistic possibility to the subjects. Also, the intersection was wide enough so that the median offset of the left-turn bay of the opposite approach was not apparent. If it had been, the through movement from the median lane would not have seemed to be a realistic possibility.

For each question to be asked, the information concerning the intersection situation and the question itself were combined on one slide. This method kept the question before the subject and minimized the possibility of the subject's forgetting part of the information.

Although a sequence of 24 intersection-question-sign sets was shown to each subject, only five combinations of intersection approach lanes and questions were used in the test. Four of the five intersection-question slides are shown in Figure 13. The

TABLE 19
SEQUENCE OF INTERSECTION-QUESTION-SIGN SETS

Set No.	Intersection-Question			Test Sign Viewing	Sign	Correct Answer
	Slide No.	Lane Position	Question [1]			
1	1	Right	c		Stop	No
2	3	Median	a		No Turn	No
3	1	Right	c		Yield Right of Way	Yes
4	3	Median	a		European No Left Turn	No
5	3	Median	a		One Way (arrow to left)	Yes
6	5	Optional	a	1	Test Sign	Yes
7	3	Median	a		No Left Turn	No
8	4	Median	b		Left Lane Must Turn Left	No
9	2	Right	d		One Way (arrow to right)	Yes
10	4	Median	b	2	Test Sign	No
11	3	Median	a		European No Left Turn	No
12	2	Right	d		No Turns	No
13	3	Median	a		One Way (arrow to left)	Yes
14	3	Median	a		No Left Turn	Yes
15	5	Optional	a	3	Test Sign	Yes
16	2	Right	d		No Turns	No
17	3	Median	a		European No Left Turn	No
18	2	Right	d		No Turns	No
19	4	Median	b	4	Test Sign	No
20	3	Median	a		One Way (arrow to left)	Yes
21	4	Median	b		Left Lane Must Turn Left	No
22	5	Optional	a	5	Test Sign	Yes
23 [2]	3	Median	a		European No Left Turn	No
24a [2]	4	Median	b	6A	Test Sign	No
24b [2]	5	Optional	a	6B	Test Sign	Yes

[1] a=Can you legally turn left from the lane in which you are now traveling?
b=Can you legally proceed straight through this intersection from the lane in which you are now traveling?
c=Can you proceed straight through this intersection without stopping?
d=Can you legally turn right from the lane in which you are now traveling?

[2] European "No Left Turn" sign and test sign explained.

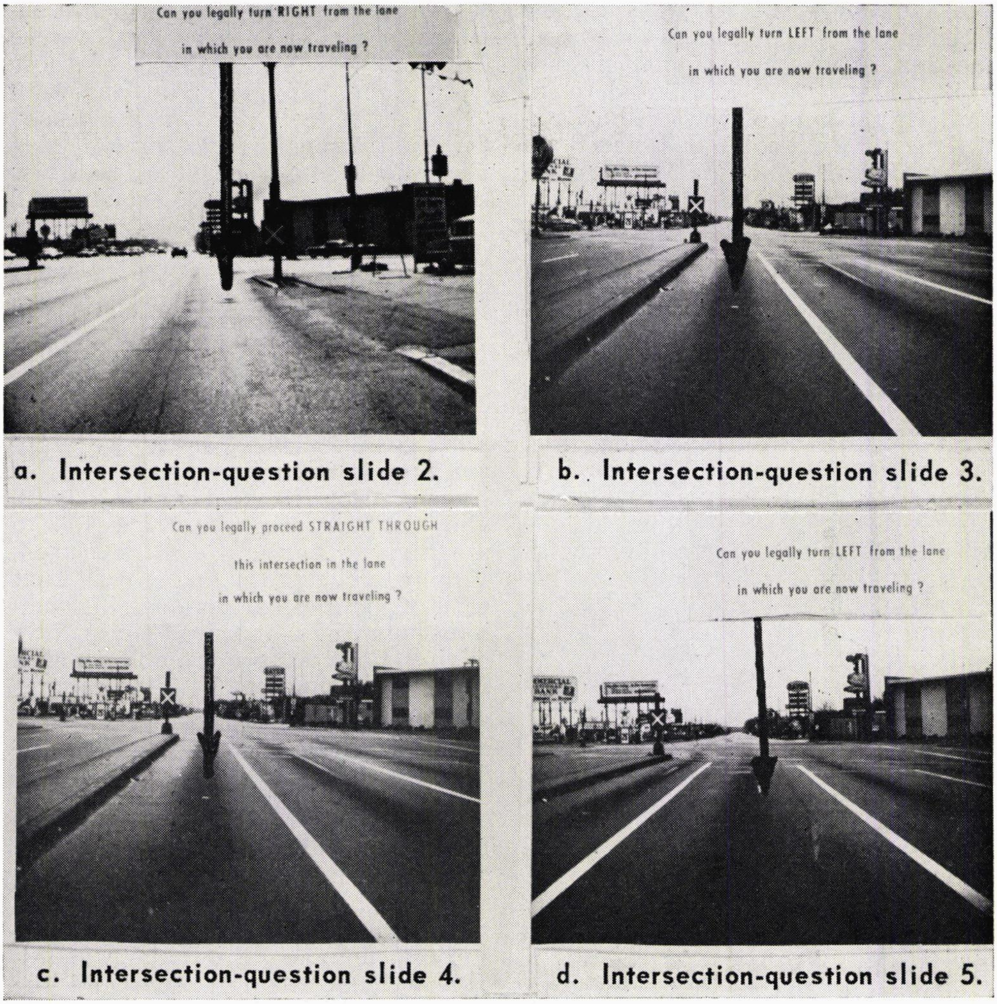
sequence of showing the 24 intersection-question-sign sets is given in Table 19.

The assumed field location of the test signs was identified on each intersection slide by a large red flag with a white X on it. The lane in which the subject's vehicle was assumed to be traveling was indicated by a downward pointing vertical arrow on the slide.

Information on the effect of learning was obtained by showing each of the subjects the same sign more than once. The response time and percent of errors should both decrease for subsequent appearances of the same sign with the same intersection-question slide. In the actual test, each subject saw one design of a double-left-turn sign six times and a European "No Left Turn" sign four times so that learning could be studied.

The projection was carried out with two basically standard Kodak slide projectors. The intersection-question slides were projected with a standard, commercially available slide projector of the Kodak 300 type. The projector that was used for the sign slides was of the Kodak 500 type but certain wiring modifications were made in this projector so that the time at which the projector light went on was recorded.

All the questions were designed for "yes" and "no" answers. A box on which two buttons—one for "yes" and one for "no"—were mounted, was placed on the arm of each of the subjects' chairs. Data on accuracy and times of response were recorded on an Esterline-Angus 20-pen recorder for each of the subjects in the group participating in a series of tests. As many as nine subjects could be given the test at the same time. The time was measured from the instant the sign slide appeared for each intersection-question-sign set.



a. Intersection-question slide 2.

b. Intersection-question slide 3.

c. Intersection-question slide 4.

d. Intersection-question slide 5.

Figure 13. Four intersection-question slides used in laboratory study.

Before each test began, the procedure was explained to the subjects. One preliminary intersection-question-sign set was shown to demonstrate the general type of situation to expect. Then each of the other four intersection-question slides were shown and explained. For each intersection-question slide, it was pointed out which lane was involved and where the applicable regulatory sign would be mounted. Each question was also read to the subjects. The subjects were also told that both accuracy of response and speed of response were of importance and to respond accordingly.

The subjects were given practice in the use of the "yes" and "no" buttons on the boxes so as to reduce the number of times the wrong button was pushed. The subjects were told that when the wrong button was depressed by mistake, this should be corrected immediately by pressing the correct button. They were also told, however, not to change their decisions on any particular sign and question. Thus, in the analysis, answer changes were attributed to inaccurate pushing of the buttons, unless the subject took more than 8 sec to change his answer. When more than 8 sec were taken to change an answer, it was assumed that the subject had been thinking about his answer and had decided to change it.

After the test was started, the subjects were told nothing until they had seen all but the last two intersection-question-sign sets. At this time, the meanings of the European "No Left Turn" sign and the particular test sign were explained to the subjects. Then they were shown two more intersection-question-sign sets—one with the European sign and one with the test sign. Aside from this explanation, the test administrator did nothing but change slides during the course of the test.

In each set, the intersection-question slide was shown first. After the subjects had had time (a few seconds) to look at the slide and get the situation and question clear in their minds, the sign slide was projected immediately to the left of the intersection-question slide.

Slide Sequence

Each subject viewed sequences containing only one test sign. There were four test signs, so one-quarter of the (136) subjects saw each sign. The change of test signs was the only difference in the sign sequences shown to the different groups. This permitted evaluation of the effects of repeated viewings of the same sign.

The purpose of the test was disguised by mixing the test sign slides in with slides of other, more familiar signs. This also tended to relieve possible frustration that might have resulted from the subjects' failure to comprehend any of the new sign designs used in the test. The signs that were spaced in the time sequence between double-left-turn signs were called "filler" signs. Most of these were standard signs. The European "No Left Turn" sign was also used for this purpose. Five signs of this type appeared in the test before a test sign was shown. These were called "break-in" signs and were placed in the test to allow the subject to become familiar with the procedure before seeing any of the test signs.

Precautions were taken in the design of the test to prevent the development of unwanted patterns. The number of slide sets containing filler signs was varied after different appearances of the sets containing test signs. Also, the sequence of "yes" and "no" answers were checked to see that no patterns had developed. Finally, a check was made to see that each of the intersection-question combinations that appeared frequently during the test was combined with some signs that yield "yes" answers and some that yield "no" answers.

The first, third, and fifth times the subjects saw the test signs they were asked if they could turn left from the optional lane and the second and fourth times they saw the signs they were asked if they could proceed straight through from the left lane. After the explanation, half of the subjects in each group were questioned about the left turn and the other half were asked about proceeding through. Contributing to the difficulty of the subjects' learning the test sign sequence was the variation in the number of filler sign sets placed between the sets including the test signs.

Subjects

Because the four groups of subjects were further subdivided by the method used in showing sign set 24 a and b, there were eight different sequences of intersection-question-sign sets (see Table 19). The results obtained with each of these sequences

TABLE 20
LABORATORY TEST SUBJECTS

Group	No. in Group	Avg. Age (yr)	Males (No.)	Licensed Drivers (No.)	Avg. Time with License (yr)	Familiar with Test Signs Before Test (No.)
High school students	64	15.3	34-36[1]	8	—	30
Army reservists	32	24.7	32	32	8.3	26
Navy reservists	40	24.9	40	37	8.9	34

[1] Two subjects did not indicate sex.

were compared with the results of the others, so that, as far as possible, the same conditions were maintained for each of the eight sequences. Relative differences in the results from the different sequences were examined. Hence, it was not necessary to maintain uniformity of subjects or conditions of testing within a sequence as long as the uniformity between sequences was preserved.

When subjects were being procured, eight groups of equal size were obtained from each source of subjects; however, the group size varied from source to source. The subjects seeing each sequence consisted of 5 men from the Naval Reserve, 4 men from the Army Reserve, and 8 students (boys and girls) from Evanston Township High School (Table 20).

Questionnaire

After each test was finished, the subjects were asked to fill out a questionnaire on personal data (such as age, sex, and driving experience) that might be relevant to the test. Comments on the test were also requested.

The subjects were also asked to select from test signs 1, 2, and 3, the sign they thought most clearly conveyed its intended message. Also, 96 high school seniors at Northwestern's Summer High School Institute were asked to rate these signs. The meaning of the signs was explained to them, as was the procedure of the laboratory study. They were then asked which of the three signs they thought should be selected as a national standard. All of the questionnaires were filled out by the subjects after the intended messages of the signs had been explained.

Comparison of Mean Response Times

For each of the four test signs, Table 21 gives data on time of response to the questions concerning legality of making certain maneuvers.

TABLE 21
RESPONSE TIME FOR QUESTIONS ON TEST SIGNS

Viewing	Question[1]	Sign	Sample (No.)	Response Time (sec)	
				Mean	Standard Deviation
1	A	1	34	3.74	2.55
		2	34	3.50	2.11
		3	32	4.10	2.71
		4	34	3.06	1.63
2	B	1	34	2.67	1.57
		2	34	2.86	1.60
		3	34	3.41	2.00
		4	34	2.74	1.37
3	A	1	34	2.18	1.33
		2	34	2.17	1.32
		3	34	2.54	1.37
		4	34	2.54	1.13
4	B	1	34	1.87	0.82
		2	34	2.24	1.24
		3	34	2.34	1.23
		4	34	2.24	1.09
5	A	1	29	2.14	1.04
		2	34	2.04	0.93
		3	34	1.91	1.21
		4	29	2.30	4.20
6	A	1	17	1.09	0.58
		2	17	1.29	0.51
		3	17	1.32	0.79
		4	17	1.63	1.47
	B	1	17	1.43	0.44
		2	17	1.11	0.69
		3	17	1.46	0.62
		4	17	1.18	1.54

[1] Question A, "Can you legally turn left from the lane in which you are now traveling?" for the optional lane. Question B, "Can you legally proceed straight through this intersection from the lane in which you are now traveling?" for the median lane.

TABLE 22

GROUP RESPONSES TO QUESTION ON THE EUROPEAN NO LEFT TURN SIGN AFTER ITS EXPLANATION

Group Seeing Test Sign	Sample (No.)	Response Time (sec)	
		Mean	Standard Deviation
1	26	1.51	0.97
2	26	1.35	0.77
3	34	1.55	0.76
4	34	1.71	0.86

TABLE 23

FREQUENCY OF WRONG ANSWERS TO QUESTIONS ON TEST SIGNS

Viewing	Question [1]	Sign	Sample (No.)	Wrong Answers	
				Frequency	Standard Deviation of Frequency
1	A	1	34	0.235	0.431
		2	34	0.088	0.288
		3	32	0.281	0.456
		4	34	0.147	0.359
2	B	1	34	0.294	0.459
		2	34	0.471	0.535
		3	34	0.206	0.409
		4	34	0.442	0.504
3	A	1	34	0.206	0.409
		2	34	0.178	0.383
		3	34	0.235	0.430
		4	34	0.118	0.327
4	B	1	34	0.235	0.430
		2	34	0.324	0.478
		3	34	0.088	0.288
		4	34	0.088	0.288
5	A	1	29	0.103	0.426
		2	34	0.118	0.327
		3	34	0.206	0.409
		4	29	0.069	0.262
6	A	1	17	0.000	0.000
		2	17	0.059	0.242
		3	17	0.000	0.000
		4	17	0.000	0.000
	B	1	17	0.059	0.242
		2	17	0.059	0.242
		3	17	0.059	0.242
		4	17	0.118	0.331

[1] Question A, "Can you legally turn left from the lane in which you are now traveling?" for the optional lane. Question B, "Can you legally proceed straight through this intersection from the lane in which you are now traveling?" for the median lane.

The table shows mean response times, standard deviations, and number of subjects in each of the four groups for each of their six viewings of test signs.

The group responses of the four groups to the questions on the European "No Left Turn" sign (after its explanation) are given in Table 22. Statistical analyses of these data, using the F and Student's t statistics (*2*) led to the acceptance of the hypothesis that each group was from the same population. Thus, it was possible to compare statistically the results from the test signs in Table 21.

The two-sided t' statistic (*2*) was then used to compare the data on mean response times for the different test signs. Generally, there were no significant differences between the mean response times of any of the different test signs for each test sign viewing. There was one exception to this. For the first viewing of the test signs, the group viewing test sign 3 had a significantly greater response time than the group viewing test sign 4 at the 95 percent confidence level.

Effects of Repeated Exposure or Learning

For each test sign, the change in mean response times for subsequent viewings of the sign was examined for similar conditions of lane placement and questions asked the subjects. The data are given in Table 21.

For the question, "Can you legally turn left from the lane in which you are now traveling?" referring to the optional lane, the largest decrease in mean response times occurred from the first to the third viewings and from the fifth to the sixth viewings (after explanation). A smaller decrease was found from the third to the fifth viewings.

For the question, "Can you legally proceed straight through this intersection from the lane in which you are now traveling?" large decreases in mean response time generally occurred from the second to the fourth viewings and from the fourth to the sixth viewing (after explanation).

Frequency of Incorrect Responses

Table 23 gives, for the group's viewing of each of the four test signs, the frequency of wrong answers to the two questions, for the six different viewings of the test signs. Also shown are the standard deviations and sizes of samples.

A two-sided t' test at the 95 percent confidence level was used to test the hypothesis that the mean frequencies of the incorrect responses were equal. For the first viewing of the test sign (when the question dealt with use of the optional lane), test sign 3 had a significantly higher frequency of incorrect answers than test sign 2. For this viewing, the frequencies for no other pair of signs were statistically different.

For the second and fourth viewings of the test signs (when the question dealt with mandatory turns from the median lane), test sign 2, showed a significantly higher frequency of incorrect responses than test sign 3. The high levels of incorrect responses for the second and fourth viewings are particularly critical because they concern the subjects' inability to determine from the sign that they must turn left when traveling in the left lane. From the safety standpoint, this is the sign's most important message, because drivers who fail to comply to this message will present accident hazards if they proceed through when vehicles in the adjacent lane (to the right) want to turn left.

Questionnaire Results

Subject preference for one of the alternative designs of the double-left-turn signs was recorded on each questionnaire. With the exception of the group from the High School Institute, each subject was asked to select the test sign that best conveyed to him its intended message; *i.e.*, which was the clearest, etc. The High School Institute group were asked to indicate the double-left-turn sign that they thought should be adopted as the national standard. The results of this part of the study were remarkably consistent. These results are summarized in Table 24.

TABLE 24
SIGN PREFERENCE

Group	Sample Size	No. Favoring			Percent Favoring		
		Test Sign 1	Test Sign 2	Test Sign 3	Test Sign 1	Test Sign 2	Test Sign 3
ETHS [1]	63	17	43	3	27.0	68.2	4.8
NHST [2]	94	20	67	7	21.3	71.3	7.4
Army Reserve	32	12	17	3	37.5	53.1	9.4
Navy Reserve	38	10	27	1	26.3	71.1	2.6
Total	227	59	154	14	24.0	68.0	6.0

[1] Evanston Township High School students.
[2] National High School Institute, Northwestern University, students.

In each of the four groups of subjects, test sign 2 received the highest preference of the group, test sign 1 received the second highest preference. Test sign 3 was the least preferred of the three signs in each of the cases. Test signs 1, 2, and 3 received 26, 68, and 6 percent, respectively, preference of all groups. Test sign 4 was not included in this phase of the study.

Discussion of the Laboratory Study

Relatively large differences (15 to 20 percent) in the mean response times of the groups viewing the different double-left-turn test signs generally were not statistically significant. One explanation of this was the large standard deviations of the response times which decreased the statistical sensitivity of the test.

Test signs 1 and 4 were very similar, so the results (mean response times and frequencies of wrong answers) might have been expected to be similar. However, the consistency of results with the two signs was not as great as might have been expected. Due to the similarity of these signs, the differences in the results might be attributed to chance and not to the signs. This points to the desirability of larger samples sizes in future tests of this type.

The results with repeated viewings of the same test signs indicate that the rate of learning is high for the first few viewings of a new design of a traffic sign, but then drops off rapidly. Apparently, most people become familiar with a sign after only a few viewings. However, some people may require many exposures to a new sign to comprehend its message. Even after five viewings and the explanation of the sign, there were a few subjects who still did not understand that the test signs prohibited crossing the intersection from the median lane.

The highest frequencies of incorrect answers to the question regarding the legality of proceeding through from the left lane occurred with test sign 2 which carried the word messages "Left Turn Only" and "Left or Thru." Even on the fourth viewing of the sign, the frequency of incorrect response was 0.324. In particular, the message, "Left or Thru," which applies to the optional lane, apparently would be confusing to a person driving in the left lane if this person did not know that only one of the sign's two messages referred to the left lane.

In the opinion survey to determine which of three signs (test signs 1, 2, and 3) was considered to convey best the intended messages, 68 percent of the subjects selected test sign 2, yet about 40 percent had given wrong interpretations to test sign 2 on the laboratory study.

The inference drawn from these results is that one must be careful in using the evaluations of new sign designs which are made by those who are fully aware of the meanings of the signs. Persons of this type may be unable to evaluate objectively these sign designs. Such evaluations, therefore, should be made primarily by persons who have no prior knowledge of the signs' meaning.

ANALYSIS

Effectiveness of Sign Designs

Several of the intersections selected for study (Lincoln and Peterson, State and Chicago, and Chestnut and Main) already had a high level of use of optional lanes for turns, obtained as the result of earlier use of other lane-use controls. It was not surprising, therefore, that test installations of roadside lane-use control signs did not produce significant increases in optional-lane use for turns at these intersections. Trials of five different designs of curb-mounted lane-use control signs at Lincoln and Peterson Avenues revealed practically no difference in optional-lane use with the five different sign designs (Table 10).

Overhead lane-use control signs, placed over each lane, produced significant increases in optional-lane use at Lincoln and Peterson Avenues, as compared with use of roadside signs (Table 7). Optional-lane use at State and Chicago, Elgin, however, did not increase significantly after installing overhead signs, due perhaps to the fact that a very high level of optional-lane use was already evident with roadside signs and markings (Table 13).

The T-intersection study provided an opportunity for studying optional-lane use at a location where little use was formerly made of optional lanes for turns. Turns made from the center lane of a 3-lane stem of a T-intersection increased significantly after authorizing markings and signs were installed; these optional-lane turns decreased when the authorizing markings wore off.

The effect on optional-lane use of length of time of exposure to controls was evident in many of the field studies, but could not be evaluated specifically. Significant changes in optional-lane use generally occurred only after a change in lane-use controls. In most cases, after installing the new controls, increases in optional-lane use continued as additional time elapsed, but the changes were not statistically significant.

A major finding in the T-intersection study was that pavement markings (arrows and word messages) were found to be quite effective in securing a high level of optional-lane use (Table 16). When roadside signs and pavement arrows were used (After IV), even higher use of the center lane for turns was found, especially for right turns.

Conditions Favoring Optional-Lane Use

The extent of use of an optional lane for turns was found to be a function of both physical and traffic conditions. Physical conditions found to be essential for efficient use of the optional lane for turns include the following:

1. Signal phasing that permits traffic from two lanes to turn on the same phase without conflict with traffic entering from the opposite direction. (Special phasing is usually needed only for double-left turns.)
2. Adequate number and width of lanes for exiting vehicles.
3. Adequate capacity for other movements entering from the same approach.

When these conditions were present and turning volumes were sufficiently heavy to cause delay to single-lane turning, motorists were inclined to turn from two lanes regardless of whether authorization had been provided for double turns.

At Lincoln and Peterson Avenues, physical conditions favoring optional-lane use for turns were present in both the Before and the After studies, and turning volumes were high; the result was high use of the optional lane for turns, even when no authorizing controls were present. In contrast, the Before conditions at Lincoln and Touhy Avenues provided a signal phasing that was not favorable to double turning, and the number of optional-lane turns was low. A change in signal phasing resulted in a doubling of optional-lane turns, even though no authorizing controls were used (Table 12).

Laboratory Study

In the laboratory study, the combination of time of response and accuracy of response to test signs was found to be satisfactory as criteria for evaluating driver response to the test signs. Since only 34 subjects were used for each sign, differences of 15 to 20 percent in mean response time generally were not statistically significant.

The laboratory study verified results of the field study in revealing that minor changes in designs of roadside signs did not produce significant differences in performance. However, both the field and laboratory studies revealed that violations and wrong interpretations were more prevalent with test sign 2 (Fig. 4-e), which carried both symbols and the messages, "Left Turn Only" and "Left or Thru," than with the other test signs.

Test sign 4 (Fig. 4-g), patterned after the new national standard for roadside lane-use control signs, had the lowest frequency of wrong answers for observances 3, 4, 5, and 6 combined (8.4 percent for sign 4; 12.5 percent for sign 3; and 16.9 percent for sign 2).

SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

1. Effectiveness of multi-turn controls at intersections can be evaluated by studying changes in the use of a second lane for turns. Observations of violations of the requirement that vehicles in the curb (or median) lane must turn, should be used to supplement studies of lane use, along with any available data on accident experience.

2. Pavement markings (lane lines, arrows, and word messages) can provide effective guidance of double-turning maneuvers, without use of signs. However, the requirements that vehicles in the curb or median lane must turn should also be posted by signs; otherwise, when markings are worn or otherwise obscured, drivers may cross the intersection from the mandatory-turn lane, and thus increase the possibility of accident.

3. Post-mounted signs were found to be effective in notifying drivers of double-turn controls, especially when the turns were made toward the direction corresponding to the side of the roadway where the signs were posted. However, pavement arrows and the standard word messages should always be used to supplement the post-mounted double-turn signs.

4. Overhead double-turn control signs, mounted over each lane, were also effective in giving notice of authorized or required lane uses. They are preferred to signs mounted at the roadside.

5. Effects of minor differences in the design of regulatory lane-use control signs are difficult to detect in field studies with the different signs.

6. The T-intersection provided the most favorable study location for field evaluation of different designs of lane-use controls, and the effects of time of exposure to controls. Results indicated that the use of the center lane at a 3-lane approach to a T-intersection was quite sensitive to presence or absence of lane-use controls.

7. The length of time of exposure to new controls is a variable needing further investigation. Fairly stable results were obtained in lane use within two to four weeks after placing a new lane-use control, but there were indications that driver response continued to change over long periods of time.

8. At intersections having heavy volumes of turning traffic, the drivers sampled in this study tended to make use of more than one lane for turns, regardless of whether such double-turning maneuvers were authorized by signs or markings.

9. In laboratory studies to simulate field situations controlled by lane-use control signs, minor differences in sign design generally did not result in significant differences in times of response to signs or in accuracy of responses. However, one design of a roadside sign (Fig. 4-e, with word messages, "Left Turn Only," and "Left or Thru") had a significantly poorer accuracy of response than the sign conforming to the Uniform Standard (Fig. 4-g). This same sign (Fig. 4-e) also produced the greatest number of violations in the field studies.

10. In the laboratory study, the proportion of drivers understanding the meaning of the signs increased rapidly after only a few viewings of the signs.

11. Further study should be given to the driver simulation method of laboratory study of effectiveness of different sign designs, using larger groups of subjects in the study.

12. The 1961 edition of Manual on Uniform Traffic Control Devices provides standards for control of multiple turns at intersections which should be satisfactory until refinements are developed from further research.

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DEPARTMENT OF SOILS, GEOLOGY AND FOUNDATIONS

A Strength Criterion for Repeated Loads

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Two criteria for the strength of fine-grained soils subject to repeated loads were postulated and studied. For the test conditions and soils employed, one of the proposed criteria was verified. Failure occurs when repeated stresses exceed critical values determined from slopes of deformation-number of repetitions curves. New facts concerning the stress level vs number of stress repetitions and the compaction conditions vs elastic rebound relationships suggest the need for a critical review of subgrade compaction specifications for highway and airfield pavements.

• THE STRENGTH and deformation characteristics of naturally deposited, as well as mechanically compacted, fine-grained soils under the action of gradually applied loads have been studied intensively by engineers and scientists for more than three decades. Considerable progress has been made in isolating and evaluating the factors that affect these characteristics.

During the last decade, there has been increased interest and quantity of work on the effects of repeated stresses. This interest has been caused by mounting evidence that the repeated application of stresses caused by wheel loads moving over highway and airfield pavements often affect the strength and deformation characteristics of the underlying soils in a detrimental manner, and that this effect may not be predicted satisfactorily by conventional strength tests.

Earlier investigations have been helpful in establishing marked difference in the behavior of various soils under the action of repeated loads and in partially evaluating the effects of certain factors such as degree of compaction, density, molding water content, and frequency and sequence of load applications. In view of its growing importance in soil engineering, the need for a strength criterion for soils subjected to repeated loads was recognized, and this study was undertaken in an attempt to develop such a criterion for compacted, fine-grained soils.

PREVIOUS INVESTIGATIONS

For most ferrous metals it is known that a plot of stress vs number of stress applications required to produce failure results in the familiar *s-n* diagram (Fig. 1). At a sufficiently low level of stress, a very large or infinite number of stress repetitions can be applied without causing failure of the metal. When the material is subjected to a series of completely reversed stresses, this critical level of stress is called the endurance limit, S_e. Determination of the endurance limit for ferrous metals usually requires that the sample withstand at least 10 million cycles of stress application.

For nonferrous metals, the endurance limit either does not exist or

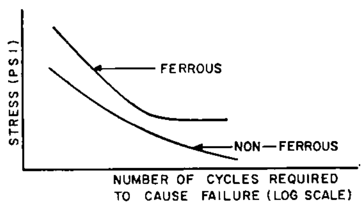


Figure 1. Typical *s-n* curves for ferrous and nonferrous metals.

is not so easily defined as in the case of ferrous metals. For example, 500,000,000 stress cycles may fail to define the endurance limit for some nonferrous metals.

The endurance limit for both ferrous and nonferrous metals has been found to be influenced by such factors as the temperature of test, speed of testing, size, shape and condition of the surface of the test specimen, medium in which the test was conducted, cold working, overstress, understress, and intrinsic stress. Further complications are added when the applied cyclic stresses are not completely reversed or where combinations of different types of stress are employed.

Repeated load or fatigue studies of such materials as portland cement concrete, asphalt paving materials, stone, and wood have been conducted and have produced a considerable amount of interesting and valuable information concerning their behavior under the action of repetitive loads. It has not been established whether these materials exhibit an endurance limit, nor have the studies led to a fundamental understanding of fatigue.

In 1943, Kersten (*1*) reported the results of both static and repeated load laboratory plate bearing tests on two statically compacted soils. Each load and unload cycle for the repeated load test required 5 min with one-half of this period devoted to each portion of the cycle. Usually no more than 300 repetitions were applied to each soil sample. Kersten's data showed that, for a given level of contact pressure, greater total penetrations were obtained under the larger bearing plates than under the smaller plates in both the static and repeated load test. For a given level of contact pressure, the initial penetration for the repeated load test was essentially the same as the corresponding penetration obtained for the static test. However, subsequent cycles of repeated loads produced increasingly larger penetrations which exceeded those obtained for the static test at all corresponding levels of applied contact pressure. For a given set of sample and test conditions, the penetration increased with larger contact pressures and with increasing number of cycles of repeated load. By selecting an arbitrary value for permissible plate penetration, Kersten was able to plot *s-n* curves (bearing value in pounds per square inch vs the number of repetitions) to show the effect of moisture content and base thickness on the shape and positioning of the *s-n* curves. These *s-n* curves resembled those for nonferrous metals, as shown in Figure 1.

Tschebotarioff and McAlpin (*2*) conducted studies of the effects of vibratory and slow repetitive loads with both controlled strain and controlled stress plunger-type loading devices. Sands, clays, and sand-clay mixtures were subjected to from 10,000 to 40,000 repetitions of vibratory and slow repetitional loads. For frequencies below the resonance range of well-graded, clean sand carrying a surcharge on the surface around the plunger, they reported that for "all densities and under all conditions of saturation, the effect on plunger penetrations of a vibratory force was several times greater than that of an equivalent static force." They found that "sand of uniform grading is particularly susceptible to the action of vibratory or slow repetitional forces. In the case of fine sand of uniform size, the continued application for 10 min of a vibratory force was found to produce deformations up to 140 times greater than those produced by a static force of equivalent magnitude" and that "the deformations produced in a soil by vibratory or by slowly repeated forces do not depend on the frequency of vibration or load repetition so long as the vibratory force itself is not magnified by resonance occurring somewhere within the vibrating system."

Buchanan and Khuri (*3*) conducted a series of repeated load triaxial tests on a lean clay soil as part of a study that dealt with soil support for rigid pavements. For one group of samples, the deviator stress was repeated while the confining pressure remained constant. For another group, both the deviator stress and confining pressure were applied and removed. Usually, no more than 40 or 50 repetitions of stress were applied to a given specimen. Their results indicated that, in the case of the lean clay, any repeated loading that exceeded the "elastic limit" caused total deformations of the soil whose "plastic" portion increased with the number of repetitions, whereas, the "elastic" portion remained fairly constant. Both the "elastic" and maximum "plastic" deformations increased with an increase in the magnitude of the applied deviator stress. However, for levels of repeated deviator stress below the soil's "elastic limit," no plastic deformations developed.

Following these earlier studies, extensive laboratory investigations were conducted by Seed and co-workers (*4* through *9*) in which compacted soil specimens were subjected to repeated applications of stress in triaxial shear. A summary of the important results of this research that are pertinent to the present study follows.

For the soils studied, and for frequencies in the range of 1 to 20 applications per min, specimen deformation depended on the number of stress applications but was independent of the frequency of applications provided the degree of saturation was not high or the soil did not possess appreciable thixotropic characteristics. For a given set of sample conditions, a given level of repeated axial load ultimately produced greater deformation than a gradually applied or static load of equal magnitude. Soil specimens may fail suddenly after having withstood a number of repeated deviator stresses of the same magnitude. The strength and stiffness of clay-like soil was increased by first applying several thousand cycles of either a repeated deviator stress of small magnitude or a repeated confining pressure. This was especially true for lower degrees of saturation even though there was no appreciable sample deformation or thixotropic effects during the load applications. For higher degrees of saturation, strength did not increase appreciably, although increased stiffness was apparent. The stiffening effect was gradually reduced as the repeated deviator stress was increased, and it was destroyed by large sample deformations. This strengthening and stiffening effect was reported to be absent or minor in sands.

After progressively increasing the repeated deviator stress on a series of identical specimens and comparing the resulting deformations with similar samples having no previous stress history, it was found that there is no simple means of evaluating the cumulative effect of a series of applications of deviator stress of different magnitudes from data concerning their individual effect. Moreover, two different soils with initial sample conditions that produced nearly identical stress-strain curves in static triaxial tests developed different total and elastic deformation in the repeated load triaxial test. It was thus apparent that for some soils, at least, the deformation characteristics, as determined by static triaxial test, were not indicative of their behavior under repeated load conditions.

Thixotropic effects were found to be an important factor in both repeated and static load tests on compacted clays having high degrees of saturation.

The complexities of the repeated stress-deformation relationships in compacted fine-grained soils suggested that rationalization of the phenomena involved might be facilitated by establishing limiting stress (rather than limiting deformation) criteria for failure under the action of repeated loads.

GENERAL HYPOTHESES

Initial efforts to obtain experimental information that would lead to a strength criterion for failure were influenced by the work of Seed, Chan, and Monismith (*4*). They had reported that a soil specimen might withstand a considerable number of repeated applications of a given deviator stress without any apparent sign of excessive deformation and then fail rather suddenly after a small number of additional applications of the same deviator stress. This indicated that an endurance strength for soil might exist that could be determined by conducting triaxial tests on a series of identical specimens subjected to increasingly higher levels of repeated deviator stress. The anticipated results from such a series of tests are shown in Figure 2. A plot of the magnitude of the applied deviator stress σ_r vs the number of stress repetitions at failure would then give the *s-n* curve as shown in Figure 3.

The belief that such a relationship might exist was strengthened by results of studies on bituminous materials conducted by Wood and Goetz (*10, 11*) and Goetz, McLaughlin, and Wood (*12*). They found that for levels of repeated axial stress equal to one-fourth of the static compressive strength, the relationship between permanent sample deformation and number of repetitions, on a logarithmic scale, was linear. For levels of repeated deviator stress equal to one-half the static compressive strength, the relationship was linear for the first few cycles but subsequently became nonlinear with the deformation increasing rapidly on the application of only a few additional stress repetitions.

At the same time, it was postulated that a stress level might exist below which no sudden increase in deformation would occur regardless of the number of stress repetitions, and that this stress level might exist even if no continuous *s-n* curve developed (Fig. 4). As the intensity of σ_r increases, it was postulated that a condition would be reached for which the deformation curve would, after the first few cycles, rise at a constant slope; *i.e.*, $d\delta/dN$ = a constant. For levels of σ_r less than this critical condition (σ_{r_4} in Fig. 4), the curves would eventually level off and $d\delta/dN$ would approach zero. For levels of σ_r in excess of the critical value, the curves

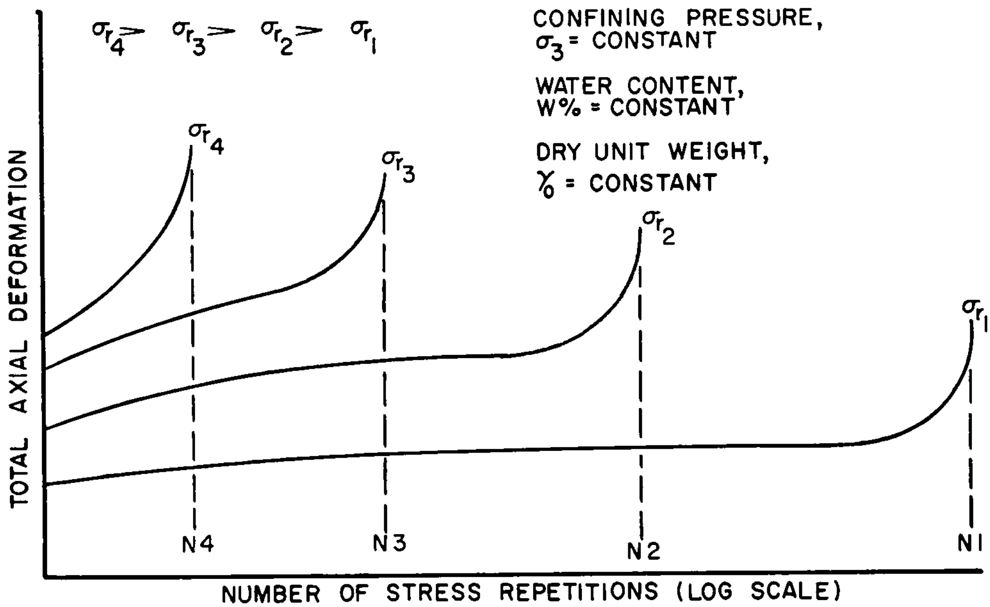


Figure 2. Hypothetical deformation vs logarithm of number of stress repetitions.

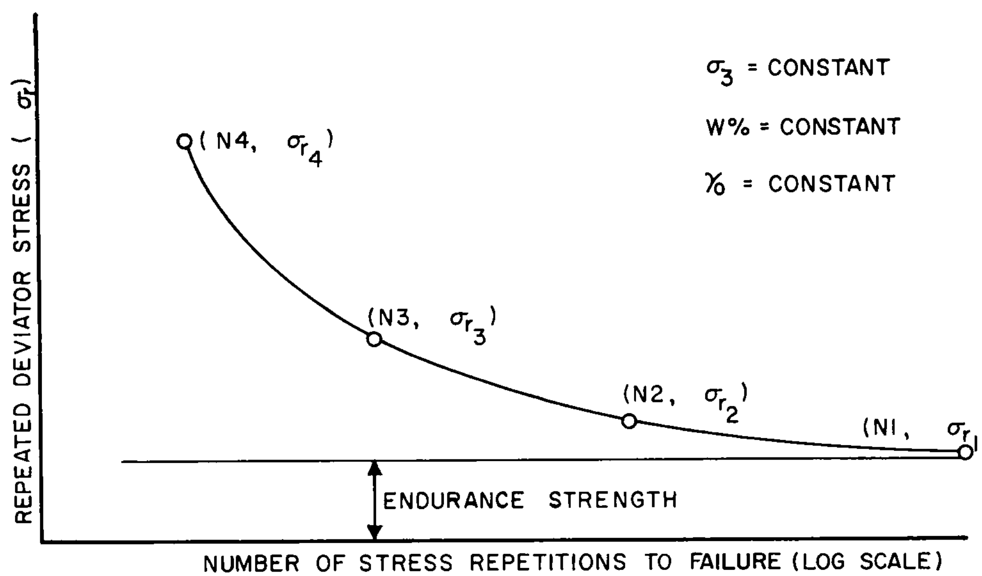


Figure 3. Hypothetical *s-n* curve for soil.

would eventually become concave upward and failure would occur either along a shear surface or by bulging.

Indirect evidence existed which indicated that the relationship shown in Figure 4 might develop for soils. Finnie and Heller (*13*) show creep strain vs time curves for two metals

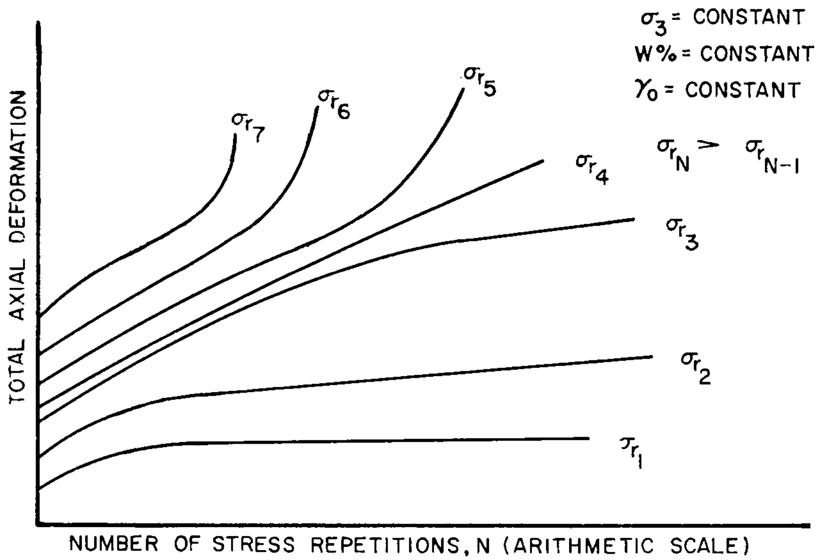


Figure 4. Hypothetical deformation vs number of repetitions.

at various levels of applied stress. These typical creep strain vs time curves, in which fracture occurs, resemble those shown in Figure 4 for which σ_r exceeds the critical stress level σ_{r4}. Moreover, Casagrande and Wilson (*14*) reported the results of creep tests on fine-grained soils in which the creep strain vs time curves for failure conditions also resembled the curves in Figure 4 at stress levels above the critical value. They stated that "failure was invariably preceded by a reversal of slope of the time-deformation curve, followed by continuous deformation at an increasing rate." Recently, after the present study was virtually completed, Vialov and Skibitsky (*15*) presented for a frozen sandy soil creep strain vs time data that have marked similarities to the Figure 4 data.

Although it was realized that creep tests may not produce the same effects as repeated load tests, the similarities were sufficient to justify further investigation. An experimental program was undertaken to check the validity of both postulated criteria.

SOILS STUDIED

Three soils were chosen for this study: a micaceous silt (Soil A) obtained in Charlottesville, Va.; a limestone residual clay (Soil B) sampled near the junction of US 11 and US 340 near Greenville, Va.; and a sand-clay (Soil C) obtained from a highway cut on Va. 639 about 6½ mi east of Ladysmith, Va.

These soils were chosen for the following reason: (a) they were typical of soils that are widespread over three or more physiographic provinces of North America and as a result are frequently encountered in engineering work; (b) the variation in physical characteristics between these three soils was sufficient to reflect a fairly broad spectrum of engineering behavior; and (c) the ease of sample preparation, due to their comparative homogeneity, and

TABLE 1
INDEX PROPERTIES AND MINERALOGICAL DATA

Soil Characteristic	Soil Type					
	Micaceous Silt (Soil A)		Limestone Residual (Soil B)		Sand-Clay (Soil C)	
Sp. gravity of solids	2.76		2.76		2.65	
Atterberg limits:						
Liquid limit	34		65		16	
Plastic limit	31		32		15	
Plasticity index	3		33		1	
Shrinkage limit	27		19		13	
Grain-size distribution:						
D_{60} (mm)	75×10^{-3}		5×10^{-3}		260×10^{-3}	
D_{10} (mm)	8×10^{-3}		—		4×10^{-3}	
D_{60}/D_{10}	9.4		—		65	
Clay Fraction	7.5		49.0		6.0	
Mineral. comp.[1]:						
Kaolinite	5.0		34		4.0	
Illite-vermiculite	1.5		10		1.0	
Hemalite	—		5		—	
Goethite	0.5		—		0.5	
Halloysite	Minor		Minor		Minor	
Calif. Bearing Ratio (soaked)	<1		5		117	
Class. unified	ML		CH-MH		SC	
Impact compaction	Stand. Proctor	Modified AASHO	Stand. Proctor	Modified AASHO	Stand. Proctor	Modified AASHO
Opt. moist. cont. (%)	16.7	12.3	29.0	22.3	8.4	6.2
Max. dry unit wt. (pcf)	105.0	118.7	89.0	104.5	127.7	140.2

[1] Approximate percent of soil fraction.

the absence of appreciable quantities of coarse particles.

Piedmont Micaceous Silt (Soil A)

Soil A is a residual soil formed from a tan-colored quartz mica schist which is a part of the Lynchburg Formation. Its pedological classification is Elioak silt-loam. It was obtained from pier excavations at depths ranging from 6 to 10 ft below the ground surface in the C-horizon. As the name implies, it contains a high percentage of fine mica particles, many of which were barely visible to the unaided eye. Soil A is a fine sandy-silt of very low plasticity. Its dry strength is low, and compacted specimens have a tendency to swell under low confining pressures.

Index properties and mineralogical data for this soil are given in Table 1. Figure 5 shows a grain-size distribution curve, and Figure 6 shows impact and static compaction curves.

Valley of Virginia Limestone Residual (Soil B)

Samples of Soil B were obtained from a depth of 1 to 2 ft below the surface and adjacent to a borrow pit that had been used in the construction of a nearby highway fill. When moist, its color is a rich red-brown and when dry, a medium red. Soil B is a highly plastic clay with a considerable fraction of silt-size particles. It has a high dry strength. In the undisturbed state, it appears to have a fragmented structure. When remolded, it is highly impervious.

The limestone rock from which it derived is a part of the Beekmantown Formation, and the soil as obtained from the pit contained a few small chert nodules which were removed by sieving. Soil B is typical of many

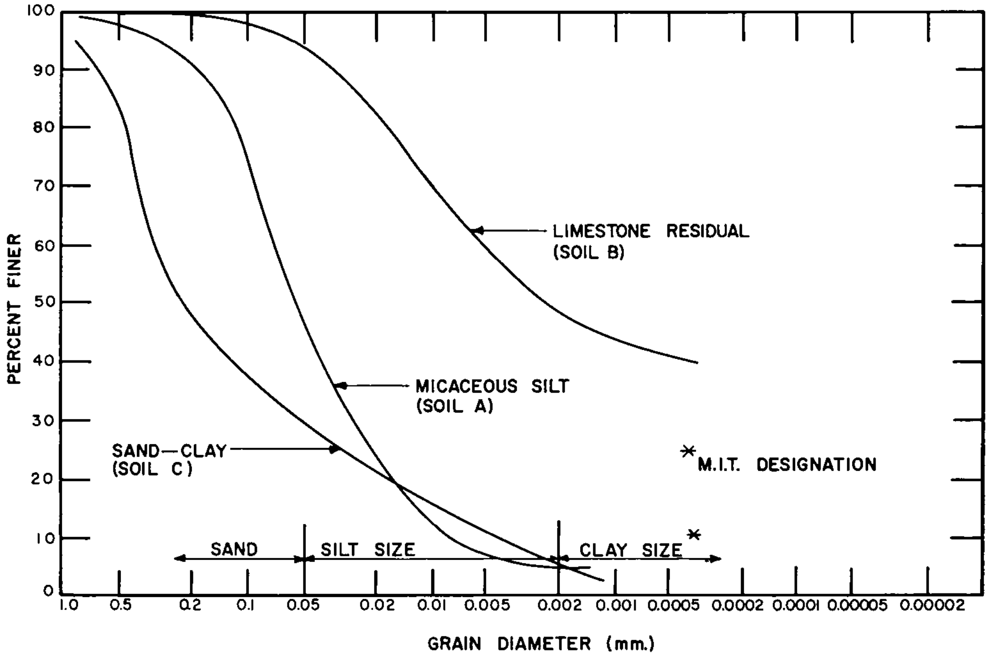


Figure 5. Grain-size distribution curves.

limestone residual soils found in the United States east of the Mississippi River. It is classified pedologically as Frederick clay-loam from the B-horizon, primarily.

Index properties and mineralogical data for this soil are given in Table 1. The grain-size distribution curve is shown in Figure 5 and impact and static compaction curves in Figure 7.

Coastal Plain Sand-Clay (Soil C)

Soil C was sampled from a depth of about 4 to 6 ft below the ground surface from what appears to be the Brandywine Terrace. In the natural state it was hard and offered considerable resistance to excavation with a hand shovel.

Soil C is a tan-colored silty sand with sufficient clay binder to impart to it moderate dry strength. The clay binder provided just enough cohesion in the moist, compacted samples to permit handling, trimming, and testing. Nevertheless, the portion of the soil that passed the No. 40 sieve is essentially nonplastic.

A small quantity of coarse sand and fine gravel present in the sample was removed by sieving.

Compacted specimens of this soil were not susceptible to appreciable swelling.

Surface soils similar to Soil C are scattered widely throughout much of the Atlantic Coastal Plain Province. They have been encountered and utilized extensively in highway and other engineering work. Pedologically it is thought to be the Sassafras soil.

Index properties and mineralogical data for Soil C are given in Table 1. A grain-size distribution curve is shown in Figure 5 and impact and static compaction curves in Figure 8.

APPARATUS AND PROCEDURE

The experimental devices and techniques employed for the preparation

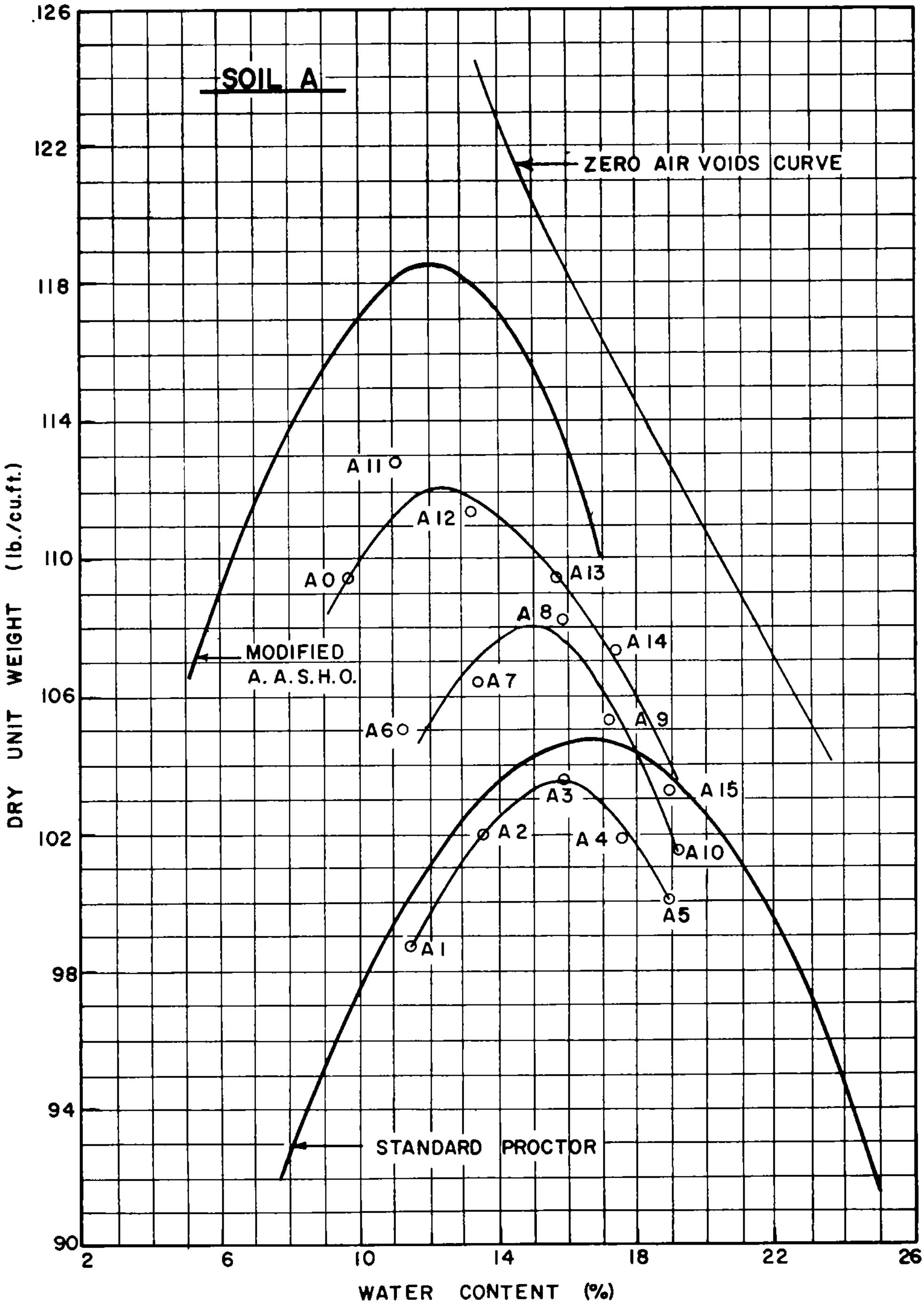


Figure 6. Compaction curves, Soil A.

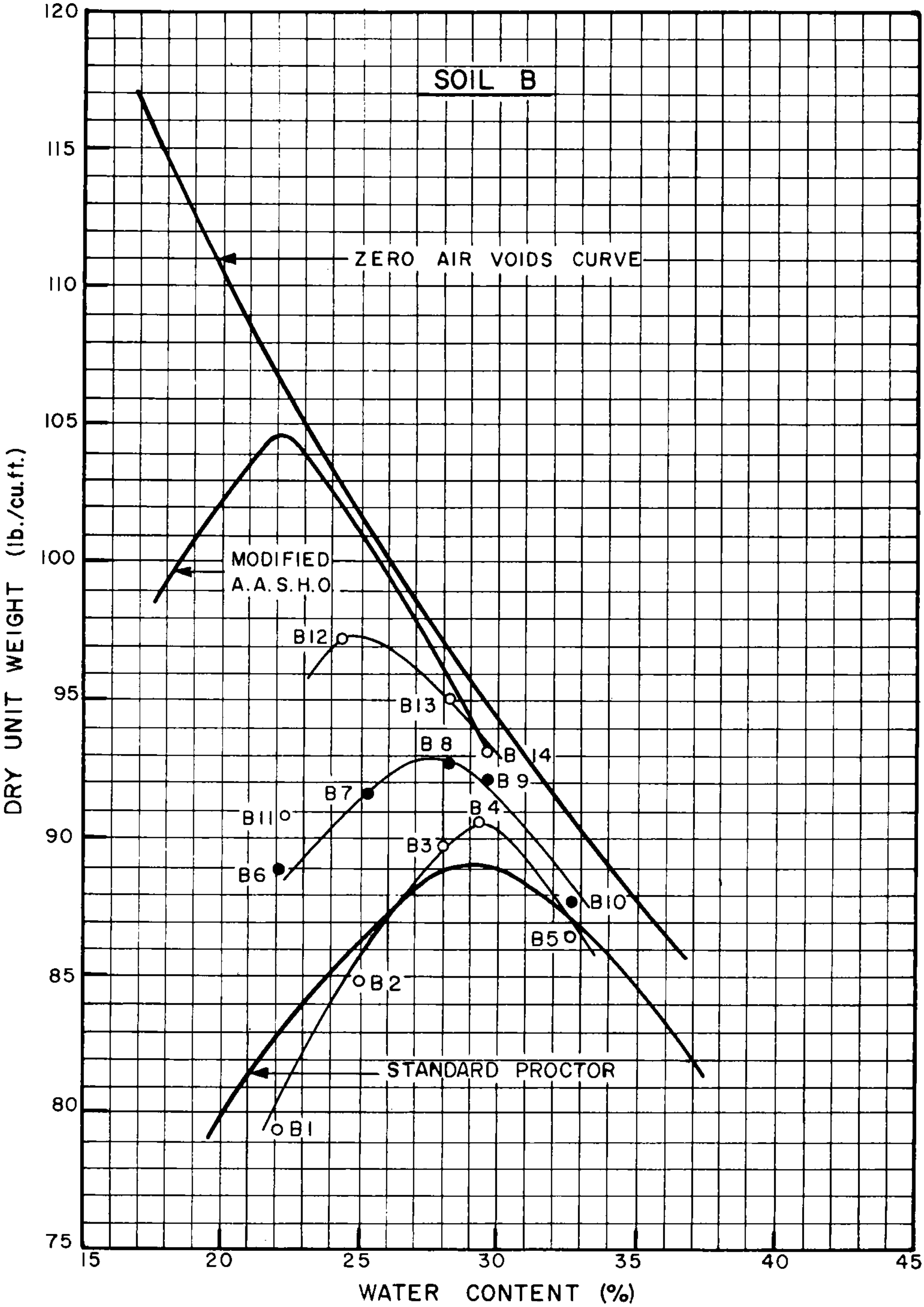


Figure 7. Compaction curves, Soil B.

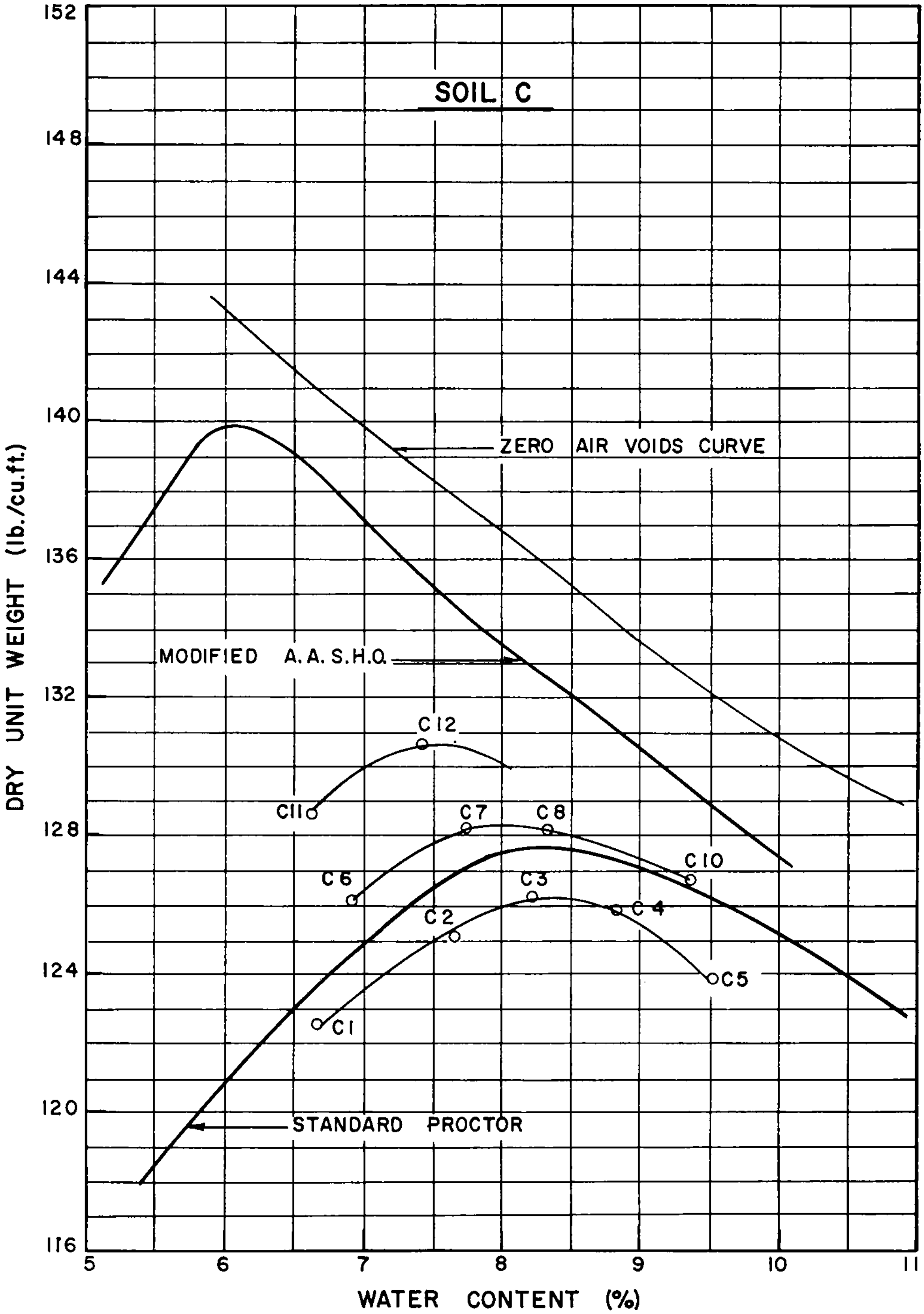


Figure 8. Compaction curves, Soil C.

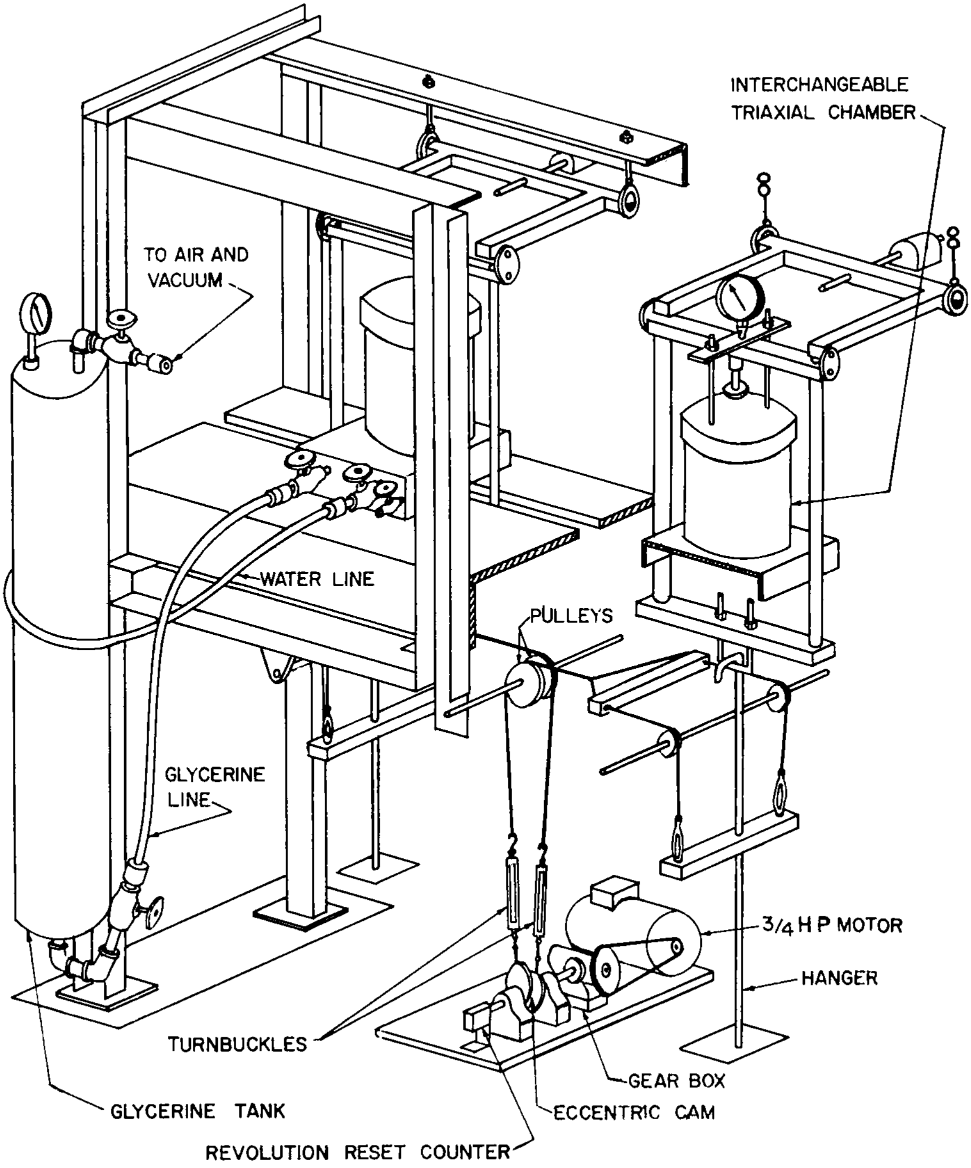


Figure 9. Repeated load triaxial device.

of raw soil, static compaction of soil cakes, cutting of duplicate specimens from soil cakes, preservation of samples, preparation of test specimens, and triaxial testing with gradually applied loads were similar in most respects to those previously employed and described by Leonards (*16*).

All triaxial test specimens were 2.80 in. long and 1.40 in. in diameter and were cut from cylindrical cakes approximately 10 in. in diameter and

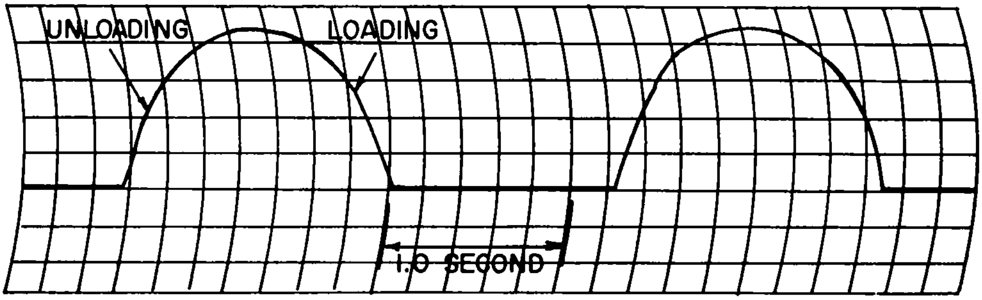


Figure 10. Loading cycle waveform.

3½ in. high that had been prepared by a static compaction process. Usually, 20 identical specimens were obtained from each of these soil cakes. From 12 to 15 of these soil cakes were prepared for each soil so that three levels of compaction and five levels of moisture content for each level of compaction were obtained for each soil. The degree of saturation of the as-compacted samples varied from 42 to 80 percent for Soil A, from 52 to 96 percent for Soil B, and from 50 to 80 percent for Soil C. The coordinates of points (such as A-2 and B-7) on the compaction curves (Figs. 6, 7, and 8) represent the dry unit weight and molding water content of each soil cake. The samples from each soil cake were stored for at least one month before being tested.

Repeated Load Triaxial Testing

The repeated load triaxial devices employed consisted of two modified double-bay triaxial testing units. The units contained necessary piping and connections for circulating water through the test specimens. Glycerin was used initially in the triaxial cell to which a confining pressure was applied from an air reservoir.

A schematic diagram of the mechanism for applying and removing the axial stress, including the cable hanger system, motor, speed-reducing pulley and gear system, pillow blocks, and cams employed in this device, is shown in Figure 9. The mechanism was designed as a compromise between convenience of operation and low initial cost. Minor repairs, such as replacing worn-out cable, were occasionally required; otherwise, the equipment was rugged and served satisfactorily. The fact that the two cams were mounted 180 deg out of phase relieved the system of much of its work, for one cam was lifting the weights while the other was lowering them, and the torques applied to the main shaft essentially canceled one another. Loads of as much as 160 kg were applied to individual specimens by this mechanism.

The frequency of load application was held essentially constant between 20 to 22 cycles per min. The form of the load-unload cycle employed was obtained with the aid of a Brush recorder (Fig. 10). The waveform can be varied by employing different shaped cams. The duration of the loading cycle can be varied by adjusting the large turnbuckle attached to the cam housing.

Some difficulty with the control of load duration was experienced in the early stages of the study. This problem was more serious for those tests in which large initial deformations developed. It resulted from the loop on the loading hanger moving downward with the deformation of the specimen while the hook on the loading rod moved up and down with respect to a fixed position. The load duration after the first cycle was, therefore, reduced until an adjustment of the large turnbuckle could be made, thereby lowering the hook on the loading rod. The problem was largely overcome by estimating the initial deformation and lowering the loop of the loading rod a corresponding amount before the start of the test. The gap between hanger and loading rod loops was set with "go" gages.

Test Procedures

The triaxial chamber containing a test specimen was filled with glycerin and the confining pressure applied. All drainage valves on the triaxial equipment were closed.

The weight comprising the axial load was then added to the hanger system, first making certain that the loading hooks were disengaged. The gear system was turned by hand until one set of the weights was raised to its highest point. Further lifting of the weights was accomplished by adjusting the large turnbuckle. Once this pair of loading hooks cleared each other, they were engaged and the weights were again lowered into position for starting the test. The gap between the two hooks was then given a final adjustment to account for the initial sample deformation. To this point no axial load had been applied to the specimen. The loading hooks for the companion specimen remained disengaged during these operations.

With the deformation dial for the first specimen and the counter set at zero, the motor was started and simultaneous readings of load application and axial deformation (to the nearest 0.001 in.), were recorded at 1, 2, 5, 10, and 20 cycles. The motor was stopped at 30 cycles and the loading hooks for the second and companion sample were positioned. The initial gap between these two hooks was set and the deformation dial gage for this specimen was set at zero. During this short interval while the motor was stopped, the axial load was applied continuously to the first specimen; however, the axial deformations observed were small and had little effect on the final results.

The motor was again started and readings of the number of load repetitions and corresponding axial deformations were recorded for both specimens. Readings were taken at predetermined intervals until failure of the specimen occurred or at least 40,000 stress cycles had been applied. Most samples were subjected to from 60,000 to 80,000 stress cycles and a few samples were subjected to over 400,000 repetitions. With few exceptions, a complete set of rebound readings was obtained for each sample. Following the completion of each test the sample was removed from the triaxial chamber for moisture content determinations.

Of the 20 identical specimens available from each soil cake, 9 as-compacted samples were subjected to repeated loads (3 samples at each of 3 levels of confining pressure). Six samples were saturated with water and then subjected to repeated loads (3 samples at each of 2 levels of confining pressure). The 5 remaining samples were subjected to conventional triaxial tests on both as-compacted and saturated samples.

For repeated load tests on soaked specimens, water was circulated upward through the specimen under a pressure equal to about one-fourth of the applied confining pressure until sample deformations had essentially ceased. Shortly before the start of repeated load applications the excess water pressure was removed and all drainage valves were closed. In all other respects the tests were similar to the as-compacted specimen tests.

A series of special tests was conducted to check the effects of thixotropic action on each of the three soils studied. Twenty identical specimens were prepared for each soil. Immediately after these samples had been prepared, 10 were subjected to a series of conventional and repeated load triaxial tests in which both as-compacted and saturated samples were employed. The 10 remaining samples in their protective wax and aluminum foil covering were stored in the humid room for two weeks and then subjected to the same series of tests.

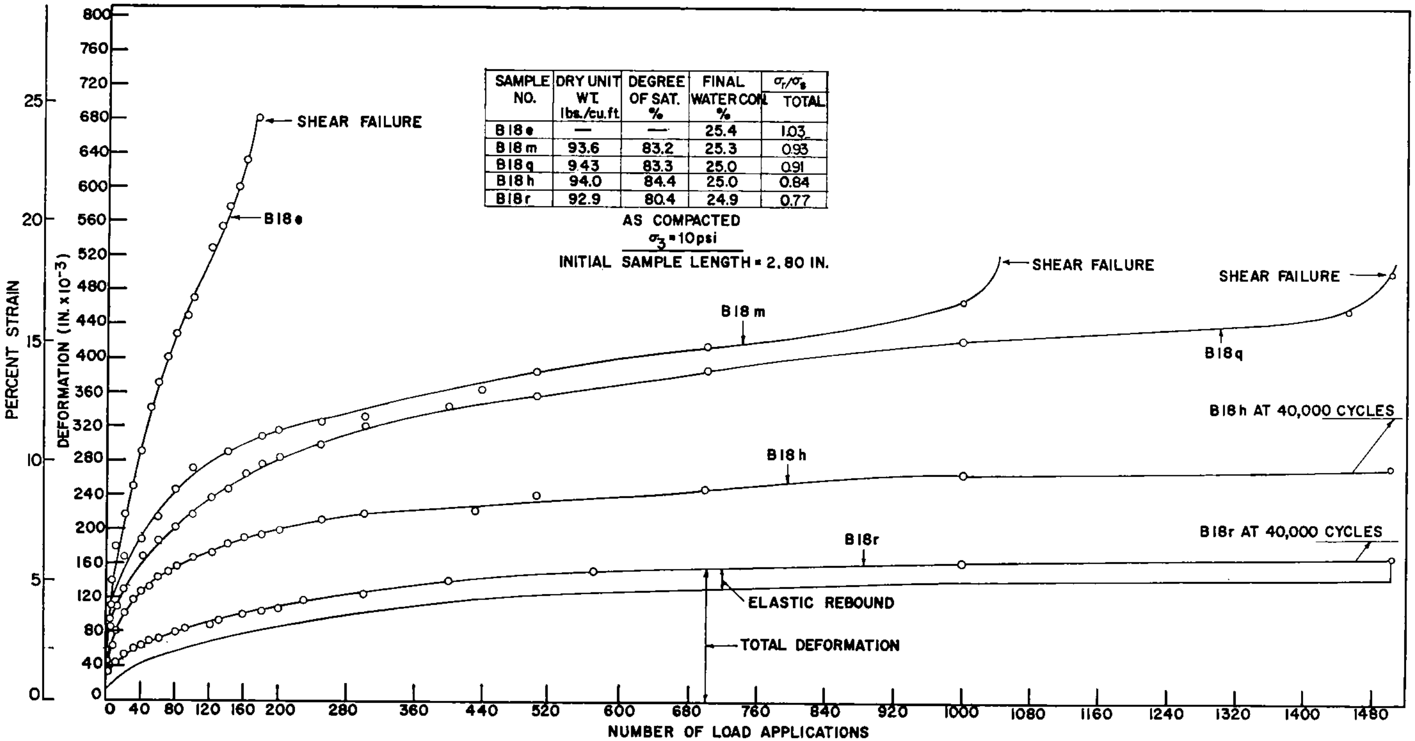
RESULTS

The test data obtained are presented in a series of eight comprehensive tables, reported elsewhere (*17*). Only a summary of the most pertinent results are included in this report. Figure 11 shows a set of deformation vs number of load repetition curves obtained for as-compacted specimens of Soil B, the limestone residual soil. Five levels of repeated deviator stress at one confining pressure are represented. The curves show that a large portion of the total deformation at any given stress level occurred during the first few hundred cycles. For samples Bl8r and Bl8h, which were subjected to lower levels of repeated deviator stress, σ_r/σ_s (ratio of the repeatedly applied deviator stress to the static deviator stress causing shear failure) equal to 0.77 and 0.84, respectively, the curves leveled out; *i.e.*, the slope $d\delta/dN$ approached zero; after several thousand cycles, the curves retained their shape even though more than 40,000 cycles were applied. For samples Bl8m and Bl8q, which were subjected to high levels of repeated deviator stress, the slopes of the curves were greater than for specimens Bl8h and Bl8r. Furthermore, after 950 and 1,500 repetitions, respectively, both Bl8m and Bl8q failed suddenly during the application of only a few additional load repetitions. Sample Bl8e was subjected to a still higher level of repeated deviator stress, $\sigma_r/\sigma_s = 1.03$. This sample developed a slip plane after 120 stress repetitions. The elastic rebound for sample Bl8r is also shown. At this low stress level, the rebound reaches an equilibrium value after a small number of stress repetitions has been applied. The ratio, σ_{rc}/σ_s, representing the postulated failure criterion is between 0.84 and 0.91.

A typical set of deformation vs number of load applications curves for the sand-clay, Soil C, is shown in Figure 12. The curve for specimen $C9^2x$ closely approximates the failure condition which was postulated earlier. The slope was essentially constant after the first few cycles. For levels of deviator stress less than the critical value, the curves leveled out; and for levels in excess of the critical value, a shear failure developed.

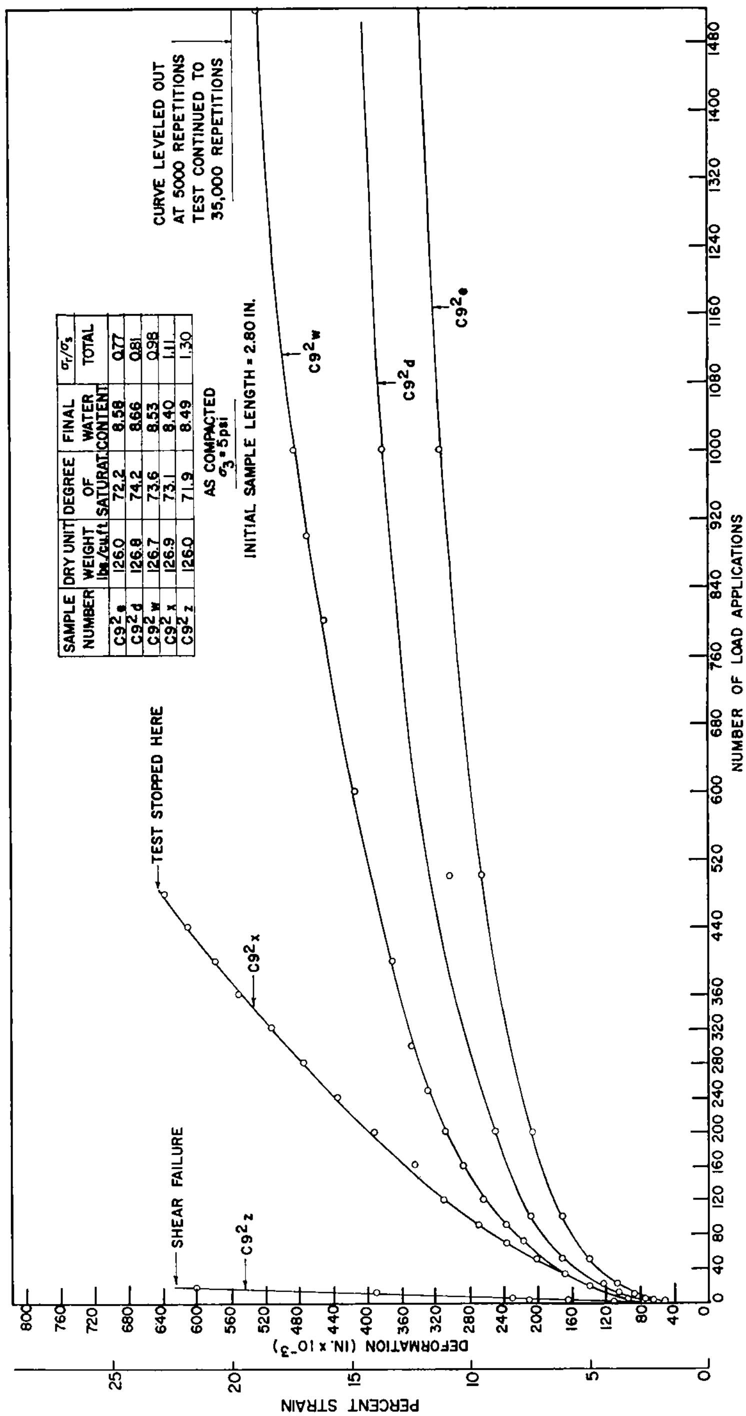
Although slight variations between specimens and slight differences in load duration made the exact determination of the critical ratio of σ_r/σ_s difficult, it was usually possible to bracket this condition closely and consistently for a given set of initial sample conditions with no more than three test specimens. By interpolation, a reliable measure of the critical ratio was obtained for both as-compacted and soaked specimens of Soils B and C at various levels of dry unit weight, water content, and confining pressure.

Curves representing the variation of the critical ratio of σ_r/σ_s with water content at two levels of compactive effort and confining pressure are shown for Soil B in Figure 13. Both soaked and as-compacted samples are represented by these curves. Average final water contents were used to plot the as-compacted curves. For corresponding initial conditions these same water contents were employed to plot the curves for soaked specimens. Values of the average final water content of soaked specimens are shown in parentheses beside each



SAMPLE NO.	DRY UNIT WT. lbs./cu.ft	DEGREE OF SAT. %	FINAL WATER CON. %	σ_r/σ_s TOTAL
B18e	—	—	25.4	1.03
B18m	93.6	83.2	25.3	0.93
B18q	94.3	83.3	25.0	0.91
B18h	94.0	84.4	25.0	0.84
B18r	92.9	80.4	24.9	0.77

Figure 11. Deformation vs number of load applications, Soil B.



SAMPLE NUMBER	DRY UNIT WEIGHT lbs./cu.ft.	DEGREE OF SATURAT.	FINAL WATER CONTENT	σ_1/σ_3 TOTAL
C9²e	126.0	72.2	8.58	0.77
C9²d	126.8	74.2	8.66	0.81
C9²w	126.7	73.6	8.53	0.98
C9²x	126.9	73.1	8.40	1.11
C9²z	126.0	71.9	8.49	1.30

Figure 12. Deformation vs number of load applications, Soil C.

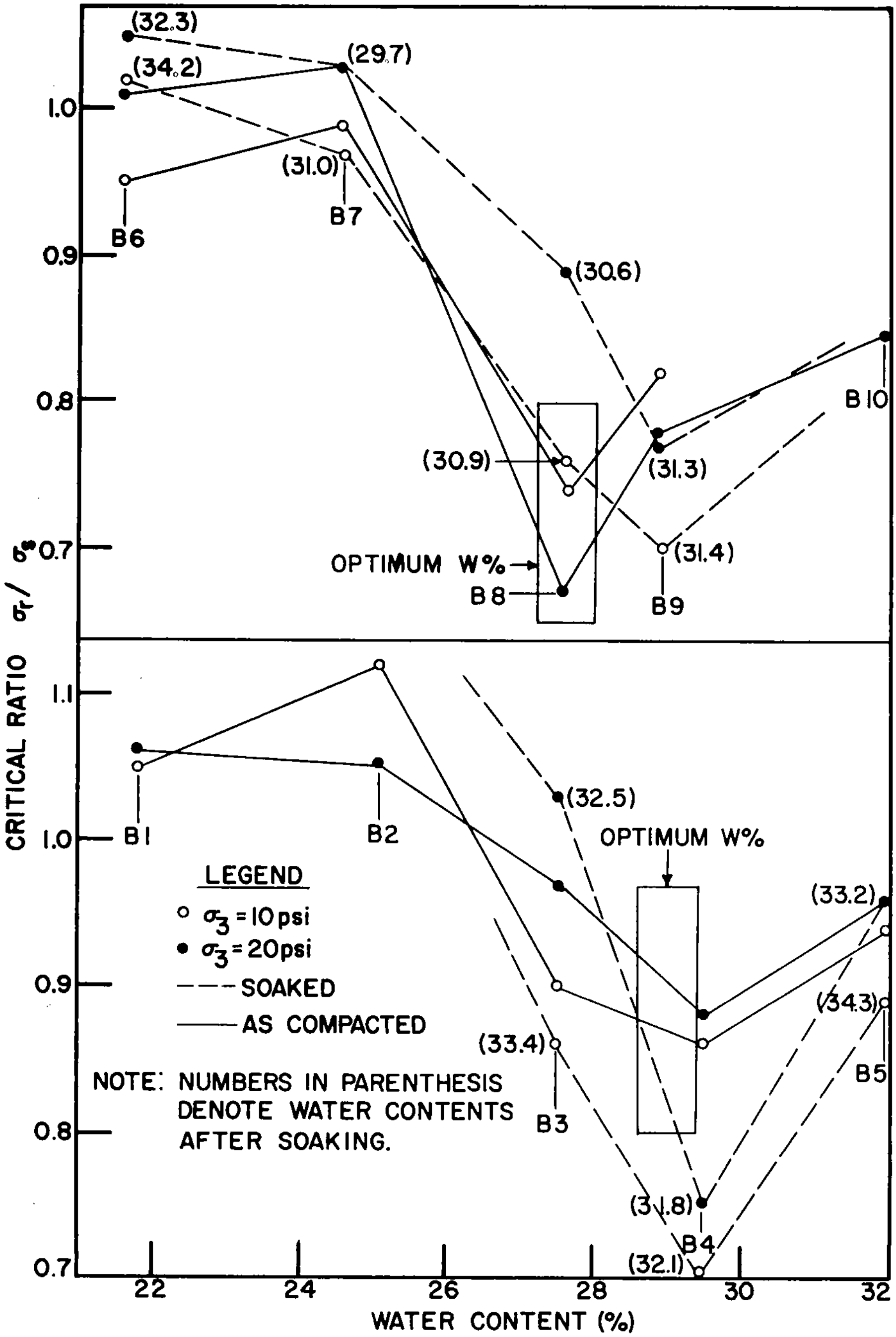


Figure 13. Critical ratio, σ_r/σ_s vs water content, Soil B.

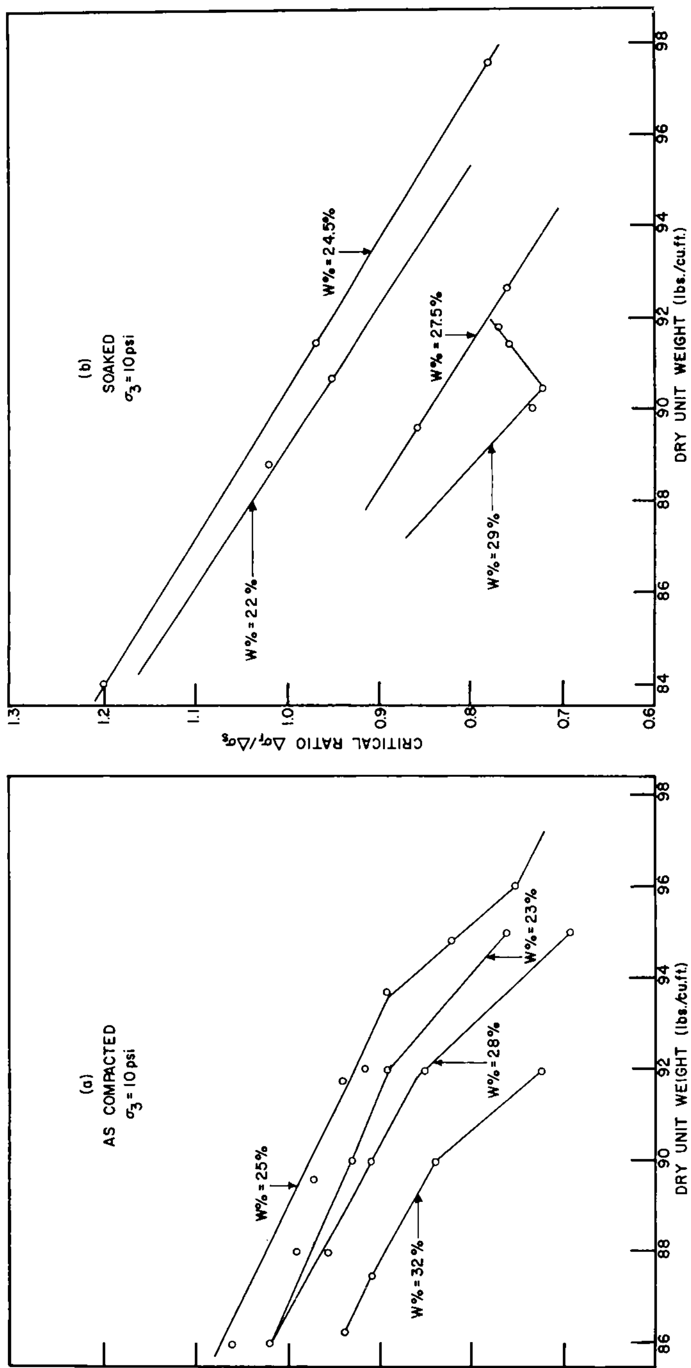


Figure 14. Critical ratio, σ_r/σ_s vs dry unit weight at constant water content, Soil B.

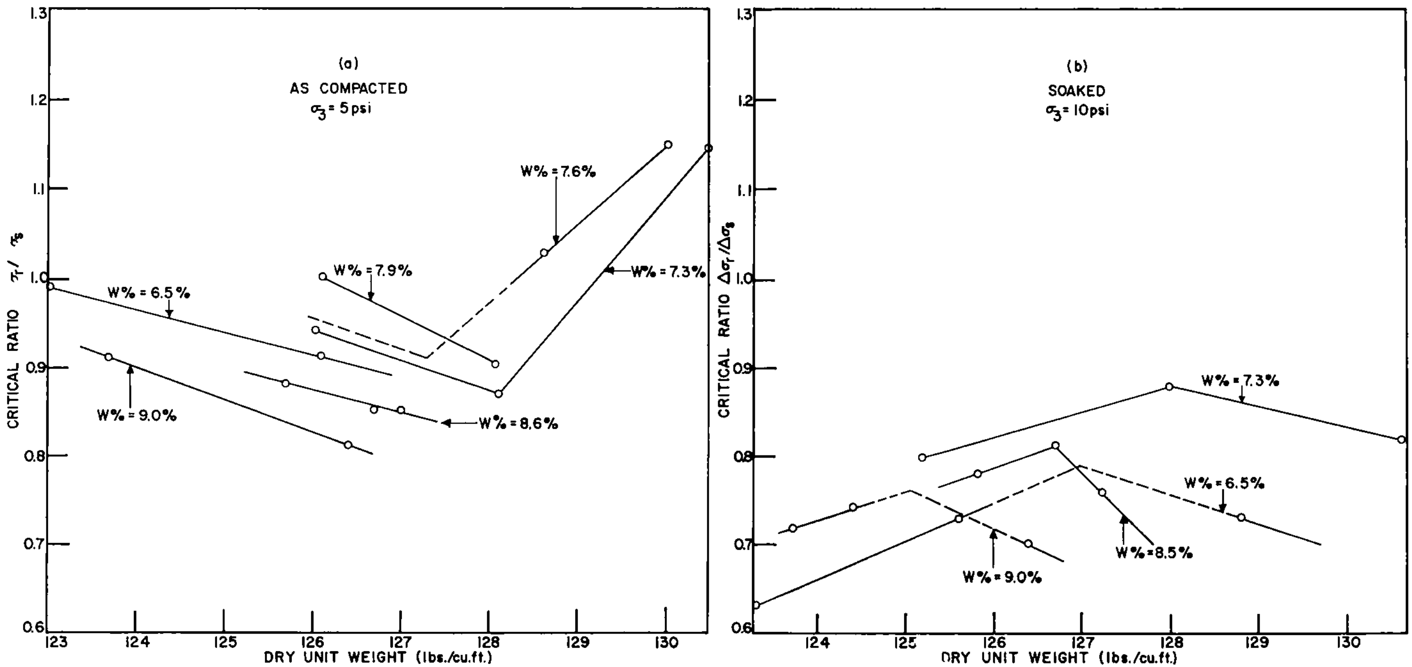


Figure 15. Critical ratio, σ_r/σ_s vs dry unit weight at constant water content, Soil C.

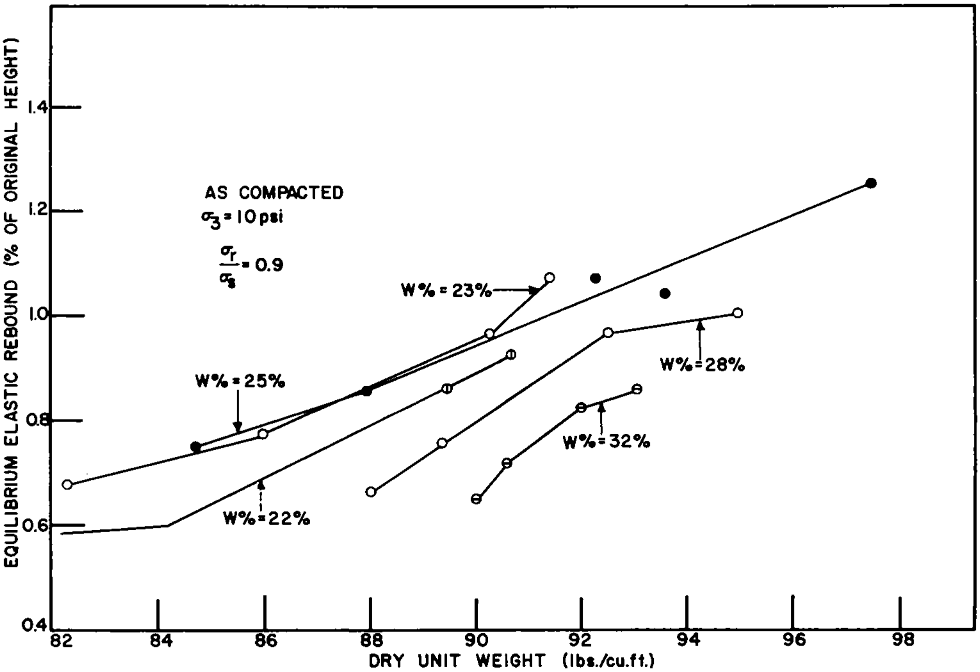


Figure 16. Elastic rebound vs dry unit weight at constant water content, Soil B.

plotted point. It is significant that the critical ratio of σ_r/σ_s drops considerably in the water content range near the optimum for the compactive effort used.

Curves illustrating the relationship between the critical value of σ_r/σ_s and dry unit weight at constant water content for both soaked and as-compacted specimens and one level of confining pressure are shown for Soils B and C in Figures 14 and 15. At any given water content, the critical ratio of σ_r/σ_s drops significantly as the compactive effort is increased in the case of Soil B. No such pattern is observed for Soil C.

An essentially complete set of rebound readings was obtained for all samples subjected to repeated stress applications. For the three soils tested and for any given set of initial conditions, the elastic rebound eventually reached a constant or equilibrium value provided the stress level was less than the strength under repeated loading. An interesting relationship between the equilibrium elastic rebound and dry unit weight (at constant moisture content) developed in the case of the limestone residual (Soil B). This is shown in Figure 16 for a single level of confining pressure and indicates that for Soil B the equilibrium elastic rebound increased with increasing dry weight at constant water content. Moreover, for a given compactive effort, the equilibrium elastic rebound reached its maximum value, for Soil B, at or near optimum moisture content. This is shown in Figure 17 for three levels of compactive effort and two levels of confining pressure. For Soil A, the micaceous silt, the equilibrium elastic rebound was not strongly influenced by either

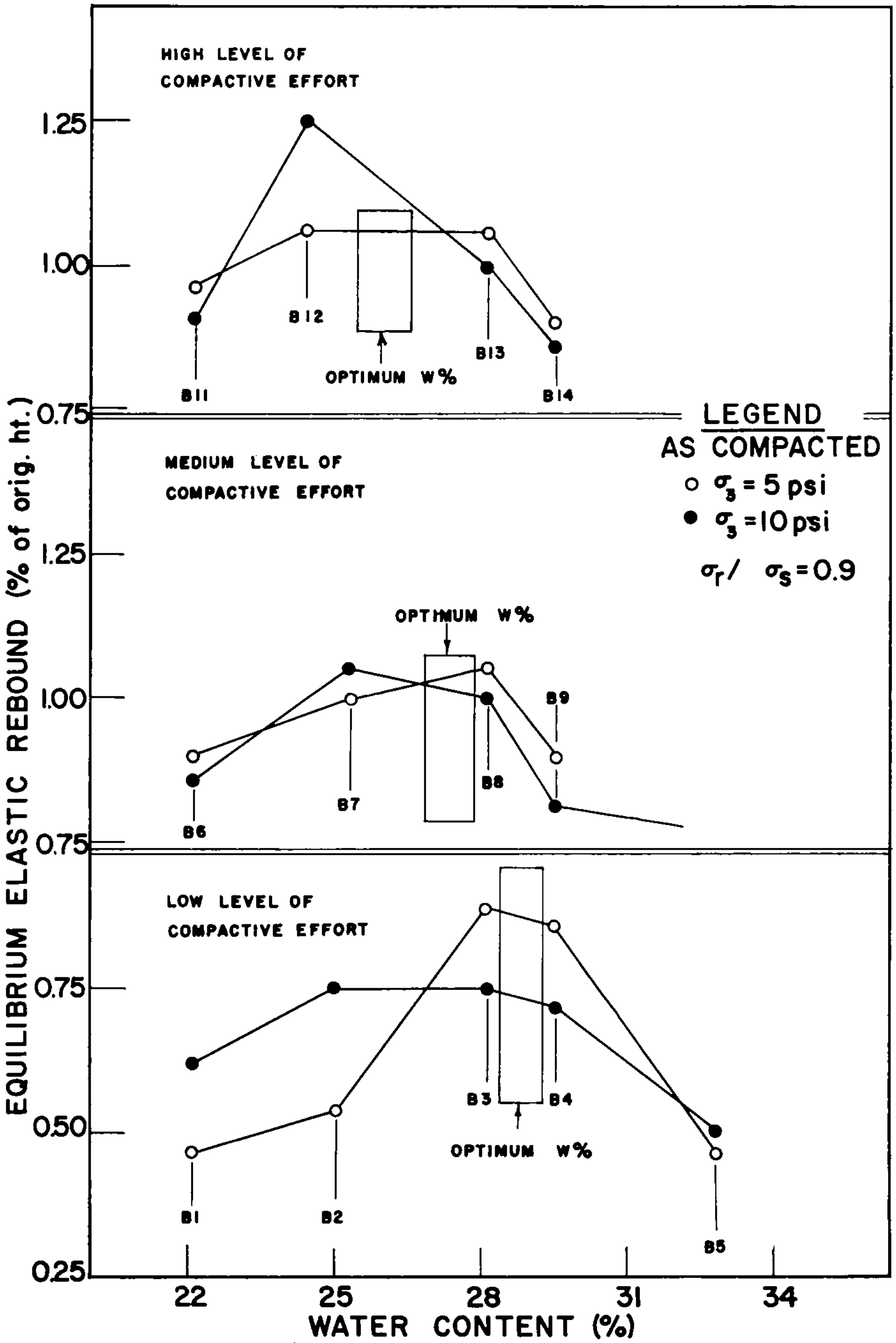


Figure 17. Equilibrium elastic deformation vs molding water content, Soil B.

dry unit weight, degree of compaction, or water content. It was, however, influenced by both confining pressure σ_3 and the ratio of repeated deviator stress to conventional or static deviator stress σ_r/σ_s as shown in Figure 18.

Special tests were conducted to determine the effects of thixotropy on the three soils employed. For the soils studied, for the degrees of saturation obtained by the compaction process employed, and for the period of sample aging used, no measurable thixotropic effects were observed.

During the course of the experimental phase of this study it became apparent that some samples were losing moisture during either the period of sample preparation or that of testing. A study conducted to determine the cause of this drying showed little, if any, migration of moisture between the ends and center of the specimen. However, a radial migration of moisture was noted; *i.e.*, the outer portions of the sample were drier than the interior. Only those specimens that remained in the triaxial chamber more than 24 hr (30,000 load repetitions) gave evidence of this radial drying. It was concluded that the glycerin, which has an affinity for water and was employed as the liquid confining medium, was the cause of this difficulty even though double membranes separated by a silicone grease were used. Distilled water was subsequently employed as the confining medium for repeated load tests; for the times involved in this testing program, no appreciable further changes in water content were noted.

The data obtained using glycerine as the confining medium showed that in all but a few instances the critical value of σ_r/σ_s was determined on samples that were not subject to the effects of this drying. That is, the pattern of specimen deformation or failure was sufficiently established in most instances well before 30,000 cycles had been applied.

ANALYSIS OF RESULTS

The performance of a highway pavement cannot be assessed solely from the behavior of a single component. Failure may occur from a lack of stability in the wearing course, from deflections in the base course due to traffic compaction, from cumulative shear deformations in the subgrade, and from temporary rebound in both the subgrade and base courses. Furthermore, the pattern of deflections produced, which is influenced by the intensity, shape, and position of the contact stresses and by the speed at which the vehicle traverses the pavement, is of critical importance in determining whether the pavement can withstand the associated deflections and rebound conditions. Nevertheless, from the standpoint of subgrade behavior, the implications of the results previously discussed are clear. These are shown in Figure 19.

At a low stress level (Curve A, Fig. 19), the deflection pattern will be too small to cause distress in the pavement. Furthermore, after a few thousand load repetitions, the additional cumulative deflections will be small and the subgrade will behave essentially as an elastic system. Thus, mathematical models that replace the subgrade by an elastic system could predict its behavior with satisfactory precision. In many cases, the required thickness of a flexible pavement to achieve this condition for heavy truck loading is considerably greater than those obtained from the design procedures used in this country. In Europe, the practice of constructing semirigid base courses (9 to 12 in. of soil cement, bituminous

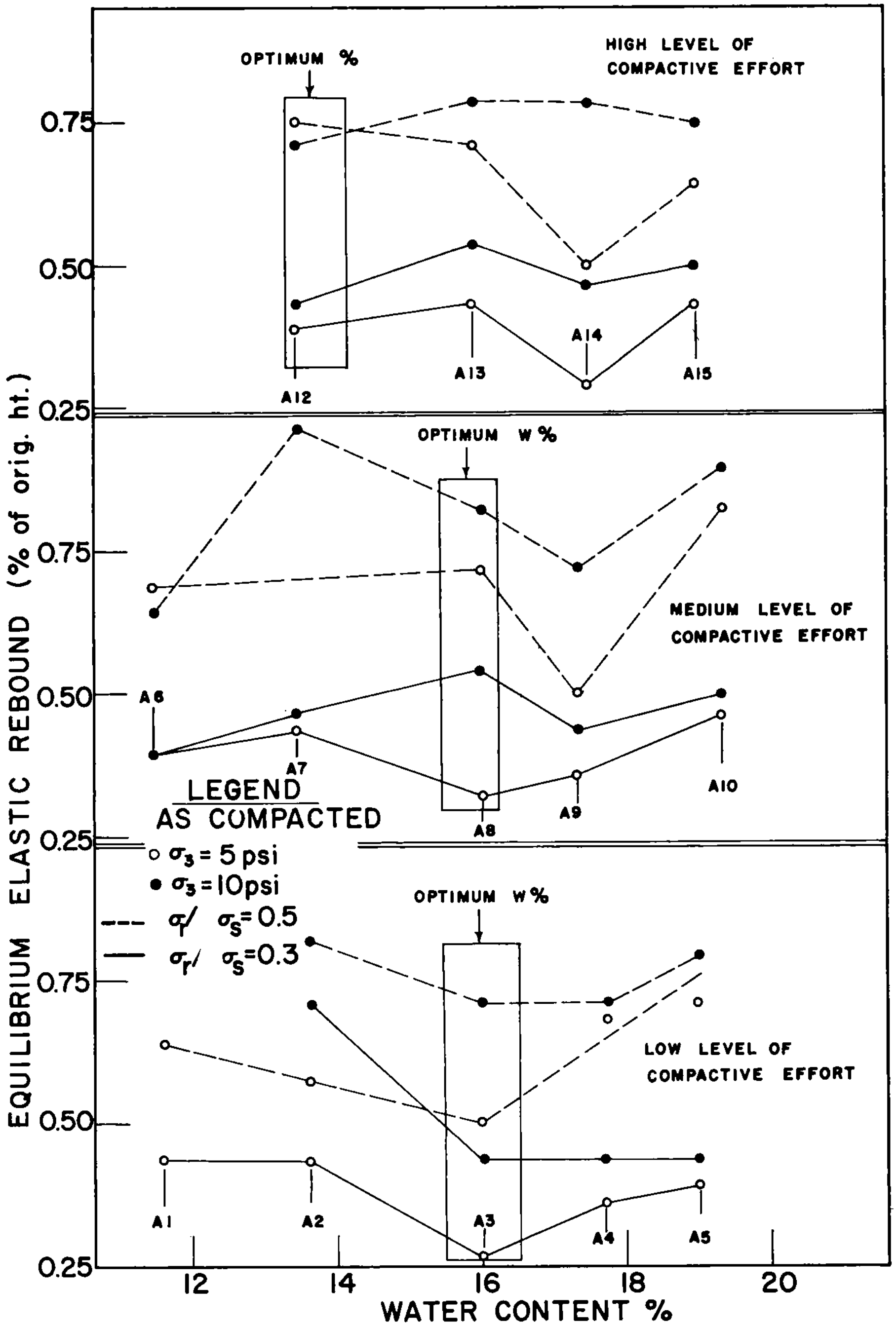


Figure 18. Equilibrium elastic deformation vs molding water content, Soil A.

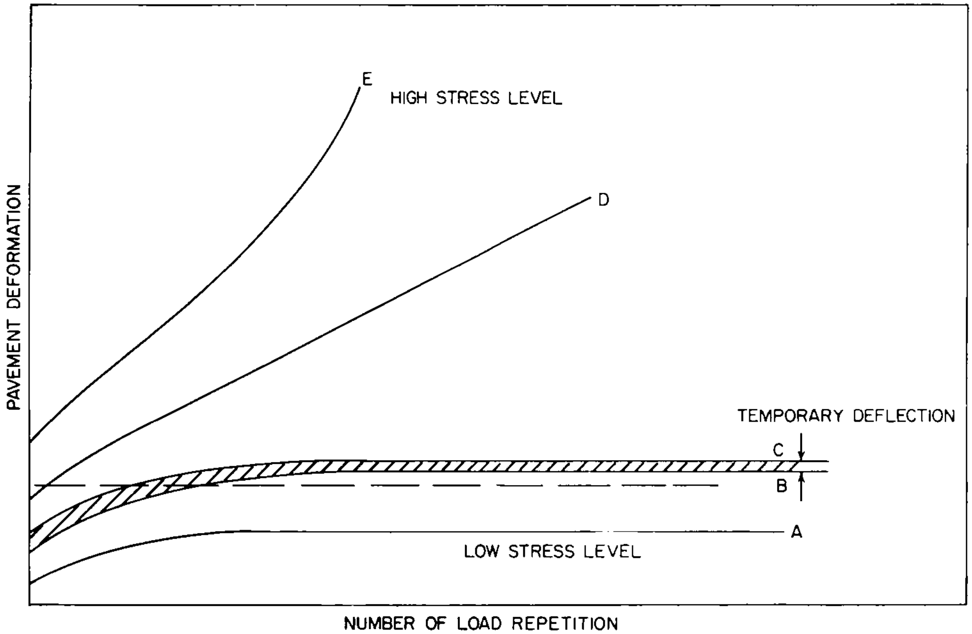


Figure 19. Schematic representation of the effect of repeated loads on the deformation pattern of flexible pavements.

bound macadam, etc.) for flexible pavements on main roads is gaining increasing favor (*18*). The semirigid base reduces the stress in the subgrade to a level shown by Curve A without necessitating the use of excessively thick pavements.

If the stress level is increased (Curve C, Fig. 19), the cumulative deflection pattern may exceed that required to cause distress in the pavement (represented by the horizontal dashed line). Nevertheless, modest repairs will be effective, as the additional cumulative deflections will be tolerable provided the temporary deflection (Curve C minus Curve B) is not large enough to cause a fatigue failure in the wearing course. Thus, if the moisture-density conditions of the subgrade (and of the base course as well, in some instances) are adversely selected to result in high temporary deflections, a pavement that might otherwise have proved adequate will experience fatigue failure. In many cases—particularly for secondary roads—it may be more economical to design the pavement with the intent of carrying out minor repairs rather than make it thick enough initially to avoid completely any signs of distress.

If the stress level is high enough to result in the condition represented by by Curve D (failure criterion), even extensive maintenance will not result in a satisfactory pavement as additional load repetitions develop a continuously increasing deflection pattern at a rate that is practically unabated. The use of elastic theory to assess this condition is entirely unrealistic; a suitable viscoelastic model may be helpful. Furthermore, the stress level in the subgrade is still below its strength as measured by any procedure applying a continuously increasing load to failure, such as the widely used CBR and triaxial tests.

If the stress level is further increased (Curve E), the pavement will rapidly deteriorate and complete replacement will be necessary soon after construction. The stress in the subgrade will be approximated by the failure condition in a triaxial or CBR test; however, a suitable factor of safety is usually applied, based on empirical correlations. As pointed out earlier, because the cumulative deflection pattern is not necessarily reflected by the static stress-strain curve, design charts based on such empirical correlations are inherently incapable of assessing the effect of repeated load and rebound conditions. Thus, they can do no better than be correct some of the time, be too conservative at other times, and occasionally result in failures.

CONCLUSIONS

For the soils studied and the test conditions employed, the following criterion of failure for compacted fine-grained soils acted on by repeated loads of constant magnitude has been established.

A critical level of repeated deviator stress, σ_{rc}, exists at which the slope of the deformation vs number of repetitions curve is constant after the first few load applications. For levels of deviator stress in excess of this critical value, the deformation curves eventually turn concave upward, their slopes continue to increase until failure occurs either by sliding along a shear plane or by excessive bulging. For levels of deviator stress less than the critical value, the deformation curves eventually approach a horizontal asymptote. It is proposed that σ_{rc} be termed the "strength" of compacted clay subject to repeated loading.

The ratio of σ_{rc} to the deviator stress at failure in a conventional triaxial test, σ_{rc}/σ_s, may be taken as a measure of the strength reduction due to the effects of repeated loads. For highly plastic clays, there are strong indications that this ratio is a minimum at or near optimum moisture content at any given level of compactive effort, and that it decreases significantly with increasing compactive effort. For such clays the equilibrium elastic rebound is also greatly increased as the compactive effort is increased. These facts warrant reexamination of current compaction specifications for highway and airfield pavements.

Design charts for flexible pavements are usually based on strength tests in which a continuously increasing load is applied to failure—modified by empirical correlations. Such charts are inherently incapable of assessing the effect of repeated load and rebound conditions on the performance of flexible pavements unless the stress level in the subgrade is sufficiently small to result in essentially "elastic" conditions.

ACKNOWLEDGMENTS

Thanks are due K. B. Woods, Purdue University; L. R. Quarles; C. N. Gaylord, and O. R. Harris, University of Virginia, for administrative assistance and encouragement in the conduct of this investigation.

G. G. Fornes and G. E. Baber, University of Virginia, assisted with the design and construction of the repeated load triaxial device.

Mineralogical determinations were made in the Bureau of Public Roads Laboratories, Washington, D. C., through the courtesy of P. C. Smith.

The authors gratefully acknowledge the financial assistance provided by the National Science Foundation, which made this study possible.

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Correlation of Load Bearing Tests on Soils

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Various load bearing test techniques are analyzed, both theoretically and experimentally, in an attempt to integrate the effects of the many variables that influence the bearing capacity and load-deformation characteristics of soils. Considerable use is made of the methods of dimensional analysis. The physical variables considered include the settlement or penetration of the loading plate or piston, applied force, size and shape of the plate or piston, method of testing, time of loading, number of load applications, and the properties of the soil being tested. Correlation between the load-deflection relations for load bearing tests and the soil stress-strain curves obtained from triaxial and unconfined compression tests are given. Laboratory and field data obtained from various locations throughout North America are analyzed and interpreted. Attention is given to the correlation of rigid plate bearing tests and the California bearing ratio test.

• A PARTICULAR FIELD of soil mechanics and highway engineering which has been a continual source of interest to numerous investigators is the determination of the bearing capacity of soils from load bearing plate tests. The problem of collecting, analyzing, correlating, and interpreting rigid bearing plate test data has long been open to much discussion and has been the motivation for numerous articles and several symposia in recent years. A satisfactory quantitative measure of the bearing capacity of soil masses has assumed increased significance in modern practical engineering.

The variables involved in this study are the size and shape of the bearing plate or piston, force applied, settlement or penetration, the number of load repetitions, time of loading, and characteristic strength and viscosity parameters of the soil. The data analyzed were obtained from various laboratory and field locations throughout North America. The variables involved are subjected to a theoretical analysis to illustrate a rational approach to this complex problem and the laboratory and field results are employed to demonstrate the feasibility of such an approach. Finally, these results are interpreted to form a rational basis for the correlation of such data.

The initial problem encountered in the analysis of such a wide range of available test data is the extreme diversification of current load test procedures. One summary of a number of procedures is presented by Housel (*1*). It is not the intent of this paper to criticize these procedures but simply to comment in general on the variables involved in each and their influencing effects on the soil response as considered from a visco-elastic viewpoint. The difficulties of including these wide variations in the test procedure in any systematic analysis become immediately evident. Furthermore, any attempt to bring the bearing capacity problem within the scope of laboratory analysis introduces additional complications. As stated by Terzaghi (*2*), an effort to solve a problem of this kind is "an attempt to uncover the responsible factors by isolating the variables and systematically determining their relative importance."

In an attempt to bring all these variables within the realm of realistic analysis, the authors have used the techniques of dimensional analysis. Such an approach enhances the interrelated transformations between laboratory models, field tests, and prototype response.

A second important element which has been found wanting in a large percentage of the literature is the inclusion of a representative soil strength parameter. Numerous attempts have been made to correlate tests on a wide range of soil types and consistencies and yet there is no measure of soil strength or consistency included. This factor alone magnifies considerably the problem involved in any attempt to correlate logically the various test procedures in which a soil strength parameter is certainly significant. Also along this same line, it appears that most tests are concluded too soon; large deformations are not obtained and it would certainly be desirable to attain the ultimate bearing capacity before the test is terminated. It is the opinion of the authors that a strength parameter for each soil is "built into" the shape of each curve obtained; that is, the shape of the curve will be dictated by the strength characteristics of the soil.

Another problem that has come through an examination of the literature is the inability of some authors to reproduce test results both in the laboratory and in the field, and the frequent incompatibility of a number of the results obtained. An example of the former is given in Figure 1 which shows the stress-strain plot for three samples of the same silty clay when subjected to a laboratory triaxial test. The maximum stress obtained by Sample 1 is about twice that obtained by Sample 3. To illustrate the second case, Figure 2 shows two similar bearing plate tests which were performed on two cohesive soils having the same uniaxial stress-strain response; yet the one curve yields plate pressures approximately three times the other for comparable deflections. It is evident that information of this type can never be reconciled in any analysis.

This paper extracts data from numerous sources in the literature, disregarding the obvious cases already mentioned, rearranges this data into a more workable form by employing principles of dimensional analysis, and then proceeds to analyze systematically the results of various tests, including strength parameters deduced from the nature of the curve. These results are then interpreted and explained at length with the ultimate result being a strong foundation for an apparent correlation between several laboratory and field testing procedures.

THEORETICAL ANALYSIS

The methods of dimensional analysis have been very successful tools in the fields of hydraulics and fluid mechanics for a number of years, but they have not been extensively used in the fields of soil mechanics and highway engineering. The senior author has been applying nondimensional techniques to a variety of problems in soil mechanics (*3* through *9*) and has found these methods to be very valuable research tools for such an experimental field. Because of the complex viscoelastic properties of soils and the complicated interaction of soil-solid systems, it is felt that the use of nondimensional techniques

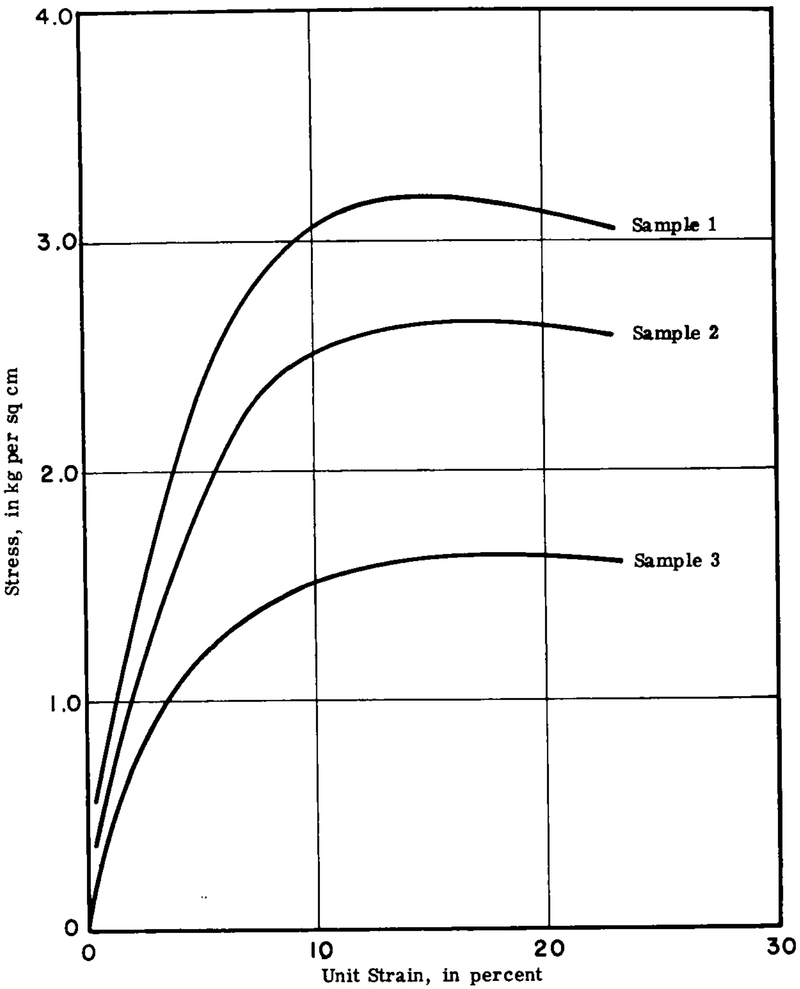

Figure 1. Stress-strain relationship, triaxial compression tests.

in both model and prototype research investigations offers definite advantages with regard to the cost, number of variables and conditions that can be studied, and time for completion of such studies.

Dimensional analysis offers a simple way to formulate a description, in functional form, of a physical phenomenon in terms of a finite number of physical quantities. Such methods, as used to formulate relationships among physical quantities, can be briefly summarized. If there are m physical quantities containing n fundamental units, which can be related by an equation, then there are $(m-n)$, and only $(m-n)$, independent, nondimensional parameters, called π-terms, such that the π-terms are arguments of some indeterminate, homogeneous function κ:

$$\kappa(\pi_1, \pi_2, \pi_3, \ldots, \pi_{m-n}) = 0 \qquad (1)$$

The general methods of dimensional analysis have been described

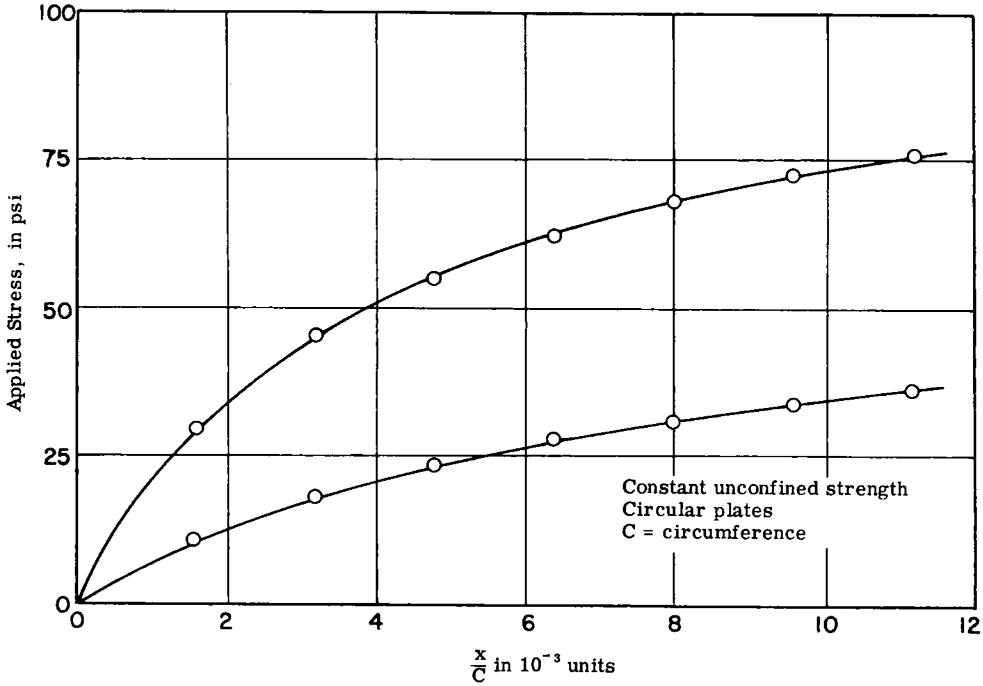

Figure 2. Comparison plot of bearing plate tests.

TABLE 1
PHYSICAL QUANTITIES IN THE DIMENSIONAL ANALYSIS OF LOAD BEARING PLATE TESTS ON SOIL

Physical Quantity	Symbol	Fundamental Unit
Surface deflection or settlement	x	L
Applied force	F	F
Number of load applications	N	FLT
Cross-sectional area of plate	A	L^2
Perimeter of plate	C	L
Time of loading	t	T
Strength parameter of the soil	q	FL^{-2}
Viscosity of the soil	η	$FL^{-2}T$
Angle of internal friction	ϕ	FLT

elsewhere (*10, 11*) and the particular problems encountered in the soil mechanics field when applying this tool have been described in detail by Kondner (*3, 4, 8, 9*) and will not be repeated in this paper. The physical quantities considered are given in Table 1 using the force, length, and time system as fundamental units.

For the problem under consideration the applied surface tractions constitute the force system of primary interest and hence the body or gravity forces are not considered. It is assumed that the material constants needed to describe the deformation characteristics of the soil are, in general, implicit in a characteristic soil strength parameter, angle of internal friction, and effective soil viscosity. The characteristic soil strength parameter is quite general in nature, and may take the form of a shear or compression modulus, unconfined compressive strength, or a relaxation modulus function, depending on the circumstances under consideration. The angle of internal friction includes the frictional resistance of sands and mixed soils, and the viscosity term controls the rate at which deformations take place in a soil. The duration of loading is important in creep and viscous response. The effect of the geometry of the bearing plate is expressed by the cross-sectional area and circumference. Some fatigue, strain hardening, and history effects are included in the number of load applications.

Because there are nine physical quantities and three fundamental units, there must be six independent, nondimensional π-terms. These π-terms can be methodically obtained by choosing n physical quantities (in this case, three) that contain all n fundamental units and cannot be formed into a nondimensional parameter by themselves (for example, F, t, and q) and combining them with each of the remaining quantities one at a time. Because of the great difficulty in experimentally determining the explicit form of the functional relation of Eq. 1, several modifications may be required in the form of the π-terms obtained. Because some of the requirements of the function κ are that it consist of independent nondimensional parameters, there is nothing unique about the forms of the π-terms. Therefore, it is possible to transform algebraically the π-terms in any way desired so long as the final π-terms are nondimensional and independent. Thus, the following π-terms can be obtained:

$$\pi_1=\frac{x}{C},\ \pi_2=\frac{F}{Aq},\ \pi_3=\frac{C^2}{A},\ \pi_4=\frac{qt}{\eta}\ \text{or}\ \frac{Ft}{A\eta},$$
$$\pi_5=N,\ \text{and}\ \pi_6=\phi \qquad (2)$$

The π-terms of Eq. 2 may be substituted into Eq. 1 to obtain the functional relationship which can be rewritten

$$\frac{x}{C}=\kappa\left(\frac{F}{Aq},\frac{C^2}{A},\frac{Ft}{A\eta},N,\phi\right) \qquad (3)$$

In Eq. 3 and hereafter, κ denotes "some function of" but not necessarily the same function for each equation. This notation is used to avoid the use of numerous subscripts and superscripts as a means of differentiating between the equations.

The nondimensional π-terms may be interpreted in the following manner. The deflection or settlement parameter x/C is the dependent variable and is a measure of the amount of deflection under an applied load. The term F/Aq is the ratio of the applied stress to the resisting stress and is called the strength ratio. The strength ratio is analogous to the Cauchy Number in strength of materials and elasticity. The term C^2/A is a characteristic shape factor. For a circular cross-section of any size the shape factor is equal to 4π ($\pi=3.1416$), and for a square shape the value is 16. The creep and viscous effects are included in the terms qt/η and $Ft/A\eta$ which are considered to be proportional to the ratio of the time of loading to a characteristic relaxation time of the soil and is called the time ratio or time factor. The form $Ft/A\eta$ can be considered as the ratio of the applied to viscous forces and may include non-Newtonian effects. Either of these last two terms controls the rate processes. The terms N and ϕ are, by definition, nondimensional and their physical significance has been given.

EXPERIMENTAL RESULTS

Although the functional relationship given in Eq. 3 is reasonably general in that it includes clays, sands, and mixed soils, the determination of the explicit form of the function for such a general case would be extremely difficult. These difficulties are compounded because of the general lack of information on the strength characteristics of the soils for the great volume of field tests reported in the literature. Strength

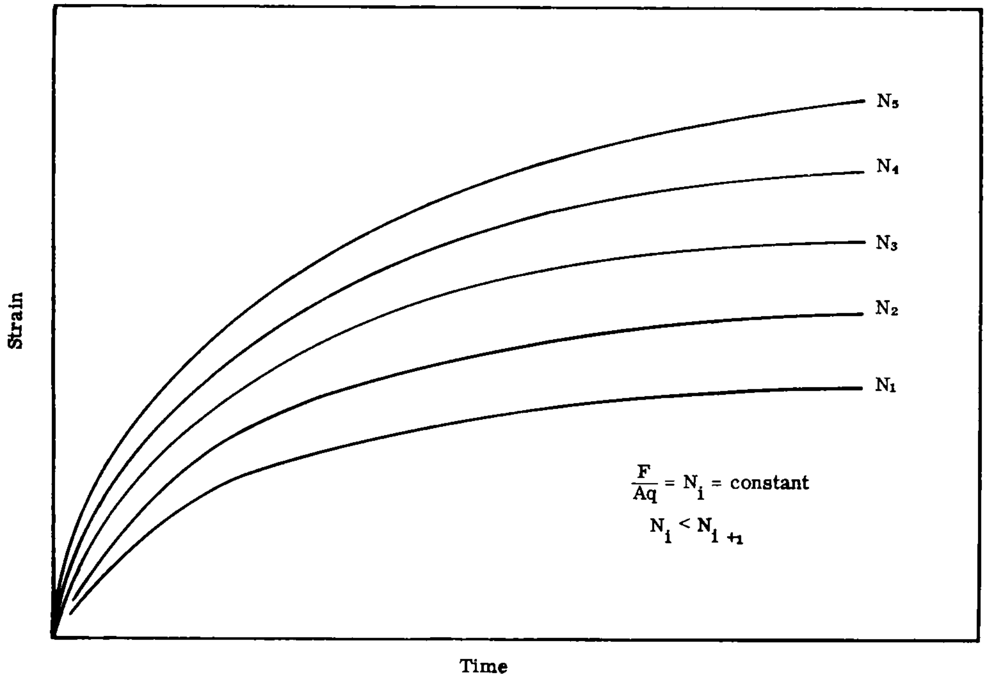

Figure 3. Typical soil creep response.

characteristics are of paramount significance and must be included in order to attempt any correlation among tests. Because of these difficulties, no attempt will be made to cross-correlate completely the various π-terms into an explicit form for Eq. 3.

Rate Effects

An equally important factor about which almost nothing is known is the soil flow characteristics that are included in the term $Ft/A\eta$. Examination of a tabular analysis of load test procedures given by Housel (*1*) and other authors indicates that most test loads are applied until the rate of settlement decreases to a specified value. It is well known that creep curves for soils are generally of the shape given in Figure 3. This shape applies for both laboratory stress-strain-time tests as well as bearing plate tests. For a procedure with a constant terminal rate of settlement the value of $Ft/A\eta$ may vary over an extremely wide range. In the case of a circular plate, $C^2/A=4\pi$, under a single load application, $N=1$, on a soil having a constant value of ϕ which may or may not be zero, Eq. 3 may be written in the form

$$\frac{x}{C}=\kappa\left(\frac{F}{Aq},\frac{Ft}{A\eta}\right) \tag{4}$$

In order to obtain a unique and compatible relationship between x/C and F/Aq, a constant value of $Ft/A\eta$ is required for each load increment. As the applied load or stress increment, F/A, is increased, the time requirement to reach a constant settlement rate increases and the viscosity η tends to decrease; that is, the soil becomes "more fluid." Thus, the term $Ft/A\eta$ increases very much

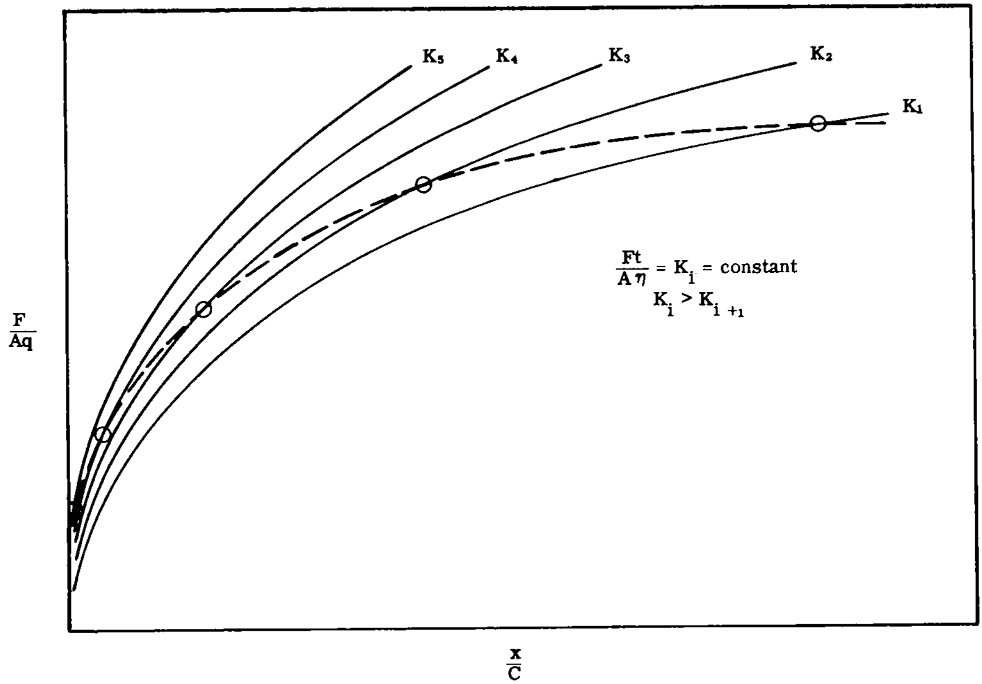

Figure 4. Influence of rate effects.

and the resulting curve relating x/C and F/Aq is much lower than the curve that would have been obtained had the initial value of $Ft/A\eta$ been maintained constant. The results of such a hypothetical test are given in Figure 4 where the unique curves for constant values of $Ft/A\eta$ are solid and the relation for the constant terminal rate of settlement test ($Ft/A\eta$ variable) is given by the dashed line.

An even more difficult problem is the correlation of bearing plate tests on soils of considerably different consistencies or strengths; for example, on soft and stiff clays. For such soils, under constant applied stress, the viscosity η may differ by a factor of a hundred or more. Thus, compatible values of $Ft/A\eta$ would be extremely difficult to obtain. The consistency effects on η are, in general, much greater than non-Newtonian effects.

A more rational way to specify procedures would be with regard to $Ft/A\eta$ by which stress increments and/or time intervals of loading could be adjusted to maintain compatibility. Unfortunately, the field of soil mechanics has not yet reached a state of development where such loading and time rates can be predicted. There is considerable need for more complete studies of the flow characteristics of soils.

Applied Stress Intensity and Soil Strength

In spite of the difficulties encountered because of viscous effects it is interesting to examine the results of studies conducted on circular plates, $C^2/A = 4\pi$, in which N and ϕ are constant and $Ft/A\eta$ has been neglected

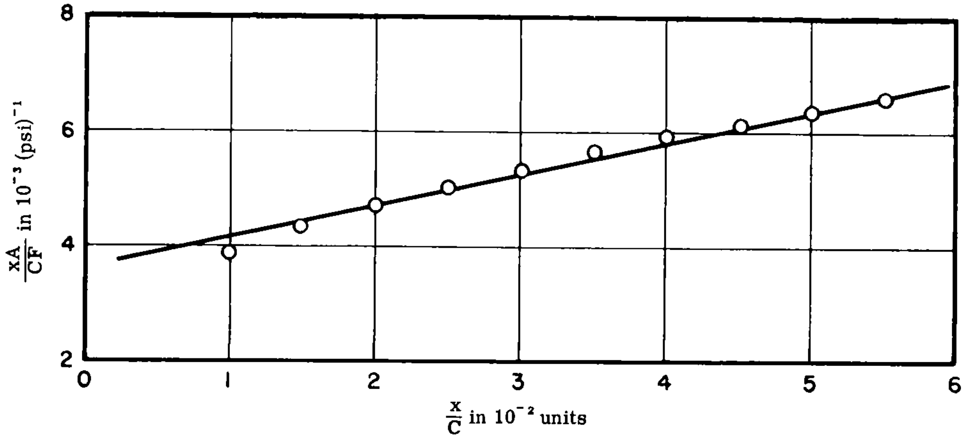

Figure 5. Model bearing plate tests on soft clay (Kondner and Krizek).

by necessity. For such cases the functional relation can be written as

$$\frac{x}{C}=\kappa\left(\frac{F}{Aq}\right) \tag{5}$$

A major difficulty in determining the explicit form of Eq. 5 from either new or previously reported experimental studies is the problem of defining and obtaining a satisfactory soil strength parameter q. It is the authors' contention that for a constant test procedure the shape of the curve of applied stress, F/A, vs deformation parameter, x/C, is uniquely dictated by the strength characteristics of the soil; that is, a strength parameter is "built into" the shape of the curve. In addition, if such a strength parameter is built into the shape of the curve, there must be a correlation between the stress-deflection relation for the bearing plate tests and the laboratory stress-strain relation.

Model Studies.—The first test results to be considered are those given by Kondner and Krizek (*5*) for model bearing plate tests on both soft and stiff clays. Because the strength parameter is assumed to be related to the shape of the stress-deformation parameter curve, the use of a secant modulus seems to be a logical approach to the analysis. The F/A vs x/C data for various diameter circular plates on soft clay were obtained from Figure 7 (*5*). These data have been replotted in the form of the reciprocal of the secant modulus, xA/CF, vs x/C and are given in Figure 5. The straight line obtained thus indicates that the explicit form of the F/A vs x/C relation is a two-constant hyperbolic equation and can be written as

$$\frac{xA}{CF}=a+b\frac{x}{C} \tag{6}$$

or

$$\frac{F}{A}=\frac{\frac{x}{C}}{a+b\frac{x}{C}} \tag{7}$$

where a and b are the intercept and slope, respectively, obtained from Figure 5. The initial slope of the F/A vs x/C curve is $1/a$ and the ultimate or "yield" stress $(F/A)_{ult}$ is $1/b$.

A similar analysis was made on the stress-strain curve obtained in unconfined compression for the soft clay and is given in Figure 6 in the form

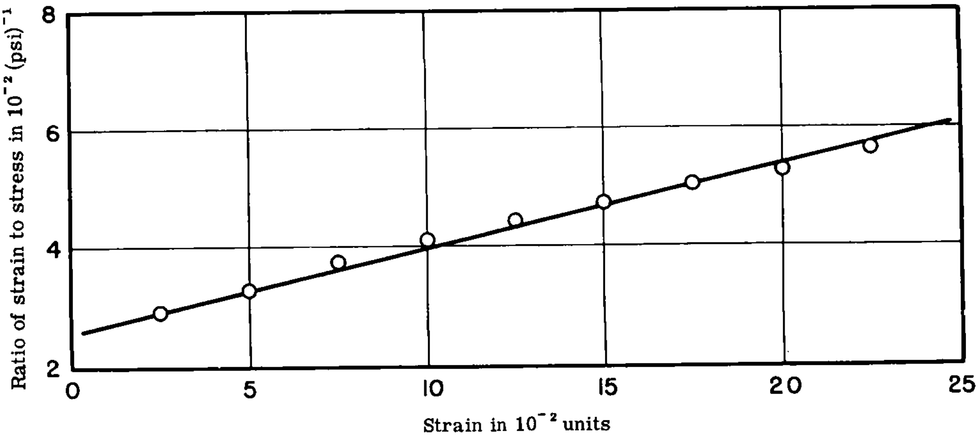

Figure 6. Stress-strain curve for soft clay (Kondner and Krizek).

of the reciprocal secant modulus, ratio of strain ε to stress σ, vs the strain. The straight line obtained indicates that the stress-strain relation is also a two-constant hyperbolic equation and, similarly, can be written as

$$\sigma = \frac{\varepsilon}{a + b\varepsilon} \tag{8}$$

where a and b are the intercept and slope, respectively, obtained from Figure 6. Because $1/b$ is equal to σ_{ult}, it must be proportional to the maximum unconfined compressive strength, q_u, of the soil. Comparison of Eqs. 7 and 8 indicates that F/A is analogous to σ and x/C corresponds in some manner to ε. Thus, the similarity of Figures 5 and 6 and Eqs. 7 and 8 seems to substantiate the authors' contention of a "built-in" strength parameter for the bearing plate test. A detailed study of the explicit form of the correlation with the laboratory stress-strain relation is beyond the scope of this paper, but such a study will be conducted in the near future.

The results of similar tests of model circular bearing plates of various diameters on a stiff clay were obtained from Figure 8 of a paper by Kondner and Krizek (*5*). These results have been replotted in the form of xA/CF vs x/C and are shown in Figure 7. The two-constant hyperbolic form of Eq. 7 is obtained where a and b are the intercept and slope, respectively, obtained from Figure 7. The unconfined compression stress-strain relation for the stiff clay is given in Figure 8 in the form of ε/σ vs ε and takes the hyperbolic form of Eq. 8. Thus, the strength parameter correlation seems to be valid for a wide range of soil consistencies for small-scale model tests.

Field Studies.—The correlation between load bearing tests and laboratory unconfined compression tests that has been presented for model studies will be extended to cover field studies.

Subsurface Tests on a Chicago Clay.—The first field study to be considered was conducted by Dix and Lukas (*12*). Three circular plate bearing tests were performed on a Chicago clay at depths of 20 to 30 ft below the ground surface. A summary of the plate sizes and depths are given in Table 2.

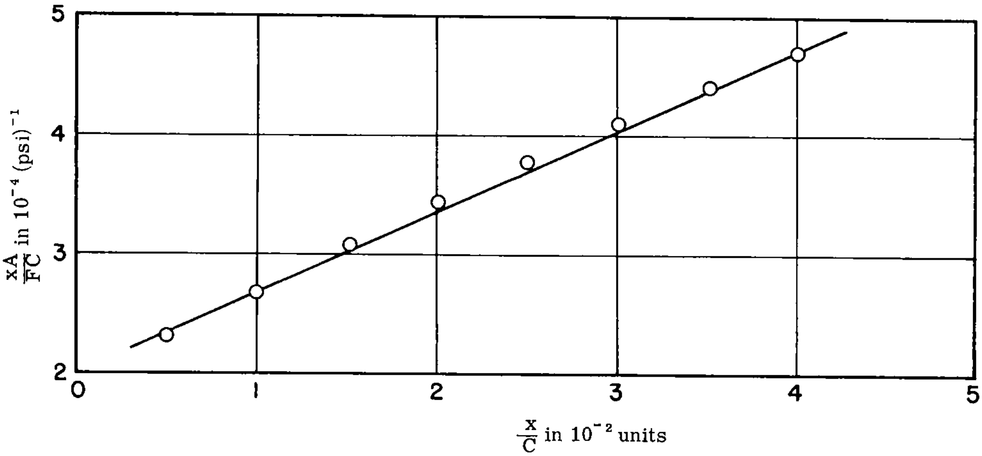

Figure 7. Model plate tests on stiff clay (Kondner and Krizek).

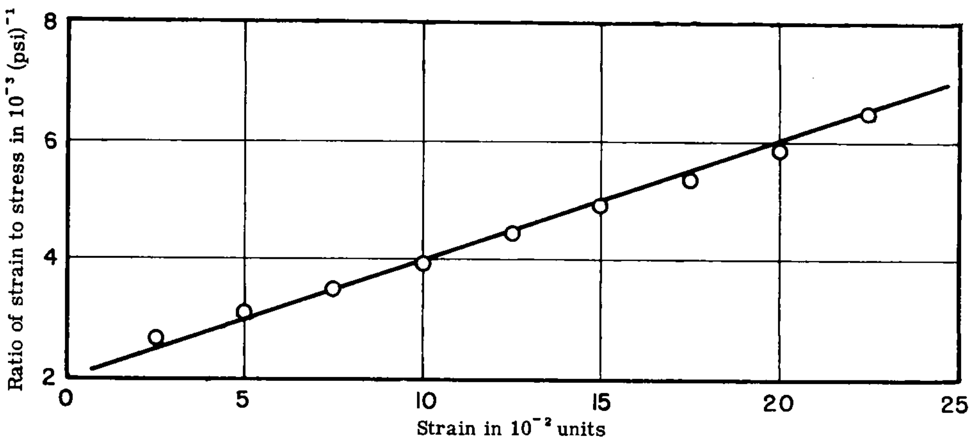

Figure 8. Stress-strain curve for stiff clay (Kondner and Krizek).

The basic test data are shown in Figure 9 by the solid curves in the form of applied stress vs deflection. Test 3, which has the largest diameter plate and greatest depth below the ground surface, starts with the highest stress per unit deflection and then crosses and remains lower than the curve for Test 2, indicating experimental difficulties. The test data were replotted in the form of the reciprocal of the secant modulus, xA/CF, vs x/C and shown in Figure 10. Once again the two-constant hyperbolic form of Eq. 7 is obtained.

TABLE 2
PLATE SIZES AND DEPTHS, FIELD TESTS

Plate Test	Diameter of Plate (in.)	Cross-Sectional Area (sq ft)	Depth Below Ground Surface (ft)
1	13.5	1.00	20.6
2	24.0	3.14	23.0
3	30.0	4.91	27.0

Both unconfined compression and triaxial compression test results have been replotted in Figures 11 and 12,

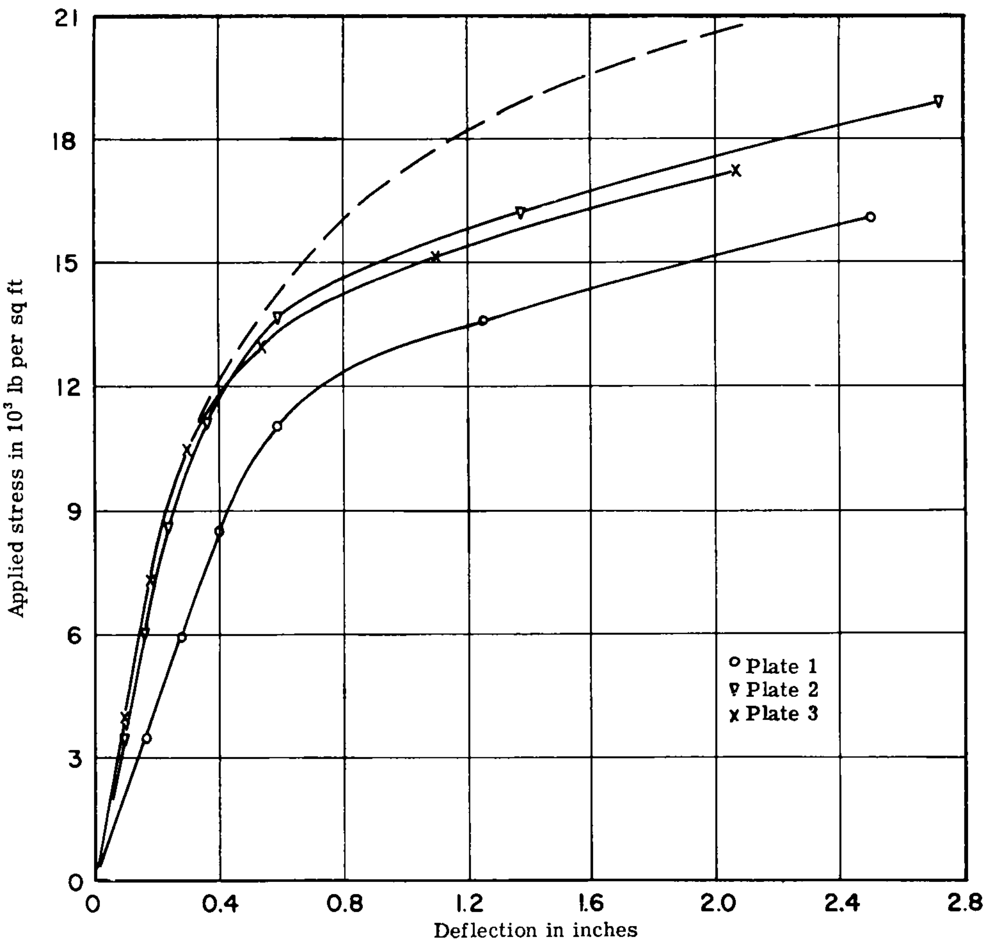

Figure 9. Bearing plate tests (Dix and Lukas).

respectively, in the form ε/σ vs ε. Once again the stress-strain relations take the form of Eq. 8. Tests 2 and 3 of Figure 10 show the same results although, as shown in Figures 11 and 12, the soil strength was greater under Test Plate 3. By relating the slopes, b, of Figure 10 with the strength curves it is possible to show that Test 3 should have followed the dashed curve of Figure 9. A complete discussion of this point is beyond the scope of the present paper.

Bearing Plate Tests.—The model tests by Kondner and Krizek (*5*) and the field tests of Dix and Lukas (*12*) have shown the correlation between the stress-strain plots for conventional strength tests and bearing plate tests. To illustrate the general nature of the two-constant hyperbolic form of Eq. 7, the following series of test results have been analyzed.

The test data reported by Benkelman and Williams (*13*) for circular plates with diameters ranging from 1 to 7 ft, conducted on subgrades at the Hybla Valley test track near

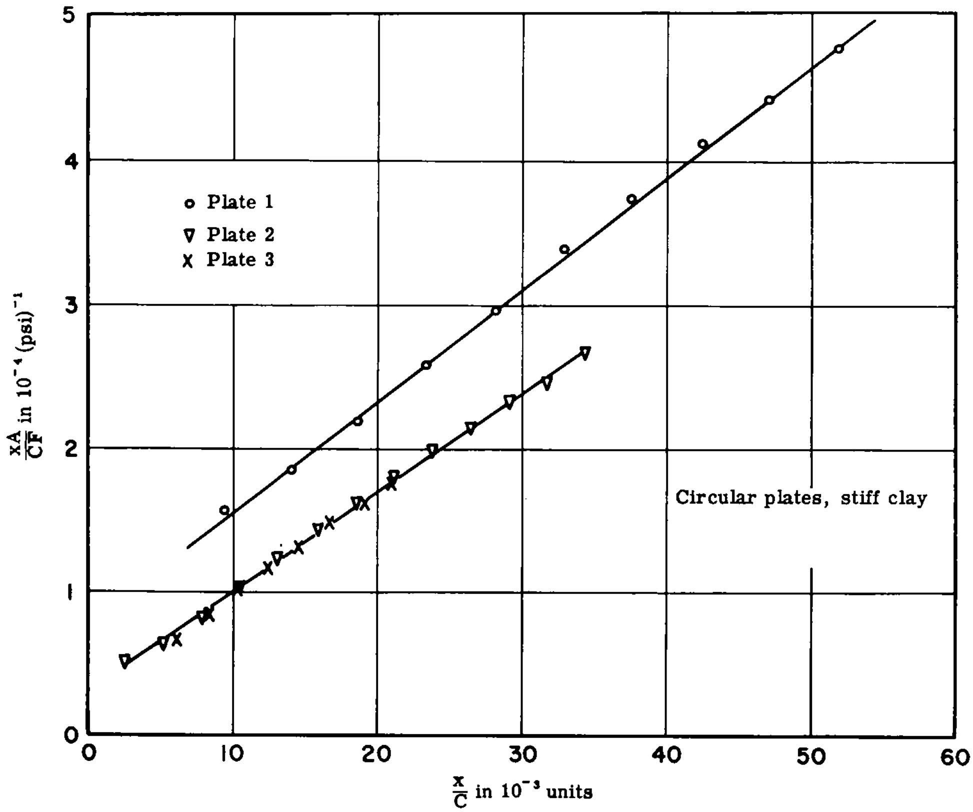

Figure 10. Bearing plate tests (Dix and Lukas).

Alexandria, Va., have been replotted and are shown in Figure 13. The hyperbolic fit is quite good.

Test data presented by Osterberg (*14*) on bearing plate tests on silty clay and buckshot clay conducted by the U. S. Army Engineers, Waterways Experiment Station, Vicksburg, Miss., are shown in Figures 14 and 15. Additional data given by Osterberg (*14*) on tests conducted at Wright Field and by Teller and Sutherland are shown in Figures 16 and 17.

Tests reported by McLeod (*15*) for circular plates of various diameters are given in Figure 18 in the form of applied stress, F/A, vs x/C. According to McLeod, the data given in the figure are typical of that obtained for eight different airfields located throughout Canada. Figure 19 is a plot of xA/CF vs x/C and fits the form of Eq. 7.

Additional hyperbolic fits of bearing plate data are given in Figures 20 and 21 for circular plates tested on a slightly plastic blue clay and a stiff sandy blue clay by Housel.

California Bearing Ratio Tests

Because the California bearing ratio test is a particular type of load bearing plate test, such tests should also give applied stress vs deformation parameter relations similar to those obtained for the model and field studies previously presented. Figures 22 and 23 show the results of two

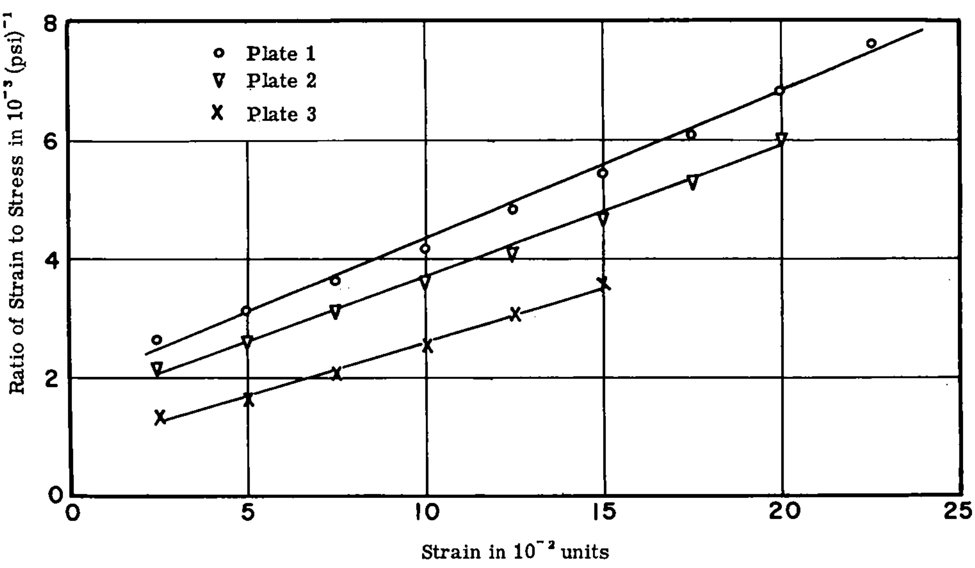

Figure 11. Unconfined compression tests (Dix and Lukas).

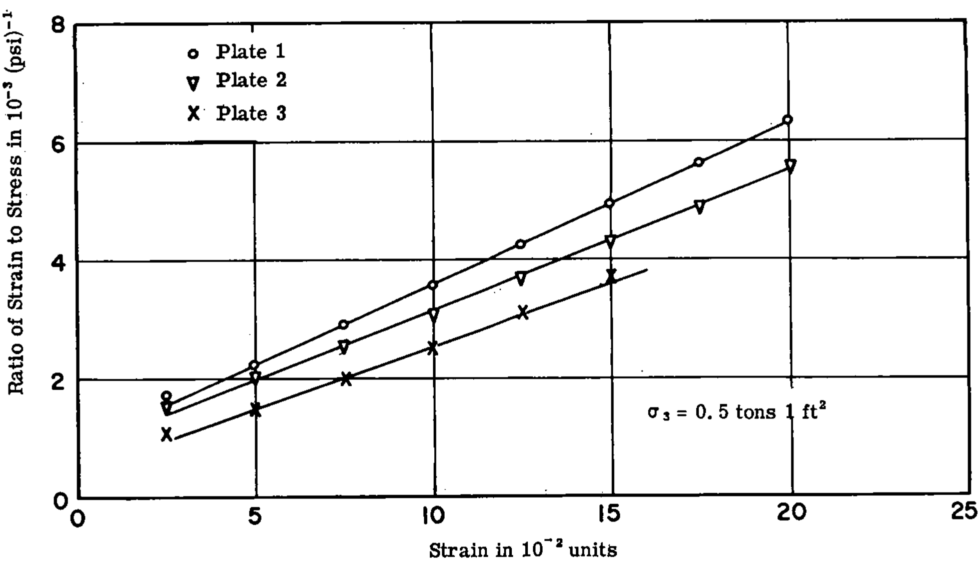

Figure 12. Triaxial compression tests (Dix and Lukas).

CBR tests conducted on a sandy loam soil and a clay loam given by Porter (*16*). Once again the hyperbolic forms of Eq. 7 are obtained. In addition, the standard 100 percent CBR curve for crushed stone can also be represented by Eq. 7 as shown in Figure 24. Comparison of laboratory stress-strain tests and bearing plate tests for the model and field studies

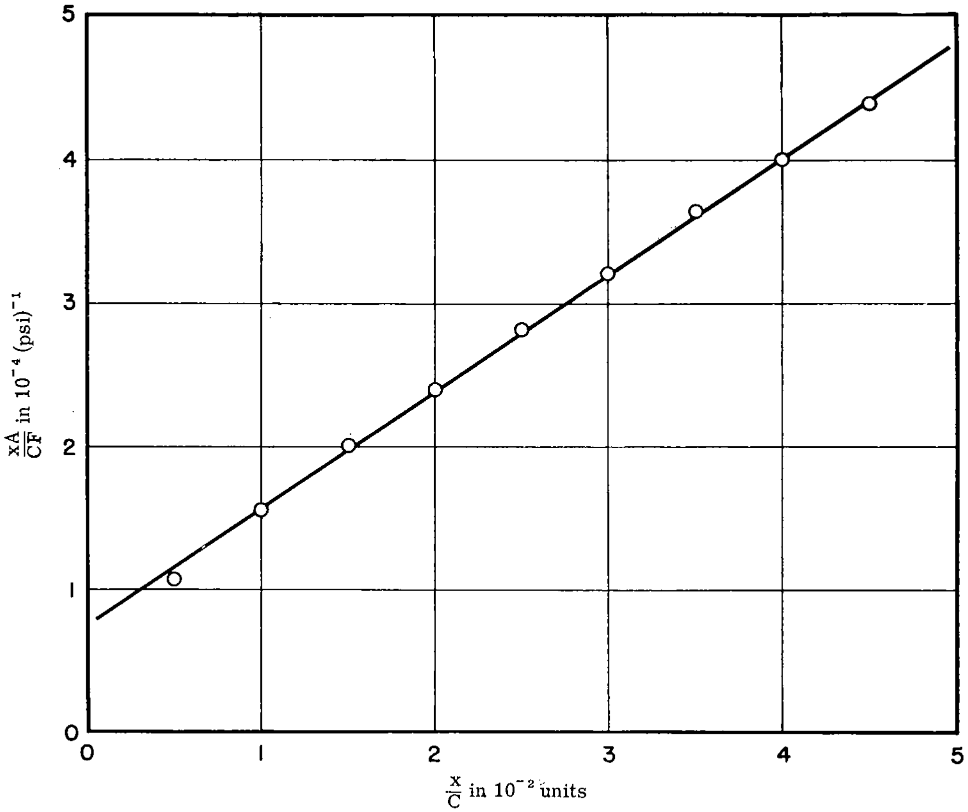

Figure 13. Bearing plate tests (Benkelman and Williams).

indicates that the CBR test is a measure of the soil strength, and as such, the CBR value is a ratio of the soil strength parameter to the strength parameter of a standard material (namely, crushed stone) at a particular value of x/C. It is important that the correlation between CBR values and bearing plate tests be made at compatible x/C values and not at equal values of x.

Shape Effects

The influence of the shape of a bearing plate on the applied stress vs deflection relation is included in Eq. 3 by the term C^2/A. Neglecting viscous, repetitional, and internal friction effects, the plate load-deflection-shape relationship as given by Eq. 3 can be written as

$$\frac{x}{C} = \kappa\left(\frac{F}{Aq}, \frac{C^2}{A}\right) \tag{9}$$

To determine the effect of shape on the variation of x/C as a function of the strength ratio, F/Aq, a series of tests were conducted on model plates of equal cross-sectional area, but with different values of C^2/A, on soils having approximately the same unconfined compressive strengths. The model plates used are shown in Figure 25. The cross-sectional areas of all of the plates were 2 sq in., but the values of C^2/A ranged from the geometric minimum of 4π for the circular shape to a value

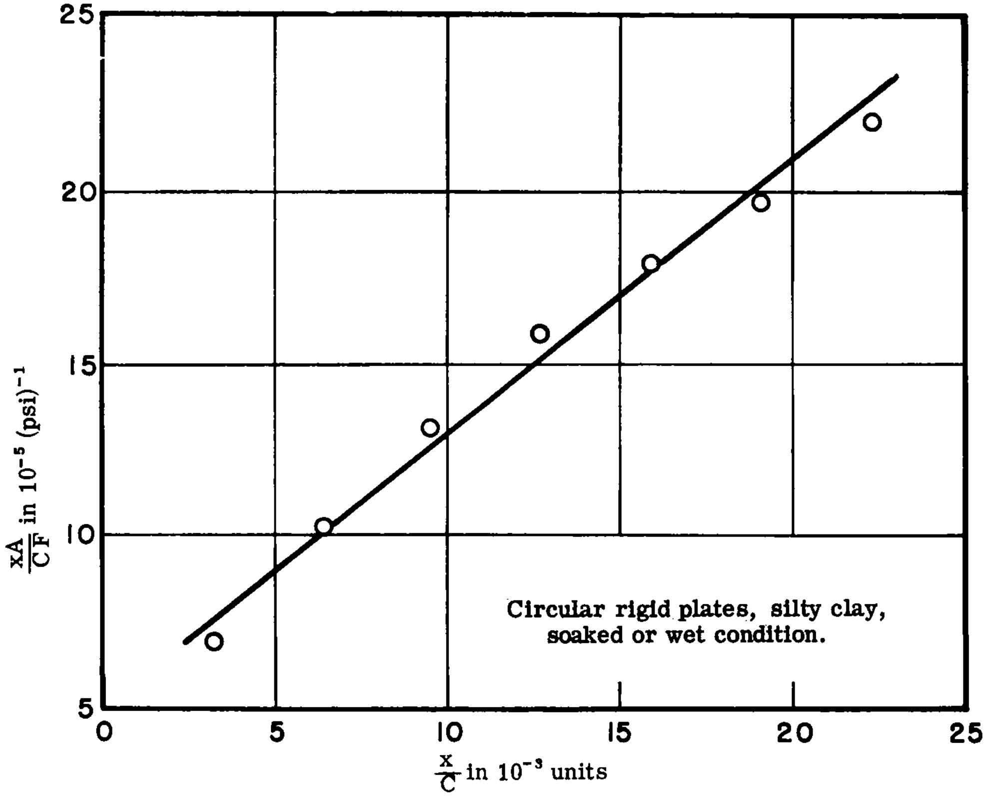

Figure 14. Tests by U. S. Waterways Experiment Station.

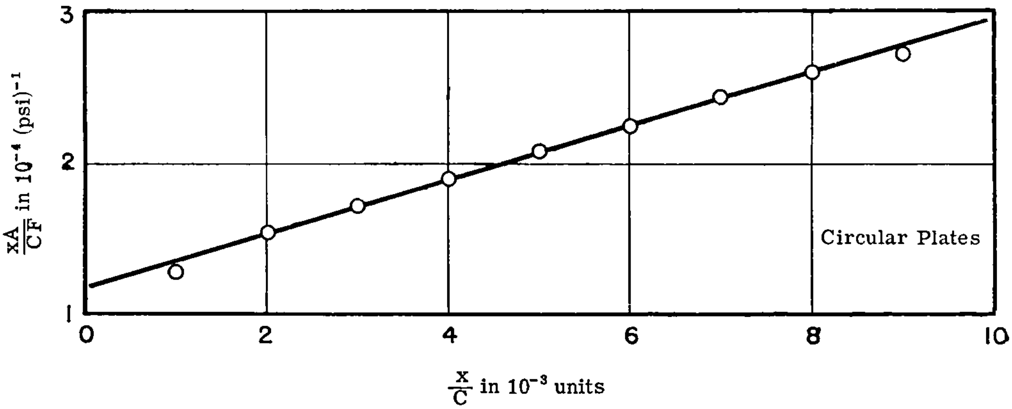

Figure 15. Bearing plate tests on buckshot clay
(U. S. Waterways Experiment Station).

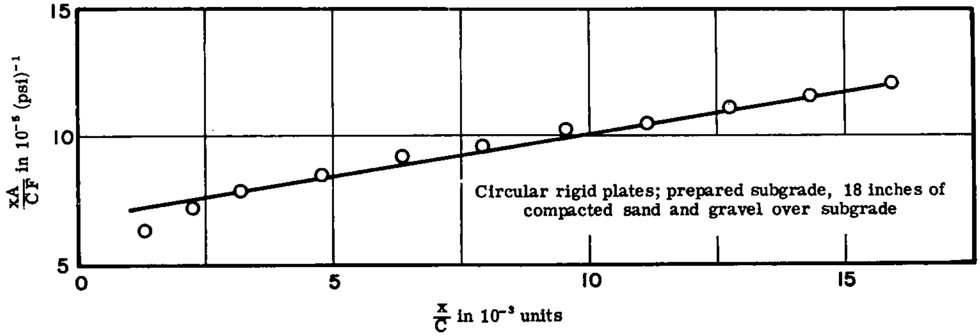

Figure 16. Average curve, Wright Field tests (U. S. Engineers, 1942).

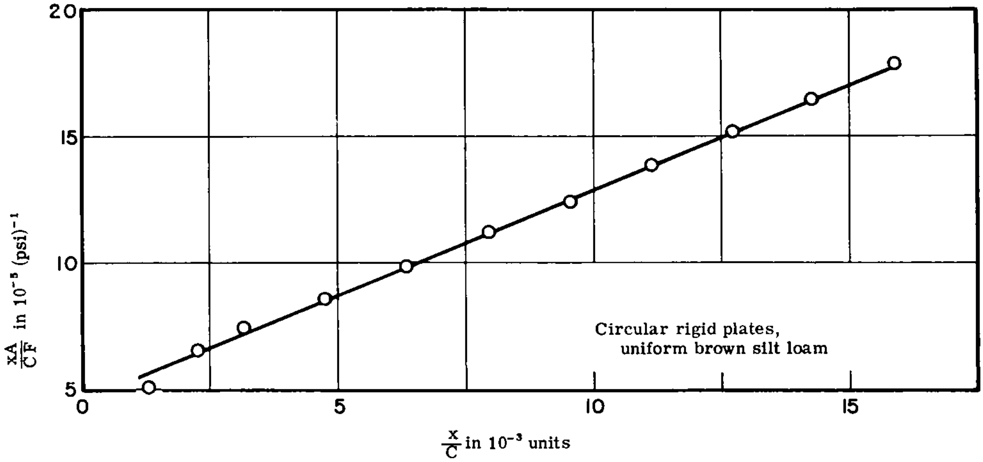

Figure 17. Average curve, tests by Teller and Sutherland, Series 3.

of 136 for the cross-shaped plate. The results of these tests are shown in Figure 26 where F/Aq has been plotted against x/C for various values of C^2/A. The strength parameter, q, used in Figure 26 is the unconfined compressive strength of the soil. Figure 26 shows a definite phenomenological influence caused by C^2/A. For a constant applied stress, F/A, the deflection parameter, x/C, increases for decreasing values of C^2/A. In comparing the results obtained using square plates of width b and circular plates of diameter d for equal cross-sectional areas on the same soil, the plotting of x/b and x/d instead of x/C can lead to the following difficulty. For constant values of x/C, the ratio of x/b to x/d is 1.273. Because the deflection parameter in Figure 26 for the circular plate is approximately 18 percent greater than the square plate, a plot of F/A vs x/b or x/d would reverse the order of the two curves by approximately 9 percent. Considering the fairly high experimental error involved in field bearing plate testing, it seems quite possible that the correlation of the results of tests for a limited number of circular

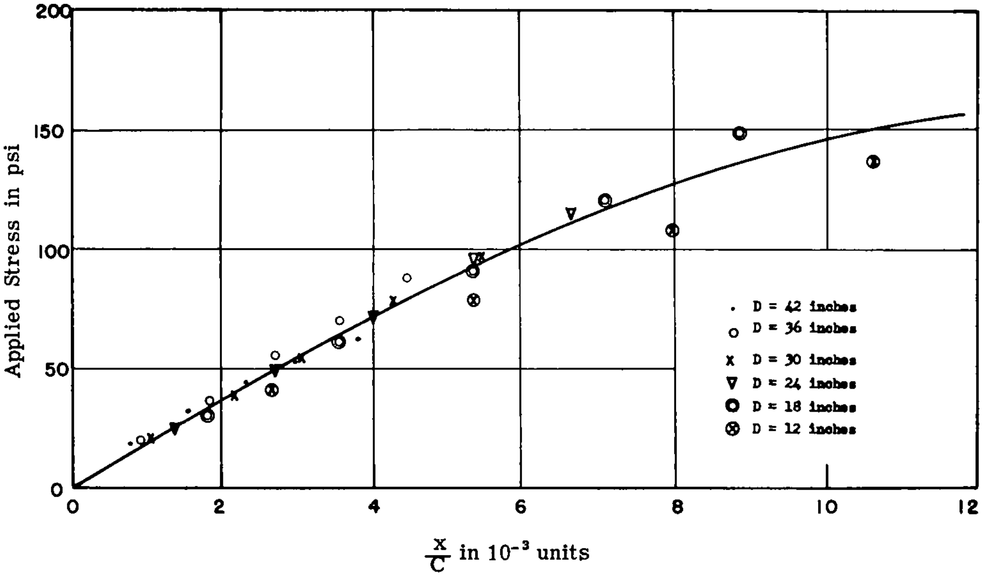

Figure 18. Typical bearing plate tests (McLeod).

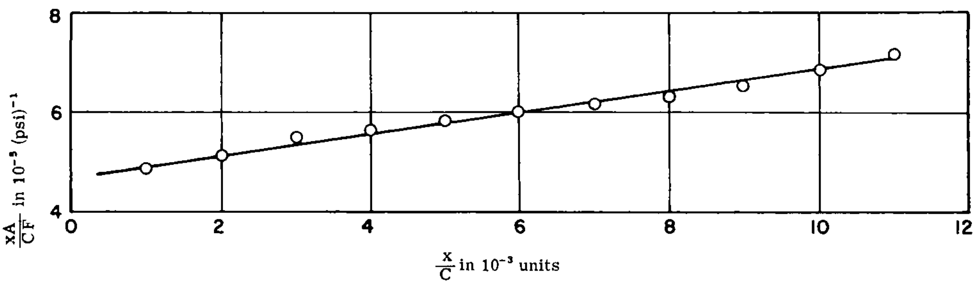

Figure 19. Typical bearing plate tests (McLeod).

and square plates would be inconclusive.

Correlation of Various Strength Indexes

In an attempt to draw some correlation between plate bearing tests and field California bearing ratio, cone-bearing, and Housel penetrometer tests, McLeod (*15*) has conducted an extensive testing program on cohesive soils at eight different airports located throughout Canada. These locations are Ft. St. John, Grande Prairie, Saskatoon, Lethbridge, Dorval, Winnipeg, Malton, and Regina. The plate tests were performed on a 30-in. diameter plate with 10 repetitions of the load, and the other tests were conducted in accordance with standard procedures. The results of this work as reported by McLeod are in the form of subgrade support, in pounds, for 0.2-in. deflection vs representative indexes for each particular test. McLeod fitted each set of data with a

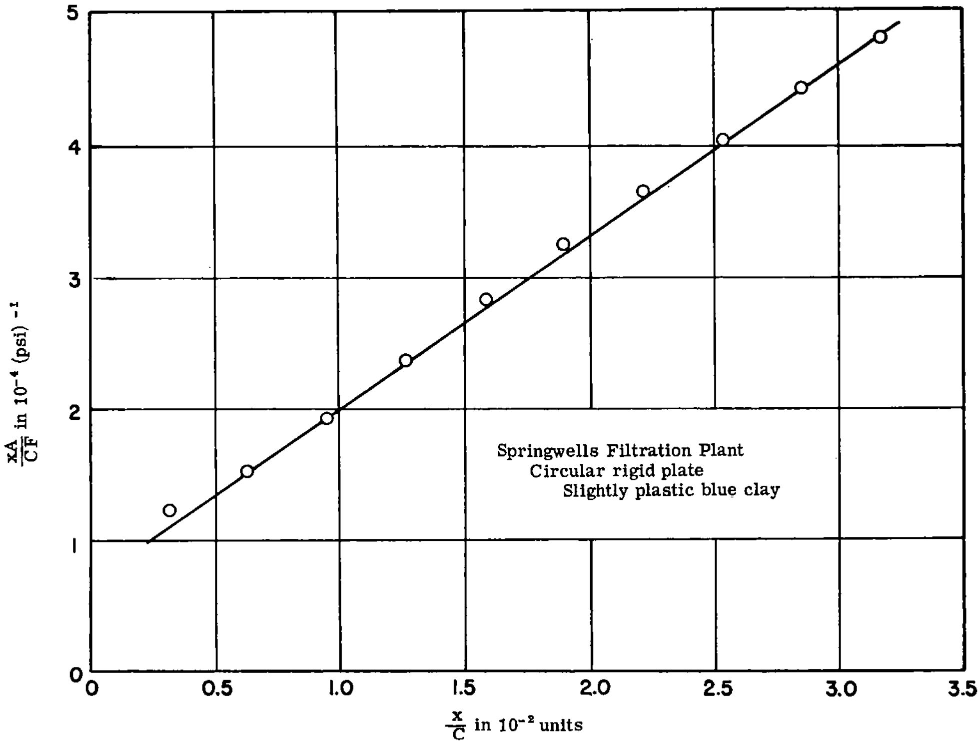

Figure 20. Bearing plate test (W. S. Housel).

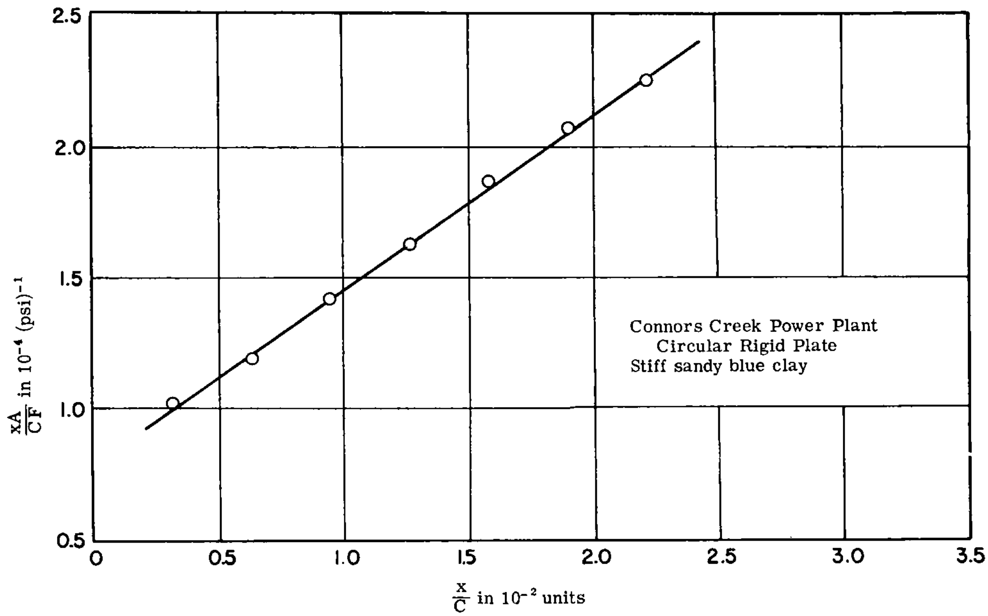

Figure 21. Bearing plate test (W. S. Housel).

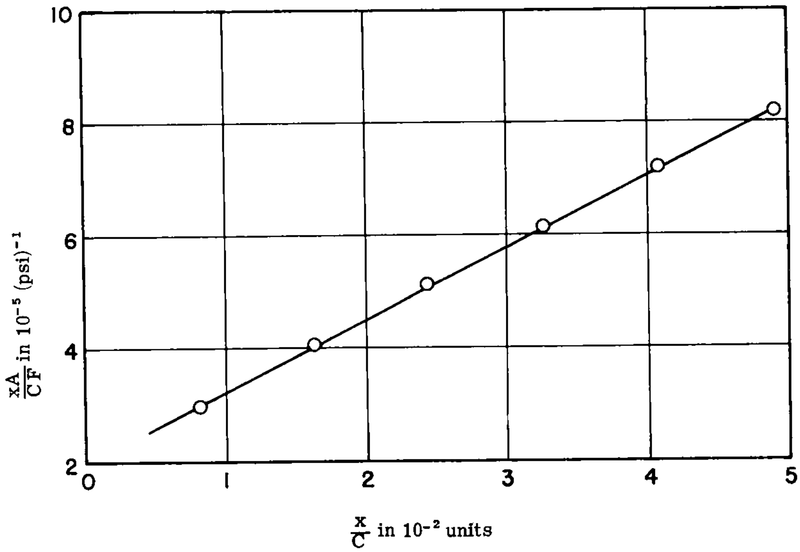

Figure 22. Typical CBR test on sandy loam soil (O. J. Porter).

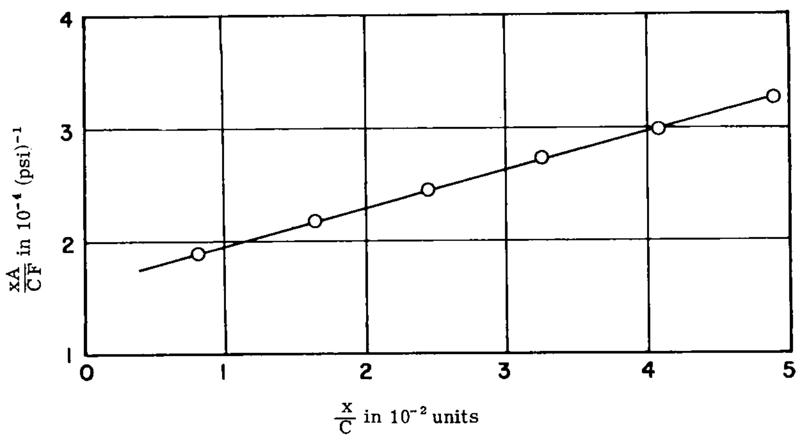

Figure 23. Typical CBR test on clay loam (O. J. Porter).

particular straight line and then correlated the straightline fits of the four types of test.

It has been observed that these same data can, in each case, be fitted reasonably well with a hyperbola. The curve for the field California bearing ratio data is shown in Figure 27; for cone-bearing, in Figure 28; and for the Housel penetrometer, in Figure 29. By referring to the hyperbolic test plot in Figure 30 for each of these three cases, it is seen that each set of data reduces to the same hyperbola when the abscissa scale is appropriately selected. This establishes a correlation on a somewhat different basis than McLeod and exemplifies the possible hyperbolic nature of this wide range of data.

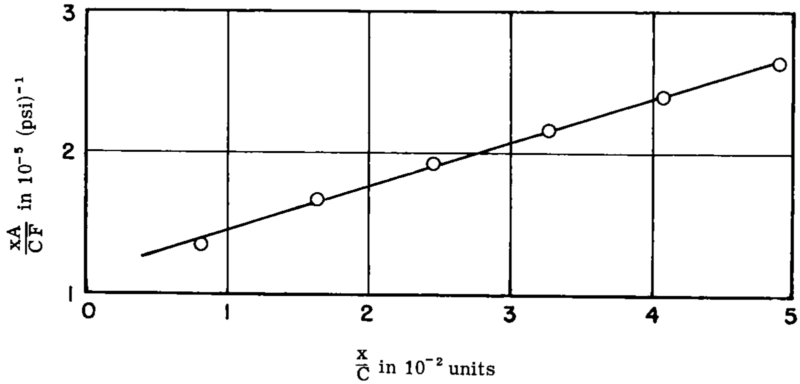

Figure 24. Standard CBR test.

The large scatter in McLeod's data as shown in Figures 27 through 29 is probably due to the wide variation in soil types and consistencies tested and to the fact that although the x/C values are constant for each index parameter, the viscosity and strength parameters have been neglected for the reasons previously discussed.

Repetitional Loading

At present most methods of pavement design are based on some index of soil strength obtained from tests in which the magnitude of the load is increased slowly and allowed to remain for some specified period of time or until some specified rate of deformation is reached. The results of such tests have been correlated empirically with soil performance and provide a reasonably reliable basis for design criteria.

The problem of repetitional loading of a soil mass, however, introduces additional variables into the analysis, and it does not follow that soil strength indexes obtained from normal static tests will provide a realistic measure of soil performance when subjected to repeated loads. A comprehensive rational approach to this problem should include not only the magnitude of the applied stress but also the number of applications, frequency, and duration. To analyze in detail the effects of the latter two factors on the deformation and strength characteristics of a soil is beyond the scope of this paper and comment will be restricted to a brief presentation of the results obtained by other investigators. The subject of number of stress applications will be analyzed in some detail, and certain observations and correlations will be pointed out. A procedure for including this variable in a general force-deformation equation will be presented.

Seed and Chan (*17*) have shown that the duration of repetitional load tests may be very important in some soils due to an increase in thixotropic effects with time. These effects become increasingly significant at smaller strains and thixotropic stiffening appears to have greater effects in these tests than in the normal type of strength tests.

The results of numerous tests by Seed, Chan, and Monismith (*18*) on partially saturated specimens of silty clay subjected to repeated applications of a constant stress in triaxial compression indicate that up to at least 100,000 stress applications, deformation is independent of frequency within the range of 3 to 20 applications per min (and possibly as low as 1 application per min) and

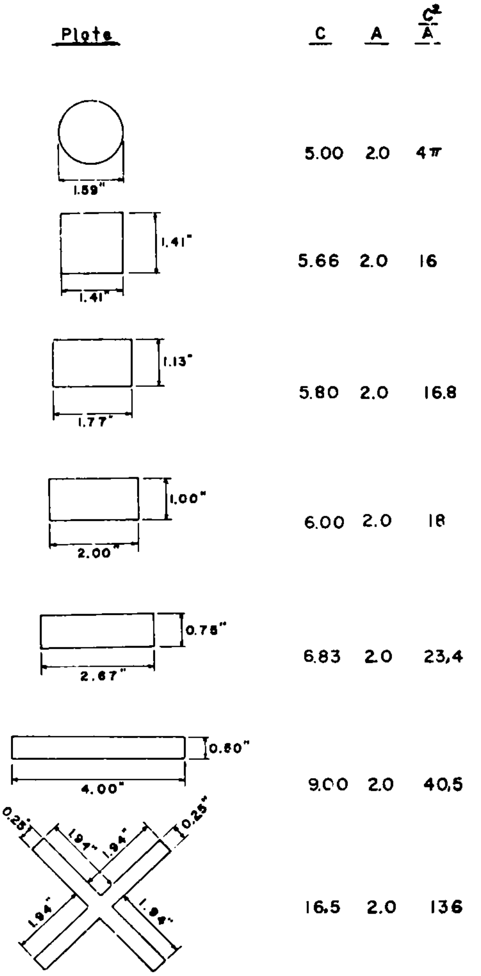

Figure 25. Plates of constant area and variable perimeter.

dependent only on the number of stress applications.

A review of the literature on repeated loading of bearing plates revealed a general scarcity of test data representing a large number of load applications. Many authors tread on dangerous ground by extrapolating the results of a few tests with a small number of load applications to predict the probable results at thousands of times the actual recorded data with the assumption of an exponential relationship. The validity of such an assumption has been questioned by McLeod (*15*) when he points out that the exponential relationship holds reasonably well up to 100 repetitions of load although "the direction of the curve has become somewhat uncertain for the last ten or fifteen repetitions."

One source of data for bearing plates subjected to repeated loads was found in McLeod (*15*) where a 30-in. diameter plate was subjected to 100 applications of a 40,000-lb load. These data have been subjected to the hyperbolic test plot shown in Figure 31 and found to satisfy the requirements for a hyperbola, and can be written as

$$\frac{x}{C}=\frac{N}{660+217N} \text{ for } 5\leq N\leq 100 \qquad (10)$$

Inasmuch as it has been shown previously in the case of the normal plate tests that there is a relationship between field plate bearing tests and laboratory stress-strain compression tests, it seems reasonable at this point to investigate the behavior of a laboratory specimen subjected to a large number of repeated loadings. Data of this type for a silty clay specimen subjected to a stress of 3 kg per sq cm at a frequency of 20 applications per min have been obtained from Seed and Chan (*17*) and plotted on an arithmetic scale as shown in Figure 32. These same data have then been plotted on a hyperbolic test plot in Figure 33. A hyperbola provides a very good fit for the data within a certain range, but no one hyperbola satisfies the full range of the test data. Thus, the strain must be given as

$$\varepsilon=\frac{N}{1.25+N} \text{ for } 3\leq N\leq 500 \qquad (11)$$

$$\varepsilon=\frac{N}{175+0.6N} \text{ for } 500\leq N\leq 10{,}000 \qquad (12)$$

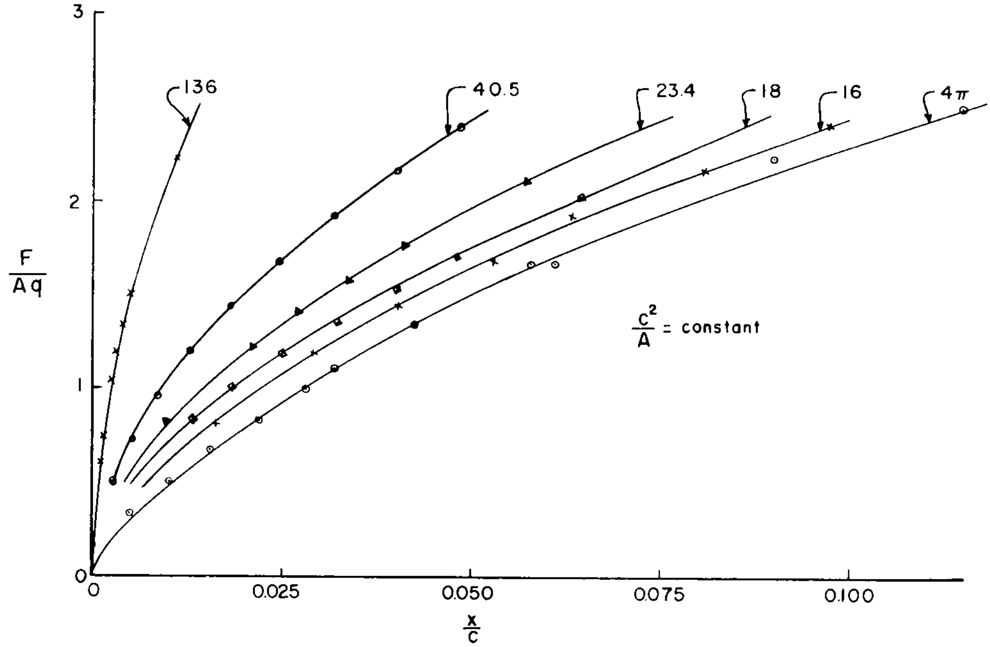

Figure 26. Nondimensional plot of x/C vs F/Aq for constant value of C^2/A.

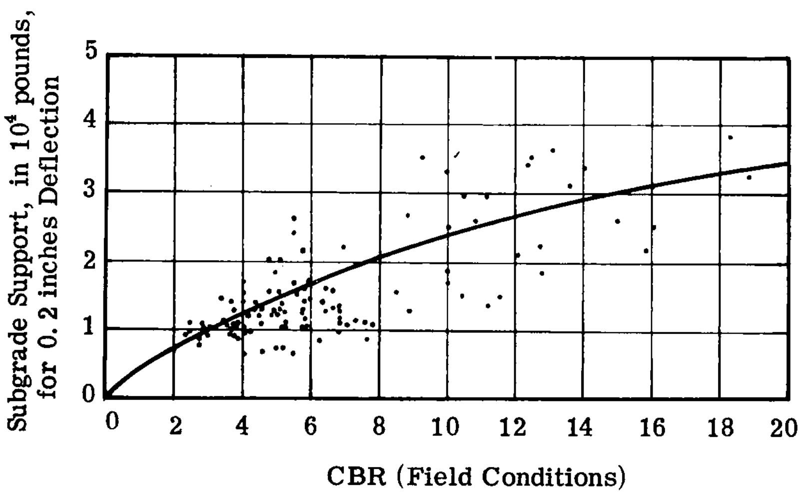

Figure 27. Subgrade support vs field California bearing ratio.

It can be verified that these data are also not satisfied over the full range by an exponential expression.

Another set of data obtained from Seed, Chan, and Monismith (*18*) is for a silty clay specimen subjected to triaxial compression with a constant lateral stress of 14.2 psi and an axial stress of 40 psi applied 10 times per min for a period of 1 sec each time.

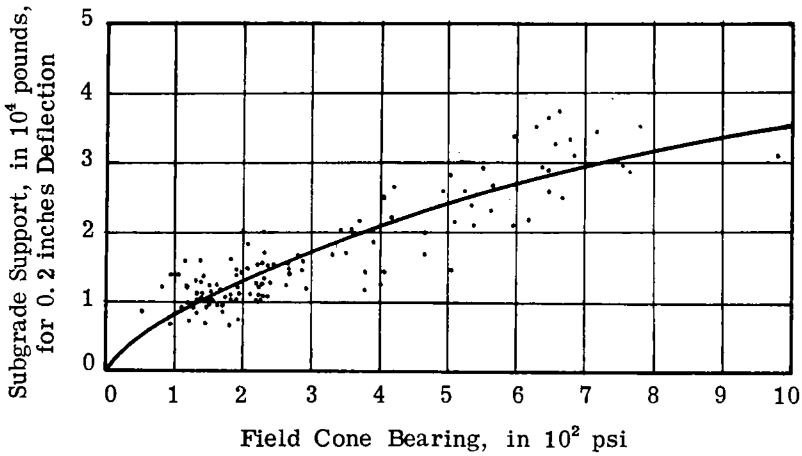

Figure 28. Subgrade support vs field cone bearing.

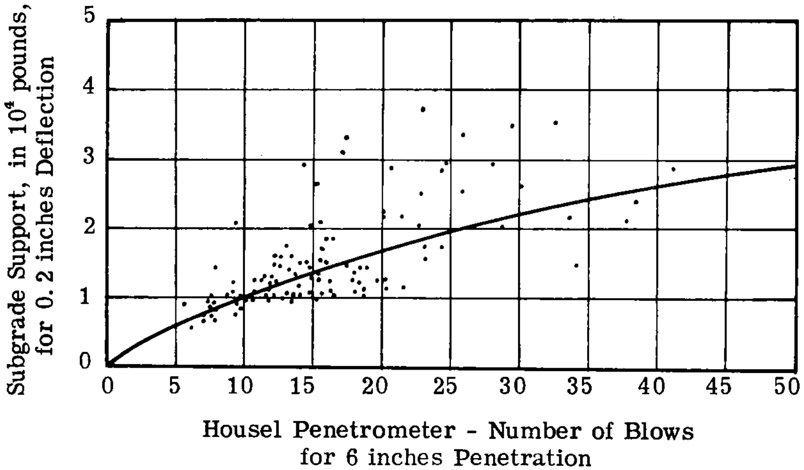

Figure 29. Subgrade support vs Housel penetrometer.

These data satisfy very well the hyperbolic test plot as shown in Figure 34 and again reveals the necessity of fitting each range with a separate hyperbola.

$$\varepsilon=\frac{N}{4+3.2N}\text{ for }3\leq N\leq 500 \tag{13}$$

$$\varepsilon=\frac{N}{550+1.92N}\text{ for }500\leq N\leq 10{,}000 \tag{14}$$

Thus, it appears that a hyperbola provides a very good fit for repetitional test data within a certain range. The test plots in each case indicate the slope of the line representing the 500 to 10,000 range to be about 60 percent of the slope of the line for the 3 to 500 range. It is possible that analysis of additional data will verify a definite relationship between these two hyperbolas. If so, this would provide a rational basis to

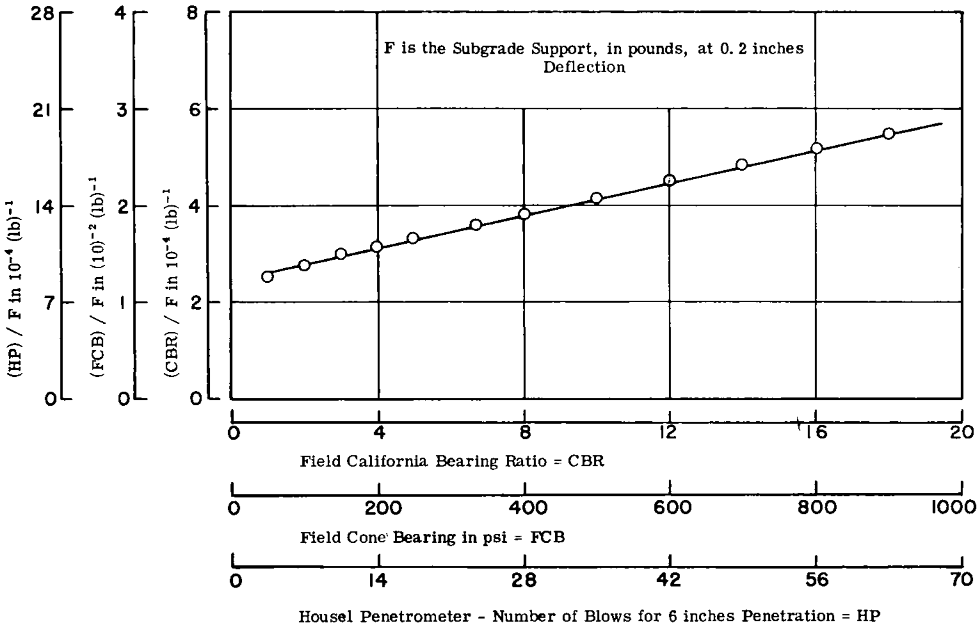

Figure 30. Correlation of plate bearing tests and various soil strength indexes.

extrapolate the results of the low range to the higher range.

CONCLUSIONS

The investigation reported in this paper leads to the following conclusions:

1. The phenomena of rigid bearing plate tests on soil can be described in functional form as

$$\frac{x}{C}=\kappa\left(\frac{F}{Aq}, \frac{C^2}{A}, \frac{Ft}{A\eta}, N, \phi\right) \qquad (3)$$

2. Load test procedures influence the applied stress vs deflection relation through the effects of the term $Ft/A\eta$.

3. Model and field bearing plate test results can be represented by the two-constant hyperbolic form

$$\frac{F}{A}=\frac{\frac{x}{C}}{a+b\frac{x}{C}} \qquad (7)$$

4. Laboratory stress-strain tests, conducted on several of the same soils tested with bearing plates, can be expressed as

$$\sigma=\frac{\varepsilon}{a+b\varepsilon} \qquad (8)$$

Comparison of conclusions 3 and 4 tends to substantiate the concept that a soil strength parameter is "built into" the shape of the load-deflection curve for a plate bearing test; that is, the shape of the curve is dictated by the strength characteristics of the soil.

5. California bearing ratio test results can also be expressed in the form of Eq. 7, indicating a correlation with plate bearing tests. Thus, the CBR value is a ratio of the soil strength parameter to the strength parameter of a standard reference material; namely, crushed stone at a particular value of x/C.

6. The influence of the shape of the bearing plate is given by the varia-

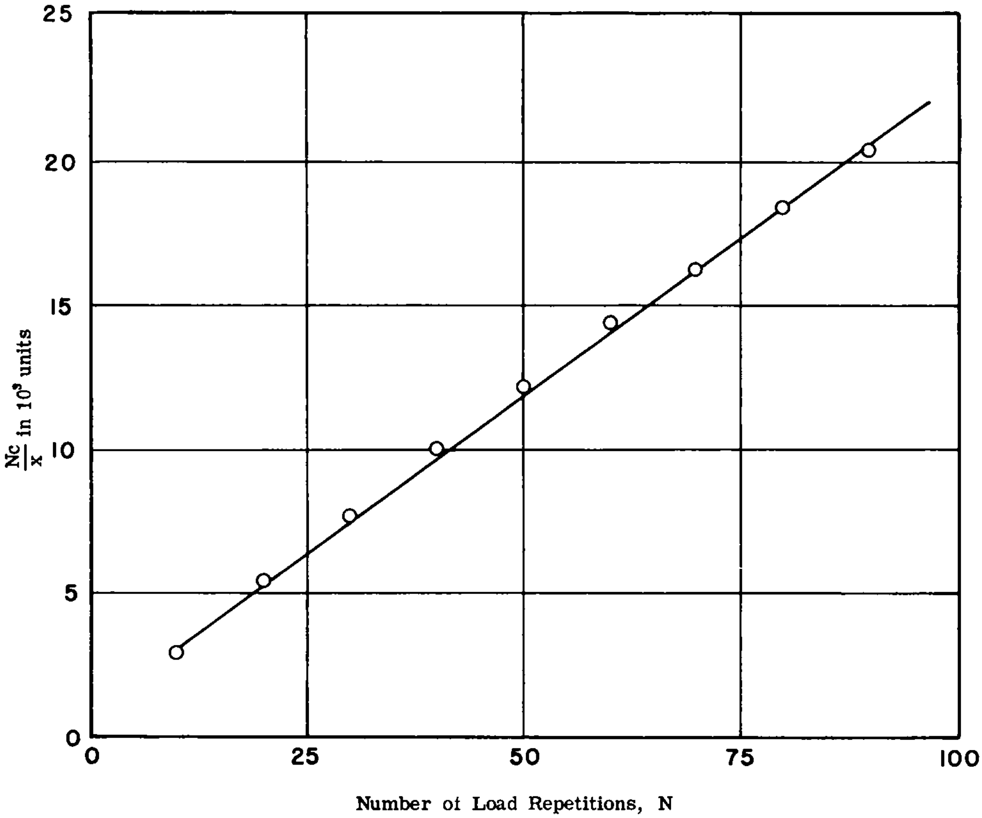

Figure 31. Bearing plate tests, load repetitions (McLeod).

tion of C^2/A, where C is the perimeter and A is the cross-sectional area.

7. A hyperbolic relation can be used to correlate plate bearing tests and field cone-bearing, Housel penetrometer, and California bearing ratio tests.

8. The results of repetitional load bearing and repetitional laboratory stress-strain tests can be expressed in the hyperbolic forms

$$\frac{x}{C}=\frac{N}{a+bN} \text{ for } \alpha \leq N \leq \beta \quad (10)$$

and

$$\varepsilon=\frac{N}{a+bN} \text{ for } \bar{\alpha} \leq N \leq \bar{\beta} \quad (11)$$

where the constants a and b depend on the range of N considered.

ACKNOWLEDGMENTS

The authors wish to express their appreciation to Jorj O. Osterberg, Professor of Civil Engineering, Northwestern University, for his assistance in providing some of the experimental data used in the analysis and for his general encouragement.

The authors are also indebted to the many researchers whose published test results have been analyzed in this paper.

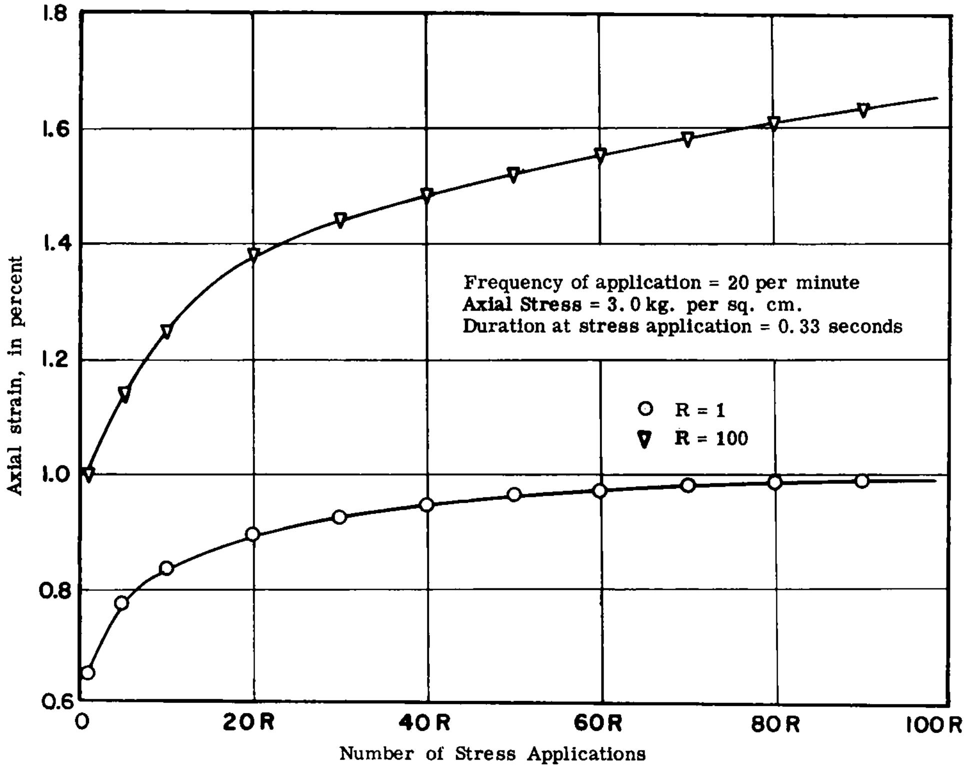

Figure 32. Axial strain vs number of stress applications (Seed and Chan).

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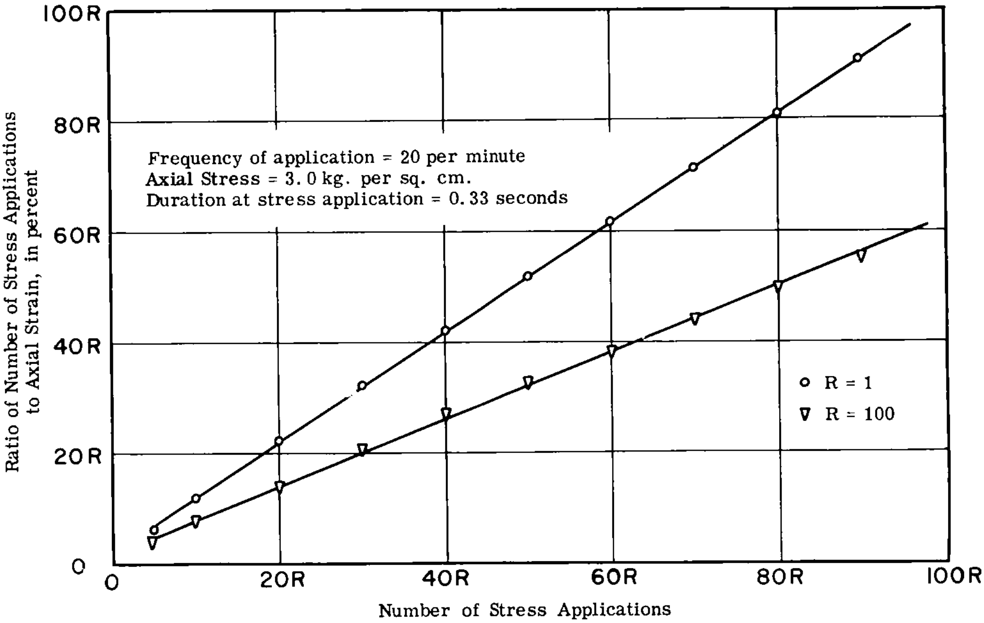

Figure 33. Axial strain vs number of stress applications (Seed and Chan).

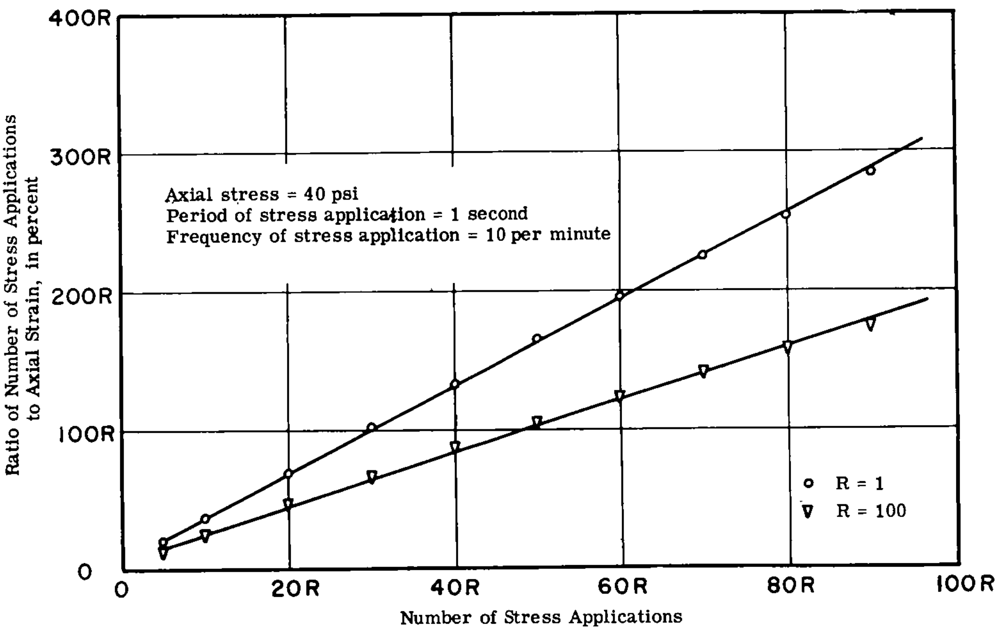

Figure 34. Axial strain vs number of stress applications (Seed, Chan, and Monismith).

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DISCUSSION

G. RAGNAR INGIMARSSON, *Research Assistant, Soil Mechanics Laboratory, University of Michigan, Ann Arbor.*—This discussion is limited mainly to the authors' conception of Eqs. 6 and 8 and the use of experimental results in support thereof.

An explicit form of the F/A vs x/C relation for bearing plate tests is

$$\frac{x}{C}\frac{A}{F}=a+b\frac{x}{C} \qquad (6)$$

Specific bearing plate tests by Dix and Lucas are shown in Figure 9, and the results plotted as $\frac{x}{C}\frac{A}{F}$ vs $\frac{x}{C}$ in Figure 10. The writer has taken

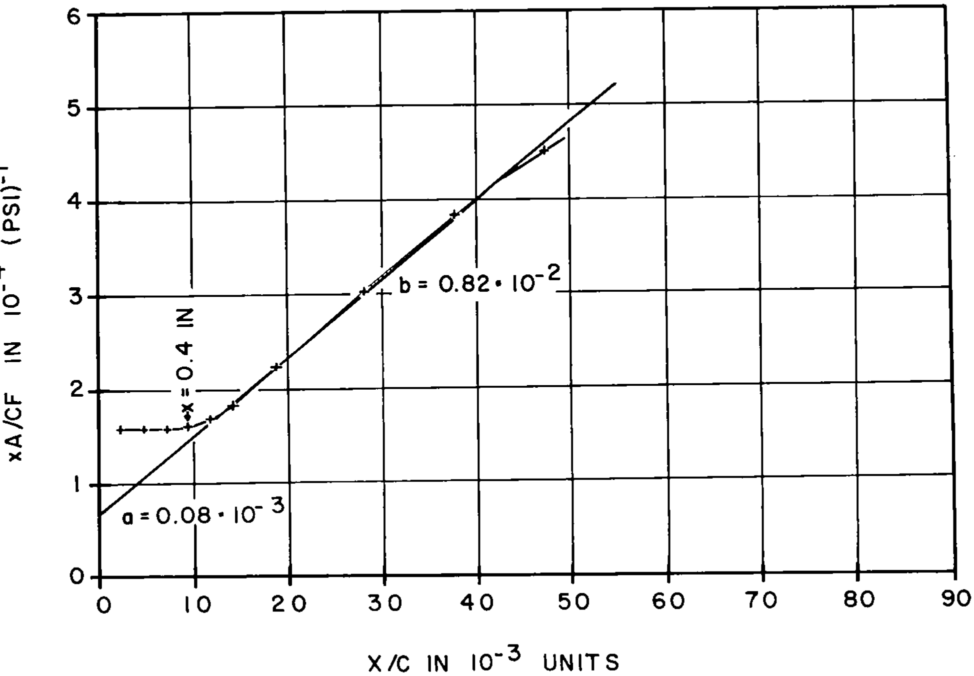


Figure 35.

the results for Plate 1 as presented in Figure 9 and given the values for x, x/C, and A/F in Table 3. Values of $\frac{x}{C}\frac{A}{F}$ vs $\frac{x}{C}$ are then plotted in Figure 35.

Comparing Figure 10 and Figure 35, it cannot be overlooked that the first point plotted in Figure 10 corresponds to $x=0.4$ in., with all values of $x<0.4$ in. thus being neglected. Referring to Figure 35, it is evident that points from $x=0$ in. up to $x=0.5$ ft would be much better represented by an entirely different straight line from that shown in Figure 10.

TABLE 3[1]

x (in)	x/C	A/F
0.1	0.00236	0.067
0.2	0.00472	0.033
0.3	0.00708	0.022
0.4	0.00943	0.017
0.5	0.01180	0.014
0.6	0.01416	0.013
0.8	0.01886	0.012
1.2	0.02832	0.011
1.6	0.03772	0.010
2.0	0.04718	0.009

[1] $D=13.5$ in.; $C=42.4$ in.; $1/C=0.0236$.

Because Eq. 6 can be rewritten

$$\frac{A}{F}=\frac{C}{x}a+b \qquad (15)$$

representing a straight line when A/F is plotted vs C/x, values of a and b could be obtained as the slope of the line and the intercept with the vertical axis, respectively. This plot has the advantage over the $\frac{A}{F}\frac{x}{C}$ vs $\frac{x}{C}$ plot in that the values plotted do

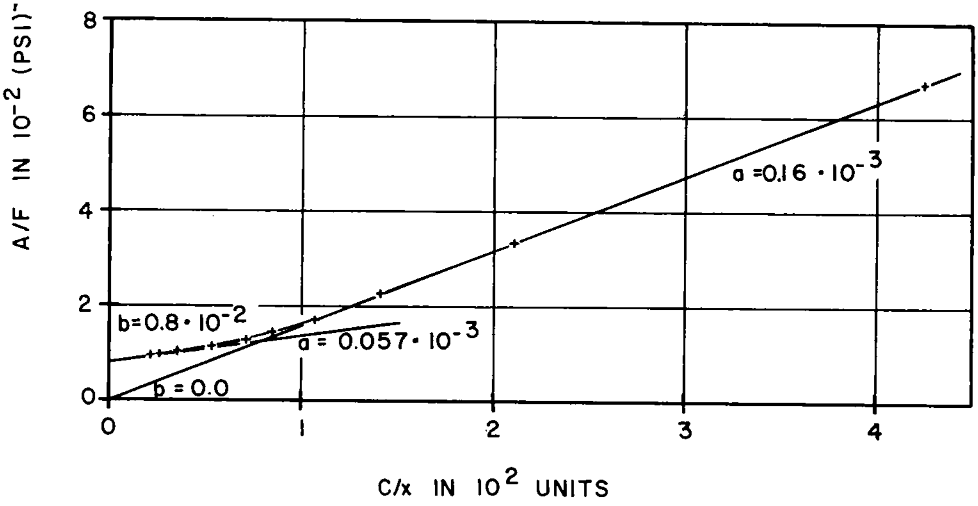


Figure 36.

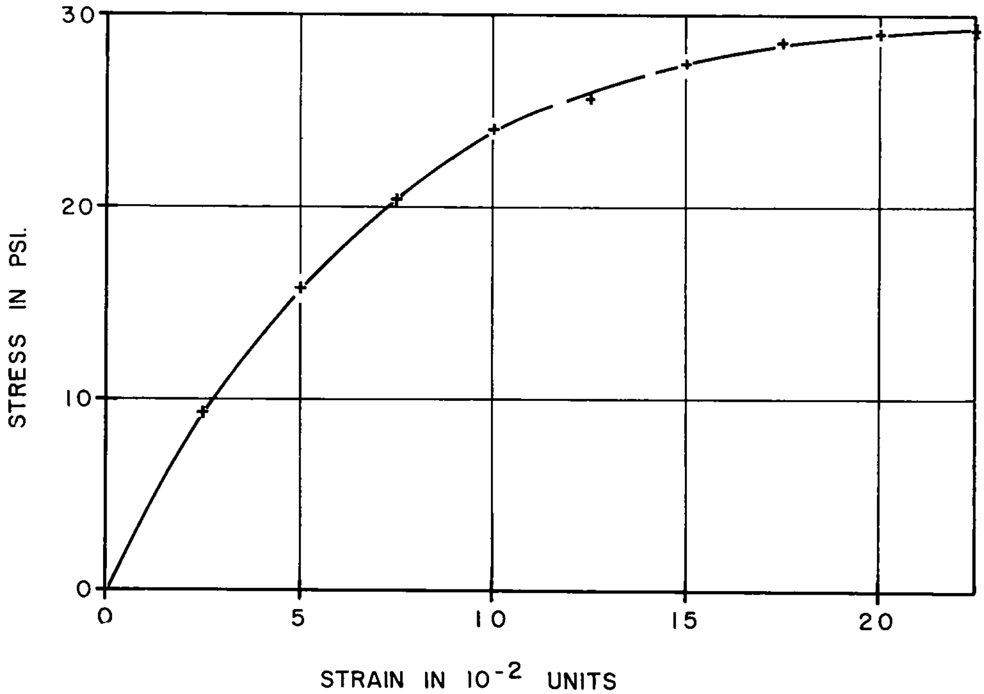


Figure 37.

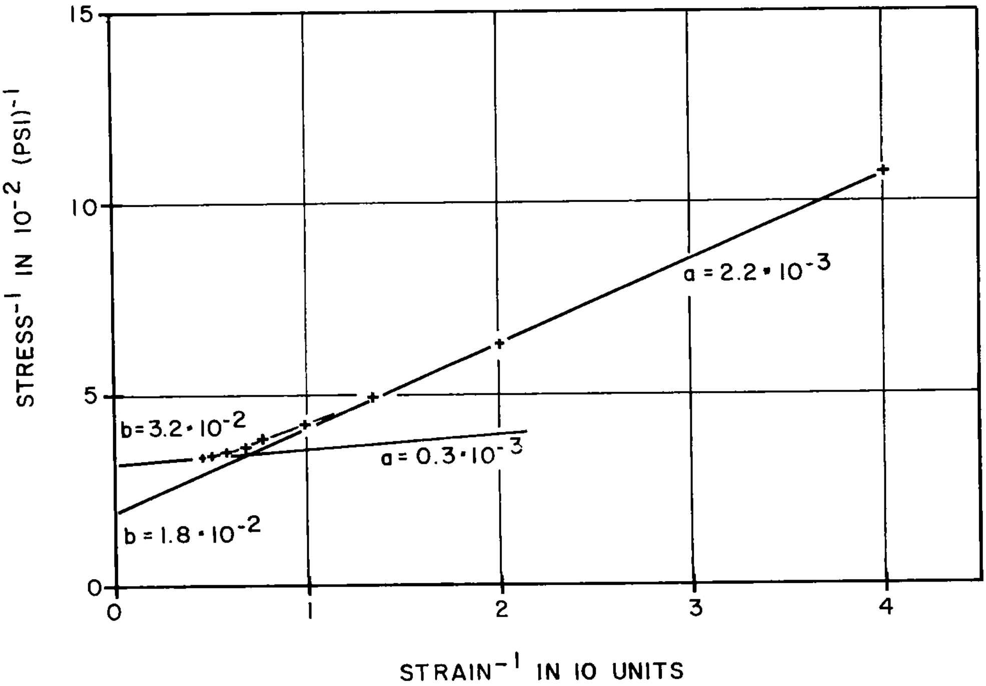


Figure 38.

not have a common factor, x/C, hence are less liable to give a deceiving picture of the correlation between the variables.

The data from Figure 9 have been replotted in Figure 36 as $\frac{A}{F}$ vs $\frac{C}{x}$. It will be noted that the points do not represent a straight line but rather a curve. Tangents to this curve would give values of a anywhere from 0.057 $\times$ 10^{-3} to 0.16 $\times$ 10^{-3}, and of b from 0 to 0.8 $\times$ 10^{-2}.

The authors further suggest that the stress-strain relation obtained from laboratory testing may be represented by

$$\sigma=\frac{\varepsilon}{a+b\varepsilon} \tag{8}$$

In support of this suggestion, results from an unconfined compression test are shown in Figure 11 as $\frac{\varepsilon}{\sigma}$ vs ε. A straight line is drawn through the points, giving b and a as the slope of the line and intercept with the vertical axis, respectively.

For the purpose of reference, the stress-strain curve for Plate 1 for the unconfined compression test has been reconstructed on the basis of the data given in Figure 11 and is shown in Figure 37.

Reviewing the data in Figure 11, it is felt rather questionable to represent the plotted points by a straight line inasmuch as they consistently form a smooth curve rather than being scattered at random around a straight line. To further demonstrate this point, Eq. 8 can be rewritten as

$$\frac{1}{\sigma}=a\left(\frac{1}{\varepsilon}\right)+b \qquad (16)$$

and the results plotted up as $\frac{1}{\sigma}$ vs $\frac{1}{\varepsilon}$.

This has been done in Figure 38 for the same points as shown in Figure 11. Depending on which portion of the curve is considered, straight lines represented by a and b varying in value from 0.3×10^{-3} to 2.2×10^{-3} and from 1.8×10^{-2} to 3.2×10^{-2}, respectively, can be selected.

On the basis of this discussion, the writer concludes that the authors' Eqs. 6 and 8 do not adequately describe the entire range of the load-deflection or stress-strain relationship. Particularly, the equations as presented are not valid over the initial portion of the load-deflection curve. It is the initial portion of this curve which is of concern to the designing engineer.

ROBERT L. KONDNER AND RAYMOND J. KRIZEK, *Closure*.—The authors wish to thank Mr. Ingimarsson for the opportunity to demonstrate the applicability of the analysis presented. It is perhaps appropriate to state at the outset that no "exact" fit of experimental data is presented in this paper nor was there the intention to do so. Because of the complexity of soil as a structural material with its observed viscoelastic type of response and the general difficulty of problems involving the interaction of soil-solid systems, as well as the complications that arise as a result of current test procedures (discussed under "Rate Effects") plus experimental error, it is unreasonable to expect an "exact" fit to assume such a simplified analytic nature as a two-constant hyperbola. Eqs. 6 and 8 have been proposed by the authors as a reasonable fit over a large range of the deflection parameter for the experimental bearing plate and stress-strain test data examined.

Regarding the Dix and Lukas test data for Plate 1, Mr. Ingimarsson points out in Figure 35 that all of the experimental points do not lie exactly on a straight line in the hyperbolic test plot. This is evident in Figure 10. In Figure 36, he replots this same data in what he refers to as a less "deceiving picture of the correlation between the variables" and then proceeds to point out the variety of straight lines that may be passed through these data. First, in Figure 36, as the force, F, and the deflection, x, go to zero, the variables plotted go to infinity; thus, the test data from $x=0$ to $x=0.3$ in. in Figure 9 lie in the abscissa range from 1.41 units to infinity in Figure 36 whereas the remaining 88 percent of the data is plotted in the relatively small region from 0 to 1.41 units. The authors feel, therefore, that Figure 36 represents the more deceiving plot and submit in support thereof the fact that it becomes much more difficult to exercise objectively the engineering judgment required to fit the data with the "best" straight line. The prevailing tendency with Figure 36 is to weight the relatively small portion of the initial data (low values of force vs deflection) too heavily because it extends to infinity. As an example, if the "best straightline" fit of the data in the Figure 36 were chosen as the line represented by $a=0.16\times10^{-3}$ and $b=0.0$, one of the possibilities suggested by Mr. Ingimarsson, this would imply that the ultimate stress F/A which may be applied to Plate 1 would be infinite $\left[\left(\frac{F}{A}\right)_{ult}=\frac{1}{b}\right]$. In his discussion, Mr. Ingimarsson has emphasized the initial portion of the data and his plot of A/F and C/x

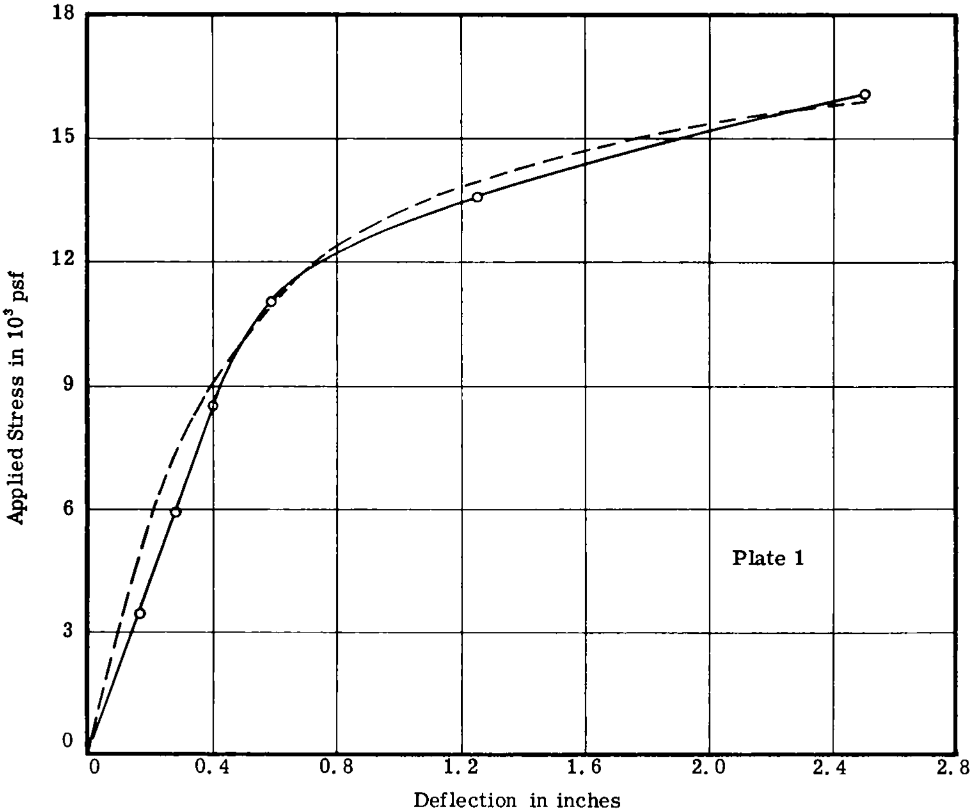


Figure 39.

strongly reflects this emphasis; however, careful examination of the nature of a hyperbolic fit will reveal that any tendency to emphasize the significance of data in the initial region will be accompanied by a corresponding overestimation of the ultimate values and this is, of course, on the unsafe side, as has been previously evidenced. In addition, the data from the initial portion of a test are often less reliable than the data from the remainder of the test due to such uncertainties as seating effects. This is particularly true for the data from Plate 1 inasmuch as the test was conducted 20.6 ft beneath the ground surface where control was less rigid. The "best straight line" as selected by the authors yields a prediction curve as shown by the dashed line in Figure 39 as compared to the actual data represented by the solid line. In light of the explanation presented, it is felt this provides a reasonable representation of the actual results obtained.

Essentially similar comments may be expressed concerning Mr. Ingimarsson's discussion of the stress-strain data presented in Figure 11 for Plate 1. Figure 38 is analogous to Figure 36 in the method of plotting. The results of the authors' fit for these data (from Fig. 11) is superimposed on Figure 37 and shown

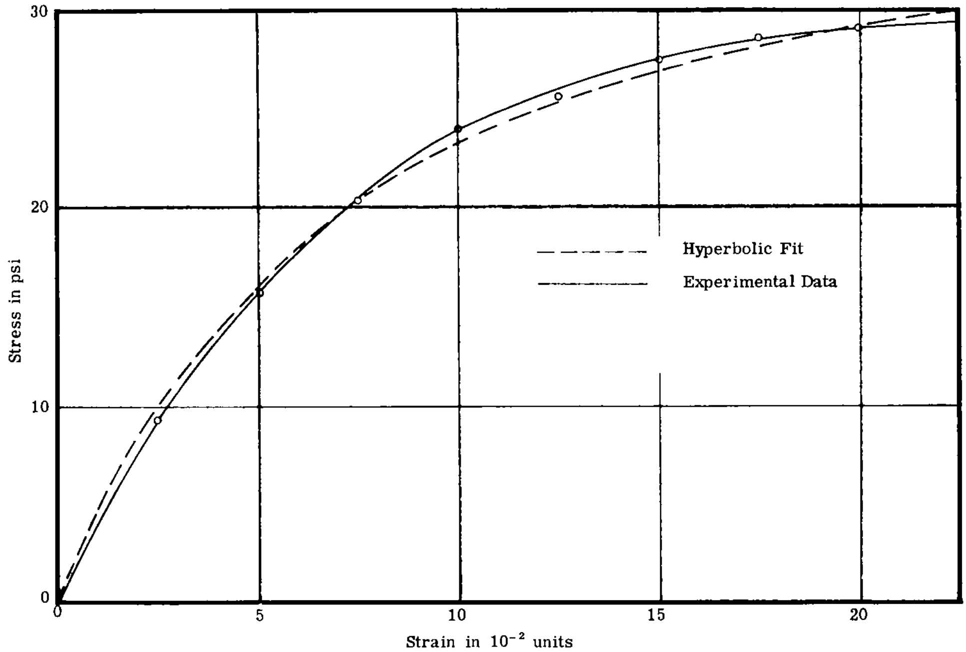


Figure 40.

as Figure 40. The close agreement between actual test data and the proposed hyperbolic fit is apparent.

In summary, Eqs. 6 and 8 are not suggested as "exact" fits for all test data, but only as good approximations over a large range of the deflection parameter for the data examined. This is felt to have been demonstrated in Figures 39 and 40, hence the authors feel the conclusions of the paper to be justified.

Construction of a Fill by a Mud Displacement Method

WILLIAM G. WEBER, JR., *Associate Materials and Research Engineer, Materials and Research Department, California Division of Highways, Sacramento*

A highway fill was constructed across an open water cove on the west side of San Francisco Bay by displacing the underlying soft mud to depths of as much as 60 ft by the weight of the placed fill. Various construction methods were attempted and a method of obtaining reasonably uniform mud displacement developed. The fill failures were analyzed and the factors affecting these failures evaluated. The measured settlements and pore pressures are presented and evaluated. The performance of the highway after three years of use is given.

• A FILL was constructed across Candlestick Cove on the west side of San Francisco Bay, displacing up to 65 ft of soft mud. The first two contracts were constructed on an experimental basis to determine how to construct the fill, and the freeway completed under four more contracts.

The soil formation in the open water area of Candlestick Cove consisted of unconsolidated sediments of recent geological age, the upper layer of which was a very soft mud with its surface at about elevation —5. The bottom of the soft mud varied from elevation —40 to —80. This soft mud was underlain by a somewhat stiffer material consisting variously of clayey sand, sandy clay, or clay. Bedrock varied from elevation —110 to —220.

The freeway alignment was 11,600 ft across the open water section. Various alternates were considered during design (*i.e.*, bridge, mud stripping, and sand drains), all of which would have been expensive. The California Highway Commission authorized in 1952 the construction of an experimental fill southward from the north end of Candlestick Cove to determine the feasibility of constructing an open water fill by end dumping methods. This was successfully accomplished for the conditions at this location. In June 1953, a contract was let to construct an experimental fill where a greater thickness of soft mud existed to study construction techniques further. The remaining portion was then constructed under normal construction contracts. Figure 1 is a general view of the area.

PROPERTIES OF BAY MUDS

The moisture content of the soft bay mud varied from an average of 90 percent at elevation —5 to 60 percent at elevation —60. The individual moisture tests were scattered in a random manner over a range of 20 percent with the moisture tending to decrease with depth as the only evident trend.

Figure 1. General view of open water fill, final grading operations in progress.

The samples obtained before construction indicated that the shearing strength of the soft bay mud was near zero at elevation —5, and increased an average of 14 psf per ft of depth.

The consolidation tests indicated that the soft bay mud was fully consolidated under its loading prior to construction. A coefficient of consolidation of 3×10^{-3} sq ft per hr was obtained for conditions existing after completion of the working table.

The vertical permeability of the soft bay mud was 1.1×10^{-5} ft per hr and the horizontal permeability was 2.2×10^{-5} ft per hr, after construction of the working table.

The liquid limit of the soft bay mud varied from 50 to 60 and averaged approximately 55. The plastic index varied from 25 to 35 and averaged approximately 30.

The soft bay mud was underlain by a stiff mud layer with sand lenses or

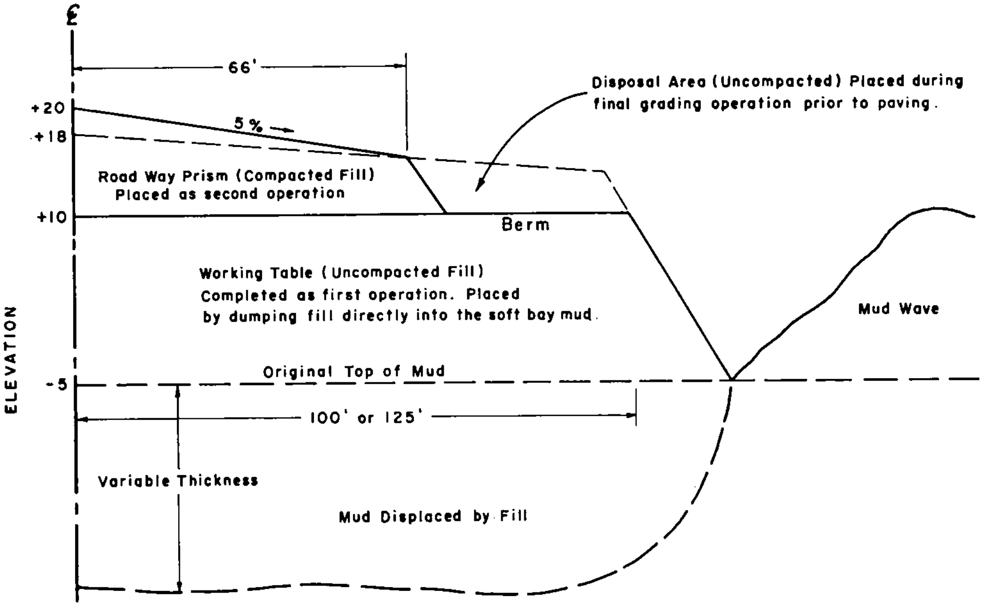


Figure 2. Typical cross-section of fill.

layers. The stiff mud layer had a shearing strength in excess of 1 ton per sq ft. Preconsolidation had occurred, perhaps due to desiccation, in recent geological times. The stiff mud layer had sufficient strength to support the fills; however, some settlement of the fill could be expected due to this layer.

CONSTRUCTION OF THE WORKING TABLE

The fill as constructed consisted of two parts, as shown in Figure 2. A working table was placed by end dumping methods and was in the uncompacted condition. A smaller compacted prism was placed on the working table and referred to as the roadway prism.

The requirement of the working table was to provide a stable platform on which to place the roadway prism. This could be accomplished by placing a wide working table, using the extra width as berms. During construction of the first unit, it was found that any height of working platform above the tide would result in large failures. It thus appeared that a stable platform could be built by using the weight of the fill to displace the soft bay mud. During construction of the first unit, mud displacement was attempted and found feasible with a reduction in the width of the working table from 400 to 250 ft. During the second unit, the construction methods were further studied and a method of operation developed where near total mud displacement was obtained.

The working table was constructed by end dumping methods, consisting of placing the fill material as close to the edge of the fill as practical and using a bulldozer to push the dirt over the side. Sections of the fill near the edge would then fail, then additional fill would be placed in the failed areas restoring them to the desired grade. The nose of the fill was maintained in a wedge shape by pushing the mud laterally. The displaced mud then formed large mud waves on each side of the fill.

The studies from the first and second units indicated several items that affect the displacement of the soft mud by the fill. The various items affecting the displacement of the soft mud were the shape of the nose, rate of placing fill, type of fill material, elevation to which the working table was carried, effect of the tide, and extent of the mud wave.

The use of a wedge-shaped nose, with the sides of the wedge at about 30 to 45 deg to centerline, effectively moved the soft mud laterally producing a reasonably uniform mud displacement up to about 100 ft from centerline. A long, slender extension of the nose along centerline resulted in large mud displacement on centerline and small mud displacement at the sides. A blunt nose resulted in a vertical sawtooth type of displacement. The shape of the nose tended to control the evenness of the mud wave built up around the nose and controlled the shape of the cross-section of the bottom of the fill as later determined by borings.

The desirable rate of placing fill was one that would maintain the top of the working table at the desired elevation and still slowly advance the fill. The 12,000 cu yd per day average used was sufficient for this purpose.

The type of fill material used had an apparent effect on the starting of failures. Rocky fills did not fail as readily as fine-grain fills. Once a failure had started, the type of fill did not appear to have any effect on the rate at which failure occurred.

The elevation of the top of the working table was related to the driving force available for displacing the soft bay mud. It was found that to keep the mud wave moving, the top of the working table had to be 3 to 5 ft above the crest of the mud wave. The planned elevation of the top of the working table, elevation +10, was increased to as high as elevation +18 to displace the mud where high mud waves occurred.

The strength of the soft bay mud was reasonably uniform in the area of the open water fill and exerted a constant effect on the fill failures.

The tide varied at Candlestick Cove, from a maximum of elevation +8 to a minimum of −2. The weight of the water at high tide acted to resist failure of the fill by supporting the mud wave. As the tide stage dropped, the mud wave would fail flowing away from the fill generally causing the edges of the fill to drop. The tide drop that resulted in fill movement was about 3 to 5 ft.

The mud wave was formed as the fill displaced the soft mud. The mud waves that were formed extended 500 to 700 ft from centerline, depending on the amount of soft mud displaced. These mud waves had a crest 20 to 40 ft from the top edge of the fill. The mud wave crest varied from elevation +5 to +16. The back of the mud wave sloped on about a 5 percent grade to elevation +2 and then extended out at about a 2 percent grade.

The mud wave acted as a berm or support for the fill, greatly increasing the stability of the fill. In the completed fill section, with the roadway prism in place, stability analysis indicated that the mud wave contributed about 30 percent of the resisting force.

The tide had considerable effect on the height of the crest of the mud wave. As the tide stage rose, the crest of the mud wave would build up if failure of the fill occurred. As the tide stage decreased, the mud would start failing within itself and gradually tend to flow away from the fill.

The strength of the mud wave was determined after the fill failures. The shearing strength of the soft mud in the crest of the mud wave was found to vary from 0.05 to 0.30 ton per sq ft depending on the type of failure

Figure 3. Nose of fill dropped about 7 ft (see Fig. 4).

that had produced the mud wave. This strength was approximately uniform to a depth of about elevation —20 and then the strength increased as before construction.

As the work progressed, construction procedures were varied to obtain maximum mud displacement. This was done by varying the factors affecting the mud displacement. A record of the amount of fill yardage placed at each station was kept so that it would be possible to estimate the amount of mud displaced. The major factor affecting the failures, and as a result affecting the mud displacement, was the height of the crest of the mud wave. The strength of the mud in the mud wave was reduced by mud blasting so that the mud wave would tend to flow away from the fill. During periods of high tide and/or high crests of the mud wave, the elevation of the top of the working table was raised in the unstable area. The rate of advance of the nose of the fill was maintained between 10 and 30 ft per day to keep the mud moving uniformly. This method of controlling the mud displacement was successful in obtaining 80 percent or greater mud displacement under the roadway prism.

FAILURES DURING CONSTRUCTION OF THE WORKING TABLE

As the nose of the fill was advanced from the stable portion of the working table, a crack would develop near the stable portion as shown in Figure 3. Regardless of how far the fill was advanced past this crack, movement would continue to occur at this location until the nose had become stable. Then the cracking would

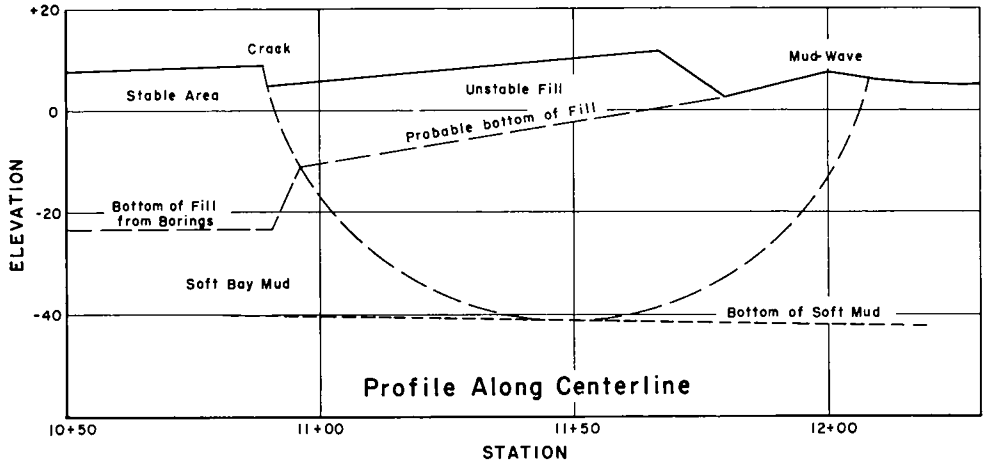


Figure 4. Typical shear-type failure.

advance to the point where the fill was again unstable. Studies were made on the failures by measuring the surface movements and running shear circles. From these measurements two primary types of movement were found to occur: (a) shear type—a rotary movement where the fill would have a large drop at the crack and little or no movement at the edge of the fill, as shown in Figures 4 and 5a; (b) squeeze type—a movement where the drop would be uniform over the failed area with a small horizontal movement occurring, as shown in Figures 5b and 6. The normal type of failure was the shear type. The mud was forced up about 20 to 40 ft from the fill and appeared to move in a vertical direction. The surface of the crest of the mud wave had a rough appearance. Occasionally a vertical face appeared on the side of the mud wave away from the fill. These shear failures appeared to follow a rotational movement.

On the few occasions that the fill was rapidly advanced, with small mud displacement, an unusual type of failure occurred that has been referred to as a squeeze-type failure. Generally, some factor (such as an extremely large build-up of the mud wave, a period of high tides, or an extended period where the nose of the fill was not advanced and then rapidly advanced) caused this condition to occur. The failed portion of the fill would crack in all directions and the settlement would occur rather uniformly. The failed area tended to settle at a slow, steady rate. The mud was pushed out from under the fill at an angle with a smooth appearance, and a high mud wave was not built up with this type of failure. When the soft mud was displaced to elevation —20 to —30, the movement would cease with the fill becoming stable. In analyzing this type of failure it was assumed that a sliding action occurred in the soft mud.

There were many factors affecting the fill failures, as previously discussed. These factors were varied to obtain maximum mud displacement. The failure resulting was of a modified shear type. In this type of failure, the fill would crack in all directions with a large drop at the edge of the fill and a small drop near the stable portion of the fill. The stable portion of the fill advanced at about the same rate as the nose of

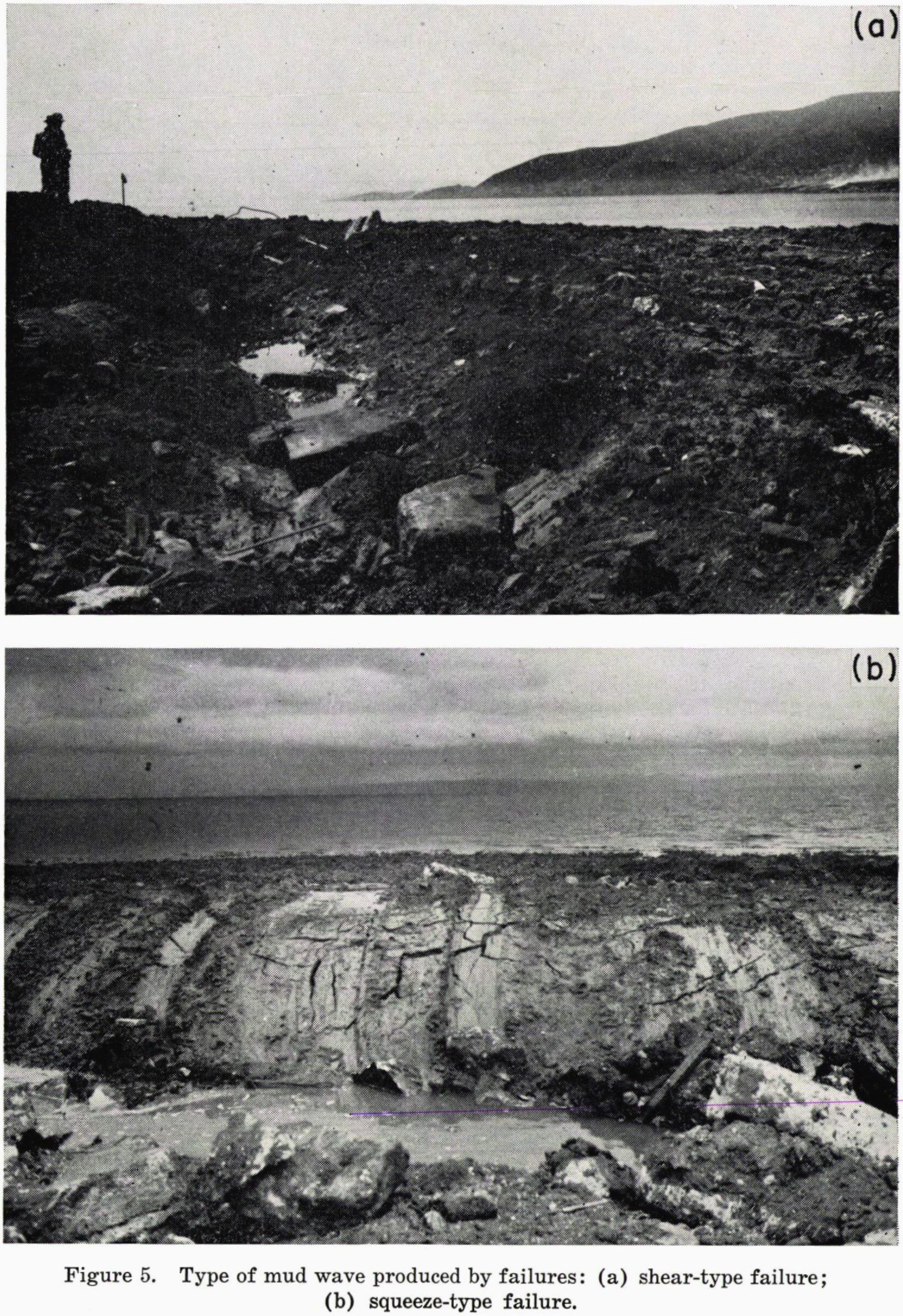


Figure 5. Type of mud wave produced by failures: (a) shear-type failure; (b) squeeze-type failure.

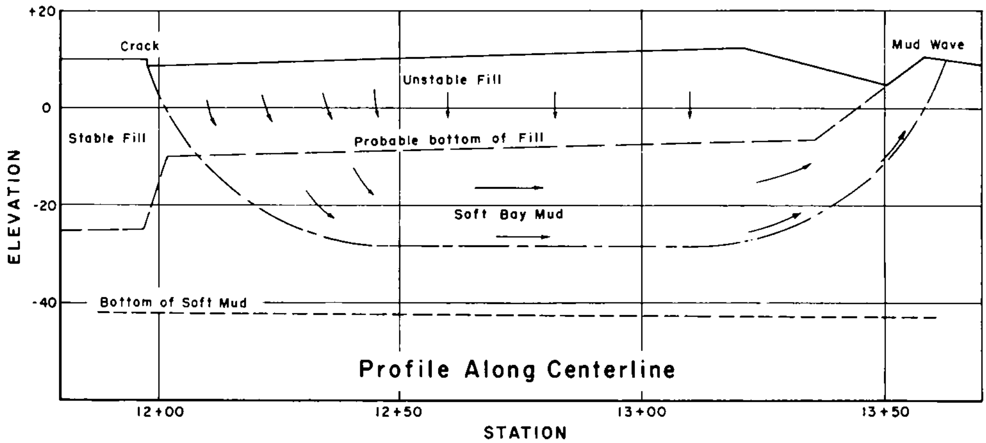


Figure 6. Typical squeeze-type failure.

the fill. This type of failure appeared to be a series of shear circle movements.

Stability analysis of the shear-type failures indicated a shearing strength of the soft bay mud of 70 to 210 psf or an average of 155 psf. The stability analysis of the squeeze-type failures indicated a shearing strength of the soft bay mud of 25 to 75 psf when the sliding surface was assumed just below the bottom of the fill. A shearing strength of 200 to 300 psf was indicated when the sliding surface was assumed just above the stiff bay mud. The sliding surface was probably 5 to 10 ft below the bottom of the fill.

Samples of the soft bay mud obtained from the mud waves soon after the fill became stable, indicated shearing strengths ranging from 50 to 200 psf. The higher strengths were obtained where shear failures had occurred and the lower strengths where squeeze failures had occurred.

Borings through the working table, made immediately after its completion, indicated shearing strengths from 150 to 250 psf immediately below the fill with the strength increasing with depth. There was an appreciable reduction in shearing strength due to the remolding of the soft bay mud by the fill failures. The shearing strength of the laboratory remolded samples of the soft bay mud varied from 50 to 100 psf.

The borings made at various times after completion of the fill indicated the shearing strength of the soft bay mud gradually increased with time, as shown in Figure 7. The time required for the soft bay mud to regain its original strength depended on the degree of remolding that occurred in the soft bay mud, and varied from one to two years. As further consolidation has occurred, the shearing strength of the soft mud increased and now exceeds its original strength. This increase in the strength of the soft bay mud has improved the stability of the main fill.

DISPLACED MUD

Borings were made, as the working table was constructed, to determine the depth to which the mud was being displaced. The borings were made at each station on centerline and about 75 ft right and left of centerline. Based on these boring data, profile and cross-sections of the bottom of the fill were determined and typical

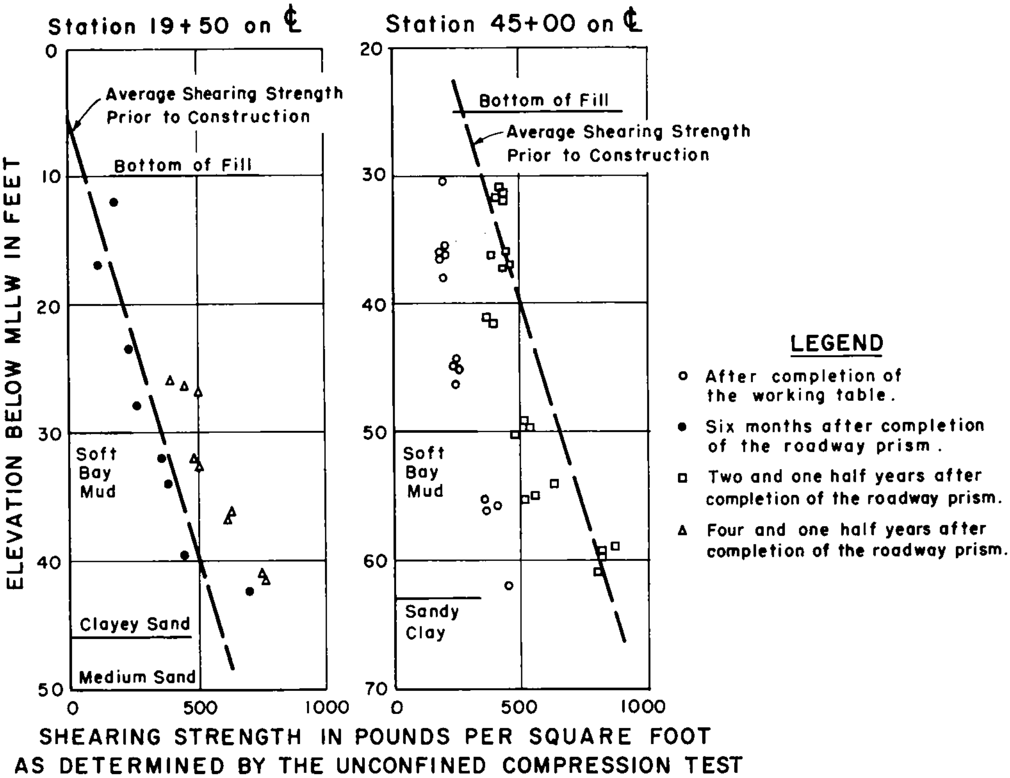


Figure 7. Shearing strength of soft bay mud at various times during and after construction of open water fill.

examples are shown in Figures 8 to 14.

Stations 6+00 to 60+00 had a great variation in the amount of displaced mud. There were several locations where the amount of soft mud remaining below the fill varied from 0 to 40 ft in a distance of 100 ft. These variations occurred in both the profile and cross-sections. The two experimental units are in this section of the roadway and account to a great extent for the variable amount of soft bay mud remaining below the working table.

The section of roadway from Stations 60+00 to 120+00 was the area where controlled mud displacement was used. Essentially total mud displacement was obtained in this area. There are fewer than 5 ft of soft mud remaining below the working table in about 50 percent of the locations where borings were made. Only one location indicated over 20 ft of soft mud remaining below the working table.

CONSTRUCTION OF THE ROADWAY PRISM

As the working table was completed, the roadway prism was placed on it. The roadway prism consisted of a 10-ft high fill placed in 8-in. compacted lifts, 130 ft wide on the top with 2:1 side slopes. Profile grade of the roadway prism was +20 as constructed. In the final grading operations, cuts and fills were made to

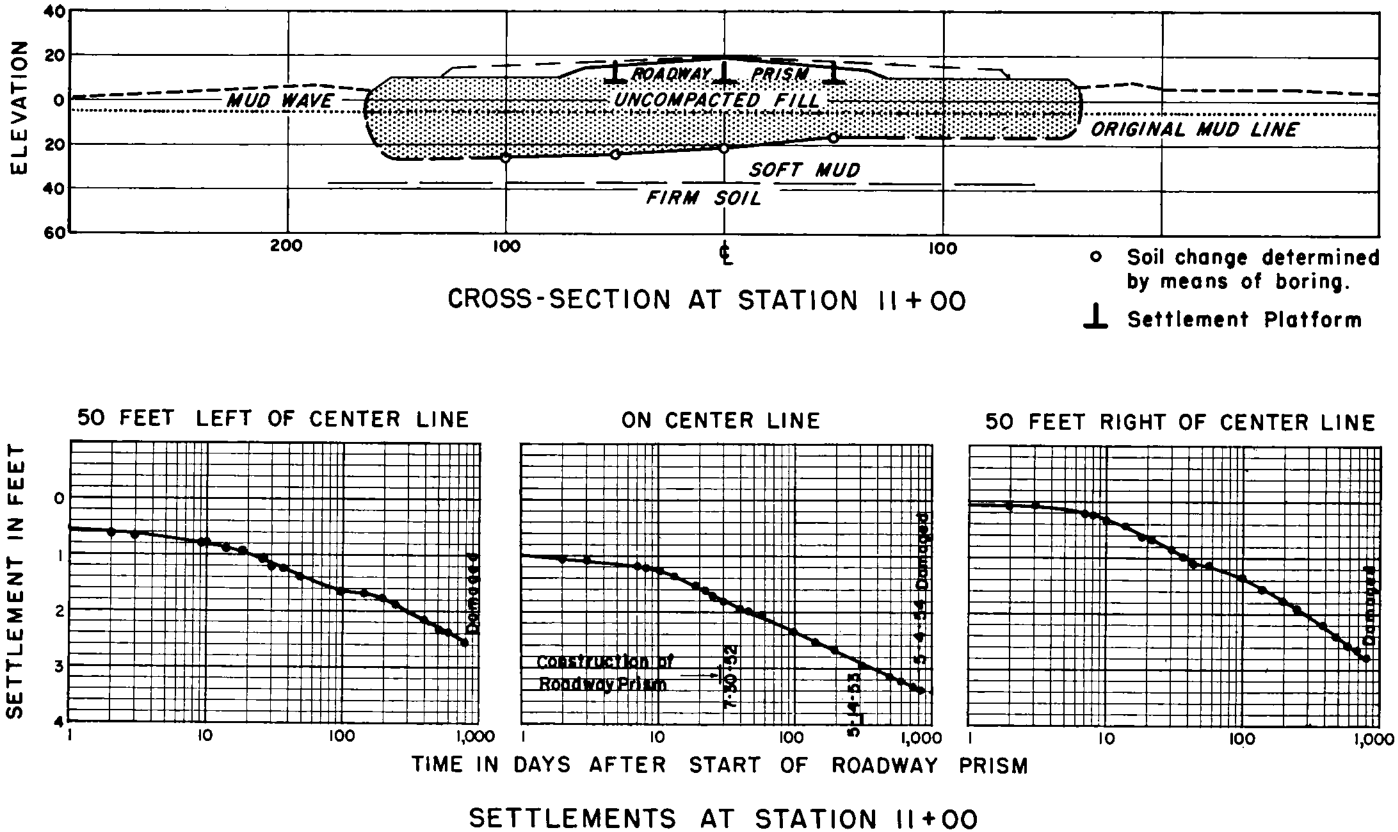


Figure 8. First unit, 300 ft wide.

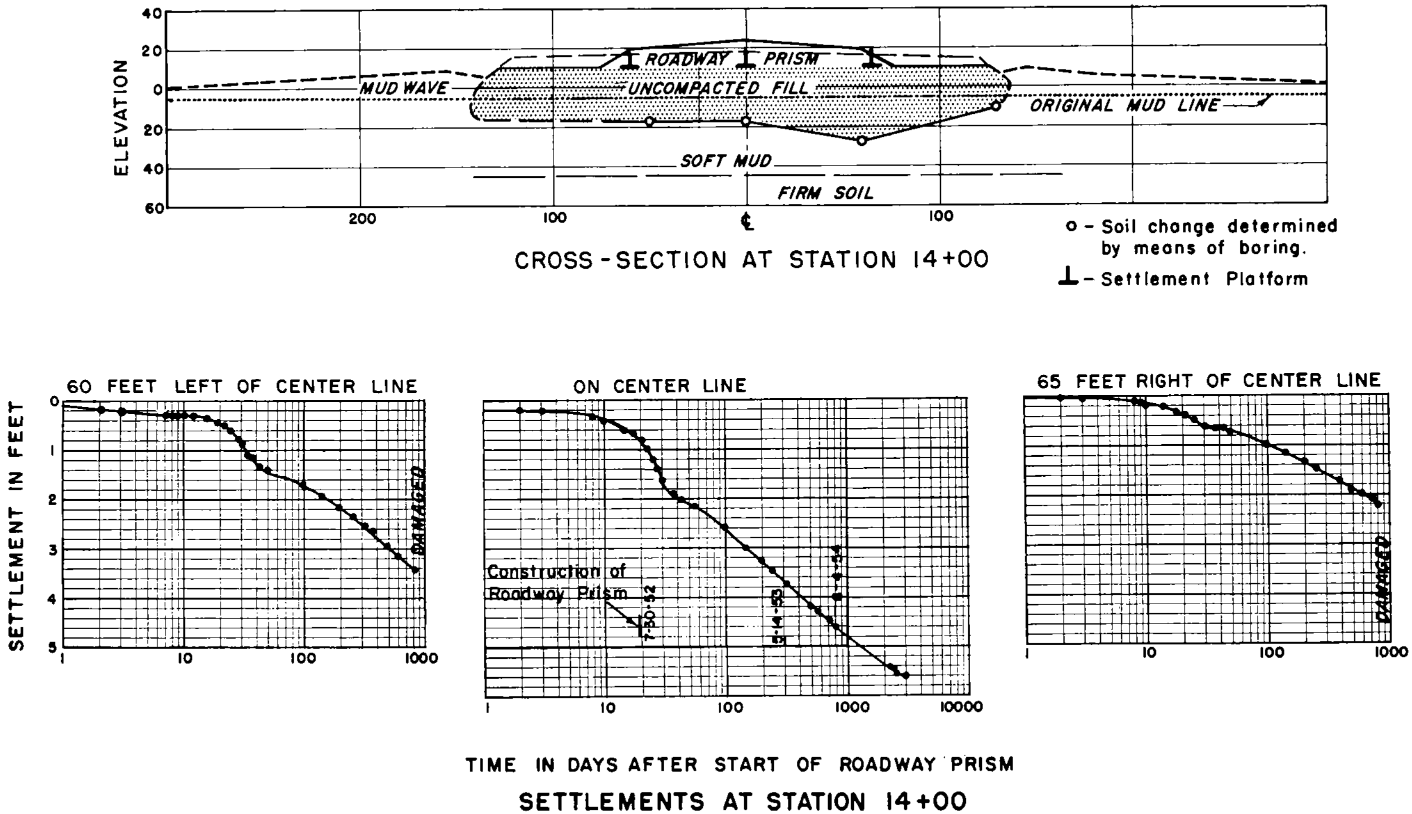
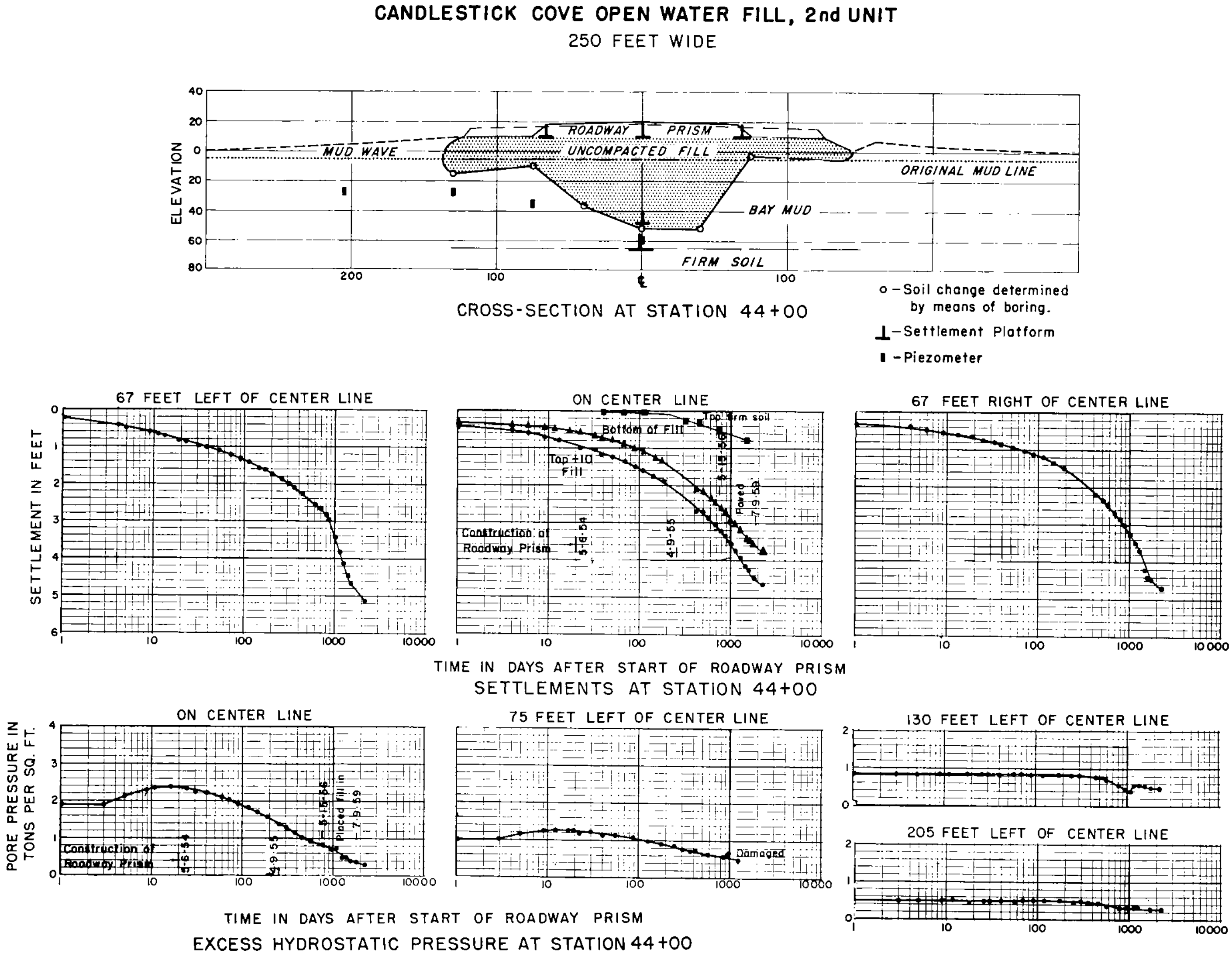


Figure 9. First unit, 250 ft wide.



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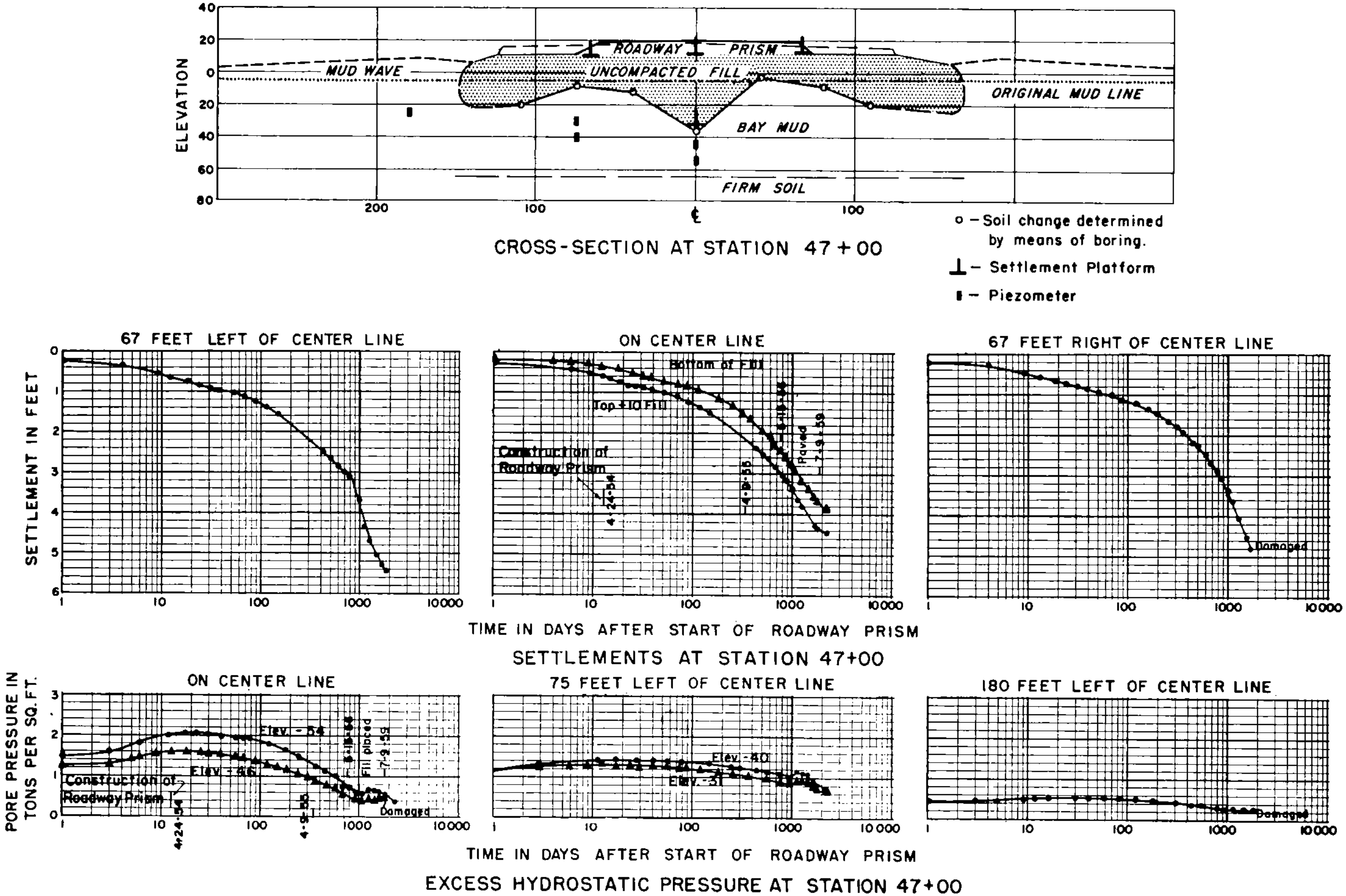

Figure 11. Second unit, 250 ft wide.

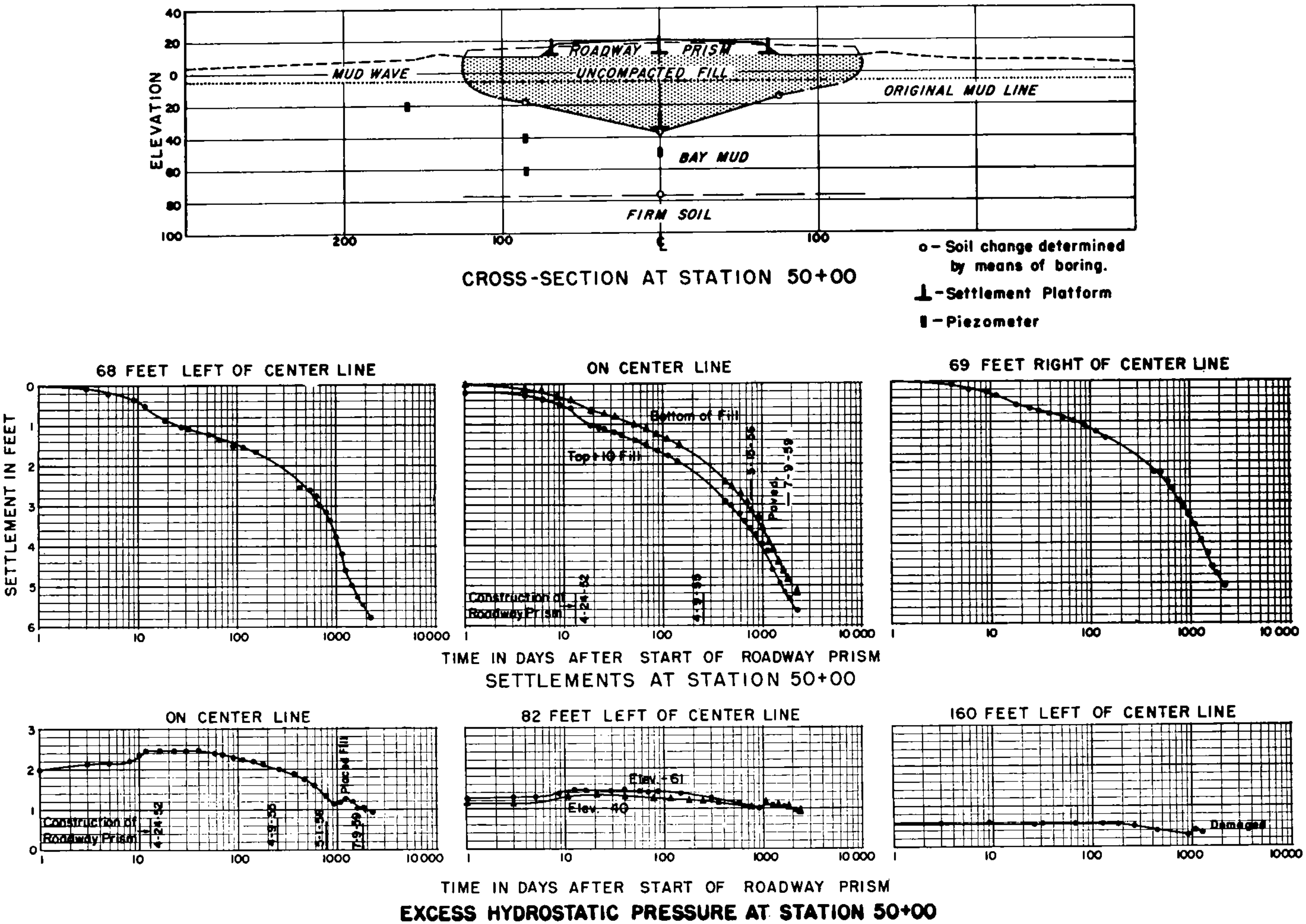

Figure 12. Second unit, 250 ft wide.

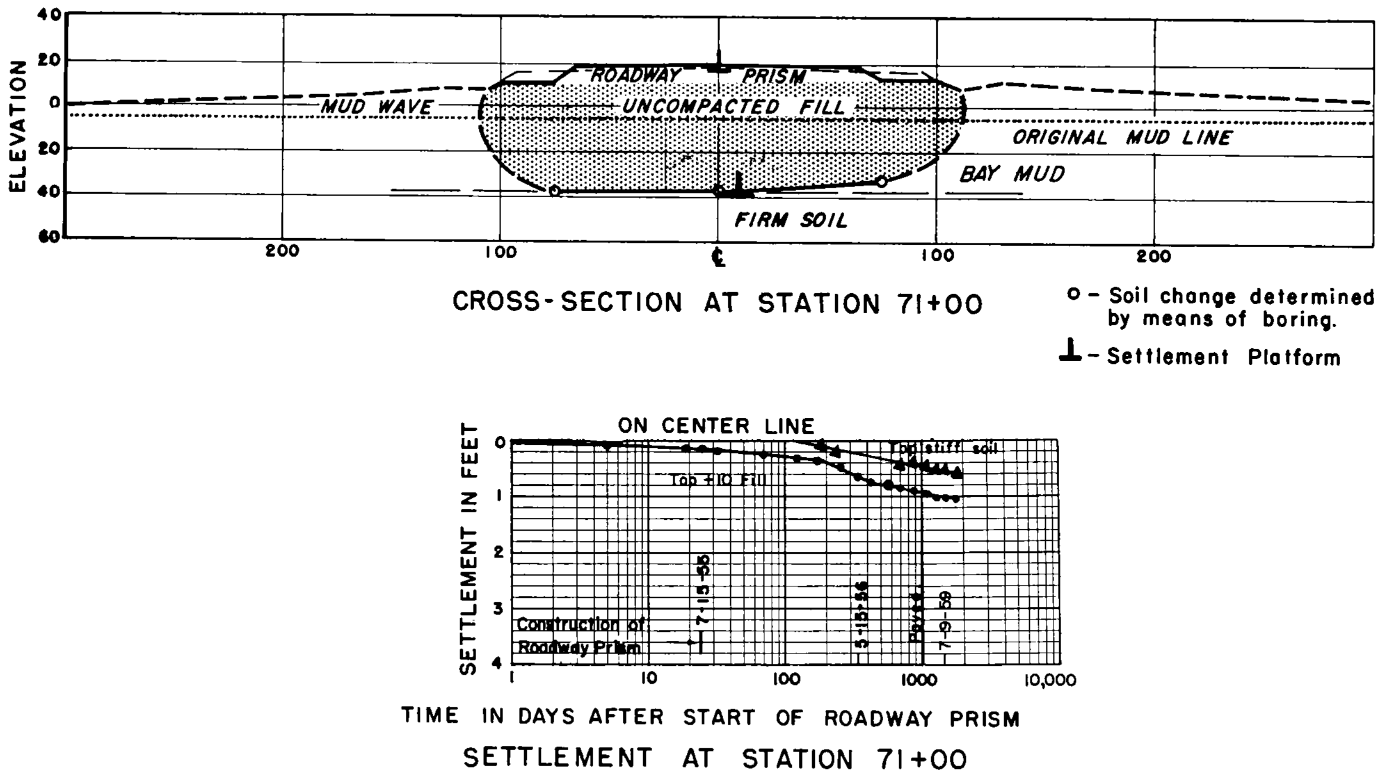

Figure 13. Third unit, 200 ft wide.

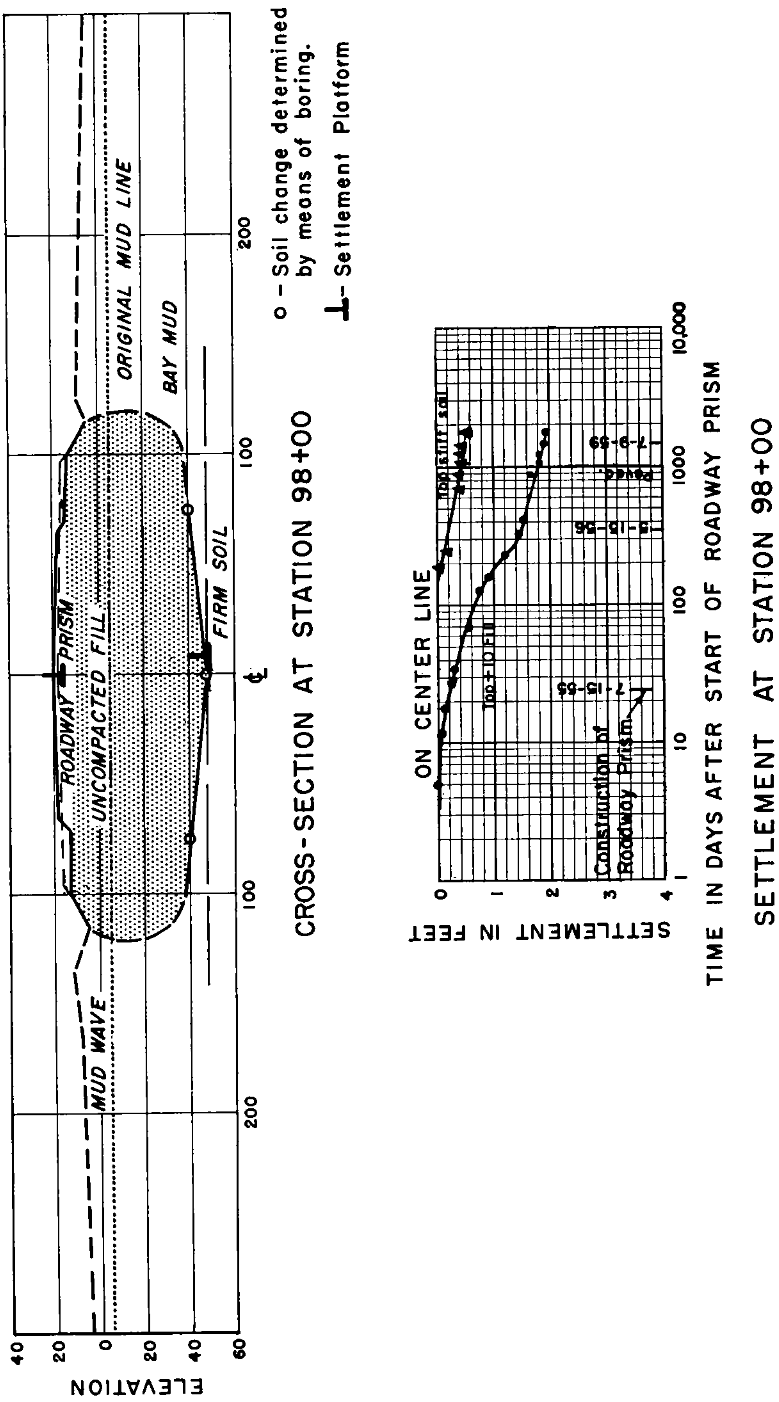

Figure 14. Third unit, 200 ft wide.

bring the profile grade to +18, with a 2-ft structural section specified.

In the two experimental units, hubs were placed 100 and 125 ft from centerline at each station to observe any surface movement of the berm during placing of the roadway prism. The settlement of these hubs was not accelerated by the placing of the roadway prism. There was about 0.1 ft of horizontal movement of these hubs away from centerline. Cracking was noted about 85 ft from centerline in the berm area during the placing of the roadway prism. These cracks were tension cracks due to the consolidation of the uncompacted fill under the loading of the roadway prism. There were no indications that shear failures occurred anywhere during the placing of the roadway prism.

At a few locations, the crest of the mud wave increased slightly in elevation and abnormally large settlements occurred 75 ft from centerline as the roadway prism was placed. At all of these locations the borings 75 ft from centerline indicated that the soft mud had been displaced to elevation—10 to —20. There were no indications of movement of the mud at other locations with the same range of mud displacement 75 ft from centerline. It is felt that such factors as the size of the mud wave, strength of the soft mud, and strength of the uncompacted fill accounted for these variations. These observations indicate that some plastic flow of the soft mud may have occurred where the soft mud was displaced to elevation —20 or less.

During the final work on the open water fill in 1957, there was a surplus of dirt. This fill was placed on the berm so as to make the berm level with the completed roadway. The fill in this disposal area is in an uncompacted condition. No failures occurred during the placement of this fill.

THEORETICAL SETTLEMENTS

There were three soil layers with varying degrees of compressibility at Candlestick Cove underlying the roadway prism. The layers were the uncompacted fill, the soft bay mud remaining below the working table, and the stiff bay mud underlying the soft bay mud. The settlement of the roadway prism was dependent on the rates of settlements of all three layers.

It was not practical to obtain undisturbed samples of the uncompacted fill; therefore, no theoretical settlement calculations could be made.

Theoretical rates of settlement of the soft bay mud were calculated for numerous locations. The settlement was calculated using the consolidation tests from the samples obtained after completion of the working table, and assuming double drainage. The theoretical settlement calculations indicate that the rate of settlement will be approximately uniform for varying periods after construction regardless of the thickness of the soft mud remaining below the fill. Where only a few feet of soft mud remain below the fill the settlement would be completed soon after construction. Settlement would continue where greater thickness of soft mud remained below the fill until the settlement was completed for that thickness of soft mud. The settlement log-time curves thus formed a close group following the same rate of settlement until the settlement was completed for various thicknesses of soft mud remaining below the fill. The differential settlement would thus be small until the completion of the consolidation for the least thickness of soft bay mud and then the thicker mud locations would continue to settle resulting in differential settlement.

Theoretical settlements of the stiff bay mud were calculated assuming double drainage even though the stiff bay mud is underlain by the Franciscan formations. The reason for this is that extensive sand layers or pockets exist in this stiff mud layer, and they probably provide some drainage. The calculated ultimate settlements of this layer are from 2 to 4 ft. The estimated theoretical rate of settlement is very slow, with an estimated 100 to 200 years being required for its completion.

MEASURED SETTLEMENT

As placement of the working table progressed, settlement platforms were placed on centerline at each station on the surface of the working table. A limited number of settlement platforms were also placed 75 ft right and left of centerline. These settlement platforms would measure the total settlement of the roadway prism. At five locations, settlement platforms were placed at the bottom of the fill and would measure the settlement due to the soft bay mud and stiff bay mud. Four settlement platforms were installed at the top surface of the stiff mud layer to measure the settlement due to this layer. Typical settlement curves are shown in Figures 8 to 14.

The total settlement curves from Stations 6+00 to 60+00 are quite similar. The slope of the log-time settlement curves is about the same and indicates that only small differential settlement is occurring between any two adjacent stations. The total settlement to date in this area is between 3 and 6 ft. The settlement since paving has varied from 0.3 to 1.5 ft and has produced long, rolling waves in the pavement. It is expected that this differential settlement will continue to increase for many years.

The total settlement curves from Stations 60+00 to 120+00 indicate that little settlement occurred during the placing of the roadway prism. The slope of the log-time settlement curve was constant during construction of the roadway prism and for about six months after its construction. About six months after construction of the roadway prism, the slope of the logtime settlement curve sharply increases for about three months, generally resulting in about one-half foot of settlement. The slope of the log-time settlement curve then, about the time of paving, assumes either of two slopes—a flat slope indicating little or no settlement, or a slope indicating minor settlements. There is no relationship to the amount of soft mud underlying the fill. A large mud displacement was obtained throughout most of this area with small settlements occurring after construction. The settlement in this area after paving varied from 0.1 to 0.6 ft with minor waviness in the pavement surface. The settlement appears to be completed in few areas.

The settlement due to the uncompacted fill has varied widely. The consolidation of the uncompacted working table was generally small and appeared to be completed within a year after construction. The rock fill consolidated about 0.1 ft and the soil fill about 1 ft under the loading of the roadway prism.

Settlements of the surface of the stiff mud were measured at only four locations, and because of these limited data, only general trends can be noted. The slope of the log-time settlement curves was small after completion of the roadway prism and then gradually increased. At two locations, the present settlement of the surface of the stiff mud has exceeded the estimated settlement for 20 years. At one location the settlement followed the estimated rate of settlement, and at the other location the settlement now appears completed. This settlement which varies from ½ to 2 ft, was probably due to the heterogeneous nature of the stiff bay mud.

Comparing the settlement after construction from Stations 60+00 to 120+00 of the surface of the roadway prism with the surface of the stiff bay mud, it was found that where near total displacement occurred, the two surfaces were settling the same amount. The settlement of the roadway prism was therefore directly related to the consolidation occurring in the stiff mud layer. With little or no soft mud remaining below the fill and the consolidation of the working table complete, the settlement of the roadway prism should be essentially the same as the settlement of the surface of the stiff bay mud. Where there was not total mud displacement, the settlement of the surface of the working table exceeds the settlement of the surface of the stiff bay mud layer.

Subtracting the settlement due to the stiff mud layer and the uncompacted working table from the settlement of the surface of the working table, it was found that the resulting settlement due to the soft bay mud was erratic from Stations 6+00 to 60+00. The nature of the erratic settlement was that in some areas the settlement was occurring faster than estimated, slower than estimated, or close to the estimated rate. Examination of the settlements and amount of soft mud remaining below the fill showed that, where near total mud displacement was adjacent to a small mud displacement area, the settlement in the high mud displacement area occurred at a higher than estimated rate. Where small mud displacement was adjacent to high mud displacement area, the settlement in the small mud displacement area occurred at a slower than estimated rate. This effect may possibly be due to arching action of the fill and extends up to 200 ft from the changes in soft mud displacement. In areas outside this arching effect, the settlement due to the soft bay mud is closely following the theoretical rate of settlement.

EXCESS HYDROSTATIC PRESSURE READING

Piezometers were installed below the working table in the two experimental sections of the open water fill. The piezometers were similar to the nonmetallic type developed by Casagrande for use at Logan Airport.

The piezometers were placed so as to obtain a half cross-section of the excess hydrostatic pressure: one or more on centerline, one or more 75 to 80 ft left of centerline near toe of roadway prism, and one under the crest of the mud wave. Four piezometers were installed in the sand to clayey sand between the soft mud and stiff mud layers. Typical excess hydrostatic pressure data obtained are shown in Figures 10 to 12.

During the period between installing the piezometers in the soft mud and the placing of the roadway prism, a slight decrease, less than 0.1 ton per sq ft, in excess hydrostatic pressure occurred. During the placing of the roadway prism, the loading on centerline was increased 0.6 ton per sq ft. This increase in weight was reflected by a 0.5- to 0.6-ton per sq ft increase in excess hydrostatic pressure on centerline and about 0.2-ton per sq ft increase in excess hydrostatic pressure 75 to 80 ft from centerline, but with no noticeable effect under the mud wave. After completion of the roadway prism, the excess hydrostatic pressure at centerline decreased slowly from about 2 to less than 1 ton per sq ft at the present time. The excess hydrostatic pressure outside the toe of the roadway prism, 75 to 80 ft from centerline, decreased at a slower rate than the excess hydrostatic pressure at centerline until the two were about equal, then they both decreased at about the same rate. The decrease in excess hydrostatic pressure under the mud wave has been small, 0.1 to 0.2 ton per sq ft, and has been very erratic. These erratic readings are believed due to tidal effects. The tide action has now destroyed all of these piezometers. The piezometers placed in the sand to clayey sand layer between the soft mud and stiff mud layers indicate a constant excess hydrostatic pressure of about 0.2 ton per sq ft in this layer.

The piezometers often indicated pressures that were in excess of the loading due to the fill directly above them. Before placing the roadway prism, the loading within 100 ft of the piezometers was investigated, and it was found that the piezometers tended to indicate the maximum pressure due to the working table within this distance. The result was a fairly uniform excess hydrostatic pressure that was not reflecting the extremes of the soft mud displacement.

The excess hydrostatic pressure measurements indicate that a rapid decrease in pressure occurred after placing the roadway prism in areas of large mud displacement until the pressures were about equal to the pressures below adjacent smaller mud displacement areas. Then the excess hydrostatic pressure in the two areas decreased at about the same rate. This effect appears to exist over an area of 100 to 200 ft and is not reflected in the rate of consolidation as measured by the vertical movements. It appears that two actions may be occurring that are affecting the settlement and excess hydrostatic pressure measurements. The excess hydrostatic pressures are being transferred in a horizontal direction up to 200 ft and the fill is acting as an arch or bridge for a distance of about 200 ft in the locations of small mud displacement.

CONCLUSIONS

The primary object of the two experimental sections at Candlestick Cove was to determine how to construct a stable fill across the open water section of this road. It was found that a 200-ft wide working table would successfully support the roadway prism without failures when the soft mud was displaced to elevation −20 or below.

During the early stages of construction the fill was placed without regard for the amount of soft mud being displaced. With this method of operation varying amounts of soft mud were trapped below the working table. A method of placing the fill was developed that effected almost total displacement of the soft mud by the fill. This total mud displacement was accomplished by controlling the shape of the nose, rate of placing and advancing the fill, elevation to which the working table was constructed, and height of the mud wave by mud blasting.

Differential settlement has occurred as a result of variable amounts of soft mud remaining below the working table. This differential settlement has produced a wavy pavement surface where large variations of the amount of soft mud exist. Where total mud displacement was obtained, only small differential settlement occurred.

ACKNOWLEDGMENTS

Acknowledgment is made to the Materials and Research Department, which is under the supervision of F. N. Hveem, and to District IV, under the supervision of J. H. Skeggs, retired, of the California Division of Highways, under whose direction this project was constructed. The author wishes to acknowledge the assistance given by A. W. Root, T. W. Smith, and W. Travis in conducting and planning the work presented.

A Preliminary Evaluation of Color Aerial Photography in Materials Surveys

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A recognized need has existed for evaluation of color aerial photographs as a tool for use in materials surveys. The opportunity to use color aerial photographs in a comprehensive materials survey presented itself in Yellowstone National Park (area about 3,472 sq mi), where a critical material shortage exists in certain portions of the area. This project was undertaken by the Federal Projects Office, Region 9, U. S. Bureau of Public Roads, and was sponsored by the U. S. National Park Service. Strip aerial photography flown along the major highway system at a scale of 1:6,000 was used for the most part. The color aerial transparencies obtained were studied stereoscopically.

Special considerations in the procurement of color aerial photography are presented, together with descriptions of field and office procedures. Comparisons are made between color and black-and-white aerial photographs. Tentative conclusions drawn from the results of the project are given.

• BLACK-AND-WHITE aerial photographs have been used by a number of State highway departments in conducting materials surveys and inventories. Private organizations and consultants engaged in mapping studies or exploration for construction materials have also used aerial photographs in the United States and in many other parts of the world. The Bureau of Public Roads, in addition to sponsoring projects in some States where aerial photographs are used for materials exploration, has conducted materials searches in Alaska and in several national forests and parks. The aerial photographs employed in these studies have been black-and-white prints available from various governmental agencies generally at scales of 1:20,000 or smaller. In some cases larger scale photographs have been used.

Although it has been considered for several years that color aerial photographs might be useful in materials surveys, their use has been delayed largely because of high cost, slow film speeds, and poor quality of color reproduction. The U. S. Geological Survey has for a number of years used color aerial transparencies for some of their geologic mapping and mineral exploration studies. The U. S. Coast and Geodetic Survey has used them for coastal mapping; the U. S. Army Corps of Engineers and other military organizations, for military intelligence studies, camouflage detection, and special terrain studies; and the U. S. Department of Agriculture, for detection of diseased crops and trees. Private commercial organizations and universities have experimented with color aerial photographs for other specialized uses.

In 1958, the Federal Highway Projects Office, Region 9 (Colorado, Utah, New Mexico, and Wyoming), Bureau of Public Roads, contracted to have some experimental color aerial photographs taken in Dinosaur National Park in Colorado and Utah. The results of this trial were promising, and possible advantages of using such photography for soil and materials surveys were recognized. Various color projects were later flown in Colorado, New Mexico, Utah, and Wyoming.

In 1961, the Bureau of Public Roads entered into an agreement with the U. S. National Park Service whereby Region 9, would conduct a comprehensive inventory of construction materials along the major highway system in Yellowstone National Park. This work was sponsored by funds provided by the Park Service. While accomplishing the objectives of the agreement for conducting a construction materials search, it was also planned to make a preliminary evaluation of the color photography in Yellowstone Park for use in materials surveys. Photography from other areas was also evaluated. Reconnaissance mapping of significant geologic units normally of concern in preliminary highway location and design was also to be done in conjunction with the materials search. Aerial photographic interpretation of special ground conditions was also to be performed in various areas.

PROCUREMENT OF AERIAL PHOTOGRAPHS

During the summer of 1961, the Bureau of Public Roads contracted to have 428 linear mi of color photography taken at a scale of 1:6,000 (1 in. to 500 ft) in single flight strips along the major road system in Yellowstone National Park. In addition, several segments were covered by two or more adjacent flight strips with standard sidelap to furnish adequate coverage. A number of side flights, mainly along minor rivers, were made where potential sources of material were suspected. Some oblique photographs were taken to supplement the vertical photographs. A small segment of the photography flown at 1:6,000 was also flown at 1:4,800 (1 in. to 400 ft) in order to make a comparison of the photographic detail. Black-and-white photographs (panchromatic) at a scale of 1:6,000 were taken for segments totaling 100 mi. Coverage in color at this same scale permitted comparison of the two types of photography. The black-and-white film used was Kodak Super XX Aerographic. Of the 428 linear mi contracted to be flown in color, 300 mi were flown during the summer of 1961. Difficulty in obtaining the required color film and inclement weather prevented the contract from being completed.

The aerial camera used to take these photographs was a Zeiss RMK 15/23 equipped with a Pleogon lens having a 6-in. focal length with a maximum aperture of f 5.6. The size of transparencies produced by this camera is 9 by 9 in. (23 by 23 cm). The average flight height above ground for 1:6,000 photography was 3,000 ft.

The color film used for this project had an exposure index of daylight-40. This is a reversal color film of the subtractive type which when processed gives a color positive transparency. Emulsion characteristics usually vary somewhat from roll to roll, as a result the exposure index as well as the color balance also varies. Because of this, it was necessary to make trial flights over the terrain for which photography was desired to determine the proper combination of shutter speed and lens opening that would give optimum exposure and color reproduction for a particular roll of film. A corrective color balancing filter was provided with each roll of film because of the variations in film characteristics.

Proper color balance must be obtained to insure relatively accurate color reproduction in the aerial transparency. To obtain this color balance and minimize shadows, aerial photographs were taken as near peak solar altitude as possible.

Haze filters are sometimes used in taking color pictures to prevent the over-all bluish hue that results from the dispersion of light by dust and water particles suspended in the atmosphere. Haze does not present a serious problem for low altitude photography, but its effect becomes increasingly apparent as the height of the aerial camera above ground is increased. The magnitude of haze also varies with time of day, season, and geographic location.

Because of the use of haze filters, the need for color balance and the slower speed of color film, aerial lenses that have larger apertures than those used in black-and-white photography must be used. Higher quality lenses having good resolution, little color or spherical distortion, and relatively even light distribution are required. It is expected with the development and use of faster color film that the lighting requirements and size of lens opening will be reduced.

Developing and processing of color film are far more exacting than for black-and-white and are extremely important in the production of acceptable color photographs. Composition of developing and fixing solutions and developing time must be carefully controlled. Temperature of developing solutions for example must be maintained within ±1 F. Considerable care must also be exercised while drying or handling the film. Special, rather costly equipment is needed for adequate processing.

Color film should be developed within 24 hr after exposing. Exposed undeveloped film should be refrigerated if it is impossible to develop it within 24 hr. Because of the tendency for the dyes in unexposed color film to change with time, particularly in hot humid climates, the film should be stored at temperatures ranging from 45 to 65 F.

The cost of color photographs should be considered not by itself, but in relation to actual savings in time, money, and increased information made available through their use. The added advantages and savings gained through the use of color are not always measurable. The costs involved in procuring either black-and-white or color aerial photographs are small, if not insignificant, compared to the final cost of constructing a highway. If a single good source of material is located for a project, or if the haul distance to a suitable source can be shortened appreciably, the cost of photography and the personnel involved may well be paid for many times. Similarly, the savings made by recognition and avoidance of unstable ground, hard bedrock, seepage areas, or other poor ground conditions in projecting a preliminary highway alignment are apparent.

The following average price ranges per linear mile for 9- by 9-in. color aerial transparencies are for photography obtained by negotiated contract. The prices include the cost of plastic envelopes in which the transparencies are placed.

Scale	*Cost ($)*
1:4,800	43.75-60.75
1:6,000	38.25-52.75
1:12,000	27.00-33.50

It has been the experience of the Federal Highway Projects Office, Region 9, that color photography may be obtained at somewhat lower prices through competitive bids, but the quality is often poor and not acceptable. Also, quoted prices are those charged for work in Region 9 and may not be the same elsewhere. Price quotations will also vary with the number of miles of photography required per job and the distances involved in getting a plane to the area to be photographed. Color photography in Region 9 has generally been taken at the same time as black-and-white photographs for photogrammetric mapping.

The cost of 1:6,000-scale black-and-white photographs printed on double weight paper having a semi-matte finish, is between $20.00 and $27.50 per linear mi. Two sets of prints and an uncontrolled photographic index are included in this price.

VIEWING EQUIPMENT

To view color aerial transparencies properly with a stereoscope, a suitable lighting system is required. Although almost any type of light box will permit viewing of transparencies, it is desirable to have a balanced light source that provides a spectrum comparable to that of natural sunlight. This is done by using fluorescent and incandescent lamps of about equal wattage. A means of varying the intensity of light is also desirable because the number of color distinctions the human eye can make varies with the light intensity. The eye can detect more color differences at lower levels of illumination. An appropriate translucent light diffuser is necessary to provide an even distribution of light throughout the transparency, to prevent "hot spots" while viewing. Ventilation holes are necessary and a small air blower is desirable to dissipate heat created by the lamps. Excessive heating of color transparencies causes them to curl and may permanently damage them. A small portable home-made light box equipped with fluorescent lights capable of being run by 115 volts ac or 12 volts dc was used on this project. It was therefore possible to view aerial photographs with a lense stereoscope in the field. A mirror stereoscope was used almost exclusively in the office.

Transparent plastic envelopes 10 by 11 in. in size were used so that annotations could be made on them with a grease pencil and to protect the aerial transparencies. The envelopes containing the transparencies were punched and placed in a three-ring notebook for convenient storage and use in the field.

PROCEDURE FOR CONDUCTING MATERIALS SURVEY

The procedure was similar to that used for black-and-white photographs. First, a search and review of published geological literature pertinent to the areas under study was made. Geologic maps, bulletins, folios, and other reports were studied. Information regarding known sources of construction materials was obtained from both Bureau of Public Roads and Park Service engineers familiar with these areas. Thus, as much background information as possible was obtained before the initial study of the aerial photographs was undertaken. The amount and value of this background information varied. Rather detailed, recently published geologic maps and reports were available for some areas and only old generalized reconnaissance-type geologic maps were available for others. This background orientation is considered an essential part of a materials investigation.

Following the literature review, a rather rapid preliminary examination of the aerial photographs, covering strips 4,500 ft wide, was made using a mirror stereoscope. This initial examination served to correlate the information obtained from the literature survey with the aerial photographic patterns observed. Specific geologic features and landforms were marked on the plastic envelopes for later investigation in the field. In this manner it was possible to plan and perform the field work more efficiently.

The ground examination consisted of examining rock outcrops, geologic materials in highway cuts, and river cut banks. Shallow holes were dug with shovels and mattock, and borings made with a hand auger to expose materials below the ground surface. Materials brought to the surface as a result of animal digging were also examined. Whenever possible, field notes taken included descriptions of soils and rocks and approximate depths of materials. Such ground examination is an essential part of this work and its importance cannot be overemphasized. Reference numbers for field observations made were placed on the aerial transparencies. Wherever possible, a stereoscopic examination was made in the field vehicle by means of a portable light box and pocket lens stereoscope. Color and black-and-white photographs were also taken from the ground during the course of the field investigation.

After the preliminary phase, the photographs were examined in greater detail, mapping units such as granite, glacial till, alluvial fans, and landslides were determined and their boundaries placed on the plastic envelopes for alternate photographs in each flight strip. Appropriate mapping symbols were developed for various rock types, landforms, and ground conditions. Boundaries or areas that were doubtful were noted and checked in the field. Mapping units were then described and the descriptions included as a part of a preliminary report for each of the segments studied. Potential material sources were noted and sites for sampling were marked on the photographs. Each of the indicated sources was examined with a stereoscope in the office by a Park Service official before any sampling or field investigation with mechanical excavation equipment was done. This insured the coordination of the work with Park Service policy and planning. All approved potential material sources were explored by means of either a truck- or a crawler-mounted backhoe, capable of excavating to a depth of about 12 ft, and representative samples were taken for laboratory testing. A brief description of the materials from the test pits was made and the approximate quantity in each potential source was determined by the field crew. Electrical resistivity equipment, although not used in this investigation, will be used in the future to help determine depths and quantities of materials. Final reports summarizing the results of the materials investigations were prepared for individual highway segments. These reports incorporate results of laboratory testing.

PRELIMINARY RESULTS AND FINDINGS

The purpose of this discussion is to point out some of the more important findings for this particular application of color aerial photographs. The findings of this study represent the result of only one summer's work using color aerial photographs for locating material sources and for generalized mapping. Undoubtedly new facts will come to light as color aerial photography is used by others.

Because the human eye is capable of distinguishing about 20,000 shades and hues of color, and color is perceived every day, it is not too surprising to find that color photographs are not only appealing but have many advantages over black and white. Persons using black-and-white aerial photographs must interpret specific ground conditions, soils, or geologic materials in terms of various photographic tones. The number of such tones or shades of gray that can be differentiated is extremely limited and many different types of soils or geologic materials of various colors may have about the same tonal expression. It should be remembered that neither photographic tones nor color are used alone in identifying specific materials or determining ground conditions. In some instances, the type of landform, gully, or drainage pattern may be the basis for recognition rather than photographic tone or color.

When using color aerial photographs for materials surveys, relative rather than absolute colors are of primary importance. Thus, slightly "off-color" photographs are as usable as those showing exact or nearly exact ground colors. This statement should not be misinterpreted as an endorsement for marginal or poor quality work. Visually, it is only possible to tell if the color registered on the photograph approximates that on the ground. The quality of the illumination also determines what colors the eye perceives, although the true colors may be registered on the film. If the illuminating source is deficient in a portion of the visible light spectrum, then these colors are not perceivable to the eye.

The requirements for acceptable color photographs that are to be used for materials surveys are not as stringent as those that have been stipulated with respect to endlap, sidelap, crab, and tilt for mapping by photogrammetric methods. In general, acceptable color photography must have good definition of images, have even light distribution, be free of clouds, and have the proper exposure and color balance.

Color photography that is either considerably overexposed or underexposed, discolored in processing, or off the designated flight lines should be rejected. Overexposed color photographs have a "washed out" appearance whereby many of the ground colors do not register on the film. The problem is particularly serious in areas with little or no vegetative cover where there is more than ample reflected light available and where light meter readings are not always reliable. A greater percentage of the ground colors register on underexposed film than on overexposed; for this reason, slight underexposure is more desirable than slight overexposure. For the most part, color photographs were properly exposed and colors recorded on the film compared well with those observed on the ground.

One obvious advantage of color aerial photographs is that cultural features such as highways, trails, buildings, and aerial targets are more readily identifiable than on black-and-white photographs. Personnel appreciate the fact that it is easier to orient oneself in the field because the terrain and culture appear natural.

Image definition is better on color aerial transparencies than on black-and-white prints because use of prints results in some loss of detail.

Wet soils, organic soils, boggy ground, and seepage zones can be easily identified on color photographs because of the green grass and other vegetative growth in these wet areas. The brownish color of the organic soils is readily recognized. Minor drainageways, poorly drained depressions or swales, and seepage areas in landslides show up quite clearly. On black-and-white photographs these areas have generally darker photographic tones, but identification or delineation is not as positive and in some instances the conditions are not apparent.

The identification and delineation of such rock types as granite, rhyolite, basalt, limestone, shale, and sandstone are greatly facilitated by means of color photography, and large boulders in glacial till are easier to identify. In one instance, the fracture pattern for a granite was so distinctive that it appeared equally well on color as on black-and-white photographs. Color is particularly helpful in instances where the rock fracture pattern or other features are not distinctive and cannot be used as a means for identification.

Although the authors lacked training in forestry, it was recognized that various vegetative types can be identified more readily on color than on black-and-white photographs. It is not always possible to correlate a particular type of vegetation with a specific type of material or ground condition but color photography will be useful wherever reliable correlations can be made. For example, aspen growing on alluvial fans in southern Colorado clearly outlined the extent of these deposits. Dense timber cover that completely obscures ground detail is a liability in aerial photographic interpretation when using black-and-white or color photographs. As an example, the detection of local, thin deposits of glacial sand overlying volcanic flows in one area of Yellowstone National Park was impossible because of a dense cover of lodgepole pine.

In several cases, the colors of certain features were not of constant density in adjacent pictures. For example, vividly colored algae growing in hot springs appeared in almost true color on one photograph, but were almost completely "washed out" in the next. This condition apparently is the result of the difference in the angle of reflected light from the ground for successive plane positions. Although this change in color density may be obvious in comparing individual photographs, it is not generally apparent when pairs of photographs are viewed stereoscopically. A similar difference in photographic tones on black-and-white photographs was also noted on sand and gravel river bars.

Several scales of color photography were flown so that a comparison could be made relative to their use for materials surveys. The scales selected were 1:4,800, 1:6,000, and 1:12,000. Previous experience in Region 9, indicated that a scale of about 1:6,000 would be the most desirable for locating materials sources. The results of this study showed this to be true. The flight strip widths for 9- by 9-in. photographs of the three scales used are as follows: 1:4,800 (1 in. to 400 ft), 3,600 ft; 1:6,000 (1 in. to 500 ft), 4,500 ft; and 1:12,000 (1 in. to 1,000 ft), 9,000 ft. A scale of 1:4,800 permits more ground detail to be observed, but the flight strip width is one-fifth less than that for 1:6,000. The advantage of having the 900 ft in additional width in the 1:6,000-scale photography, more than offset any slight advantage gained by use of the larger scale. The additional detail obtained from the 1:4,800-scale photographs did not help appreciably in the detection of materials sources or in the determination of ground conditions. The 1:12,000-scale photographs show insufficient ground detail for optimum use, although the smaller scale does provide greater width of coverage and permits a broader overall view. Examination of photographs at this scale requires greater study time than at 1:6,000 and leaves many uncertainties in the interpreter's mind regarding actual ground conditions. Scales ranging from 1:6,000 to 1:8,000 should provide sufficient ground detail and width of coverage for optimum use in materials surveys.

If a color aerial transparency is either lost or damaged, there is no possibility of replacement as with prints made from black-and-white film negatives. This is an obvious disadvantage, and as a result, greater care is needed in handling and working with transparencies. Reproduction of color transparencies is rather costly and color reproduction of the originals is not reliable. A suitable illuminating source is also required for viewing color transparencies in the office. A special portable light box adapted for battery operation is needed if transparencies are to be viewed in the field. Field use of color aerial transparencies under extremely hot and dry climatic conditions results in curling of the transparencies. Under similar conditions black-and-white photographs printed on double weight paper also curl and become brittle.

CONSIDERATIONS AND RECOMMENDATIONS

The considerations and recommendations indicated are based primarily on the results and findings of the work done in Wyoming, Colorado, Utah, and New Mexico during summer 1961. Future work will undoubtedly uncover many new facts in the use of color photographs in engineering materials and soils surveys. Some of the present objections should be eliminated with improvements in faster color films having greater contrast and exposure latitude.

The type of camera and lens used to take aerial color pictures is highly important. The quality of such a camera should be equal to or exceed that of the Zeiss RMK 15/23 equipped with a Pleogon lens. There has been a tendency for commercial organizations to use lenses of longer focal length, such as 8¼-in. or 12-in. rather than 6-in. lens to obtain color photographs. Although use of longer focal-length lenses results in better light distribution, giving more uniform density in the photograph, the flying height required is greater to obtain a given scale. This increased height of the aerial camera above the ground increases the haze effect that gives the photographs an undesirable bluish hue. High quality lenses of shorter focal length do not produce noticeable vignetting. Haze is a lesser problem due to lower flying heights required.

Equipment used to process color film is rather expensive and some commercial organizations have neither the equipment nor experience in taking or processing color photographs necessary to produce a good quality job. For this reason, color photography has been secured in Region 9 by negotiated contract because the objective is to obtain a high quality product at a reasonable price. Unless a State highway department has had previous experience with a commercial organization, it is recommended that the organization concerned be required to submit in advance examples of color photographs at the desired scale of the area to be photographed. Examples of photographs at other scales, or the same scale, taken in other parts of the country should not be accepted as evidence that the organization is capable of producing acceptable photographs for the area in question.

The season and time of day are important in the procurement of color as well as black-and-white photography. Color photographs should be taken preferably during the time interval between complete snow disappearance and leafing of deciduous trees or from the time the trees are bare to the first snow. Photographs taken in the fall have longer shadows than spring or summer photography because of the lower sun angle. This is particularly apparent in photographs taken in the higher northern latitudes. The intensity and emitted wave lengths of light from sunlight also vary with latitude and time of day. For obtaining proper color balance and minimizing shadows, aerial photographs should be taken as close to peak solar altitude as possible.

The selection of Kodak Ektachrome Aero Film, used on this project, was based on the previous success attained by the commercial organization contracted to do the work in producing acceptable color aerial transparencies with this film. Because the evaluation of various types of color aerial films was not a part of this study, no attempt was made to use other commercial films for this materials investigation.

Color aerial prints, rather than color aerial transparencies, are not acceptable for use in materials surveys because of excessive cost and lack of good color reproduction. One price recently quoted was $4.00 to $5.00 per print, when ordering 50 prints or more at a time. The high cost of color prints is a direct reflection of the skill and experience required and the difficulty in producing color pictures of acceptable quality. Relatively large emulsion shrinkage of color prints causes them to curl with small changes in temperature and humidity.

This investigation did not include the use of color aerial photographs for conducting engineering soils surveys. Casual observations regarding the identification of soil differences by means of photographic interpretation appears encouraging. An evaluation of color photographs for a broad range of geologic materials and a variety of soil forming conditions is needed.

Both black-and-white and color aerial photographs can be used to economic advantage in conducting comprehensive materials surveys. Color photographs are particularly useful in this respect because they generally permit information to be obtained that cannot be seen on black-and-white photographs. Furthermore, color photographs permit the job to be done more efficiently and reliably.

An adequate job by trained personnel may take considerable time and effort in the office and field. The education, training, and experience of the aerial photographic interpreter is significant. A background in the earth sciences, training and experience in aerial photographic interpretation, and a knowledge of highway construction materials, photogrammetry, and highway engineering are highly desirable.

The possible application of color aerial photographs for mapping by photogrammetric methods appears encouraging. One problem is in obtaining a stable film base to which color emulsions will adhere. Once this technical problem is resolved, it is likely that color photographs will be worthy of consideration and evaluation for mapping by photogrammetric means.

Undoubtedly, black-and-white photographs will continue to be used in the search for construction materials. However, good quality color aerial photographs can be produced and employed advantageously in conducting material searches. About 700 linear mi have been photographed in color for the Federal Projects Office, Region 9, Bureau of Public Roads, since 1958. As a result of past experience and the findings of this study, an additional 250 linear mi of color photography are planned to be flown in 1962.

ACKNOWLEDGMENTS

The authors express their thanks to the U. S. National Park Service for making this project possible, and to D. E. Winsor, Federal Highway Projects Division, Bureau of Public Roads, Washington, D. C., for initiating this investigation. Appreciation is extended to D. C. Harrington, R. A. Bohman, W. Fitzer, and other personnel of the Materials Laboratory of the Federal Highway Projects Office, Region 9, Bureau of Public Roads, for their help and guidance in conducting the work. The authors and Region 9 wish to express their appreciation to Continental Engineering Company, Denver, Colo., for its interest and cooperation in this project.

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Variability of Engineering Properties of Brookston and Crosby Soils

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Engineers have always assumed that soils derived from the same parent material and under the same environmental conditions would have similar engineering properties. To ascertain the extent to which this is true a study was conducted on two soils obtained from Madison and Tipton Counties, Ind., and pedologically classified as Brookston and Crosby.

Twenty borings were obtained from each county—10 from Brookston soils and 10 from Crosby soils. Samples of these soils were subjected to the following tests and the results analyzed statistically: Atterberg limits, Standard AASHO compaction, Hveem stabilometer and swelling pressure, California bearing ratio, grain-size distribution, and unconfined compression.

X-ray diffraction tests were conducted on 8 samples—4 from the rises and 4 from the depressions.

From the statistical analysis, utilizing analysis of variance techniques, it was found that soil variability is a function of the property being measured. The variability of the soils, as defined by the parameters of these tests, was large. The consequences of such variation as it pertains to pavement design were considered.

Diagrams are presented which relate the number of borings required to predict the mean value of a given test parameter to a desired degree of precision.

• WHEN DEALING with relatively large areas, two broad aspects of soil sampling need be investigated: accuracy of soil tests for a given soil type, and determination of the number of soil samples required to define the soil within certain specified limits. The latter pertains to pedological soil classification as well as classification based on landforms.

As an example, a highway crosses a typical glaciated area. By the use of airphotos, agricultural soil maps, and other tools at the disposal of the engineer, the general soil types can be delineated. Next, information on the uniformity of the deposit can be obtained by detailed exploration. The variability among random samples may be great. Clarification of the random variability of soil can be of great value to the soils engineer.

Another phase of the problem is the variability from one soil area to another of the same classification. These data in this regard would be of of great value for setting up "average" soil property values which can be adopted for design.

Data from this last phase can be used by the soils engineer and researcher alike for preliminary pavement design. Correlation studies of pavement performance would also be enchanced if typical strength values were known.

To find the optimum solution to the problems stated above the disciplines of soil mechanics, statistics, airphoto interpretation, and pedology were utilized.

PURPOSE AND SCOPE

The primary purpose of this study was to determine the variation that could be expected in the engineering properties of soils derived from the same parent material and under similar conditions of climate, vegetative cover, age, and topography. Also, the number of samples required to reliably predict these properties was determined.

The areas selected for this study are located in Tipton and Madison Counties, Ind. The parent material is late Wisconsin drift and is illitic in nature. The soils formed from this parent material belong to the Miami-Crosby-Brookston Catena. The Crosby (rise) existing on 0 to 4 percent slopes and the Brookston (depression) existing in depressional areas were used in this study.

Twenty borings were made in each county—10 in elevated positions and 10 in the depressions. The A-, B-, and C-horizons were samples in each boring. However, only moisture content and Atterberg limit determinations were performed on the soil from the A-horizon. The soils from the B-horizon and C-horizon, in addition, were subjected to grain-size analysis, California bearing ratio (CBR) tests, compaction tests (dynamic and kneading), unconfined compression tests, and Hveem stabilometer and swelling tests.

The data from the tests were subjected to statistical analysis to estimate the variance of the soil properties and the number of samples required to define these properties. In regard to the variance of soil properties, two questions were answered:

1. Is there a significant difference between the physical properties of the soil taken from horizons in the same soil series in two counties?
2. Is there a significant difference between the results obtained from the various borings within a given county?

Finally, it was hoped to discover useful relationships among the previously listed properties that would provide information for the preliminary design of structures.

PROCEDURE

Pedologic maps and soil surveys were not available for the counties considered in this study; therefore, it was necessary to make the selection of the boring sites on the basis of airphoto patterns. After studying the airphotos of five Indiana counties, it was decided to use Madison and Tipton Counties because of the similarity of their airphoto patterns. In particular, an area just south of the Union City moraine, in each county, was chosen.

The parent material is Wisconsin drift. However, to negate the effect of the moraine the sampling sites were chosen so that they were equidistant from the moraine (approximately 5 mi).

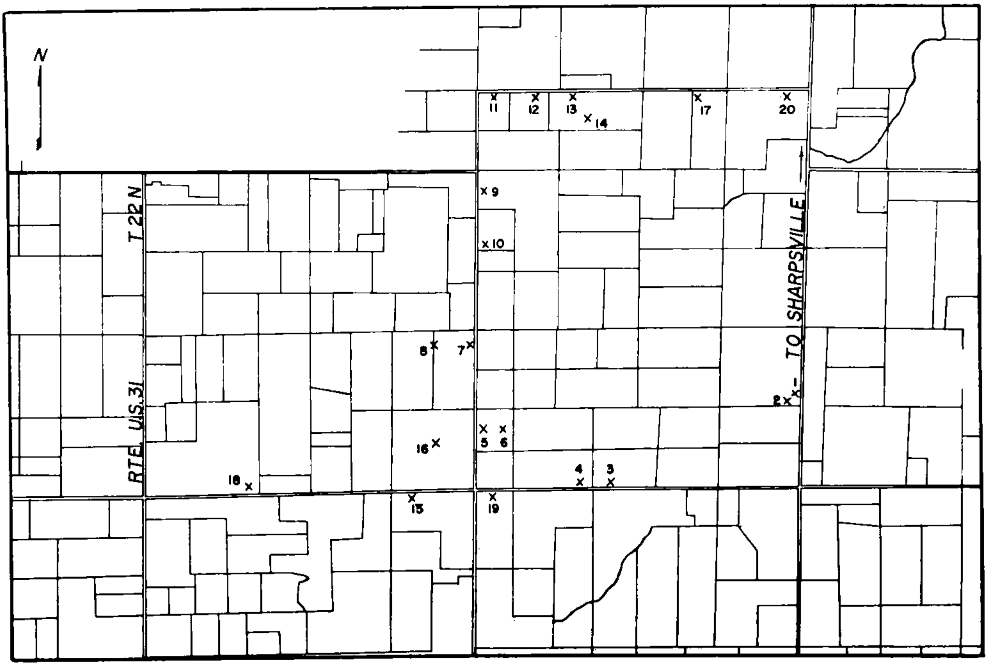

Figure 1. Boring locations, Tipton County.

On the basis of airphoto pattern, the soils of the area were divided into two categories—rises and depressions. Possible boring sites were chosen in the office, after which a field check was made and the final boring locations determined (Figs. 1 and 2). A total of 20 borings was made in each county—10 in the rises and 10 in the depressions (see Fig. 3 for generalized soil profiles based on boring logs).

Samples were obtained by hand augering. Approximately 300 g of soil was taken from the A-horizon of each boring, and values of the Atterberg limits and natural moisture content were determined. Because the A-horizon is often wasted in engineering construction, extensive testing was not warranted.

In addition to samples for the Atterberg limit and natural moisture content tests, approximately 100 lb were taken from both B- and C-horizons of each boring. The latter samples were air dried and quartered into sizes necessary to perform the following tests:

1. Grain-size distribution and specific gravity;
2. Standard AASHO compaction;
3. Hveem stabilometer and swelling pressure;
4. California bearing ratio;
5. Unconfined compression; and
6. X-ray diffraction.

Natural Moisture Content

Moisture content samples were taken from each horizon in each boring, selecting the sample from the same depth below the ground surface—the depth at which these samples were taken depended on whether the boring in question was located in a rise or a depression. No quantitative analysis of these data was attempted. Only one moisture content sample was taken per horizon.

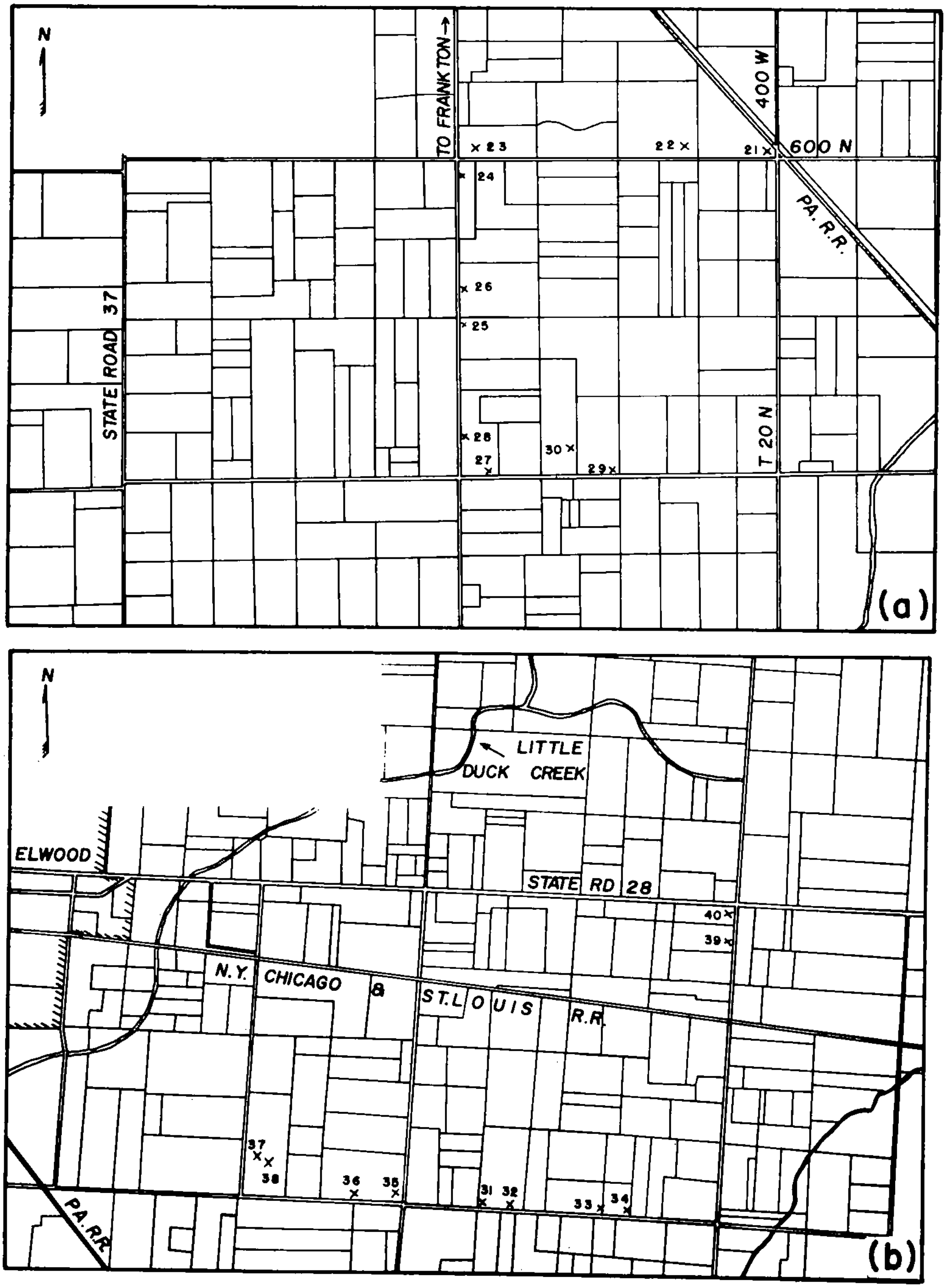

Figure 2. Boring locations, Madison County.

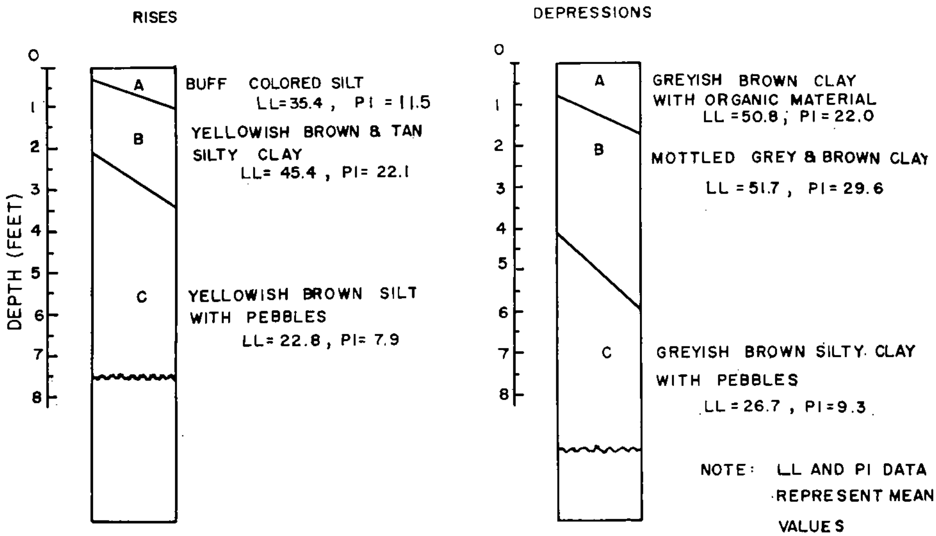

Figure 3. Boring results.

Atterberg Limits

The liquid and plastic limits were determined in accordance with ASTM Designations D423-54T and 424-54T, respectively, with the exception of the method of preparation of the samples. The tests were conducted on samples at their natural moisture content. Such a procedure would best indicate plasticity properties of the in-situ materials. Two determinations were made in each horizon.

Grain-Size Distribution and Specific Gravity

The procedure for determining the specific gravity of the soils is that given in ASTM Designation D854-58. In regard to the grain-size analysis, ASTM Designation D422-54T was employed with the following variations:

1. A constant temperature bath was not used.
2. Two grams of the water conditioner "Calgon," manufactured by the Calgon Company, Pittsburgh, Pa., per 50 g of soil was used as a deflocculating agent.

Compaction Tests

Standard AASHO compaction tests were run according to Method A of ASTM Designation D698-58T.

Hveem Stabilometer and Swelling Pressure Test

Hveem stabilometer and swelling pressure tests were conducted in accordance with test method California 301-B, State of California Division of Highways. Molding moisture content was considered critical and was the controlled variable. This molding moisture content was chosen on the basis of the kneading compaction curves.

The kneading compaction curves were established by the compaction procedure given in test method California 301-B with three variations:

1. All moisture was added to the sample the day before testing.

2. Compaction curves were determined for compaction foot pressures of 350, 250, and 150 psi (see Fig. 8 for typical curves).

3. The compactor foot pressure used to get the soil into the mold was 75 psi instead of 15 psi as prescribed in the test method.

On the basis of the first series of compaction tests, it was determined that the compaction foot pressure that would give densities approximating the standard AASHO results was 150 psi. Thus, the remainder of the tests were run using the 150-psi foot pressure only.

Because it was not feasible to run compaction tests on samples from each horizon, the samples were grouped according to the density obtained from the standard AASHO compaction test. A sample of each group was then subjected to a compaction test using the kneading compactor. The stabilometer specimen from each horizon was then molded at the optimum moisture content (OMC) determined from tests on the sample representative of its density group.

Borings 3, 25, and 12 were used as the standard. For the C-horizon the density groups represented by these samples were more than 120 pcf, 117 to 120 pcf, and less than 117 pcf, respectively. However, in the B-horizon the density range was much narrower and it was necessary, in many instances, to use logic and intuition in assigning a molding moisture content to a given sample. The criteria for determining whether the proper moisture content was assigned were density and the action of the soil under the compaction foot. If a density approximating the standard AASHO was obtained and if there was not significant shoving of the surface during the compaction process, the assigned moisture content was assumed satisfactory.

TABLE 1
OPTIMUM MOISTURE CONTENT

Boring	Horizon	OMC (%)
3	B	16.5
	C	11.0
12	B	18.0
	C	14.2
25	B	17.0
	C	12.0

The moisture contents used for molding the specimens are given in Table 1. The average moisture contents of the samples were controlled to within ±0.5 percent.

CBR Test

CBR tests were conducted in accordance with the U. S. Army Corps of Engineers test procedure given in EM 1110-45-302, Appendix III, 1957, part 5 with the exception that the standard AASHO compactive effort was used. Also, the average molding moisture content was controlled to within ±0.5 percent of the standard AASHO optimum moisture content.

Unconfined Compression Tests

Unconfined compression tests were run on specimens molded with the Harvard miniature compactor. The compactive effort was five layers at 15 blows per layer using a 40-lb spring.

The soils from each horizon were divided into groups according to density and compaction tests conducted on a representative sample of each group to determine the OMC. The same density groups as cited in the discussion of the Hveem tests were utilized. Borings 11, 33, and 24 were taken to represent the high, medium, and low density groups, respectively (based on the density of the C-horizon).

On the basis of these tests, the OMC of the groups are given in Table 2. These average moisture contents were within ±0.5 percent of the desired moisture content.

TABLE 2
OPTIMUM MOISTURE CONTENT

Boring	Horizon	OMC (%)
11	B	16.5
	C	11.6
33	B	18.0
	C	13.0
24	B	17.0
	C	13.0

TABLE 3

County	Boring No. Rise	Depression
Tipton	4, 12	1, 14
Madison	21, 35	24, 28

The rate of strain, used for the unconfirmed compression tests, was 0.07 in. per min. Also, after molding, the samples were wrapped in aluminum foil, placed in a sealed container, stored overnight, and tested the following day.

X-Ray Diffraction Tests

X-ray diffraction tests were run on the B- and C-horizons of 8 borings. Two borings were selected from the rises and 2 from the depressions of each county.

The basis of the selection of the borings to be used was unusual behavior as exemplified by the CBR and Hveem stabilometer data. The samples chosen produced higher CBR and/or stabilometer (R) values for the B-horizon than for the C-horizon. This situation is just the opposite of the normal trend, and it was felt that a knowledge of the clay minerals present might help to explain the reason for this behavior. With this in mind, borings representative of the group of soils in which this event occurred were chosen. Table 3 gives their topographic position and county.

The slides for the X-ray diffraction test were prepared from a portion of the soil quartered for the hydrometer analysis test. Fifty grams of the soil were mixed with approximately 700 cc of water and 2 g of the water softener Calgon. The suspension was then mixed in a mechanical stirrer for 3 min, after which the soil was allowed to settle out of suspension. After a period of time, a sample was taken from the suspension at a depth, based on Stokes' law, where 2-μ particles would be located. This portion of the suspension was placed on a glass slide and allowed to dry.

Statistical Analysis

Following completion of these tests, the data were analyzed using analysis of variance techniques. Table 4 gives the data layout for the analysis of variance studies. With the exception

TABLE 4
DATA LAYOUT FOR ANALYSIS OF VARIANCE

Boring No. / Horizon	Tipton County Depressions	Tipton County Rises	Madison County Depressions	Madison County Rises
	1 3 6 8 20	2 4 5 7 19	22 24 40	21 23 39
A				
B				
C				

Variables to be analyzed:
Two observations per cell
1. Liquid limit
2. Plastic limit
3. Plasticity index
4. Optimum moisture content
5. Optimum density

One observation per cell
6. CBR values (soaked)
7. Percent passing No. 200 sieve
8. Percent < 0.002 mm
9. Swelling pressure
10. Stabilometer values
11. Unconfined compressive strength

of the Atterberg limits, only the B- and C-horizons are considered.

RESULTS

The analysis of variance model for the test results is

$$Y_{ijklm} = U + C_i + D_j + CD_{ij} + B_{k(ij)} + H_l + HC_{il} + HD_{jl} + HCD_{ijl} + HB_{lk(ij)} + E_{m(ijkl)} \quad (1)$$

in which

Y_{ijklm} = the value obtained from a given test;
U = the true mean value for the population;
C_i = the between-counties true effect;
D_j = depression vs rise true effect;
$B_{k(ij)}$ = between-boring true effect in the C-D cells;
H_l = between-horizons true effect; and
$E_{m(ijkl)}$ = error true effect of repeat measurements.

The other terms denote interactions between the main effects listed. As regards the main effects, C, D, and H are fixed while B and E are random. The subscripts may assume values as

$i = 1, 2$
$j = 1, 2$
$k = 1, 2, 3, \ldots\ldots, 10$
$l = 1, 2, 3$
$m = 1, 2$

The variation in the results of the borings may be represented as

$$\sigma_T^2 = \sigma^2 + \sigma_B^2 + \sigma_{HB}^2 \quad (2)$$

in which

σ_T^2 = the total estimated variance between borings;
σ^2 = the variance due to laboratory procedure;
σ_B^2 = the variation from boring to boring; and
σ_{HB}^2 = the variation in boring results due to differences in the properties of the horizons.

The standard deviation of the mean of the borings can be written

$$\sigma_{\bar{X}} = \sqrt{\frac{\sigma^2 + \sigma_B^2 + \sigma_{HB}^2}{n}} \quad (3a)$$

Therefore, if it is desired to predict the mean value of the population to any specified degree of precision, L, then

$$L = t\sigma_{\bar{X}} \quad (4)$$

in which

L = the limit of accuracy, and
t = the value obtained from the normal distribution and is a function of the α level desired.

The normal t can be used because the estimate of $\sigma_{\bar{X}}$ contains a great many degrees of freedom.

In this study an α level of 0.05 is used, which means that, on the average, 95 percent of the time the true mean values will fall within the limits indicated for the given value of n. Also, for $\alpha = 0.05$, $t = 1.96$.

The statistical analysis is based on the assumptions that the variance is not significantly affected by a change in operators, there is no significant change in variance with horizon, and there is normality of dependent variables.

In the text and the analysis of variance tables the following abbreviations are used:

DF = degrees of freedom;
MS = mean square; and
EMS = expected mean square.

Atterberg Limits

Liquid Limit.—Table 5 summarizes the results of the analysis of variance. Each main effect and interaction was tested for significance utilizing the F-test for the ratio of two variances (*1*). From these tests it was determined that a significant difference existed between the rises and depressions, between borings

TABLE 5
SUMMARY OF ANALYSIS OF VARIANCE—LIQUID LIMIT

Source of Estimate	DF	Sums of Squares	MS	EMS
Between counties (C_i)	1	$S_i=C_i-C=57.53$	57.53	$\sigma^2+6\sigma_B^2+120\sigma_C^2$
Depression vs rise (D_j)	1	$S_j=C_j-C=4483.36$	4,483.36	$\sigma^2+6\sigma_B^2+120\sigma_D^2$
CD_{ij}	1	$S_{ij}=C_{ij}-C_i-C_j+C=23.00$	23.00	$\sigma^2+6\sigma_B^2+60\sigma_{CD}^2$
Between borings in C-D cell, $B_{k(ij)}$	36	$S_{k(ij)}=C_{ijk}-C_{ij}=1795.27$	49.87	$\sigma^2+6\sigma_B^2$
Horizons H_l	2	$S_l=C_l-C=25{,}085.67$	12,542.83	$\sigma^2+2\sigma_{HB}^2+80\sigma_H^2$
HC_{il}	2	$S_{il}=C_{il}-C_i-C_l+C=85.93$	42.96	$\sigma^2+2\sigma_{HB}^2+40\sigma_{HC}^2$
HD_{jl}	2	$S_{jl}=C_{jl}-C_j-C_l+C=1457.52$	728.76	$\sigma^2+2\sigma_{HB}^2+40\sigma_{HD}^2$
HCD_{ijl}	2	$S_{ijl}=C_{ijl}+C_i+C_j+C_l-C_{ij}-C_{il}-C_{jl}-C=26.46$	13.23	$\sigma^2+2\sigma_{HB}^2+20\sigma_{HCD}^2$
$HB_{lk(ij)}$	72	$S_{lk(ij)}=C_{iklj}-C_{ijk}-C_{ijl}+C_{ij}=3629.75$	50.41	$\sigma^2+2\sigma_{HB}^2$
$E_{m(ijkj)}$	120	$S_{m(ijkl)}=\Sigma_{ijklm}\ X^2_{ijklm}-C_{ijkl}=684.78$	5.71	σ^2
Total	239	$SS=\Sigma_{ijklm}\ X^2_{ijklm}-\frac{T^2}{N}=37{,}329.27$		

within the different combinations of county and rise vs depression; that is, in the *C-D* cells and between horizons. Also, it was found that the interactions between the horizons and borings in the *C-D* cells tested significant. Significance indicates that the effect being considered makes a major contribution to the variation in the test results.

The analysis of variance and the significance tests also showed that there was no significant difference between counties and that no interaction terms involving counties tested significant. This indicates that the data need not be subdivided on the basis of counties. From Table 5 the following values for the variance estimates can be obtained:

$$\sigma^2=\frac{685}{120}=5.71$$

$$\sigma_{HB}^2=\frac{50.41-5.71}{2}=22.35$$

$$\sigma_B^2=\frac{49.87-5.71}{6}=7.36$$

therefore,

$$\sigma_T^2=5.71+22.35+7.36=35.42.$$

Based on this value of σ_T^2 the number of borings required to predict the LL to a given degree of precision was determined. Figure 4 shows this relationship. Precision (limit of accuracy) is expressed in percentage points of moisture. Thus, for an average of 8 borings, 95 percent confidence limits will be, on the average, ±4 percent. In Figure 4, the ordinates for liquid limit are

$$1.96\sqrt{\frac{35.40}{n}}$$

because 95 percent of a normal curve area is from −1.96 to +1.96.

Plastic Limit and Plasticity Index.—To conserve space and aid reading, the analysis of variance tables for the plastic limit, plasticity index and all measured variables subsequently referred to are omitted in this report. If such information is desired, see Hampton (*3*).

The results of analyses of variance of both plastic limit and plasticity index data proved no significant difference between the two counties but all other main effects tested significant; *i.e.*, borings in the C-D cells, horizons and rise vs depression (topography).

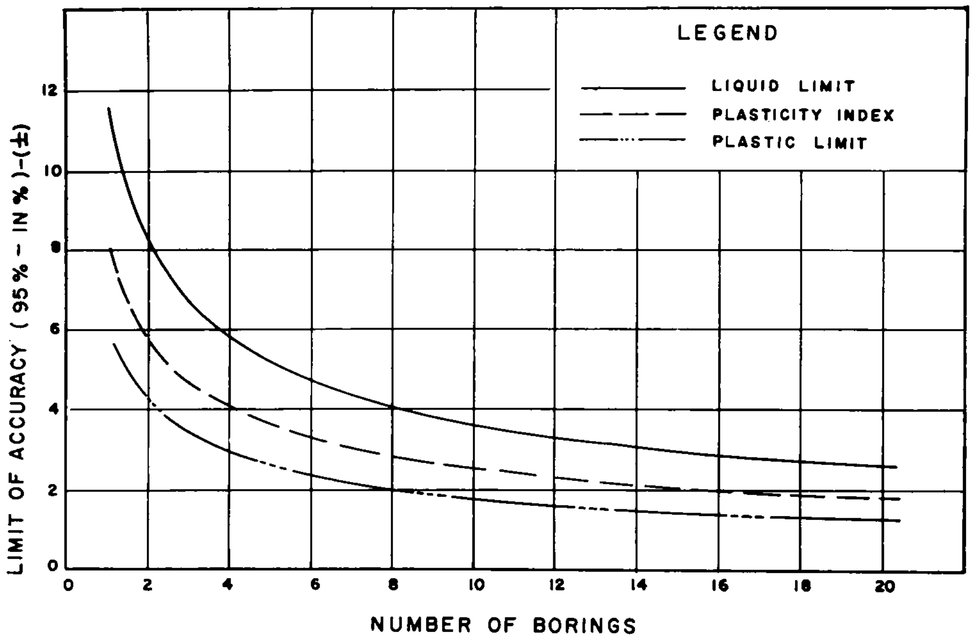

Figure 4. Limit of accuracy *vs* number of borings, Atterberg limits.

As to the plasticity index (PI), all interaction terms tested significant with the exception of county-depression (CD) and horizon-county-depression (HCD) interactions. Considering the plastic limit, only the HCD interaction was not significant.

Considering in the light of Eq. 2 the plasticity index

$$\sigma_T{}^2 = 5.22 + 3.75 + 7.69 = 16.66.$$

Therefore,

$$\sigma_{\bar{X}} = \sqrt{\frac{16.66}{n}} \tag{3b}$$

As to the plastic limit,

$$\sigma_T{}^2 = 1.03 + 6.08 + 2.16 = 9.27$$

and

$$\sigma_{\bar{X}} = \sqrt{\frac{9.27}{n}} \tag{3c}$$

Based on these values of $\sigma_{\bar{X}}$ the number of borings required to predict the plastic limit and the plasticity index to any desired degree of precision can be computed (see Fig. 4). The limit of accuracy (precision) is in terms of percentage points of moisture.

From Figure 4, considering absolute values, the liquid limit is the most variable and the plastic limit the least. The absolute variability of the plasticity index lies between that of the aforementioned properties.

Figure 5 shows the classification of the soils from the borings used in this study. This plot is based on the Unified Soil Classification System. Some of the points represent more than one boring. Also, the points represent the average of the two determinations for each horizon in a given boring.

The results for a given horizon departmentalize themselves very well. Looking at the over-all picture the A-horizon results lie below the A-line

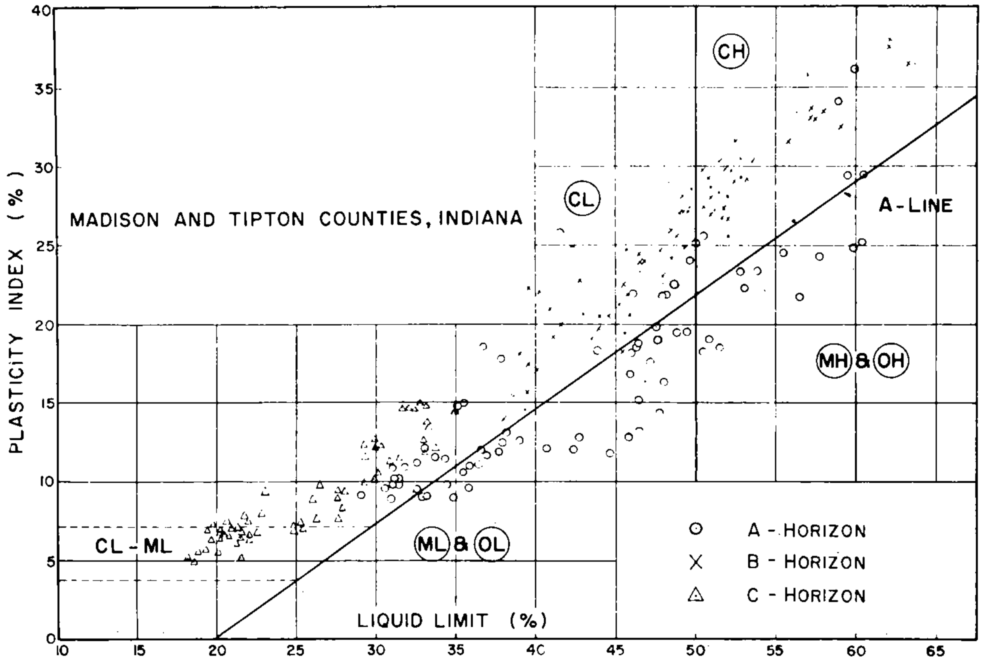

Figure 5. Summary of Atterberg limit data.

in the majority of cases. Furthermore, although it does not show in Figure 5, all the depressional soils had a liquid limit greater than 41 percent though only two samples from the rises had a liquid limit above this value.

For the B-horizon, all the results plotted above the A-line. A slight majority of the samples were classified CL with the remainder CH. Only two of the depressional soils had a liquid limit less than 49 percent though five of the rise soils had liquid limits above this value.

Finally, the C-horizon soils all plotted above the A-line with the majority being classified CL and the remainder CL-ML. A liquid limit of 25 percent appears to be the boundary between the rises and depressions —the latter lying above this value.

The Atterberg limit data were subjected to a linear regression analysis to determine the equation of a line that would represent the data. Considering the B- and C-horizons the regression line representing these data had a slope equal to 0.72, which is approximately equal to the slope of the A-line. However, when considering all three horizons the slope of the regression line is 0.66, which is much less than the slope of the A-line.

Table 6 gives a summary of the Atterberg limit data. It contains the maximum, minimum, and mean values of the liquid limit and plasticity index. This table shows that the mean values of these properties for a given horizon are not greatly different for the two counties.

Compaction Tests (Standard AASHO)

An analysis of variance was conducted on the optimum moisture content (OMC) and the optimum density

TABLE 6

SUMMARY OF ATTERBERG LIMIT DATA

County	Topography	Horizon	Liquid Limit (%)			Plasticity Index (%)		
			Min.	Max.	Mean	Min.	Max.	Mean
Tipton	Rise	A	31.4	44.7	35.5	9.0	18.6	11.7
		B	39.5	53.2	46.7	17.1	29.8	23.8
		C	19.4	33.1	22.8	5.2	14.8	8.2
	Depression	A	43.9	60.4	51.1	12.8	25.2	20.5
		B	49.2	63.2	54.3	27.4	37.9	20.8
		C	14.5	33.7	27.4	NP	14.7	9.6
Madison	Rise	A	29.1	46.5	35.3	8.9	15.2	11.3
		B	38.0	51.9	44.1	13.9	30.3	20.5
		C	18.5	33.2	22.9	5.0	13.5	7.6
	Depression	A	41.6	60.5	50.5	12.8	36.1	23.4
		B	45.7	65.3	51.0	23.9	38.4	28.4
		C	18.1	38.4	26.0	5.2	15.5	9.0

(OD) values using the data from the Standard AASHO compaction tests. Considering the optimum density data, the variance components obtained are $\sigma^2=1.02$, $\sigma_B{}^2=3.22$, and $\sigma_{HB}{}^2=4.94$. Therefore, from Eq. 2

$$\sigma_T{}^2=1.02+3.22+4.94=9.18$$

Using this value of $\sigma_T{}^2$ and Eq. 4 the upper curve of Figure 6 is obtained. The curve represents the relationship between number of borings and limit of accuracy. For example, to predict the mean optimum density of the population within the limit of ± 3 pcf it would be necessary to make four borings.

The components of the total variance of the optimum moisture content data are $\sigma^2=0.50$, $\sigma_B{}^2=0.74$ and $\sigma_{HB}{}^2=1.01$; therefore, $\sigma_T{}^2=2.25$. Based on this value of the total variance,

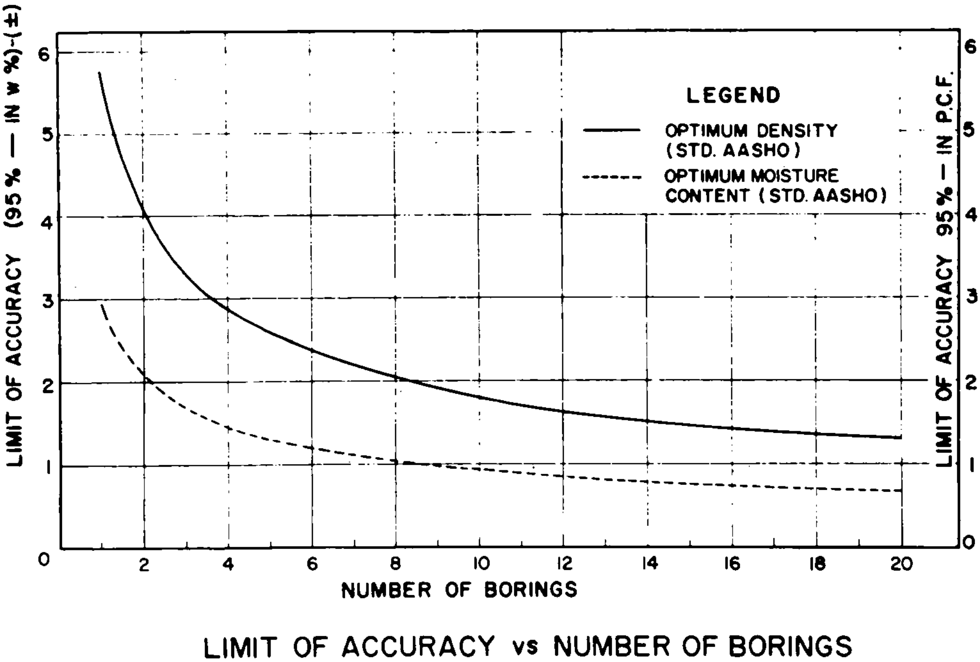

Figure 6. Limit of accuracy *vs* number of borings.

TABLE 7
SUMMARY OF COMPACTION TEST (AASHO) DATA

County	Topography	Horizon	Opt. Moist. Content (%)			Optimum Density (%)		
			Min.	Max.	Mean	Min.	Max.	Mean
Tipton	Rise	B	17.0	23.0	19.0	99.5	108.2	103.6
		C	10.9	13.7	12.1	117.0	123.0	121.0
	Depression	B	17.2	22.8	19.3	97.7	108.0	103.2
		C	10.1	16.3	13.2	110.0	123.1	117.2
Madison	Rise	B	16.0	20.2	18.2	102.1	108.0	104.3
		C	9.0	15.4	13.2	113.1	125.0	117.5
	Depression	B	16.2	20.4	18.7	102.6	107.3	104.8
		C	10.5	15.1	12.7	114.2	122.7	119.0

σ_T^2, and $t=1.96$—for significance level of 5 percent, $\alpha=0.05$—the lower curve of Figure 6 is obtained.

The factors that tested significant for both the optimum moisture content and density are the horizon and between-boring main effects and the horizon-boring interaction. In addition, the county-topography and the horizon-county-topography interactions tested significant as to the optimum density data. Thus, the absolute variability of the optimum density data is greater than that of the optimum moisture content. This can also be observed from a comparison of the magnitude of the mean squares of the variance estimates, as well as the relative position of the curves of Figure 6.

Table 7 gives a summary of the compaction test data. It contains the maximum, minimum, and mean values of the optimum moisture content and optimum density data. The closeness of the results when horizon is held constant and the wide disparity when it is allowed to vary show why the factors tested significant.

A linear regression analysis was made on the optimum density and plastic limit data. From this analysis it was found that the equation representing the linear relationship between the OD and the PL is

$$OD = 152.6 - 2.1\ (PL) \qquad (5)$$

in which

OD = optimum density (lb/ft^3); and
PL = plastic limit (%).

Figure 7 is a graph of Eq. 5. Each point represents the average of the two tests run per sample. No segregation of results based on county and/or topography was observed, but the data did group themselves according to horizon.

Hveem Stabilometer and Swelling Pressure Tests

As described previously, the samples were first grouped according to the optimum density obtained from the Standard AASHO compaction tests. Next a representative sample from each group was subjected to compaction with the kneading compactor to determine the OMC and OD. The samples for the stabilometer and swelling pressure tests were then compacted at the optimum moisture content representative of the group to which it belonged. Figure 8 is typical of the kneading compaction curves from which the optimum moisture content was determined for each group. The 150-psi curves were the basis for this study.

Analyses of variance were conducted on the stabilometer (R-Value) and swelling pressure values. Considering the stabilometer values (R-values), the only factors that may possibly be significant are the between-boring variance (σ_B^2) and the horizon-boring interaction (σ_{HB}^2). As to the swelling pressure, horizons (σ_H^2) and the horizon-topography interaction definitely tested significant,

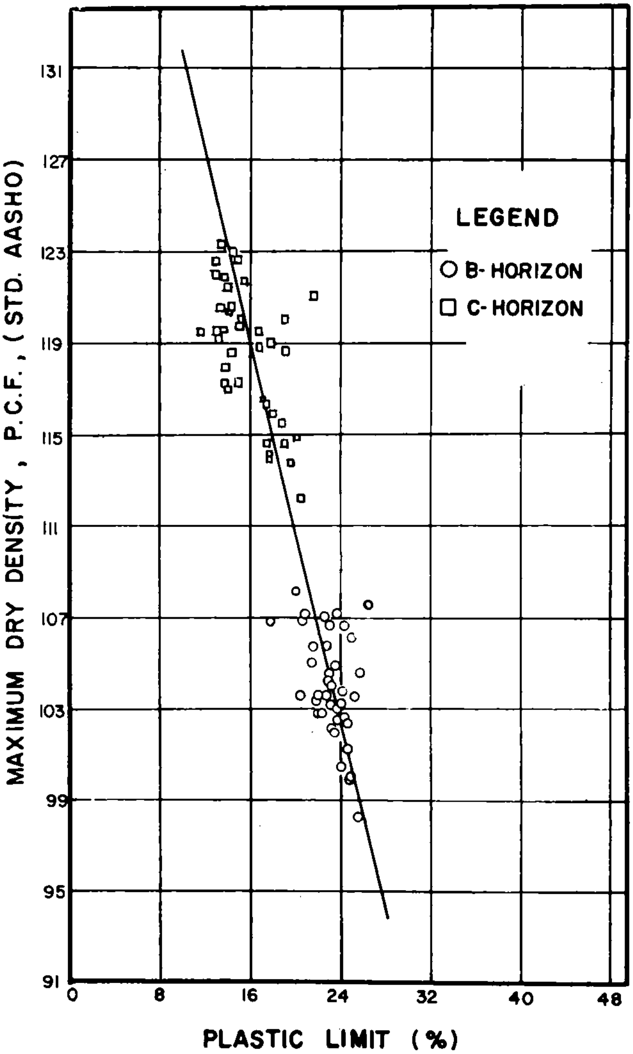

Figure 7. Plastic limit *vs* maximum dry density (Standard AASHO).

and the possibility remains that $\sigma_B{}^2$ and $\sigma_{HB}{}^2$ would test significant.

Inasmuch as there is only one measurement per cell it is impossible to obtain a statistical estimate of the error mean square (σ^2). This makes it impossible to obtain an independent estimate of $\sigma_B{}^2$ or $\sigma_{HB}{}^2$.

Unless independent statistical estimates of the properties in Eq. 2 can be obtained, it is not possible to predict accurately the number of borings required for a given degree of precision. However, to obtain an estimate of the relationship between borings and precision, upper and lower limiting values of σ^2 were assumed. On the basis of experience it is felt that the lower limit should be $\sigma^2=4$ which would give $\sigma_{HB}{}^2=89.41$ and

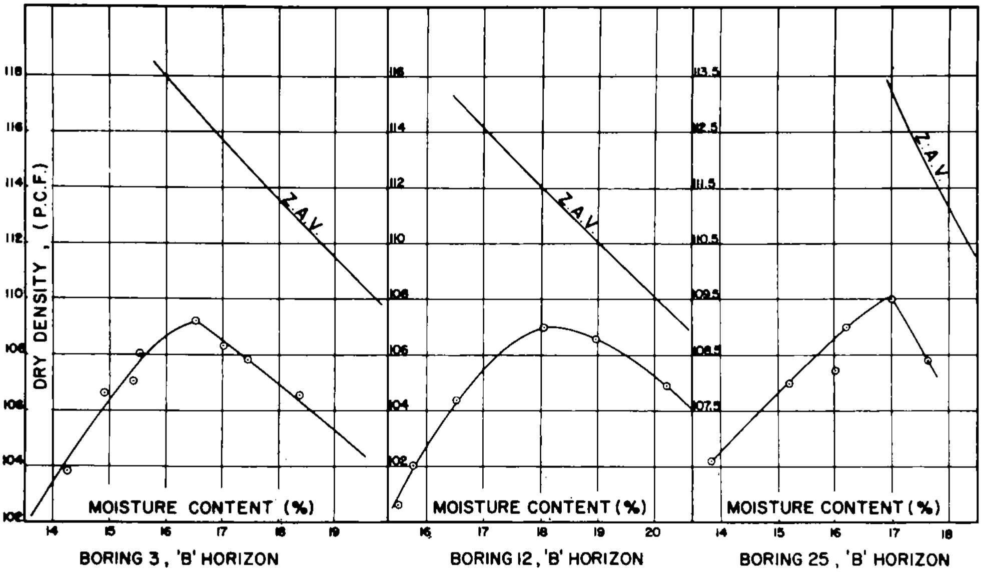

Figure 8. Moisture content *vs* dry density, kneading compaction curves; compactor foot pressure = 150 psi.

$\sigma_B{}^2 = 51.06$. The upper limit is considered to be $\sigma^2 = 36$, giving $\sigma_{HB}{}^2 = 57.41$ and $\sigma_B{}^2 = 70.12$. Thus, for the lower limiting value,

$$\sigma_T{}^2 = 4 + 89.41 + 51.06 = 144.47$$

and for the upper limiting value of σ^2

$$\sigma_T{}^2 = 36 + 57.41 + 35.06 = 128.47$$

Based on these values of $\sigma_T{}^2$ the curves of Figure 9 are obtained.

In Figure 9 the limit of accuracy is expressed in terms of both *R*-value and pavement thickness. It is apparent that pavement thickness is relatively insensitive to small changes in *R*-value. Also, it is evident that the variation in σ^2 produces a relatively insignificant change in the number of borings required for a given degree of precision.

Considering the swelling pressure, it was estimated that the maximum value of σ^2 would be 0.50 psi^2 and the minimum value 0.1 psi^2. Thus, the values obtained for the total variance, $\sigma_T{}^2$, are

$$\sigma_T{}^2 = 0.5 + 1.5 + 1.13 = 3.13$$

and

$$\sigma_T{}^2 = 0.1 + 1.9 + 1.33 = 3.33$$

It is apparent from Eq. 4 that there will be no significant difference between the number of borings required based upon the limiting values of σ^2. The curve shown in Figure 10 is for $\sigma_T{}^2 = 3.33$.

The limit of accuracy is expressed in terms of both pounds per square inch and pavement thickness required to prevent swell. It is evident that a small change in swelling pressure causes a large change in the pavement thickness required to prevent swell. For example, if there is an error in the swelling pressure of 0.8 psi, the estimate of the thickness required to prevent swell may be in error by as much as 10.8 in.

CBR Test

Only six samples showed a CBR of more than 12, and the great

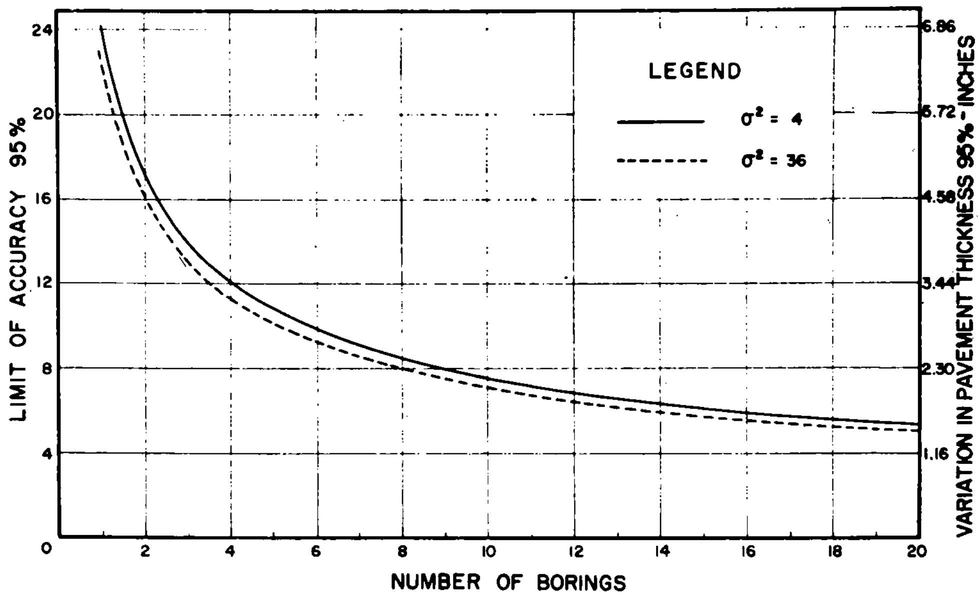

Figure 9. Limit of accuracy *vs* number of borings, *R*-value.

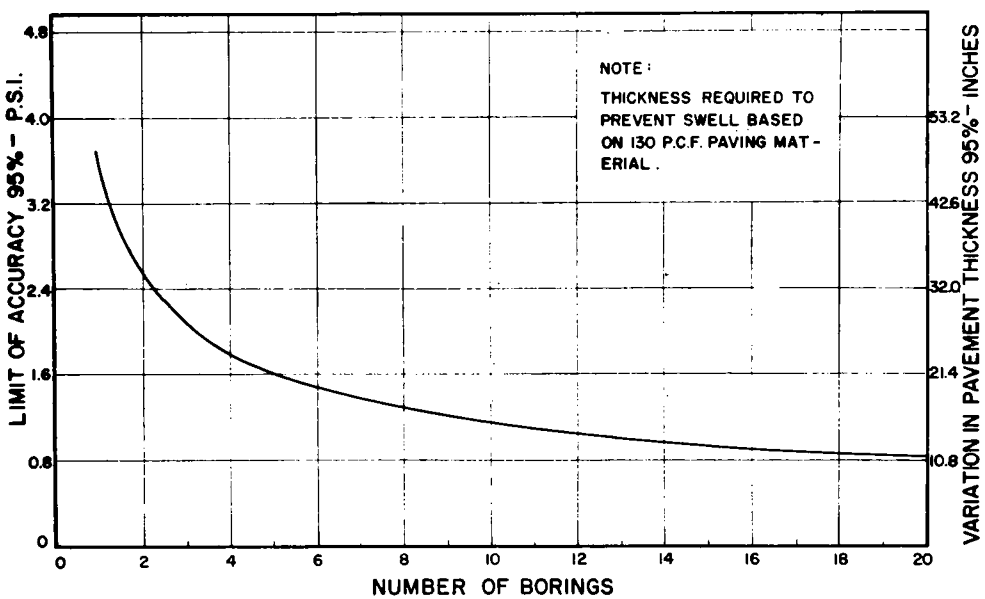

Figure 10. Limit of accuracy *vs* number of borings, swelling pressure.

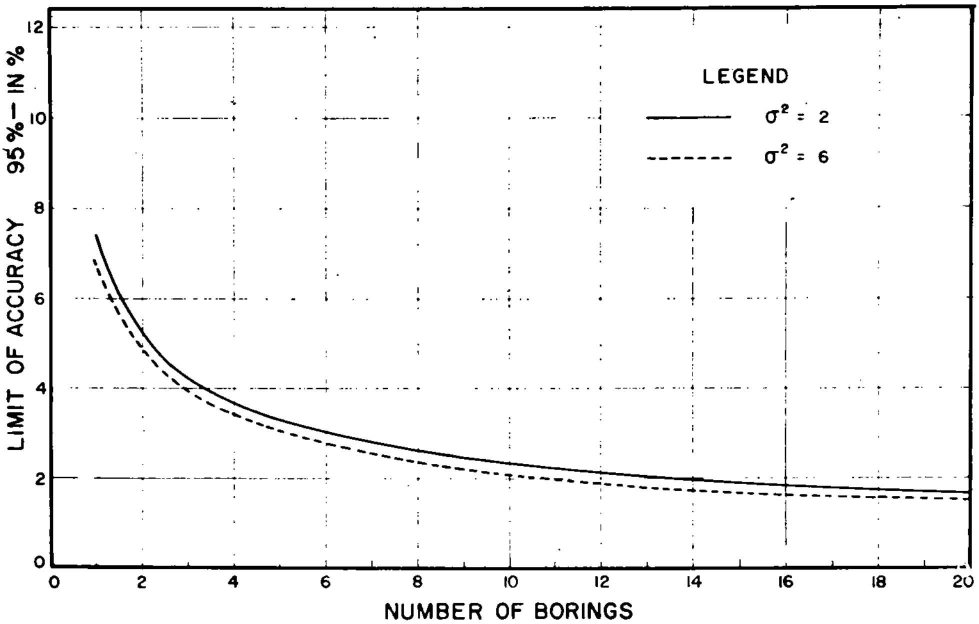

Figure 11. Limit of accuracy *vs* number of borings, CBR.

majority had CBR values less than 10. Of the samples that had CBR values greater than 12, five were from the C-horizon.

In some instances, the CBR value from the B-horizon was greater than that for the C-horizon. This will be explained in the discussion of results.

An analysis of variance was conducted on the results and the relatively small values of the mean squares were noted. This indicated that the variability in the test results was low. Also, only the county-topography interaction tested significant.

Assuming that the maximum value of $\sigma^2=6$ and the minimum value of $\sigma^2=2$, then

$$\sigma_T^2=6+4.32+1.37=11.69$$

and

$$\sigma_T^2=2+8.32+3.37=13.69$$

These values are then used in establishing the curves of Figure 11. It is evident that the magnitude of σ^2 has a nominal effect on the number of borings required for a given degree of precision.

In practically all cases some swell occurred, the magnitude of the swell being greatest for the B-horizon.

Grain-Size Analysis

The data from the grain-size analysis are in two parts: the percent of material finer than 0.074 mm (No. 200 sieve) and the percent of material finer than 0.002 mm. A summary of this information is given in Table 8, which gives the maximum, minimum and mean values of these properties. It is apparent that the soils are fine grained and that the mean values for the measured properties do not vary greatly with county. However, the range (maximum less minimum values) seems to be greater for the rises than the depressions, when comparing counties.

From an analysis of variance on the percent of material finer than

TABLE 8
SUMMARY OF GRAIN-SIZE DISTRIBUTION DATA

County	Topography	Material Finer Than (mm)	Horizon	Percent Passing		
				Min.	Max.	Mean
Tipton	Rise	0.074	B	67.7	95.7	87.1
			C	46.9	75.5	62.8
		0.002	B	25.4	37.5	31.7
			C	16.0	22.5	21.2
	Depression	0.074	B	84.5	97.0	91.8
			C	62.7	87.2	70.9
		0.002	B	26.0	39.4	33.8
			C	13.5	31.0	22.8
Madison	Rise	0.074	B	75.1	96.4	86.8
			C	59.0	94.0	67.3
		0.002	B	23.0	35.0	29.0
			C	11.6	25.5	21.2
	Depression	0.074	B	81.1	96.8	89.9
			C	53.6	80.1	61.6
		0.002	B	28.5	38.0	33.7
			C	17.0	24.0	20.8

0.074 mm it was evident, that the mean square is highly variable due to the magnitude of its values. Also, the factors that tested significant are horizons and the horizon-county-topography interaction. Based on the magnitude of the "Horizon" MS, it was apparent that this effect must be held constant to obtain a reasonable degree of accuracy.

Because there is only one measurement per cell it is not possible to obtain a statistical estimate of the error mean square σ^2. Therefore, to estimate the number of borings required for a given degree of precision it is necessary to assume values of σ^2. To bracket the proper value of σ^2, it was assumed that its maximum would be 25 and its minimum, 4. On this basis, the estimates of the total variance are

$$\sigma_T{}^2 = 25 + 13.73 + 30.22 = 68.95$$

and

$$\sigma_T{}^2 = 4 + 34.73 + 40.72 = 79.45$$

These values along with Eq. 4 are used to establish the relationships shown in Figure 12. It is apparent that variations in σ^2 do not have a large effect on the number of borings required for a given degree of precision.

Considering the data for the percent finer than 0.002 mm, the only two effects that tested significant were horizons and topography.

Based on the expected mean square of the between-boring main effect, the maximum possible value of σ^2 is 16.85 for data obtained. However, it is felt that a more realistic maximum value would be 9 and the minimum value 1. On this basis the estimates of the total variance become

$$\sigma_T{}^2 = 9 + 50.20 + 3.92 = 63.12$$

and

$$\sigma_T{}^2 = 1 + 58.20 + 7.92 = 67.12$$

Due to the closeness of the square root of these two values, there is a negligible difference between the curves of limit of accuracy vs the number of borings for the two cases considered. Therefore, only the curve for $\sigma_T{}^2 = 67.12$ was plotted (Fig. 13).

From Figures 12 and 13 the order of variability of the grain-size distribution properties may be determined. The more variable grain-size property is the percent finer than 0.074 mm, followed very closely by the percent finer than 0.002 mm.

Unconfined Compression Test

The soils were divided into three groups, based on Standard AASHO

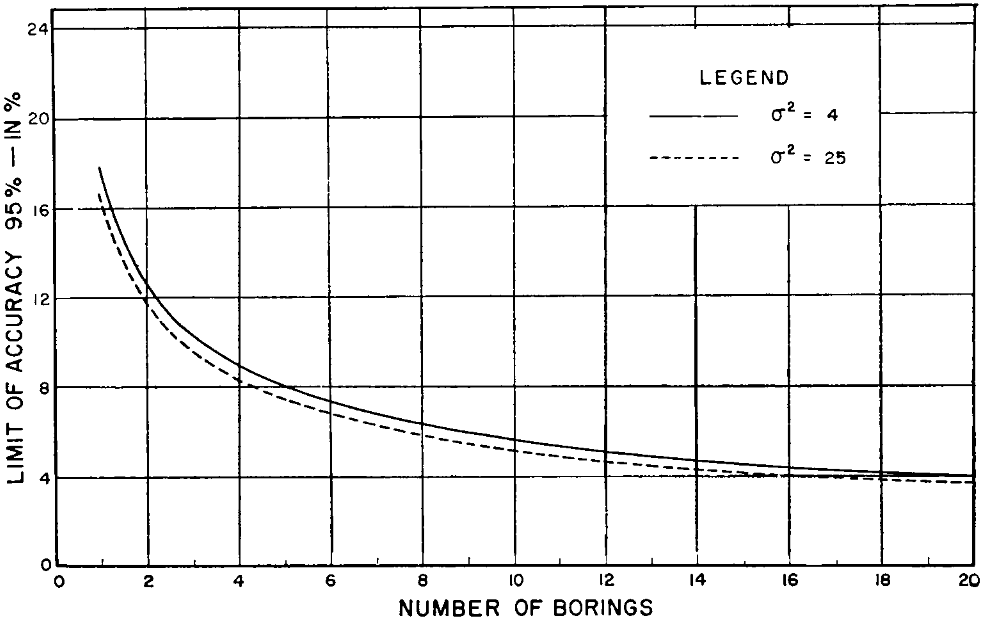

Figure 12. Limit of accuracy *vs* number of borings, percent finer than 0.074 mm.

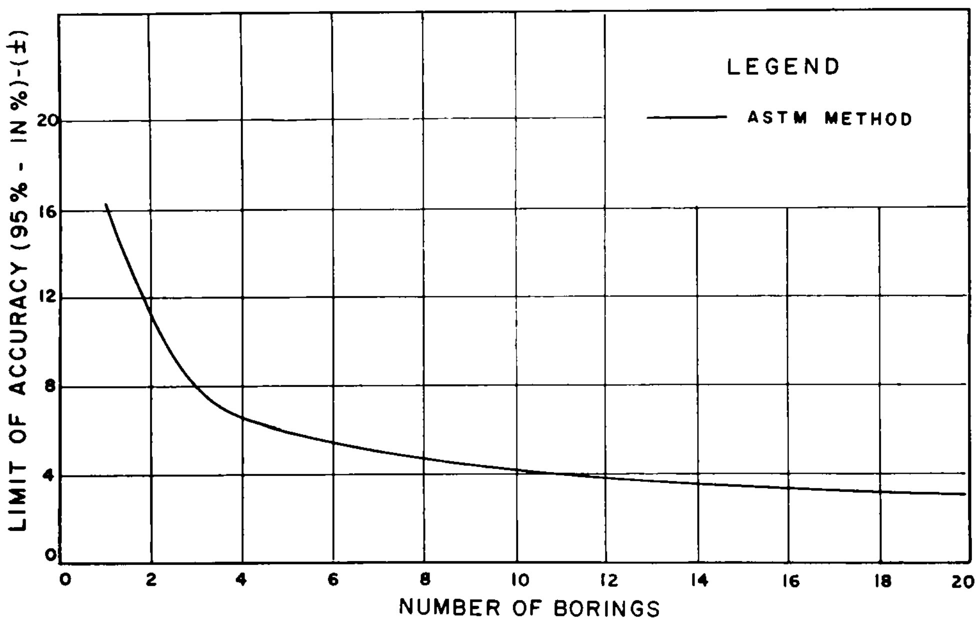

Figure 13. Limit of accuracy *vs* number of borings, percent finer than 0.002 mm.

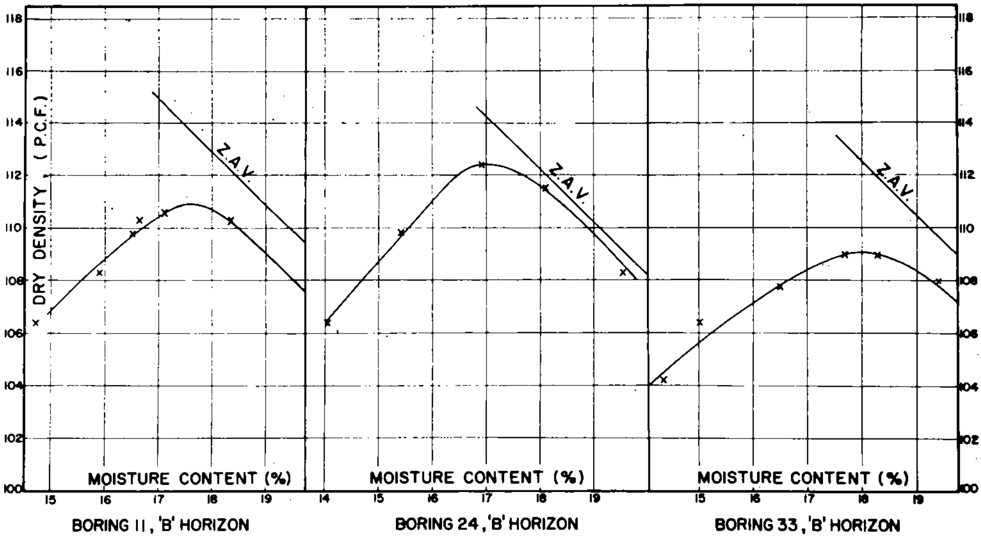

Figure 14. Moisture content *vs* dry density, kneading compaction curves; Harvard miniature compactor, 40-lb spring.

density. A compaction test was conducted on a member of each group to determine the optimum moisture content for that group (see Fig. 14 for typical curves). Subsequently, unconfined compression test specimens were molded at the moisture content representative of the group in which it was a member.

The main effects that tested significant, based on an analysis of variance, were depression vs rise (topography) and horizons. The only interaction term that proved significant was the horizon-county interaction. More factors did not test significant because of the large values for the horizon-boring and between-boring effects.

It was not possible to determine the error variance because only one test was run per sample. Therefore, it was necessary to assume a maximum estimate of the error variance of $\sigma^2 = 6$ and a minimum value of $\sigma^2 = 1$. Based on the maximum value, $\sigma_T^2 = 186.90$, and for the minimum value, $\sigma_T^2 = 204.41$. From these estimates of the total variance the relationship between the number of borings and the limit of accuracy was determined (Fig. 15).

The unconfined compressive strength of the B-horizon was greater than that of the C-horizon. Also, in comparing a given horizon, the unconfined compressive strength of the depressions exceeded that of the rises. No definite trend could be established as to the relative strengths of the soils in Madison County vs the soils in Tipton County.

ANALYSIS OF DATA

Atterberg Limits

The mean squares (MS) of the various estimates are indicators of the relative contribution of these effects to the variance. Considering the effects that tested significant, the liquid limit (LL) is much more variable than the plasticity index and the plasticity index is much more variable than the plastic limit (the magnitude of the MS decreasing for a given effect from the former to the

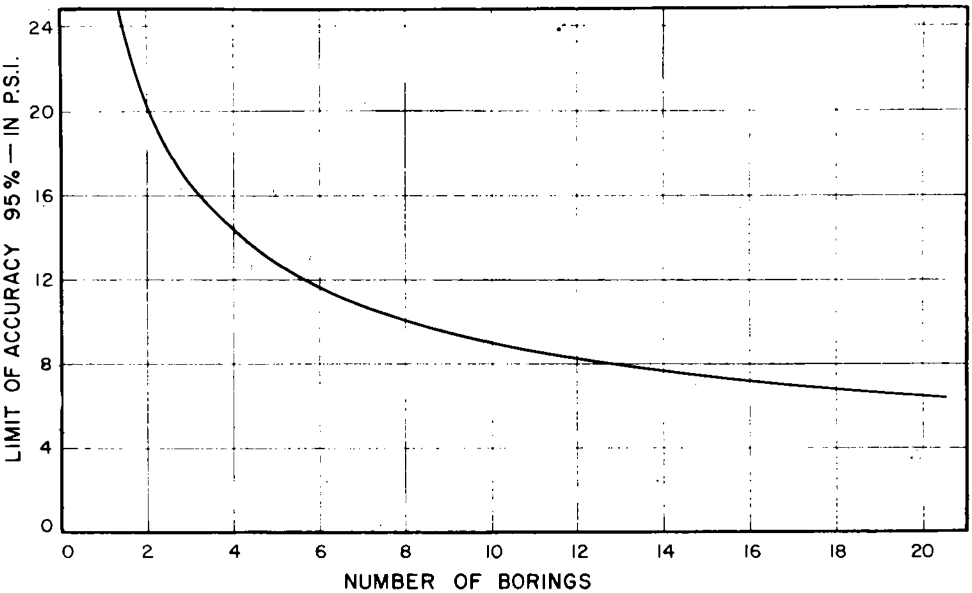

Figure 15. Limit of accuracy *vs* number of borings, unconfined compressive strength.

latter). This indicates that the plastic limit (PL) is relatively constant for the given parent material area even though the values of the LL and PI may vary over a large range. Thus fewer borings would have to be made to determine the PL to a required degree of precision than either of the other two.

As an example, assuming four borings are taken in the areas under consideration, the LL could then be predicted within approximately ±5.8 percentage points of moisture content, the PI within ±4 percentage points and the PL within ±3 percentage points. This difference in the limit of accuracy only decreases slowly with an increase in the number of borings.

The most important factor contributing to the variation in results is horizon. This factor is much more important than any other factor, as is indicated by the extremely large value of the MS.

The second most important contributor to the variation in the results is topographic position; *i.e.*, whether the soil came from a rise or a depression. The third is the interaction variation due to the relationship betwen topographic position and horizon.

There is not much difference between the other two factors that tested significant (between borings in the *C-D* cells and the horizon-boring interaction).

Because only one of the factors that tested significant is used to determine the relationship between the number of borings and the precision (horizon-boring interaction), the other factors should be kept constant in future sampling procedures to predict the mean value of the Atterberg limits. For example, data from the B- and C-horizons should not be used to predict the mean value of the B-horizon. This is to be expected from a knowledge of soil profile development.

On the basis of the analysis of variance for the Atterberg limits, it was observed that the error mean square $E_{m(ijlk)}$, is relatively large for the LL and PL (5.71 and 5.22, respectively. This signifies that an error of as much as ± 2.39 percentage points of moisture, in the case of the LL, may be introduced as a result of the test method and operator effect.

At this point, it is necessary to consider the factors, other than boring location, topography, and horizon that contributed to the variance of the Atterberg limit results in this study. Four factors are initial moisture content, operator, depth at which the sample was obtained, and clay mineral content.

Natural Moisture Content.—It has been established that drying a soil sample before testing significantly alters the Atterberg limits. This is particularly true if the drying is allowed to progress below the shrinkage limit. Consequently, the values of the Atterberg limits determined by conducting tests on soil at its natural moisture content may be significantly different from the values obtained from tests conducted on air-dry soil. The amount of the difference depends on the degree of plasticity of the soils; *i.e.*, the greater the degree of plasticity the greater the difference.

The natural moisture contents of the C-horizon were found to be significantly greater than the plastic limit, for the depressions. However, in most instances, the natural moisture content for the rises was approximately equal to or less than the plastic limit. The reason for this is no doubt due to the position of the water table. In the depression borings, water was encountered in practically every hole, though borings in the rises intercepted water in only one instance.

As to the B-horizon, in Tipton County the natural moisture content of the depression soils exceeded the plastic limit, in practically all cases and in Madison County it was less than or equal to the plastic limit. This is directly related to the position of the water table. In Tipton County the water table lies much closer to the surface of the ground than in Madison County. Therefore, considering capillary effects the expected natural moisture content for the B-horizon soils of Tipton County would be greater than for those of Madison County.

The A-horizons of both counties had natural moisture contents, in most cases, less than the plastic limit. This is to be expected because it is in this horizon that ambient temperature changes have their greatest effect. Also, this is the horizon in which the greatest fluctuation in moisture content occurs; as one goes deeper below the surface, the moisture content of the soil becomes more stable.

On the basis of this information, inasmuch as the Atterberg limits were conducted on samples that were not air dried, a portion of the variance was due to the variation in the natural moisture content of the samples.

Operator.—A certain portion of the variance is due to the fact that different operators were used. The number of tests conducted is as follows: Operator 1, 75; Operator 2, 165; total, 240. However, the possibility exists that there is a significant difference between Operators 1 and 2. Such is indicated by the relatively large value of the error mean squares of the LL and the PI, and was shown to be so on the basis of an analysis of variance.

Depth of Sampling.—An attempt was made to obtain each Atterberg limit sample (for a given horizon) at the same depth below the surface of the ground. This control may not have been sufficient because it does not take into consideration the thickness of each horizon. For example, the clay content of the sample, which is one of the major factors in determining the value of the Atterberg limits, is a function of the depth below the surface of the horizon at which the sample is obtained. For example, a sample obtained near the upper surface of the B-horizon will be less plastic than one obtained from the lower boundary of the B-horizon. Consequently, if the thickness of the horizons are not taken into consideration a variability in the results will be introduced. Whether this variation is significant is debatable.

In the C-horizon it was not always possible to take the Atterberg limit samples at the same depth. The interface of the B- and C-horizons was determined by applying hydrochloric acid to the soil as it was removed from the hole. When the acid was placed on material from the C-horizon a noticeable reaction took place. The initial reaction sometimes occurred below the normal sampling depth. Thus, a greater variability of sampling depth was present in the C-horizon.

Compaction Test (Standard AASHO)

Due to the factors that tested significant, for the best results, it is necessary to keep horizon and topography constant, considering the OD. Such a procedure will result in the fewest number of samples being required to predict the population mean value because it eliminates the variability due to the interactions that tested significant.

Considering the optimum moisture content data, the only factor that tested significant and is not considered in the total variance is the horizon effect. Thus, as far as obtaining the total variance, for a given horizon it would not be necessary to discriminate on the basis of topography or counties. In other words, for a given horizon there is no significant difference between the total variance of a rise and that of a depression regardless of county. However, horizons, topography, and counties should be held constant for the maximum degree of accuracy for a given number of borings. It is recognized that the optimum moisture content and optimum density are determined simultaneously for a given soil. Nevertheless, from the standpoint of establishing construction requirements, it minimizes the need for making a large number of compaction tests.

Hveem Stabilometer and Swelling Pressure Tests

Compaction.—Stability numbers (R-values from the Hveem stabilometer) and swelling pressures are a function of the method of compaction, the compacted moisture content, and density. Moisture content was considered one of the most important variables. An attempt was made to compact the samples with ± 0.5 percent of the optimum moisture content.

At moisture contents slightly in excess of the optimum, and in some instances at the optimum value, there was appreciable shoving of the surface under the action of the compactor foot (150-psi foot pressure). Whenever this situation occurred, it took place toward the latter phase of the compaction process. Thus, the possibility exists that as the compaction process progressed there were created large positive pore pressures and that, with time, these became sufficient to produce shear failure, under subsequent action of the foot.

This method of compaction may result in a nonhomogeneous sample. This is mainly due to the fact that compaction occurs from the top down. Consequently, one would expect a variation in compacted density with depth. This no doubt affects the strength, compressibility, and swelling characteristics of the compacted soil.

R-*Values.*—Due to the relatively small range of mean squares, no single effect had a dominant roll in determining the R-value. However, due to this relative "uniformity," the total variance estimate is much higher than for any of the other measured properties (with the exception of the unconfined compressive strength). Thus, speaking in absolute terms, the number of borings required for a given degree of precision is much greater (see Fig. 9).

In essence, the stabilometer test is a triaxial test. Consequently, the factors that affect the shearing resistance as determined by triaxial test should affect the R-value (port pressures, mineralogy, density, etc.). Therefore, considering a given parent material group, it appears reasonable to expect the variance estimates to be homogeneous.

Figure 9 shows the relationship between the number of borings, limit of accuracy, and pavement thickness. Though the R-value may vary widely, the resulting change in pavement thickness is relatively small.

According to the Hveem method of pavement design, the thickness of pavement required is determined (*5*) by

$$T=\frac{K'(TI)(90\text{-}R)}{5\sqrt{C}} \qquad (6)$$

in which

$K'=0.095$;
$TI=1.35\ \mathrm{EWL}^{0.11}=8.71$ (assumed), EWL is the total number of equivalent 5,000-lb wheel loads anticipated for the design life;
$R=$resistance value (R-value); and
$C=$cohesiometer value$=200$ (assumed).

Based on Eq. 6,

$$T=0.286\ (90\text{-}R).$$

It is evident that there can be a relatively large variation in R-value with only a nominal change in design thickness. Thus, even though the stabilometer values show large variation from hole to hole, the effect as regards pavement thickness is much less variable due to the fact that traffic is the primary control of pavement thickness. K' and TI are a function of traffic.

The variation in R-value encountered in this study as well as the fact that the R-values for compacted soil from the B-horizon, in some instances, exceeded that of the C-horizon may possibly be due to the effect of pore pressures. Because the swelling pressure test preceded the stabilometer test, the samples were tested at a high degree of saturation. Drainage was not allowed during application of the load, and the shear deformations caused an increase in the pore water pressure.

Those soils whose strength is primarily due to internal friction may have low R-values depending on the rigidity of the soil skeleton and the degree of saturation. If the soil structure deforms little at values of the vertical normal stress less than 160 psi (stress at which the R-value is determined) then the magnitude of the pore pressures will be small and the strength component due to internal friction will be large. Naturally, in the case of a compressible soil skeleton or high degree of saturation the converse is true and one might obtain a low R-value.

For soils whose strength is derived principally from cohesion, the situation may be different. In such cases, the effect of pore pressures can be much less if the strength that results from cohesion is not as greatly dependent on the effective stress on the failure plane at failure as is the strength component due to friction. Depending on the magnitude of the strength contributions from cohesion and internal friction, the degree of saturation, the clay minerals present, and the rigidity of the soil structure, it is quite possible to have the R-value for the B-horizon exceed that for the C-horizon.

Also, the optimum moisture content for each sample was not available. It was assumed that the OMC as determined from a representative sample was appropriate for all the members of the group from which it was selected. The assumption is reasonable, but the degree to which it is valid, in all probability, had an effect on the results.

Swelling Pressure.—Factors that affect the swelling pressure may be listed under two general categories—physiochemical and mechanical. Seed, Mitchell, and Chan (*4*) have shown that the mechanical aspect of the swelling phenomena may at times be of such magnitude that it cannot be neglected. However, because all samples were prepared in the same manner it was assumed that the mechanical aspect of the swelling phenomena could be neglected when considering the variation between samples.

The horizon variance tested significant as did the horizon-topography interaction. Considering the physiochemical aspects of the clay minerals present in these soils, such is to be expected. The quantity of a given type of clay mineral present in a sample depends on the horizon from which the sample was obtained. Also, if the minerals of one horizon have a greater affinity for water than the other, then the greatest amount of swell would be expected in the soil with the higher affinity.

The fact that the horizon-topography interaction tested significant was anticipated. In a rise, the soil is well drained, and in a depression, it is poorly drained. The nonexpanding lattice clays are predominant in the rises, and in the depressions expanding lattice clays are in the majority since they are generated best in environments where there is an abundance of moisture.

The exact quantitative relationship between the quantity of a given clay mineral and the amount of swell was not determined because of the heterogeneity of the amount of clay minerals that may exist at a given point in a given soil mass and the variations in chemical composition and in the weathering stage. Nevertheless, the effect of both quantity and type of clay minerals on the swelling properties of a given soil can be estimated qualitatively.

On the basis of the swelling pressure test it was found that this factor varied greatly with change in moisture content. In some instances, a change in moisture content of 1 percent caused a change in the swelling pressure of as much as 3 psi. Such a change results in a change of flexible pavement thickness required to prevent swell of 40 in. This represents an extreme circumstance, but a difference in thickness of one-tenth this amount is intolerable. Consequently, in those circumstances where the soil may come into equilibrium with free water, it is necessary that its swelling characteristics be adequately defined. Correspondingly, if the soil is to be used as borrow, its compaction moisture content should be specified in such a manner that difficulty from excessive swell will not arise.

The moisture content at which these samples were molded is representative of the OMC of the sample. Compaction of a soil at optimum moisture content and its corresponding density generally yields satisfactory results in regard to swell under prototype pavements.

In addition to satisfying stability requirements, it is necessary to insure that the pavement will not heave when coming in contact with free water. Both requirements are satisfied if the thickness of pavement is adjusted so that thickness by R-value is made equal to thickness by expansion pressure. This will usually result at a molding moisture content different from the optimum value. Nevertheless, in most instances the thickness required for stability at the OMC is less than the thickness required to prevent swell. Consequently, the desirable placement moisture content in the field in all probability is greater than the OMC obtained in the laboratory.

The data suggest that, in spite of the small hole-to-hole variation in thickness indicated by the stabilometer test, the combined effects of swelling and R-value may result in extreme variation. Figure 10 shows that a small change in swelling pressure means a relatively large change in thickness required to prevent swell.

CBR Data

In many instances, the CBR value for compacted soil from the C-horizon proved to be less than the value for the B-horizon. This is contrary to the normal trend and the difference, although not large, was consistent throughout much of the program.

The most probable causes of the event must lie in the degree of saturation of the upper inch of the sample and/or the difference in quantity and type of clay minerals present in the B- and C-horizons. Although the mineralogy of the soils may have contributed to this effect, a definite relationship could not be established on the basis of available data.

Of the 29 borings in which the CBR value of the C-horizon was found to be less than that of the B-horizon, the moisture content of the upper inch of the sample was much closer to the liquid limit for the C-horizon samples. Because the strength of a soil at the liquid limit is very low (approximately 25 g per sq cm) and is much greater at the plastic limit, the CBR value for the B-horizon is expected to be greater than for the C-horizon.

For CBR values equal to or less than 12 the following equation was used to determine the required thickness of pavement (*2*):

$$t = \sqrt{P\left[\frac{1}{8.1(\text{CBR})} \frac{-1}{p\pi}\right]} \tag{7}$$

in which

t = design thickness of the pavement structure in inches;
P = total wheel or (equivalent wheel) load in pounds; and
p = tire pressure in psi.

However, for CBR values greater than 12, the curve representative of Eq. 7 was extended, as shown in Plate 1 (*2*).

Total wheel load was assumed to be 5,000 lb and the tire pressure 70 psi. Also, the thickness obtained from Eq. 7 is for 5,000 coverages.

To keep the effect of repetition of load on pavement thickness approximately constant for both the stabilometer and CBR tests, it is necessary that the CBR requirement for thickness be adjusted for a number of coverages equivalent to 23.3 million repetitions of a 5,000-lb wheel load.

Table 4.4 (*5*) shows that there are approximately 2.2 trips of a 5,000-lb wheel load required for one coverage. Therefore, the thickness obtained from Eq. 6 should be adjusted for 10.6 million coverages. The adjustment in thickness will be made in accordance with Plate 3 (*2*). Based on an extension of this plate, it is found that 176 percent design is required for 10.6 million coverages. Thus the pavement thickness determined on the basis of 5,000 coverages must be increased by 76 percent.

Comparing the thickness by CBR with the thickness by stabilometer, for the same number of coverages no definite trend could be established for all the data. With the B-horizon, the greater thickness of pavement was obtained in some instances using the CBR method and on about an equal number of occasions using the

Highway Research Board, 33rd
Annual Meeting, 1954.
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stabilometer. However, with the C-horizon, the CBR method produced the greater thickness in the great majority of cases.

Finally, the total variance of the CBR is relatively small. However, at low values of this parameter a small variation in CBR value produces a large variation in thickness (see Eq. 7).

Unconfined Compression Test

The large variability of the unconfined compressive strength is possibly due to variations in cohesion and moisture content. The former is also a function of the quantity and type of clay minerals present in a given sample.

A certain amount of cohesion is required for stability of unconfined compression samples. This cohesion allows a greater time to reach the failure load and hence a greater strength. There is a greater quantity of clay in the B-horizon than the C-horizon and it was anticipated that the former had the greater strength. The aforementioned factors also tend to explain why the unconfined compressive strengths of the depression soils were greater than the rises. On the basis of this, because the unconfined compressive strength is very sensitive to the amount of cohesion, the variability of the results is expected to be large.

The unconfined compressive strength of a soil varies with its compacted moisture content. Moisture density curves were not established for each sample and therefore this may have introduced a small error.

As a result of the factors that tested significant, it is necessary to hold topography and horizons constant when using this test as a measure of variability. However, due to the large value of the total variance the unconfined compression test is a good measure of variability. At the same time it is too sensitive for practical use. For example, a soil would have to be exceptionally homogeneous before the variation in results would allow a reasonable number of samples to be taken to define adequately this property over a relatively large area.

SUMMARY OF RESULTS AND CONCLUSIONS

The method of selecting boring sites and the number of borings depends on the factors that tested significant in the analyses of variance. For the most precise results, the factors that tested significant and are not included in the determination of σ_T^2 should be held constant:

Property	*Factors To Be Held Constant*
Liquid limit	Topography and horizons
Plasticity index	Topography and horizons
Plastic limit	Topography and horizons
Optimum density	Topography and horizons
Optimum moisture content	Horizons
R-value	None
Swelling pressure	Horizons
CBR	Topography
Percent finer 0.074 mm	Horizons
Percent finer 0.002 mm	Topography and horizons
Unconfined comp. str.	Topography and horizons

County never tested significant for any of these properties. Theoretically this means that one could sample the soils in Tipton County and use the results of tests on these samples to predict the properties of soils in Madison County. However, this is not too safe, because failing to find significance does not prove that there is no difference: there simply was no reliable evidence of any difference. If there is a difference between counties it is likely to be relatively small. Hence, to obtain a more accurate estimate it would be better to base the estimate on samples from both counties. For example, if it is desired to define certain properties of a soil within a specified limit and 10 borings are required, if the areas of interest are far apart it would be better to base estimates on 5 samples from each area rather than 10 samples from one area. The aforementioned is based on the assumption that the soils in the areas are of the same pedologic classification and have similar airphoto patterns.

In using the total variance estimates to determine the number of borings required to define certain properties to within specified limits, one must consider the effect of an error in classification. The total variance estimates contained in this report are based on soils pedologically classified as Brookston (depressions) and Crosby (rises). Consequently, the variance estimates are strictly valid for these soils alone. If the data were applied, by mistake, to soils that did not fit either of these classifications error might result. However, the magnitude of this difference cannot be ascertained without similar research projects on soils of various classifications.

It was assumed that the variance of the measured properties was independent of horizon. This is logical because the B-horizon soils were derived from the C-horizon soils. However, it was not posible to check this assumption because the B- and C-horizon samples were obtained from the same boring. This correlation cannot be taken into consideration statistically.

There are several approaches to the use of information on the variability of soils for design. If the mean value of the design parameter is used, then in general the structure will be overdesigned 50 percent of the time and underdesigned 50 percent of the time. If this situation is not satisfactory it can be altered by using the computed standard deviation of the mean with the proper significance level. The procedure is as follows:

1. Determine the standard error of the mean, as shown in Eq. 2.
2. Based on the significance level chosen, establish the relationship between the number of borings and the limit of accuracy, as shown in Eq. 4.
3. Subtract the limit of accuracy from the mean value obtained from n number of samples.
4. Determine the pavement thickness required on the basis of the value obtained from step 3.

This procedure will insure that the pavement on the average will prove satisfactory $100(1-\alpha)$ percent of the time. In the preceding statement, α is the significance level chosen. In this study, $\alpha=0.05$. Naturally, if in step 3 the limit of accuracy were added instead of subtracted, the resulting design would be unsatisfactory $100(1-\alpha)$ percent of the time. This method assumes normality in the distribution of the measure in question.

Based on the information presented in this report the following conclusions appear justified:

1. To minimize the variation in results due to differences in weathering stage of the clay minerals, all samples should be taken from the same depth below the surface of the horizon under consideration.
2. The low variability of the optimum moisture content data indicates that the number of samples required for construction control would be few.
3. To give a realistic value for the areas under question, a minimum of six samples will normally suffice. Actually, the number of samples required depends on the degree of precision required for the properties of interest. However, with the exception of the highly variable properties, six samples should suffice.
4. The Atterberg limits are affected by the amount of drying to which the samples have been subjected. Consequently, if facilities are not available in which the soils can be maintained at a constant moisture content, it would be best to air dry all samples before conducting the test. This would reduce the variability of the results.

5. Assuming good laboratory technique, the effect of the operator and testing procedure depends on the magnitude of the total variance. For large values of the total variance the effect of large variations in the error mean square on the number of samples required for a given degree of precision is small. However, to increase the accuracy of variability studies it would be best to use just one operator for a given series of tests.

6. Due to the magnitude of the error that may be introduced into the results of Atterberg limit determinations as a function of the test procedure and operator effect, it appears that a one-point method of determining the liquid limit is justified.

7. The Hveem method of flexible pavement design in relation to stability is relatively insensitive to the strength properties of the soil as determined by the *R*-value. Large variations in *R*-value can occur with only a relatively small change in pavement thickness required for stability. This is due mainly to design thickness being principally controlled by traffic considerations.

Conversely, the variation in the swelling pressures is relatively small. However, a small change in the swelling pressure results in a large change in the thickness required to prevent swelling. Because both stability and swelling requirements must be satisfied in the Hveem method of design, there may occur large variations in required pavement thickness for a given area.

8. The variance of the CBR values was relatively small. However, they are in the low CBR range with the result that a small change in the CBR value necessitates a large change in pavement thickness.

9. Based on the variability of the reported data, designing on the basis of soil classification or some other simple procedure is justified. This is due to the large variation in design thickness which within a given area will occur because of the variation in the parameter that forms the basis for the design. Also, such variation in results strongly suggests the use of a statistical approach to pavement design.

10. Disparity in variability between the unconfined compression, CBR, and stabilometer tests is probably due to the failure criteria and to the fact that the latter two tests are run on soaked samples.

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GENERAL

Awards

HIGHWAY RESEARCH BOARD AWARD

THE HIGHWAY RESEARCH BOARD AWARD was instituted in 1940 for the purpose of giving recognition to the authors of papers of outstanding merit presented at the annual meetings of the Board. The recipients of the award have been:

1940 WILFRED OWEN, for the paper "Trends in Highway Financial Practice," presented at the Nineteenth Annual Meeting.

1941 F. V. REAGEL, for the paper "Freezing and Thawing Tests of Concrete," presented at the Twentieth Annual Meeting.

1942 M. B. RUSSELL AND M. G. SPANGLER, for the paper "The Energy Concept of Soil Moisture and Mechanics of Unsaturated Flow," presented at the Twenty-First Annual Meeting.

1943 RALPH A. MOYER, for the paper "Motor Vehicle Operating Costs, Road Roughness and Slipperiness of Various Bituminous and Portland Cement Concrete Surfaces," presented at the Twenty-Second Annual Meeting.

1944 DONALD J. BELCHER, for the paper "The Engineering Significance of Soil Patterns," presented at the Twenty-Third Annual Meeting.

1945 DONALD LOUTZENHEISER, for the paper "Proposed Design Standards for Interregional Highways," presented at the Twenty-Fourth Annual Meeting (*unassembled*).

1946 K. B. WOODS, HAROLD S. SWEET AND T. E. SHELBURNE, for the paper "Pavement Blowups Correlated with Source of Coarse Aggregate," presented at the Twenty-Fifth Annual Meeting.

1947 NORMAN W. MCLEOD, for the paper "Airport Runway Evaluation in Canada," presented at the Twenty-Sixth Annual Meeting.

1948 C. R. HANES, for the paper "Some Practices Used by Ohio in the Salvaging of Old Pavements," presented at the Twenty-Seventh Annual Meeting.

1949 F. N. HVEEM AND R. M. CARMANY, for the paper "The Factors Underlying the Rational Design of Pavements," presented at the Twenty-Eighth Annual Meeting.

1950 ROY E. JORGENSEN AND ROBERT G. MITCHELL, for the paper "Accident Analysis for Program Planning," presented at the Twenty-Ninth Annual Meeting.

1951 THOMAS J. CARMICHAEL AND CHARLES E. HALEY, for the paper "A Study of Vehicle, Roadway, and Traffic Relationships by Means of Statistical Instruments," presented at the Thirtieth Annual Meeting.

1952 HUGO C. DUZAN, WILLIAM R. MCCALLUM AND THOMAS R. TODD, for the paper "Recent Trends in Highway Bond Financing," presented at the Thirty-First Annual Meeting.

1953 EARL C. SUTHERLAND AND HARRY D. CASHELL, for the paper "Structural Effects of Heavy-Duty Trailer on Concrete Pavement," presented at the Thirty-Second Annual Meeting.

1954 C. A. ROTHROCK, for the paper "Urban Congestion-Index Principles," presented at the Thirty-Third Annual Meeting.

1955 CARL C. SAAL, for the paper "Operating Characteristics of a Passenger Car on Selected Routes," presented at the Thirty-Fourth Annual Meeting.

1956 CHESTER MCDOWELL, for the paper "Inter-Relationship of Load, Volume Changes, and Layer Thickness of Soils to the Behavior of Engineering Structures," presented at the Thirty-Fifth Annual Meeting.

1957 GEORGE M. WEBB AND KARL MOSKOWITZ, for the paper "California Freeway Capacity Study—1956," presented at the Thirty-Sixth Annual Meeting.

1958 ALAN M. VOORHEES, for the paper "Peak Hours of Travel," presented at the Thirty-Seventh Annual Meeting.

1959 CHARLES J. KEESE, CHARLES PINNELL AND WILLIAM R. MCCASLAND, for the paper "A Study of Freeway Traffic Operation," presented at the Thirty-Eighth Annual Meeting.

1960 KENNETH A. STONEX, for the paper "Roadside Design for Safety," presented at the Thirty-Ninth Annual Meeting.

1961 GEORGE HAIKALIS AND HYMAN JOSEPH, for the paper "Economic Evaluation of Traffic Networks," presented at the Fortieth Annual Meeting.

A. S. LANG AND DAVID H. ROBBINS, for the paper "A New Technique for Prediction of Vehicle Operating Costs in Connection with Highway Design," presented at the Fortieth Annual Meeting.

ROY W. CRUM DISTINGUISHED SERVICE AWARD

The Highway Research Board, in 1948, established an award to be made in recognition of outstanding achievement in the field of highway research. This award was known as the "Highway Research Board Distinguished Service Award" until 1952 when the Executive Committee redesignated it the ROY W. CRUM DISTINGUISHED SERVICE AWARD as a memorial to the Board's late director, Roy Crum, who served as head of the HRB staff from 1928 until his death in 1951. Outstanding achievement consists of distinguished service, production of fundamental or developmental research or the administration, promotion or fostering of outstanding research, which in the judgment of the Executive Committee is worthy of the Award. Recipients of the Award have been:

CHARLES H. SCHOLER	1948
FRANK H. JACKSON	1948
KENNETH B. WOODS	1949
O. K. NORMANN	1949
FRED V. REAGEL	1950
ALBERT T. GOLDBECK	1950
ROY W. CRUM (*posthumously*)	1951
HERBERT S. FAIRBANK	1952
PREVOST HUBBARD	1953
CHARLES R. WATERS	1953
BURTON W. MARSH	1954
RALPH A. MOYER	1954
WALTER H. ROOT (*posthumously*)	1954
EARL FOSTER KELLEY	1955
TILTON E. SHELBURNE	1955
STANTON WALKER	1955
FRANCIS N. HVEEM	1956
EDWARD H. HOLMES	1957
HARMER E. DAVIS	1958
GUILFORD PAYSON ST. CLAIR	1959
MERLIN G. SPANGLER	1959
ROBERT R. LITEHISER	1960
HAROLD ALLEN	1960
ELMER M. WARD (*posthumously*)	1961
REX M. WHITTON	1961
WILLIAM H. GOETZ	1961

ELMER M. WARD

The Highway Research Board has named Elmer M. Ward to receive posthumously its annual *Roy W. Crum Distinguished Service Award* in recognition of his outstanding record of fifteen years with the Board, as a Correlation Service Engineer of Materials and Construction and of Maintenance, and later as Assistant Director of the Board.

Born in Anthon, Iowa, Mr. Ward spent much of his boyhood in South Dakota, then returned to Iowa to attend Iowa State College at Ames, where he graduated in 1927 with a degree in Civil Engineering. It was there that he studied under Roy W. Crum.

Immediately after graduation, he entered his professional career with the Iowa State Highway Commission as District Engineer of Materials and Head of the laboratory for the Materials and Testing Department at Mason City. His experience there focused his interest on the limitations in the knowledge of road building materials, gave him an interest in research, and inspired him to seek improved knowledge of their characteristics and how to utilize them to the best advantage in construction and maintenance.

During the War years, from 1941 until 1946, Mr. Ward was with the Roads and Airfields Branch of the Engineer Board, Corps of Engineers, U. S. Army, at Fort Belvoir, Va., and Yuma, Ariz. While with the Corps of Engineers, he contributed many ideas utilized in the development of methods and equipment for building roads in desert terrain—methods and equipment used by the armed forces in the campaign in North Africa during World War II.

He joined the technical staff of the Highway Research Board in 1946 as Engineer of Maintenance, and in 1949 was also made Engineer of Materials and Construction. While serving in these capacities in connection with the Correlation Service work of the Board he provided thoughtful and helpful liaison with State highway departments, colleges and universities, and many research agencies throughout the United States. His Correlation Service work and his contribution to the work of the Board's committees during this period were preeminent.

In recognition of his administrative ability, Mr. Ward was named Assistant Director of the Highway Research Board in 1954. He served in this capacity with distinction until his death on August 27, 1961. His untimely death has saddened his many friends throughout the country and has made them more acutely aware of the sterling qualities he possessed as an excellent administrator in the field of highway research and as a true friend.

He was elected a member of Phi Kappa Phi in 1956, during which year he was named as "Engineer of the Year" by the Iowa Society of Professional Engineers. He was a Registered Professional Engineer, a Fellow of the American Society of Civil Engineers, and a Member of the Iowa Society of Professional Engineers, the National Society of Professional Engineers, and the "Road Gang," an informal group of top highway engineers in the Washington area.

His loyalty and unswerving devotion to

the work and principles of the Highway Research Board of the National Academy of Sciences—National Research Council went far beyond the call of duty. His efficient and untiring efforts have immeasurably advanced the cause of highway research, and enhanced the stature of the Highway Research Board.

REX M. WHITTON

The Highway Research Board has chosen Rex M. Whitton to receive its annual *Roy W. Crum Distinguished Service Award* in recognition of his outstanding service to highway research and development over a long period of years.

This service has been rendered in two general areas: First, as a State highway official for more than 40 years and as Federal Highway Administrator for the past year; and second, for his direct contributions to the work of the Highway Research Board dating back to 1937.

He is now serving as an *ex officio* member of the Executive Committee of the Board and was a member of the Executive Committee from 1954 to 1961. He was elected Vice Chairman of the Board in 1956 and served as Chairman in 1957. He was a member of the Department of Maintenance from 1938 to 1954 and was Chairman in 1950-51. He has also served on committees of the Department of Economics, Finance and Administration, the Department of Design, and the Department of Traffic and Operations.

To assume his post as Federal Highway Administrator in February 1961, Mr. Whitton resigned as Chief Engineer of the Missouri State Highway Commission after more than 40 years of uninterrupted service with the highway department. Born in Jackson County, Missouri, he attended both grade and high school there before entering the University of Missouri, where he received a degree in civil engineering in 1920.

He began his service with the Missouri State Highway Department in 1920 as a member of a survey party. Advancing through the ranks, he served in various positions until he was elevated to the post of Chief Engineer in 1951.

He was named a member of the Executive Committee of the American Association of State Highway Officials in 1954, and reappointed a member in 1957. He served as President of the AASHO for the year 1956.

He was the recipient of the George S. Bartlett Award in 1958 for outstanding service in highway progress in the Nation.

He was selected by the American Public Works Association as one of the "Top Ten Public Works Men of the Year" in 1960.

He was the 1960 recipient of the Thomas H. MacDonald Award for continuous outstanding service in the highway engineering field.

In June 1961 he received an honorary degree of Doctor of Science from his alma mater, the University of Missouri.

His long record of service to the highway program at the State and National levels, as well as in the work of the Highway Research Board, eminently qualifies Mr. Whitton to receive the *Roy W. Crum Distinguished Service Award.*

WILLIAM H. GOETZ

The Highway Research Board has chosen William H. Goetz to receive its annual *Roy W. Crum Distinguished Service Award* in recognition of his outstanding record of accomplishment, both in research and education, in the field of highway engineering. His concentration has been in the area of highway materials, with particular emphasis on flexible pavements. Professor Goetz has, through his research efforts, made numerous personal contributions toward the understanding and improvement of bituminous materials and mixtures. He has also been the personal mentor of many graduate students who themselves have gone on to professional careers of significant contribution in highway engineering.

A native of Ann Arbor, Mich., Professor Goetz attended the University of Michigan, receiving the Bachelor of Science degree in Chemical Engineering in 1936. After two years employment with the Michigan State Highway Department he joined the staff of the newly formed Joint Highway Research Project in the School of Civil Engineering at Purdue University, where he also received his Master's degree in Chemical Engineering. For many years he has been in charge of studies of bituminous materials and mixtures in the Joint Highway Research Project. As a member of the academic staff he has been responsible for the development of graduate programs of instruction and research, whereby many men have gone on to the Masters and Ph. D. degrees under his direction.

Professor Goetz became a member of the Highway Research Board in 1939 and has attended every Annual Meeting since 1941. His first contact with Highway Research Board committees came in 1941 when he became a member of the Committee on

Paints and Marking Materials. His record of service has been noteworthy. Professor Goetz has at various times held membership on five committees of the Board and been chairman of two. He currently is chairman of the Committee on Bituminous Surface Treatments. In 1956 he was made a member of the Department of Materials and Construction, followed in 1960 by his appointment as Chairman of the Bituminous Division. In the latter post he has effected a reorganization of the Division's committee structure, a change that has just recently become effective but which is already proving its value.

Professor Goetz also has demonstrated his technical and organizational abilities in other technical societies. In the American Society for Testing and Materials he holds membership on and is First Vice Chairman of Committee D-4, Road and Paving Materials. Within this committee he is active on several subcommittees and is chairman of the group dealing with the effect of water on bituminous-coated aggregates. He also is a member of ASTM Committees E-10, Radioisotope and Radiation Effects; E-17, Skid Resistance; and D-1, Paint.

In 1958 Professor Goetz was elected to the presidency of the Association of Asphalt Paving Technologists, climaxing a long period of active participation in the activities of that organization.

Honors are not unknown to him. He received the award for the best paper presented at the 1956 meeting of the Association of Asphalt Paving Technologists and in 1960 received the Richard L. Templin award of the American Society for Testing and Materials.

Professor Goetz is the author or co-author of more than 40 technical papers, and an associate editor, with K. B. Woods and D. S. Berry, of the *Highway Engineering Handbook*, published in 1960.

Throughout his career William H. Goetz has demonstrated ability, leadership, and adherence to the ideals of the profession. His record is one of distinguished public service in the search for knowledge and in the transmission of this knowledge to others.

GEORGE S. BARTLETT AWARD

THE GEORGE S. BARTLETT AWARD was established in 1931 by a group of friends of George S. Bartlett, with the purpose of perpetuating the spirit of friendship and helpfulness which he brought into his work in the highway field.

It is conferred annually upon an individual who has made an outstanding contribution to highway progress, the recipient being selected by a Board of Award composed of one representative of each of the following organizations:

AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS
AMERICAN ROAD BUILDERS' ASSOCIATION
HIGHWAY RESEARCH BOARD

The recipients of the award have been:

THOMAS H. MACDONALD	1931
ARTHUR N. JOHNSON	1932
JAMES H. MACDONALD	1933
FRANK F. ROGERS	1934
EDWARD N. HINES	1935
THOMAS R. AGG	1936
C. A. HOGENTOGLER	1937
FRED R. WHITE	1938
ROBERT MOSES	1939
FRANK T. SHEETS	1941
PAUL G. HOFFMAN	1942
H. S. MATTIMORE	1943
CHARLES H. PURCELL	1944
FREDERICK E. EVERETT	1945
CHARLES M. UPHAM	1946
H. S. FAIRBANK	1947
G. DONALD KENNEDY	1948
M. J. HOFFMAN	1949
R. H. BALDOCK	1950
C. S. MULLEN	1951
ROY W. CRUM (*posthumously*)	1951
SAMUEL C. HADDEN	1952
D. C. GREER	1953
JAMES A. ANDERSON	1954
WILLIAM RANDOLPH HEARST, JR.	1955
PYKE JOHNSON	1956
C. D. CURTISS	1957
REX M. WHITTON	1958
GEORGE H. FALLON	1959
W. A. BUGGE	1960
BERTRAM D. TALLAMY	1961

Minutes of Annual Business Meeting

Washington, D. C., January 12, 1962

• THE 1962 MEETING of the Highway Research Board was called to order by Chairman R. R. Bartelsmeyer at 2:00 P.M. on January 12, 1962, with the following in attendance:

Executive Committee. R. R. Bartelsmeyer, *Chairman;* C. D. Curtiss, *First Vice-Chairman;* Wilbur S. Smith, *Second Vice-Chairman;* E. W. Bauman; Donald S. Berry; Mason A. Butcher; J. Douglas Carroll, Jr.; Harmer E. Davis; Duke W. Dunbar; John T. Howard; J. B. McMorran; Burton W. Marsh; Oscar T. Marzke; Glenn C. Richards; C. H. Scholer; K. B. Woods; W. A. Bugge; A. E. Johnson; Pyke Johnson; Louis Jordan; Rex M. Whitton.

Staff. Fred Burggraf, *Director;* W. N. Carey, Jr., *Assistant Director;* M. Earl Campbell, *Engineer of Economics, Finance and Administration;* Ray E. Bollen, *Engineer of Materials and Construction;* Adrian G. Clary, *Engineer of Maintenance;* Franklin N. Wray, *Engineer of Design;* Kenneth G. McWane, *Engineer of Traffic and Operations;* A. W. Johnson, *Engineer of Soils and Foundations;* Emmett W. Harris, *Executive Assistant;* Herbert P. Orland, *Editor;* Ross D. Netherton, *Counsel for Legal Research;* Paul E. Irick, *Research Statistician;* W. B. McKendrick, Jr., *Project Director, AASHO Road Test.*

Department and Special Committee Chairmen. Guilford P. St. Clair, *Department of Economics, Finance and Administration;* T. E. Shelburne, *Department of Design;* John H. Swanberg, *Department of Materials and Construction;* H. E. Diers, *Department of Maintenance;* Fred W. Hurd, *Department of Traffic and Operations;* Miles S. Kersten, *Department of Soils, Geology and Foundations;* Burton W. Marsh, *Special Committee on Night Visibility;* Morgan J. Kilpatrick, *Special Committee on Highway Equipment;* Pyke Johnson, *Special Committee on Urban Transportation Research;* Fred J. Benson, *Ad Hoc Committee on Research Problems of Mutual Interest and Concern to Users and Producers of Asphaltic Materials;* Ross D. Netherton (representing J. H. Beuscher), *Special Committee on Highway Laws;* John E. Moore, *Special Committee on Public Dissemination of Research Findings;* David M. Schoppert, *Ad Hoc Committee on Driving Simulation.*

Member Organizations. American Association of State Highway Officials, A. E. Johnson; American Association of Landscape Architects, Robert T. Walker; American Automobile Association, B. W. Marsh; American Concrete Institute, E. A. Finney; American Concrete Pipe Association, J. A. Ruhling (representing H. F. Peckworth); American Iron and Steel Institute, C. A. Willson; American Public Works Association, G. E. Bodien (representing D. M. Smallwood); American Road Builders' Association, J. E. Wiley; American Society for Testing Materials, K. B. Woods and W. S. Housel; American Society of Civil Engineers, R. A. Moyer; American Trucking Associations, J. V. Lawrence; The Asphalt Institute, A. S. Wellborn and J. M. Griffith; Association of American Railroads, H. H. Hale and E. R. Feldman; Automotive Safety Foundation, J. O. Mattson and C. E. Fritts; Bureau of Highway Traffic, Yale University, F. W. Hurd; Bureau of Indian Affairs, U. S. Department of the Interior, R. J. Trier; Bureau of Public Roads, U. S. Department of Commerce, R. M. Whitton; Calcium Chloride Institute, W. E. Dickinson; Federal Aviation Agency, T. Wilkinson (representing H. J. Lichtefeld); Institute of Traffic Engineers, E. G. Wetzel; Institute of Transportation and Traffic Engineering, University of California, H. E. Davis; National Bureau of Standards, U. S. Department of Commerce, B. E. Foster; National Crushed Stone Association, J. E. Gray; National Park Service, U. S. Department of the Interior, H. C. Vollmer; National Ready Mixed Concrete Association, D. L. Bloem; National Sand and Gravel Association, D. L. Bloem; National Slag Association, E. W. Bauman; Office, Chief of Engineers, Department of the Army, T. B. Pringle; The Port of New York Authority, E. G. Wetzel; Portland Cement Association, J. D. Piper; Rail Steel Bar Association, W. H. Jacobs; Society of Automotive Engineers, K. A. Stonex; Texas Transportation Institute, Texas A&M College, F. J. Benson and C. J. Keese.

1. Minutes, Meeting of January 13, 1961

The minutes of the Annual Business Meeting held January 13, 1961, were approved as printed in Volume 40 of the

PROCEEDINGS of the Highway Research Board.

2. Report on Letter Ballot on Nominations for the Executive Committee

The Nominating Committee, composed of representatives from Member Organizations and chaired by Pyke Johnson, a member of the Executive Committee, recommended the following to be members of the Executive Committee: William A. Bugge, Mason A. Butcher, Harmer E. Davis, K. B. Woods, Donald S. Berry, J. Douglas Carroll, Jr., John T. Howard, and Oscar T. Marzke. The list of candidates was sent to the Member Organizations, and they voted uniformly for the eight candidates as follows: Affirmative, 34; Ballots not returned, 12. These men were appointed by the Chairman of the Division of Engineering and Industrial Research with the approval of the President of the National Academy of Sciences.

3. Report of Executive Committee on Awards

The ROY W. CRUM AWARD FOR DISTINGUISHED SERVICE was given to Rex M. Whitton, William H. Goetz, and Elmer M. Ward (posthumously).

The HIGHWAY RESEARCH BOARD AWARD was given to George Haikalis and Hyman Joseph for their paper "Economic Evaluation of Traffic Networks," and to A. S. Lang and David H. Robbins for their paper "A New Technique for Prediction of Vehicle Operating Costs in Connection with Highway Design." Both papers were presented at the 40th Annual Meeting (see "Awards").

4. Reports of Department and Special Committee Chairmen and on Special Projects

(a) DEPARTMENT OF ECONOMICS, FINANCE AND ADMINISTRATION

G. P. St. Clair, Chairman

On December 1, 1961, there were 179 men in this Department and its 16 committees. Overlapping memberships bring the total of memberships to 226. These overlapping memberships are somewhat of a problem. They reflect in part the diverse interests and skills of the membership and in part an over-optimism on the part of individuals as to the amount of committee work they can do.

At the 41st Annual Meeting the Department sponsored five sessions, at which 21 papers were presented. In addition, a number of less formal papers were presented at committee meetings.

An insight into the work of the Department and its prospects for future accomplishments can be gained by highlighting the activities of its three component divisions, as follows:

Division I
Administrative and Management Studies

W. L. Haas, Chairman

Division I is sponsoring a conference on planning in highway administration to be held March 26 and 27, 1962. This is the fourth in a series of so-called workshop conferences to be held by this Department. However, the word "workshop," which seems to have been overworked in recent years, has been dropped. If this conference is as successful as planned, it is hoped that through the cooperation of the Highway Research Board and the American Association of State Highway Officials its results may form the basis of regional conferences on the same subject during the year following.

Through its *Committee on Highway Organization and Administration*, the Division has proposed that a research project be undertaken, and offered for financing under the cooperative research fund plan recently announced, for a review of principles and practices in modern organization and management. This research would be productive of a report or treatise which would reflect not only the best of current literature and developments in this field, but also the results of the highway management seminars that have been held in recent years.

The *Committee on Motor Vehicle Registration and Titling Practices* has expanded the scope of its membership. It has also placed a grant obtained from the Automotive Safety Foundation with the Highway Traffic Safety Center, University of Illinois, for a pilot study of motor vehicle registration and titling practices. The committee has also been active during 1961 in liaison work involving activities with the American Association of Motor Vehicle Administrators.

The *Committee on Land Acquisition and Control of Highway Access and Adjacent Areas* is completing a study of liaison practices involving public utilities and highway departments, in connection with highway projects involving public utility relocations. A good questionnaire response has already been received. The committee is also supporting severance damage studies now being undertaken in 42 States. In this connection, during 1961 the committee has sponsored a special study of the admissibility of severance damage studies in right-of-way litigation, which will be reported on at the 1962 Annual Meeting. Cooperation also has been given to the AASHO

Committee on Right-of-Way in its efforts to update land acquisition practices.

The *Committee on Education and Training of Highway Engineering Personnel* particularly emphasized its desire to complete in the coming year the study of career ladders for both professional engineering and engineering technician work in the highway field. It also received approval for a survey of State highway department needs in the management training field. The committee is working with the NHUC-AASHO Management Advisory Committee on this project.

Division II
Finance, Taxation, and Cost Studies

J. W. Martin, Chairman

The organization of Division II, which was delayed somewhat by questions about the creation of some of its committees, has been completed. Two of the most promising fields of activity are that of the *Committee on Highway Programing*, which is undertaking a thorough going study of principles, techniques, and practices in that field; and that of the *Committee on Highway Needs*, which is initiating a research aimed at disclosing both the merits and the defects of past and current studies of highway needs. This study should develop techniques for the improvement of highway need studies in the future.

The *Committee on Highway Taxation and Finance* has yielded its responsibility for research in certain specific fields to new committees organized more nearly on a task force basis.

The *Committee on Equitable Allocation of Highway Costs* has taken over that important subject and the subject of credit financing has been assigned to the Committee on Financing Highway Facilities Through Borrowing. The domain of continuing interest of the parent committee was expressed in a revised statement of objective and scope, including the following:

> The province of the Committee includes all phases of financial planning to meet mounting highway needs except as they are otherwise assigned to special project committees set up for the purpose. Proper subjects for study include, among others, the behavior and outlook for various classes of highway expenditure, the distribution of highway funds, the phenomena and problems of property taxation, road-user taxation, toll financing, and other means of pricing highway services, and also the highway tax problems imposed by the interstate movement of vehicles.

Division III
Economic Studies

R. G. Hennes, Chairman

Also delayed somewhat by difficulties in the formation of new committees, Division III is now operating with five committees. The *Committee on Highway Engineering Economy* is undertaking a study of the extent to which the economic analysis of engineering projects is now being utilized by highway departments. A subcommittee composed of State and Provincial highway engineers has been formed to solicit the information about present practices in this field. After a preliminary questionnaire has been compiled and tested, the country will be canvassed. The dual purposes of this project are (a) to improve the general knowledge of present practices and techniques and (b) to demonstrate the practical value of economic analysis.

The *Committee on the Economics of Motor Vehicle Size and Weight* approved the report on the operating costs of line-haul freight carriers prepared by Hoy Stevens, and this report will soon be published by the Board. The second part of this committee's mission, that of developing the relation of highway costs to motor vehicle size and weight, will be carried on in cooperation with the Committee on Highway Costs (Division II) and will utilize the AASHO Road Test results.

The *Committee on Economic Forecasting* has collaborated with the Virginia Council of Highway Investigation and Research in pursuing its program objectives under three headings:

(a) Assemble, peruse, and record the bibliography on economic forecasting, especially in relation to highways.
(b) Evaluate references in the bibliography.
(c) If possible, recommend one or more methods of forecasting.

The *Committee on Indirect Effects of Highway Improvement* has successfully carried on and extended the work on economic impact of highway improvements formerly sponsored by more than one committee of the Board. As the work progresses and experience in the study of indirect benefits is gained, more sophisticated methods of data gathering and analysis are being utilized. It has become apparent that it will be necessary to undertake some work in the field of socio-econometrics.

The most recently organized is the *Committee on Intercity Freight Transporta-*

tion, with John C. Kohl as Chairman. A well-balanced membership has been developed for this committee and it is felt that it will provide a useful extension of the work on motor freight transportation begun by the *Committee on Economics of Motor Vehicle Size and Weight.*

At its annual business meeting on January 11, 1962, the Department instructed its chairmen to appoint an *ad hoc* committee with the duty of drawing up a research framework study for the Department. This committee will review the current work of the Department and prepare problem statements for high priority research in the field of the Department, that has not yet been provided for. It is hoped that this effort will be productive of a number of worthwhile proposals for consideration as projects in the cooperative research program of the Board and the American Association of State Highway Officials.

(b) Department of Design

Tilton E. Shelburne, Chairman

The Department of Design is composed of 24 individuals, including the chairmen of the eleven committees. It serves in an advisory capacity, assisting the Board in the formation and coordination of committee activities, the encouragement of research, and the development of information (both theoretical and practical) applicable to highway design. Work of the Department is carried on by eleven active committees varying in size from 9 to 33 members and totaling approximately 250. Through committee activity the Department suggests, plans, and examines research needs in the field of highway design and reviews and evaluates papers and reports.

Publications sponsored by the Department and released by the Board in 1961 included 56 design papers representing the work of 76 authors. These papers comprised eleven bulletins and 185 pages of Vol. 40, Proceedings, making a total of 720 pages. Nine of the 59 sessions at the 1962 Annual Meeting were sponsored by the Department. Committee activities are presented in the following reports.

Committee on Roadside Development— The committee continues to research the many aspects of roadside development for the purpose of benefiting the development of the complete highway.

For 1961 the theme of the committee's program emphasized highway construction, design and maintenance as they relate to roadside development. At the 41st Annual Meeting the committee conducted three business meetings and three program sessions. Fifteen papers were prepared and presented individually or jointly by 19 persons on the general topics of noise abatement, turf establishment, use of plant growth control chemicals, erosion control practices, planting for control of drifting snow, National Park and Parkway roadsides, roadside rests requirements on the Interstate system, responsibilities of the landscape architect in highway design prior to and during construction, complete highway design standards including roadside development, esthetic criteria in freeway design, vehicle collision with roadside objects, roadside maintenance practices on the Interstate system, and equipment for roadside use. During 1961 the committee published its Annual Report and four distributions of material from its clearinghouse.

The program for 1962 will be primarily a continuation of the theme for the past year. The Program Subcommittee will consider, among others, such additional topics as a symposium on trees and utilities, and roadside design standards. In addition, 15 circulars will be prepared and distributed on subjects such as fertilizers, noise abatement, snow barriers, firebreaks, and salvage of materials.

Committee on Road Surface Properties Related to Vehicle Performance—As a result of recent interest and activity in the measurements of roughness and skidding on highway pavements, ten research papers were presented at the 1962 Annual Meeting of the Board.

On June 26, 1961, at Atlantic City, N. J., a meeting was held in which this committee and ASTM Committee E-17 outlined plans to conduct a comprehensive program of correlation skid tests aimed at standardization of test equipment, test tires, methods of testing, and evaluation of test results. T. E. Shelburne was made chairman of a group that will carry out the required field tests. Several committee members are serving on the Advisory Committee for this ASTM project.

The business meeting this year was attended by all members of the committee except one. Fifteen visitors, attending at the invitation of Chairman Moyer, took part in discussion relating to proposed research programs on road roughness and skid resistance.

Five of the papers presented at the 1962 Annual Meeting described some of the latest developments in measuring road roughness and in the evaluation of roughness which relates to the serviceability index and expected service life of any pavement. Also discussed were the dynamic forces of loads moving over pavements affected by surface roughness. These same studies are being followed closely by the reactivated Committee on Pavement Condition Evaluation.

Five of the papers provided excellent coverage of methods of testing and equipment to measure skid resistance in terms of locked-wheel friction values and coefficients of friction at incipient skidding. The Board was honored by having Dr. Gösta Kullberg of the National Swedish Road Research Institute present his paper covering extensive research on factors influencing skidding, particularly on wet pavements and pavements covered with snow or ice. Dr. Kullberg is one of the world's leading authorities, having worked more than 20 years in research on skid resistance of highway and airport pavements.

Chairman Moyer presented a paper on skid resistance measurements at the University of California. It described a newly developed torque device utilizing truck-trailer equipment for testing skidding by different tires. The need for correlating results obtained by different skid-test methods was emphasized. Professor Moyer cited research in Great Britain proving that where standards of skid resistance have been adopted and corrective deslicking treatments have been provided the savings in accident costs offset many fold the cost of making the tests and supplying the necessary non-skid treatments.

In the plan for the cooperative program for the correlation of skid test equipment and methods under the purview of the ASTM and HRB committees and the U. S. Bureau of Public Roads, several States and other agencies who are interested will be invited to enter their skid test equipment for purposes of comparison.

Committee on Pavement Condition Surveys—This committee, reactivated during 1961, held two meetings, one at Purdue University on July 6 and 7, 1961, and the other at Washington, D. C., on January 8, 1962.

The first meeting, entirely organizational in nature, was attended by 15 members and 5 guests; the second, by 14 members, one proxy, and 14 guests.

One of the first items considered was the scope and purpose of the committee's activities. A draft statement prepared at the Purdue meeting was approved as amended at the January meeting. It was generally agreed that the purpose of the committee should include pavement evaluation and a study of the uses to which survey data can be put. Therefore, action was taken at the January meeting to change the name to *Committee on Pavement Condition Evaluation.*

Projects undertaken by the committee during the past year include (a) determination of the committee purpose and scope; (b) a review of literature relating to pavement condition; (c) participation in, and preparation of, a report summarizing the results of a correlation of data obtained by various instruments for measuring pavement roughness.

For its second (Washington) meeting the committee heard reports on (a) a survey of the literature search; (b) a New York study of surveys using photographic techniques; (c) a BPR study using the CHLOE profilometer; and (d) cooperative condition surveys on US 20 in Indiana.

Committee on Flexible Pavement Design—During the year three members retired or resigned from the committee and two new members were added. Thirty of the 33 members and about 50 visitors attended the annual business meeting.

The six papers sponsored by the committee at the 40th Annual Meeting were published in HRB Bulletin 289. The Subcommittee on Rating Flexible Pavements presented the final report on its work to the main committee. A copy of the report will be transmitted to the Committee on Pavement Condition Surveys.

The Subcommittee on Load-Deflection Testing prepared a purpose and scope statement and formulated preliminary plans for future activity. A meeting was held during the 41st Annual Meeting.

The following papers were sponsored by the committee at the 41st Annual Meeting:

1. "Flexible Pavement Performance Studies in Arkansas," by M. C. Ford, Jr., and J. R. Bissett.
2. "Flexible Pavement Design: A Complex Combination of Theory, Testing and Evaluation of Materials," by Chester McDowell.
3. "The Flexure of a Road Surfacing, Its Relation to Fatigue Cracking, and Factors Determining Its Severity," by G. L. Dehlen (South Africa).
4. "Significance of Layer Deflection Measurements," by R. D. Walker, Eldon J. Yoder, Robert R. Lowry, and W. T. Spencer.
5. "Flexible Pavement Research in South Dakota," by Robert A. Crawford.

Liaison was continued with the Committee on Composite Pavement Design and the Committee on Pavement Condition Surveys, and with the AASHO Committee on Design.

The chairman and secretary attended a special meeting with the Department chairman and the chairmen and secretaries of the Committee on Rigid Pavement Design and the Committee on Composite Pavement Design to develop definitions for the three pavement types and to discuss the purpose and scope of the three committees.

Committee on Rigid Pavement Design—During 1961 the committee arranged for four papers for presentation at the 1962 Annual Meeting, as follows:

1. "Laboratory Studies of Progessive Bond Failure in Continuously-Reinforced Concrete Slabs," by Joseph H. Moore and Albert D. M. Lewis.
2. "Experience in Texas with Terminal Anchorage of Concrete Pavement," by M. D. Shelby and W. B. Ledbetter.
3. "Concrete Pavement Designs in Five Countries of Western Europe," by Gordon K. Ray.
4. "Investigations of Prestressed Concrete for Pavements," by Bengt F. Friberg.

The Subcommittee on Prestressed Concrete Pavements has been actively engaged in preparation of a comprehensive report on the basic concepts, design variables and construction procedures in connection with prestressed pavements. This work is now in its final stages and it is anticipated that the report, which will also include recommendations relating to the planning, design, and pertinent observations of experimental prestressed highway pavements, will be available for publication this year.

On April 19, 1961, the chairman and secretary met in LaSalle, Ill., with officers of the Committees on Flexible Pavement Design and Composite Pavement Design, to discuss, define and agree on the specific types of pavement which fall within the scope of each of these committees.

At the regular annual business meeting held on January 8, 1962, at least five papers and a symposium on the corrosion of load-transfer devices were listed as possibilities for sponsorship at the 1963 Annual Meeting.

Comments relating to this business meeting are as follows:

1. The committee urges that immediate action be undertaken to implement the research outlined in a statement prepared by the committee, in January 1959, entitled "Urgently Needed Research in Connection with Rigid Pavements."
2. A subcommittee is to be established to review the AASHO Road Test report on rigid pavement research, when available.
3. A subcommittee is to be established to review the AASHO guide for the design of rigid pavements, when available.
4. A subcommittee is to be established to develop a procedure for conducting the research recommended in the previously mentioned statement on urgently needed research.
5. The committee agreed to hold an interim meeting in Michigan in September.

Committee on Geometric Highway Design—A year ago the committee reaffirmed that its major roles are to identify needed research in the field and to stimulate its bit-by-bit accomplishments through a usable report stage, recognizing that much of the actual research will be done by others. To this end the committee has been expanded from 13 members to its present strength of 23. The committee now has a good balance between the automotive industry (1 member), educational institutions (2 members), consulting and advisory firms (4 members), and governmental bodies (3 Federal, 13 State). Sixteen are directly engaged in geometric design and the others in work concerning design standards and teaching.

The 1962 HRB program had more than 40 papers in the field of geometric design. More than one-half of these are on subjects on which the committee has made some identification as to the current need for research.

The committee meeting on January 10, 1962, was attended by 13 members and about 25 visitors. On further discussion it was concluded that while revision and updating of Special Report 12 (1953) would be a desirable goal, a thorough job was not practicable of accomplishment by the committee. Consideration will be given to a review of Special Report 12 for the purpose of determining the extent to which research has been done on the list of projects and needs for continuing emphasis.

Most of this year's meeting was devoted to statements by the committee members as to specific current design problems wherein research effort might afford guidance toward solution. These included (a) needed sight distance at diamond interchanges; (b) climbing lanes on 2-lane and on multilane highways; (c) applicability of and treatment for narrow medians; (d) design for controlling traffic at the ends of reverse roadways on freeways; (e) radius, grade, and superelevation for interchange ramps; (f) standardization for design of speed-change lanes; (g) merging areas for major traffic streams; (h) flush versus raised shoulders on freeways; (i) limits for control of access along crossroads at interchanges; and (j) relation between vehicle speeds (speed profile) and design speed.

It was agreed that during the coming year a small working group within the committee should review these problem areas and establish those on which reasonably attainable research studies might be made. The full committee will then prepare concise statements as to the needed research in the selected areas and the likely manner to make studies. Efforts will be continued to locate sponsors for the needed research, and to assist in outlining the areas and kinds of studies needed.

Committee on Surface Drainage of Highways—The committee held a meeting at Colorado State University on November 16 and 17, 1961, attended by 14 of the 18 members. The purpose of the meeting was to draft a report on the needs for research in the field of highway drainage. The first day was also attended by 13 hydraulic engineers from the Bureau of Public Roads, who contributed their suggestions on research needs.

On the second day the many problems were classified into a list of 17 projects. The committee then took a ballot to obtain a consensus as to the relative need for research without respect to how the work would be done or by whom. These projects with their rankings are as follows:

Project	*Rank*	*Weighted Rank*
Hydrology-estimation of peak rates of runoff and related problems in both rural and urban areas	1	1
Control of erosion at culvert outlets	2	3.2
Design of stable channels	3	3.5
Conveying water down steep slopes	4	6.3
Scour around bridge piers and abutments	5	7.1
Open channel confluences at high velocity	6	8.4
Transitions into and out of contracted bridge waterways (includes spur dikes)	7	8.5
Coordination of hydraulic design problems between agencies	8	8.7
High-velocity flow in stormdrain systems	9	9.3
Stormdrain inlets	10	10.1
Erosion protection of slopes	11	11.2
Highway crossings of estuaries	12	11.5
Sediment transport through multiple-barrel culverts	13	11.5
Drainage law	14	11.6
Bridge deck drainage	15	12.1
Watertightness of conduits	16	13.8
Conduit outlets on beaches	17	14.2

Attention is called to the fact that research on peak rates of runoff was rated as the No. 1 problem by everyone on the committee. The need is especially acute with respect to watersheds under 10 sq mi in both urban and rural areas. The attack on this problem needs to be made by both a long-range and a short-range program. The long-range program would include establishment of gaging stations to obtain more data on small watersheds. The short-range approach would involve an intensive analysis of existing data and an attempt to arrive at synthetic methods of estimating both the peak rates and the volume of runoff.

The committee then broke up into four task forces to draft detailed statements on the projects having the top four rankings, including what the problem is, why research is needed, and, in most cases, how the research might be undertaken. Subsequent to the meeting, statements were submitted on the remaining projects.

Eleven members or their proxies were present at the meeting in Washington, D. C., on January 8, 1962. The report on needed research was submitted as amended to the Department.

The committee also drafted a report on the status of current research in highway hydraulics.

The committee intends to hold the next meeting about the middle of September at St. Paul, Minn.

Committee on Bridges—At the business meeting on January 10, 1962, 12 of the 17 committee members were present, plus 23 visitors.

Mr. Hogan reported on the experimental project in New York where reinforcement with 60,000-psi yield point is being used at a working stress of 30,000 psi. The design, including instrumentation, is complete. Contracting is expected in the spring of this year. The principal objective is to observe the adequacy of the design in the control of cracking at the high unit stress. Mr. Fountain reported on a similar project in Texas, where conventional proportioning of beam sections is being used.

The possibility of follow-up work extending the Road Test work was discussed. It was felt that, given the physical characteristics of the material and the volume and weight of traffic, a good estimate of the fatigue life of steel structures could be made. The Traffic Committee will be asked to give an estimate of future traffic broken down into weight groups for which structures should be designed. It was suggested that fairly good data could be had by installation of gages on members (structural parts) of existing bridges, these gages to be connected to an accumulator which would record the stresses in groups of various stress level. This might give an idea of the number of loadings at different stress levels that should be expected.

Mr. Dill reported for the Subcommittee on Extra-High-Strength Structural Steel. The fatigue testing that was hoped to be completed last year is not yet reported. It is expected that by next year's meeting a usable criterion for this material can be presented. It was reported that ASTM is working on a specification which will replace the five specifications now used to cover the products of the mills producing this material.

A report was received on pile hammer tests in Michigan, with a complete report expected next year. The vibrating hammer being developed was discussed. This equipment has not reached a state of development where it can be considered for general use.

Mr. Fountain reported briefly on the investigation of concrete bridge deterioration new being made by the Portland Cement Association in cooperation with the BPR and several of the States. This will include a visual inspection of 150 bridges in each of 10 States, followed by a detailed investigation of 10 to 18 structures in each State, including core samples from each bridge.

Committee on Guardrails and Guide Posts—The committee activities during 1961 included the continuation of efforts to develop proposed testing procedures for guardrail systems. Meetings of the subcommittee were held during the year and the "proposal" was submitted to the full committee meeting held on January 8, 1962.

A subcommittee has also been engaged in the development of warrants for installation of guardrails and guide posts. As this matter requires a great deal of research, the work of the subcommittee will continue. A report will be available no later than the next Annual Meeting.

As a result of recent dynamic testing of guardrails, it has been indicated that certain modifications to presently used guardrail systems improves performance. A subcommittee is now at work assembling this information. The required sketches will be developed, along with a report outlining the various items. This may take some time to complete, but will be ready no later than the end of this year.

During the past year, several committee members attended tests being conducted at Niagara Falls by Cornell University for the State of New York. Members will continue to follow that testing program.

At the committee meeting of January 8, 1962, it was made known by at least three producers of guardrail elements that they would probably be undertaking testing programs during the year. The committee expects to cooperate on those tests to fill in gaps in the general knowledge of guardrail systems.

Committee on Photogrammetry and Aerial Surveys—Chairman D. J. Olinger resigned late in December 1961. He will be missed greatly because of his excellent leadership and guidance of the committee.

During 1961 the committee had two meetings. The first, held in two sessions on January 9 and 12, 1961, constituted the ninth meeting of the committee since its organization in August 1956. There were 21 members present or represented by proxy at these sessions. The committee now consists of 27 members. The second meeting was held in Edgewater Park, Miss., June 21-23, 1961, at which 19 committee members were present or were represented by proxy. At both meetings there was lengthy discussion regarding current and specific applications of aerial surveys to the solution of highway engineering problems and to the various phases in which research is desirable to improve old methods and devise new techniques or methods.

The committee sponsored two sessions at the 40th Annual Meeting. At the first session, four papers were presented outlining specific applications of aerial photography and photogrammetry in the highway engineering field by the States of Georgia, Arizona, California, and Ohio. At the other, papers were presented on "Control Surveys for Interstate Highways," and "Digitizing Stereo-Plotter Output," as practiced in Region 9 of the Bureau of Public Roads and by the Ohio Department of Highways, respectively.

At the June meeting, a program was undertaken for presentation of several papers at the 41st Annual Meeting. It was initially intended that this program would be more extensive than was actually realized. Nevertheless, eight papers were prepared—three on separate and vital topics of interest and value to highway engineers, pertained to horizontal control staking by triangulation for a major highway construction project in Austin, Tex.; the findings of value in utilization of the Zeiss Stereotope for measurement of cross-sections by photogrammetric methods for highway design and measurement of construction quantities; and a new control survey method utilizing a remote baseline for measurement of supplemental horizontal and vertical control, essential for leveling and scaling stereoscopic models in the utilization of photogrammetery wherever precision work is required for highway engineering purposes. The remaining five papers constituted a symposium on the utilization of aerial photography for right-of-way determination, evaluation, boundary descriptions, and procurement through negotiation and/or condemnation.

During its eleventh meeting at the 1962 Annual Meeting the committee planned to hold a midyear meeting in June or July 1962. The California Division of Highways invited the committee to hold this meeting in San Francisco. Sixteen members of the committee were present or were represented by proxy at the meetings held January 8 and 10, 1962.

At the committee meeting in June 1961 it was decided that a questionnaire should be prepared to obtain knowledge of current practice in highway departments in the utilization of aerial surveys. The purposes of this questionnaire are threefold:

1. To apprise the highway departments of the over-all and detailed way in which aerial surveys can be utilized.
2. To achieve a comprehensive summarization of current practice for evaluation and dissemination to highway departments.

3. To procure essential information to enable the committee to better formulate its program of assistance and advice on research which would be desirable in the photogrammetry and aerial surveys field to improve methods of application in the solution of highway engineering problems.

The questionnaire has been submitted to members of the committee for review and comment and will be ready for submission to the HRB by March 1, 1962. The committee's intent is to complete an analysis and report from the questionnaire replies and prepare a memorandum thereon for distribution before the 42nd Annual Meeting.

Committee on Composite Pavement Design—The major emphasis of the committee's activities during 1961 was the development of a prospectus whose purpose is to provide direction and impetus for further experimentation with composite pavements. Present plans call for submitting the prospectus to the Board for consideration as a circular. The present timetable calls for completion of the committee's part by mid-April 1962.

The committee held its third meeting at LaSalle, Ill., on Sept. 21-22, 1961, and its fourth meeting on January 7, 1962, at Washington, D. C.

Work is continuing on definitions and terms. The preliminary task assignments have been completed so that only two subcommittees are now working—one on definitions and one on theory of design.

(c) Department of Materials and Construction

John H. Swanberg, Chairman

The Department has had a good year. At the 41st Annual Meeting it sponsored 50 formal papers and 20 informal discussions. This involved presentations at 15 sessions, of which 5 were symposia and 3 were conference or off-the-record sessions. These conference sessions have been valuable and worthwhile contributions to the program. Although not involving formal papers, a number shortly after the meeting become formalized for publication and some develop into papers at later HRB meetings.

The work of the Department is accomplished through the Bituminous, Concrete, Construction and General Materials divisions, composed of a total of 23 committees. All committees and divisions held working meetings during this annual meeting. During the past year the Bituminous Division has been reorganized and two new committees have been added. The Department's Administrative Committee, consisting of the Department Chairman and the Division Chairmen and Vice-Chairmen, met during the 41st Annual Meeting with all members present. Such matters as accomplishments during the past year, program of activities for the coming year, personnel changes, changes in scope of committees, new committees, and policies, were discussed. On January 12, 1962, the Department met with 31 of its 42 members present. At this meeting the actions of the Administrative Committee were discussed and the proposed program for the coming year was presented.

It is proposed that two committees be added. One will be an *ad hoc* committee of the Bituminous Division to report on the status of knowledge and current research programs relative to rapid methods for measuring thickness, density, and bituminous content of bituminous mixtures. This results from recommendations from the *ad hoc* Committee on Research Problems of Mutual Interest and Concern to Users and Producers of Asphaltic Materials. Approval will also be requested for a committee in the General Materials Division on adhesives and bonding agents other than portland cements, asphalts and tars.

There has been an increasing number of subcommittees formed in committees of all divisions, with the objective in mind of substantially and effectively increasing the accomplishments of such committees.

The outlook for papers, reports and other activities during the coming year is good. The program of upgrading the quality of papers is continuing.

(d) Department of Maintenance

H. E. Diers, Chairman

The new committees, organized in 1960, expanded their membership in 1961 and made further progress in defining their scope of activity and initiating needed work. Nine of the ten committees held their annual business meetings on the occasion of the 41st Annual Meeting. Six papers presented at the Annual Meeting were sponsored by the Department.

The *Committee on Maintenance of Concrete Pavements as Related to the Pumping Action of Slabs* reviewed its past activities and discussed present problems. C. E. Vogelgesang resigned as chairman and W. T. Spencer was appointed as his successor. During the coming year the committee will endeavor to complete a resume of its activities and to list needed future studies to be developed along with recommendations for assignment of those needs to other committees or for the fruitful continuation of this committee.

The *Committee on Maintenance Costs* surveyed various State highway departments to obtain data to assist in evaluating unit work costs on seal coat application, traffic striping, guardrail painting, and roadside mowing. This information will be published during 1962. Further work of a similar nature will be conducted during 1962, with greater emphasis on obtaining detailed information on job condition and character of work performed.

The *Committee on Maintenance Personnel* collected and published its annual joint report with the AASHO committee. The form has been slightly modified for the 1962 report.

The *Committee on Salvaging Old Pavements by Resurfacing* sponsored four of the papers presented at the 41st Annual Meeting. HRB Bibliography 21, published in 1957, will be brought up-to-date and a report on undersealing is expected.

The *Committee on Snow and Ice Control* meeting was well attended and showed an increased membership over the previous year. Discussion centered on the revision of the 1954 HRB Recommended Practice Manual No. 9-3R. A target date of December 1, 1962, was set for distribution of the revised manual.

The *Committee on Maintenance of Bituminous Pavements* has now been formed and is actively engaged in work leading toward a symposium at the 42nd Annual Meeting on the subject of thin bituminous concrete retreatments.

The work of the *Committee on Maintenance of Controlled Access Highways* has resulted in publication in early 1962 of a tabulation of cost experience on limited access highways of typical States and toll authorities. It is expected that similar information will again be assembled to give an up-to-date, continuing, experience record on the subject.

A special subcommittee was appointed to delineate problem areas, specific committee aims, and objectives for the near future.

Discussions were held on snow and ice control on a closed system, need for separate maintenance organizations for Interstate maintenance, median treatments, guard fence, mowing, motorist's services, cost tabulations on a lane-mile basis, scheduling of maintenance operations, frontage roads, and standardization of traffic control devices.

The *Committee on Maintenance of Structures* heard reports on the PCA surveys of concrete bridge decks and the Georgia bridge maintenance organization and held a symposium on the use of latex-modified portland cement mortar for repairing deteriorated concrete bridge decks. Representatives of the Dow Chemical Co. and the Texas, Kentucky and Maine Highway Departments presented papers at the symposium. There was insufficient time to hear a report on hurricane damage on the Gulf coast.

The *Committee on Maintenance of Portland Cement Concrete Pavements* held its first meeting for the purpose of organization and planned for work to be undertaken during the coming year.

The *Committee on Maintenance Equipment* was formed during the year. Immediate objectives will be to develop pertinent information on the use of various types of equipment designed for the use, such as rotary, reel and hammerknife mowers related to terrain, acreage, and rate of cutting.

(e) Department of Traffic and Operations

Fred W. Hurd, Chairman

The purpose of the Department is to suggest, plan, encourage, correlate, and evaluate research within the broad field of the use of streets and highways (including their terminals) and to assist in outlining sound procedure for conducting such research. The scope of the Department generally covers those aspects of research bearing on the ability of the highway to properly serve its purpose of transporting persons and goods expeditiously and safely. The objectives of the Department are carried out largely through the work of its 13 committees, which collect, study, and analyze data, and prepare bibliographies within their fields of interest, and encourage preparation of significant research papers for presentation at the Annual Meetings.

At the 41st Annual Meeting the Department sponsored 10 sessions, in which 52 technical papers were presented. During this meeting the committees of the Department held business meetings at which 65 preliminary papers were evaluated and discussed. A meeting of the Department attended by 27 committee chairmen and members was held in August to discuss activities of the committees and to consider the quality of more than 100 papers offered for the Annual Meeting.

The Department utilized three *ad hoc* committees during the year. The *ad hoc Committee on Purpose and Scope* was reappointed to continue its work toward reducing duplication of effort and up-dating areas of research in view of expanding knowledge and new technology. A new *ad hoc Committee on How to Obtain Pertinent and Adequate Discussion of Papers* proposed procedures for inviting written discussions for a selected group of Annual Meeting papers. The discussions are to be prepared and reviewed in advance of the paper presentation. A third *ad hoc* committee is concerned with the problem of *Staggering Terms of Office of Committee Chairmen,* made desirable by the six-year limitation on the term of committee chairmen imposed by the Executive Committee in 1961. This committee has recommended annual terms for committee chairmen during 1962 and the establishment of a review committee within the Department to suggest appointments and reappointments for the following year.

Highlights of individual committee activities during the year are as follows:

The *Committee on Parking* focused attention on the desirability of establishing practical guides for ratio of parking need to different types of traffic generators. It was decided to begin this general investigation by concentrating on the parking requirements at hospitals.

The *Committee on Origin and Destination* held an all-day business meeting attended by about 130 persons. At this meeeting 14 preliminary papers related to projects in progress of interest to the committee were discussed and a symposium was conducted on the general subject of a nationwide intercity transportation study.

The *Committee on Shoulders and Medians* completed the "Rest Area Study Procedure Guide" and distributed it to committee members for review prior to its publication. The work of the Task Force on Shoulders has resulted in a proposal to the Bureau of Public Roads to undertake the research required.

The *Committee on Highway Capacity* continued work on the revised edition of the Highway Capacity Manual. Special meetings of the entire committee or subcommittees were held on October 13-14, 1961, in Los Angeles; on October 16-17, 1961, in New York City; and on January 5-7, 1962, in Washington, D. C. The committee made concentrated effort during the year to advance completion of the revised manual to the earliest possible date.

The *Committee on Traffic Control Devices* coordinated its activities with the research needs of the National Joint Committee on Uniform Traffic Control Devices. The Subcommittees on (a) Criteria for New Traffic Signal Warrants, (b) Supports for Traffic Control Devices, and (c) Pavement Edge Markings, were active in either initiating projects or conducting continued research. The Subcommittee on Traffic Signal Lenses was disbanded, having completed its objectives.

The *Committee on Highway Safety Research* devoted a portion of its business meeting to a discussion on traffic safety research needs and priorities, which brought together the viewpoints of individual researchers and the various organizations interested in supporting research.

The *Committee on Quality of Traffic Service* continued preparation of both a general and an annotated bibliography. Work was started on an outline for a research project to relate subjective and objective definitions of "quality" of service.

The *Committee on Vehicle Characteristics* encouraged and furthered research that made available techniques from which performance and costs of commercial vehicle operations may be predicted.

The *Committee on Theory of Traffic Flow* completed a bibliography, which has been published as a Correlation Service Circular. A special committee formed from this committee with representatives from the National Research Council Division of Mathematics continued work on a publication of selected information on traffic flow theory.

The *Committee on Freeway Operations* completed a bibliography, which was published by the Board. Reports on ten current technical projects related to freeway operations were reviewed and a two-day meeting for exchange of operational study results was arranged.

The *Committee on Road User Characteristics* encouraged research on human factor application to traffic problems by reviewing technical papers and providing advice to researchers.

The *Committee on Channelization* continued with the revision of HRB Special Report 5, including collection, review and discussion of channelization "examples." This project was made possible by a grant from the Automotive Safety Foundation to Texas A&M College and the report will be published by the Board.

The *Committee on Speed Characteristics* offered advice and counsel to a number of researchers on the subject of speed, including the development and coordination of several projects. Work on the summarization of existing knowledge relative to speed was initiated.

(f) Department of Soils, Geology and Foundations

Miles S. Kersten, Chairman

The Department of Soils, Geology and Foundations has had an active year and its committees have continued to advance their programs. At the start of the year there was a considerable change in membership. The Department added 13 new members and discontinued 8. In view of the Board plans for definitive time limits on chairmanships, new chairmen were appointed for five of the thirteen committees. Those retiring from these positions were D. P. Krynine, Olaf Stokstad, George McAlpin, and Fred Benson; also one chairman was lost in the death of Harold Clemmer. The service of these men is acknowledged with thanks; all had served their committees faithfully for considerable lengths of time —Dr. Krynine for more than 20 years.

Three of the committees had meetings during the year. The *Committee on Soils-Calcium Chloride Roads* met in May at Pinehurst, N. C., and again in October at Healing Springs, Va.; the *Committee on Soil-Sodium Chloride Stabilization* had a meeting in May; and the *Committee on Landslides* had a two-day session at Steubenville, Ohio, on May 17-18.

The Department membership had the first interim meeting in its history. Twenty of the total membership of 32 met in Chicago for the two days of work and discussion on September 21-22, 1961. The main items considered were the program of papers for the technical sessions at the Annual Meeting and the presentation of a preliminary report by a committee appointed in January 1961 to study a possible reorganization of the Department's committee structure.

The technical program presented at this year's meeting consisted of nine sessions, comprising 46 papers and one conference session on rock slopes. The principal subjects for these sessions were stresses in earth masses, physico-chemical phenomena, soil-portland cement stabilization, lime and lime-fly ash stabilization, and frost action; these five subjects comprised about two-thirds of the soils program. The review of papers at the September meeting gave good control of the program. Only two changes were made after that date; 100 percent of the Department's papers were reported in the synopsis issue of "Highway Abstracts."

Some of the activities of the project committees are summarized in the following:

The *Committee on Subsurface Drainage* held discussions on the status of electro-osmotic drainage, the relative merits of depressed or raised medians, curbs, and the desirable permeability of shoulders and bases. Plans are made for future discussion of mole drain installation.

The *Committee on Compaction of Embankments, Subgrades, and Bases* is reviewing material for a second bulletin on compaction, planned as a companion to HRB Bulletin 272; the material has been prepared by A. W. Johnson and John R. Sallberg.

The Committee on Soils-Calcium Chloride Roads at its two interim meetings has had progress reports on the following subjects: calcium chloride studies in North Carolina, Maryland, and Alabama; base project, Stillwater, Minn.; Maryland base study; use of calcium chloride in lime-fly ash stabilization; use of calcium chloride in soil-cement construction; North Carolina secondary roads study; Kentucky maintenance study; and National Crushed Stone Association study.

The *Committee on Stress Distribution in Earth Masses* has formed four subcommittees. These are:

1. Foundations for overhead signs (this group has cooperated with the Ohio State Highway Department in planning a test program for foundations for overhead signs).
2. Preparation of a bulletin on stress distribution problems. An annotated bibliography is the first project.
3. A committee to delineate and outline needed solutions in the stress distribution field.
4. A committee to summarize current knowledge of the methods of measuring stresses and displacements in earth masses.

The *Committee on Frost Heave and Frost Action in Soils* will present in 1963 a Symposium on Highway Design in Frost Areas, Part II. Eight task forces are already at work on this selecting authors and it is planned to have the papers assembled for initial review at an interim meeting during the summer of 1962.

The *Committee on Surveying, Mapping, and Classification of Soils* this year dedicated Bulletin 299, which contained 9 papers presented at the Soil Mapping Symposium last year, to the memory of Frank R. Olmstead, former Chairman of the Department. A review of subsurface exploration methods of the highway departments is being tabulated for publication. Also, information collected from States on the need to revise the AASHO Soil Classification System was prepared by this committee and sent by the Board to AASHO officials for consideration.

The *Committee on Physico-Chemical Phenomena in Soils* observed this year the 25th anniversary of its founding. A short history of the committee and its work was published in an RCS circular. The committee has received congratulations from a number of organizations and persons on this event. During the year the transactions of the Soil-Water Conference held in 1960 were put into final form for publication. The Soil Science Society of America has invited the cooperation of the committee in its program and one activity being planned is for the committee to take over an issue of *Soil Science* for coordinated papers on a soil theme of importance to agricultural, highway, and other types of soil engineers and scientists.

The *Committee on Landslides* is studying a scheme for a team study of Ohio River Valley landslides, and has started a revision and updating of the 1951 landslide bibliography. This committee this year conducted the first conference-type technical session that the Department has had for several years.

The *Committee on Lime and Lime-Fly Ash Soil Stabilization* published during the year its first RCS Circular (No. 448), "Recommended Lime Stabilization Practices."

The *Committee on Soil-Portland Cement Stabilization* after 2½ years of work completed its task of preparing a selected annotated bibliography containing the significant references on the stabilization of soil with portland cement, 1931-1961. It also has subcommittees active in thickness design, construction, and mix design.

The *Committee on Soil-Sodium Chloride Stabilization* has undertaken the task to clarify the matter of mechanics of stabilization with sodium chloride. The work is being accomplished in three steps:

1. Conduct a questionnaire survey on the use of sodium chloride.
2. Prepare a statement regarding mechanics of stabilization with sodium chloride, with specific reference to the present state of the art.
3. Prepare a statement of research needs.

It is contemplated that these three steps will be completed in 1962.

The *Committee on Soil-Bituminous Stabilization* at its meeting received an informative report on the early history of soil-bituminous stabilization by Hans F. Winterkorn, and K. O. Anderson of the University of Alberta described some research work being done there. Contacts are to be made wtih engineers on soil-bituminous projects in various parts of the country.

(g) Special Committee on Night Visibility

Burton W. Marsh, Chairman

Increasing development of the Interstate System and of other mileages of freeways, with the inevitable increases in night traffic which will result, emphasize further the need for greatly stepped-up research on problems of night driving, major among which are those involving night visibility. It must be remembered that night fatalities account for three-fifths of all traffic deaths.

The committee is composed of 43 well-qualified persons with a good balance between those whose main interest is in basic research (nearly one-third university people) and those having interests mainly in applied research. Both are important.

A lively, productive, and well-attended open meeting with numerous guests heard nine papers or progress reports. Two regular meeting sessions heard nine other papers and reports. One feature, which has for several years highlighted the committee's program, was another splendid review of the research literature by Oscar W. Richards.

Planned for next year is a symposium or panel presentation to bring up to date what is known about seeing at night driving levels, lively discussions of reasons for large differences in professional views as to the level of light needed in fixed highway lighting emphasizes the need for research and the committee is pleased that one of the research areas proposed by it and approved for the coming year in the new AASHO program includes justification of and warrants for lighting of freeways.

(h) Special Committee on Highway Equipment

Morgan J. Kilpatrick, Chairman

The committee during 1961 sponsored two papers which were presented at the 1962 Annual Meeting. Also during the year, the committee, with endorsement from the Department of Maintenance, sponsored publication of Special Report 65, the "Iowa State Highway Maintenance Study."

One paper presented at the Annual Meeting related to aggregate heating and drying for bituminous concrete production. This paper summarizes the findings of several years of basic research. The results are encouraging, even though the phenomena may not be fully understood. Additional work on the quantitative phases of aggregate drying and heating is planned.

The Iowa State Highway Maintenance Study report has generated favorable comment. The committee and others concerned with the subject believe that this publication marks the beginning of an era of increased attention toward and recognition of maintenance problems.

There has been a delay in releasing several reports which the committee expected to recommend for publication as part of the Highway Research Board's continuing series of Road Research Releases. These and other reports are planned for issuance during the coming year.

Among topics discussed at the committee business meeting was one which received enthusiastic support. Briefly, the committee has sponsored issuance of many short reports, called Road Research Releases, for which there is a continuing demand. Many requests for these publications have an origin in the engineering schools. It is planned that all issues to date be consolidated and published under one cover.

(i) Special Committee on Electronic Research in the Highway Field

O. K. Normann, Chairman

During 1961 the committee decided to publish a report on electronic developments or techniques that have application to highway transport and may also include statements of highway problem areas in which electronics research may be desirable or promising. This bulletin will be published on an irregular basis as the quantity of information dictates. The first issue will be published in the spring of 1962.

At the 41st Annual Meeting the committee sponsored two sessions concerned with electronics research in highway transport. A total of 8 papers were presented ranging from electronic weighing of vehicles to automatic vehicle guidance systems. This was the largest number of papers ever presented at a Board meeting relating electronics to highway transport.

The annual business meeting of the committee, held on January 10, 1962, was mainly concerned with discussions of the publication of the reports. In addition, a subcommittee was appointed to examine the status of speed measurement by radar in terms of its accuracy and reliability, and also any history of research on such devices.

(j) Special Committee on Urban Transportation Research

Pyke Johnson, Chairman

On September 18-19, 1961, this committee sponsored a conference of some 60 qualified authorities on urban transportation research. The affair, held at the National Academy of Sciences, was broken down into five panels chairmaned by Henry Fagin, Douglas Carroll, Don Wagner, Morton Schussheim, and Paul Opperman. Subjects discussed were potential research projects in five areas—land use, personal desires, economic impacts, public programs, and local government.

Out of this discussion came reports on each subject which were later discussed at a second conference of the group held during the 41st Annual Meeting. It is planned to put this material together in the form of a report which will complement the work of the E. H. Holmes committee (HRB Special Report 55).

Meanwhile, the committee hopes to establish a clearinghouse for all urban transportation research activities and reports, to add to its own staff, and to continue to hold further conferences.

(k) ad hoc Committee on Research Problems of Mutual Interest and Concern to Users and Producers of Asphaltic Materials

Fred J. Benson, Chairman

The committee held two meetings during the year, one in Montreal in May, the second in Chicago in November. Both of these meetings were well attended.

A Correlation Service Circular issued in October 1961 carried the summary report of the committee's major actions for the three-year period 1958-1961. This report presents the committee's thinking concerning the most important problems of asphaltic materials and their use on which research should be in progress. The committee hopes that the circular will stimulate additional research studies seeking information concerning these problems. The committee also has in the process of publication a discussion of needed research in the problems of the surface chemistry of asphaltic materials prepared by Vaughn Smith, a member of the committee.

Another activity of the committee has been the promotion of regional meetings of users and producers of asphaltic materials for the discussion of matters concerning specifications and other items of interest. The programs are patterned after the effective user-producer organization in existence on the West Coast. A conference was held at Kansas City this year and three others are in the planning stage.

The committee has made a number of suggestions for needed specific research studies; these have been forwarded through channels to the appropriate committee of the Highway Research Board or the American Association of State Highway Officials. Several studies initiated by the committee are also being carried forward by the Asphalt Institute or some of its member companies.

Since the ad hoc committee's function is defining of research problems and attempting to interest qualified research organizations in working on these problems, the committee has no opportunity to point to its own research accomplishments. It does feel, however, that it has made and is continuing to make progress in securing valuable research attention for the problems of asphaltic materials and their use.

(l) Special Committee on Highway Laws

J. H. Beuscher, Chairman

The committee held a business meeting on January 10, 1962, in Washington, D. C., at which time three recommendations were approved. In connection with Research Correlation Service Circulars dealing with highway laws, it was recommended to include notice and digests of current legal writings, State legislative studies, and other similar forms of research. In connection with HRB Special Reports on highway laws, the committee recommended that a form of binder for storing and using this series of reports be made available for purchase by users of this series. Finally, in accordance with the committee's program and also pursuant to sentiment expressed by the AASHO Committee on Legal Affairs, the committee recommended that HRB, acting alone or jointly with other appropriate parties, sponsor a workshop for State highway counsel during 1962. This workshop would be for the purpose of exploring ways for greater utilization of HRB highway laws studies and other recent legal research in current operating problems of highway counsel.

Two sessions of the 1962 Annual Meeting were devoted to nine papers sponsored by the committee. These sessions were organized around recent sign advertising regulation under the Federal-Aid Highway Acts.

During the year the committee welcomed four new members. To enable the committee to carry on its expanding functions, Ross D. Netherton was appointed to the position of HRB Counsel for Legal Research.

During 1961 the following were published by the committee:

HRB Bulletin 294, "Highway Laws: 1961," contains the annual report of the committee, the program for future committee activity, and the four papers presented at the 1961 Annual Meeting.

HRB Special Report 64, "Traffic Engineering: A Legal Analysis," is another in the series of special reports dealing with the legal aspects of highway functions.

"Highway Programming: An Analysis of State Law," and "Law of Turnpikes and Toll Bridges: An Analysis," have been completed, reviewed by the States, and are in the process of being printed. Work is progressing on the following reports: "Maintenance and Drainage"; "Highway Administration"; "Highway System Classification, Part II"; "Highway Planning"; "Highway Financing."

Highway definitions, highway construction, and grade crossing elimination are three more subjects that probably should be reported on, but no work on them has been started. In addition, there are several subjects which are not extensive enough to warrant full-scale report, but which do not fit in with other reports. They are public relations, landscaping, regulatory power, law enforcement authority, and general grants of power and authority. The possibility of treating these as several individual reports under one cover is being considered.

(m) Special Committee on Public Dissemination of Research Findings

John E. Moore, Chairman

The committee has been in existence for approximately 18 months. A complete report on its activities during the first year was given by the first chairman, John Gibbons. Since the 1961 Annual Meeting the committee has undertaken the following projects:

1. Reviewed all of the Board's publications to determine their publicity potential.
2. Written and distributed 25 special releases.
3. Handled press relations for the 41st Annual Meeting, including a press room, and a special press conference on the AASHO Research Program and HRB's role as administrator.

As a result of advance publicity on the meeting, some 33 reporters and editors from the general and highway press registered as covering the sessions.

Press reception to news releases from the committee has been excellent throughout the year.

(n) ad hoc Committee on Driving Simulation

David W. Schoppert, Chairman

For many years, several committees of the Highway Research Board (notably the Committees on Road User Caracteristics, Highway Safety Research, and Electronics) have been interested in driving simulation as a research technique. This interest goes back to the Councilwide Committee on Highway Safety Research, which recommended the development of a research-quality simulator as a result of the first Highway Safety Research Correlation Conference in June 1952. The committee reviewed the recommendations of that conference and selected certain research priorities, one of which was: "As a research tool, development of an accurate simulator for automobile driving."

During the years that followed individual members of the various committees became interested in driving simulation and development of a driving simulator was recommended to the Board's ad hoc Committee on Research Priorities (see Special Report 55). At the 39th Annual Meeting three papers on driving simulation were presented.

These, together with others on the general subject of electronics, were published in HRB Bulletin 261.

Several members of HRB committees participated in the Williamsburg Conference sponsored by the President's Committee for Traffic Safety. The report of that conference proposed driver environment simulation as a means for conducting research into driver behavior in a variety of normal and stressful situations.

During 1960 the Public Health Service, the Bureau of Public Roads, and the Automotive Safety Foundation created a Steering Committee for a National Conference on Driving Simulation. Many members of the HRB committees previously mentioned served on this committee, and subsequently participated in the conference, held February 27-March 1, 1961, in Santa Monica, Calif.

Of special interest to the Board was the following motion, adopted at the National Conference on Driving Simulation:

> "Moved that the Chairman approach the Highway Research Board with the request that it (HRB) establish a means of encouraging the development and application of driving simulation in any way feasible, specifically as a beginning to organizing a clearinghouse of information in this area and to undertake to the extent possible to coordinate the many independent efforts in this field of such great potential."

As a next step the Board appointed an ad hoc committee to:

> "Consider the role of the Board in the field of driving simulation and if deemed advisable to recommend and suggest ways and means by which the Board can take a more active part in the development of this technique."

The type of driver environment simulator envisioned by the Williamsburg Conference and Special Report 55 would be a high-fidelity device capable of simulating a wide variety of driving situations. It might more properly be described as a research laboratory. In fact, the report of the Williamsburg Conference refers to the "large-scale simulation device" as "a large driving laboratory."

In addition, there is the concept of driving simulation as a research technique, which a particular scientist would use or not depending on the experiment he planned to carry out. This might involve simulating only part of the driving task, or part of the vehicle, or some task similar to driving.

These concepts are not in any sense contradictory. Many of the techniques used are similar, and advances in the ability to simulate part of the task could advance the possibility of simulating the whole.

In addition to research, driving simulation has been used for several years for driver training and is being used experimentally for driver examination. Improved means of simulating the task are expected to yield better driver training, selection, and examining methods.

All of these concepts have much in common. The technology is similar, instrumentation of the driver is similar, the variables to be measured and the units of measurement are similar. Thus, any new information related to work in one of these on either the techniques or devices used or on the driver-vehicle-road system is of value to others working in any of the areas.

Recent years have seen a spread in interest in simulation techniques which is continuing at an accelerating rate. This, coupled with rapid advances in the technology, has made it difficult if not impossible for individuals to keep abreast of the work that is going on and the improvements in techniques and instrumentation which are taking place. Experience in the fields of space, defense and industry provide a considerable backlog of information not always readily available to others outside these fields. Thus the need for a forum and clearinghouse of information is apparent. The potential of simulation is also clearly apparent.

The question of "coordination of the many independent efforts" is another matter. The need for coordination in its compulsory sense is, at best, debatable and the means for achieving it are not immediately apparent. On the other hand, voluntary coordination naturally flows from the free exchange of information and ideas. In that sense, the Board could provide the means for coordination.

The question then arises whether this might best be done by an existing committee or by a new committee. Considering the wide variety of interests represented, the emerging nature of simulation, and the high degree of specialization involved in many of the techniques, it would appear that a special committee could best carry out the program which is indicated.

The ad hoc committee recommends that a Committee on Driving Simulation be organized as a special committee of the Highway Research Board. It would be the purpose of this committee to stimulate research, development, and application of driving simulation techniques and devices and to serve as a clearinghouse for information in the field.

There is a wide variety of interest in driving simulation. This comes from the fields of safety, public health, highway design, traffic operations, driver education, traffic law enforcement, driver examination and licensing, motor vehicle design and from universities and industry. The membership should include interested persons from as many of these fields as possible.

(o) Calcium Chloride Stabilization

This project in its entirety consists of a continuing series of projects ranging from basic studies of the chemistry of the interaction between calcium chloride and various types of pure clay to observations of the effectiveness of calcium chloride in facilitating construction and maintaining of traffic-bound roads and granular base courses. Special funds for some of the activities are furnished by the Calcium Chloride Institute.

(p) The AASHO Road Test

W. B. McKendrick, Project Director

The year 1961 was a "finish-the-job" year at the AASHO Road Test. The final phases of research work were carried out and the vast majority of the project's activities were brought to a successful conclusion.

Psychologically, at least, the process of closing out the project was more difficult than the organizational phase which ocurred a few years ago. Disposition has been made of the bulk of the project's physical plant. Personnel has been reduced to about a dozen people who are engaged in final clean up and the preparation of files which will be retained by the Highway Research Board. In this connection it is gratifying to note that many of the people who have left the project during the past year have succeeded in securing excellent positions with Federal and State government agencies, universities, and industry. Those remaining at the project will complete their work in a few weeks, and all Highway Research Board activity at the site will be ended by February 1, 1962.

The main research phase of the project had been completed by this time last year and work had already begun on certain of the planned post-traffic tests. During the winter, accelerated fatigue tests were carried out on seven of the thirteen surviving test bridges. In these tests a mechanical oscillator was used to excite the bridges in such a way as to produce stresses approximating the average condition observed during the regular test traffic. Repetitions of stress cycles were continued until the bridge failed or until the total cycles, including those produced by test traffic, had reached 1,500,000. Many valuable data were collected during these studies.

The main post-traffic special studies were carried out during April, May, and June. These included a performance study of certain pavement sections on Loop 2 which had previously been subjected to 2,000- and 6,000-lb single-axle loads. In this special study the pavements were subjected to traffic by military M-52 tractor-semitrailers loaded to 32,000 lb on a tandem axle. In one lane of the loop the vehicles were equipped with normal tires; in the other, with the special low-pressure, low-silhouette tires. Also carried out were a tire pressure-tire design study, a study of the effects on pavement of certain commercial construction equipment, special suspension systems, and various types of military vehicles.

Although all these studies provided many interesting data, it soon became apparent that the Road Test experiment design and instrumentation were not capable of detecting the effects of such variables as tire pressure, tire design, and vehicle suspension. The studies did, however, serve to point out the direction that future research should take.

Further tests on bridges also were run during this period. All surviving bridges were subjected to tests with increasing loads until failure or until the load capacity of the vehicle was reached. Again, many valuable data were collected.

While the various post-traffic special studies were under way, the major task of 1961 was also proceeding. This, of course, involved the final steps in data analyses and the writing, reviewing, and editing of the final reports.

The final analyses of data from the main Road Test experiments produced certain revisions, refinements, and improvements in the mathematical equations contained in the preliminary report submitted to the Bureau of Public Roads as of July 1, 1960. Thus, a "Second Preliminary Report on the AASHO Road Test" was produced and forwarded to the Highway Research Board as of May 15, 1961. The purpose of this and the previous report was to provide information for use in the Bureau's Highway Cost Allocation Study and in the current work of the AASHO Design and Highway Transport Committees.

Once the major analyses were completed, the data analysis branch and the research branches concentrated on analyses of data from the various side studies conducted in conjunction with the main tests. It was not until these many analyses were largely completed that work could begin in earnest on preparing the final reports.

The first two of the projected seven major reports were prepared by a special report-writing branch of the staff. Report One, "History and Description of the

Project," has been published; Report Two, "Materials and Construction," is in process of publication.

The major research reports have each been prepared by the staff branch directly concerned. These are: Report Three, "Vehicle Operations and Pavement Maintenance"; Report Four, "Bridge Research"; Report Five, "Pavement Research"; and Report Six, "Special Studies."

Initial drafts of each report were prepared by personnel of the appropriate branch. The draft was then reviewed by a staff committee composed of the Chief Engineer for Research, the Chief of Data Analysis, and the Chief of Public Information.

Following the on-project review a first draft was mailed to members of a small review committee. Such committees were appointed by the Board to review each of the reports. The committee was allowed sufficient time—usually a month to two months—for a complete and careful study of the report. Written comments were forwarded to the committee chairman, and a meeting was held to discuss all suggested revisions.

The committee review sometimes resulted in considerable revision of the draft report. Often it entailed additional data analyses, rewriting, and drafting of figures and tables.

The revised report was then mailed to all members of the National Advisory Committee, the Regional Advisory Committees, and the Executive Committee of the Board. Again, an extended period was allowed for review, and written comments or approval were invited. In most cases, however, the thorough review by the smaller committee forestalled any major changes at this stage.

Upon approval by the National and Regional Advisory Committees, a third draft of each report was prepared for submission to the Board for final review, editing, and publication.

It is obvious that the process of writing, reviewing, revising and editing the various reports consumed considerable time. However, the opinions and suggestions of the various reviewers have proved invaluable to the project staff, and have resulted in excellent finished products.

At the present time, Reports One, Two, and Three have been submitted to the Board. Reports Four, Five, and Six will be submitted later this month.

The seventh report, which will be a summary of the six previous reports, is in the first stages of preparation. It will be completed by Road Test staff members now working in the Washington offices of the Board.

Although the major task in 1961 was the preparation of reports, the project's service branches continued to perform other duties throughout much of the year.

The data analysis branch was, of course, directly concerned in many phases of report preparation. In addition, the branch had the task of setting up data systems, IBM card files, catalogs, etc., so that the most important data from the project could be conveniently stored by the Board. This data will be available for future use by the Board and by other interested parties who will wish to make further analyses.

The materials branch functioned on a reduced scale during much of 1961. Its main duty consisted of conducting so-called "trench" studies; that is, studies of subsurface conditions in the pavement test sections. An extensive program was carried out after the completion of regular test traffic, and other studies were made at various times during the post-traffic special studies period.

The pavement maintenance branch was responsible for an extensive maintenance program on Loop 2 prior to the start of performance study traffic in the spring. The branch also carried out necessary maintenance during the special studies.

Activity of the operations branch declined rapidly following the end of regular test traffic. However, a small force was maintained in the operations office and garage through the special studies period. The activities of the branch ended soon after the completion of the post-traffic special tests.

The instrumentation branch continued to be responsible for the maintenance of instruments used in the special studies. In addition, the branch performed further development work on the CHLOE profilometer, a simplified version of the rather complicated instrument developed for use at the Road Test. Three of these instruments have been built and are in use. Two are currently being used by the Bureau of Public Roads in a nationwide survey designed to determine the serviceability level at which major maintenance is performed on pavements.

The public information branch was concerned throughout much of 1961 with the editing of reports. Some normal public information work did continue. There were no general news releases, primarily because there was nothing new to report, in view of the fact that test results were restricted. Such releases as were made came about as the result of special requests, and were handled on an individual basis. Several local releases were prepared to keep the community informed of developments at the test site; and thus prevent rumors and misinformation. In the radio-TV field, the branch assisted in preparation of a short public service film for showing on the U. S. Steel Hour in May. More recently, the branch has been enaged in work on a series of films on the project. These films, produced with the cooperation of the Bureau of Public Roads, are expected to be valuable additions to the printed reports.

The Army Transportation Corps Road Test Support Activity was allowed to decline in strength through the final few weeks of regular test traffic in 1960. Following the end of traffic the unit was cut to a small nucleus of officers and enlisted men who remained on the project through the post-traffic special studies period. All military personnel had left the test site by July.

In summary, 1961 saw much activity at the test site. Some of this activity was concerned with further research, but the bulk of the work involved the final reports, classification and filing of data, disposal of property and equipment, and the various duties connected with the termination of the project. The staff believes that all of these tasks have been successfully completed or are nearing completion, and that the six years of activity at Ottawa have resulted in major advances in knowledge which will be invaluable to the entire highway industry.

This will be the last yearly report to the Board on the AASHO Road Test. It has been a remarkable project in many ways and it has been fortunate that it was possible to assemble the very fine staff that conducted the work in Ottawa. All of this group have now assumed other responsibilities in all parts of the country. In most cases they will continue in research work.

5. Report of Director

The sudden passing of Elmer M. Ward, Assistant Director, extremely saddened the Board. Mr. Ward had joined the technical staff in 1946 as Engineer of Maintenance, and later served also as Engineer of Materials and Construction. He was appointed Assistant Director in 1954, and served in this capacity until his death on August 27, 1961. His many years of faithful and devoted service to the Highway Research Board will be long remembered.

The 41st Annual Meeting of the Board was a successful one, as attested by the following:

Number of sessions	59
Number of papers and reports (Note: 412 authors and co-authors, of which 211 (51 percent) presented papers for the first time)	295
Number of committee and department meetings	99
Registration	2,447

Classified Registration at 41st Annual Meeting

Organization	Registration
State highway departments	493 [a]
Colleges and universities	353 [b]
Industry, trade associations	681
Bureau of Public Roads	274
Other Federal agencies	148
Foreign countries	140 [c]
Consultants	122
City public works	50
County highway departments	29
Others	157
Total	2,447

[a] Representing 47 state highway departments.
[b] Representing 87 colleges and universities.
[c] Representing 38 foreign countries.

Publications—The papers and reports published by the Board originate primarily from presentations at the annual meetings. Exceptions are special reports of committees, reports of findings from special projects administered by the Board, and other contributions including those of the staff.

During the calendar year 278 papers and reports, 678 abstracts and 25 circulars were distributed. These publications totaled 5,704 pages.

Committee Activity—During the year the Board's six departments and 95 technical committees were active in conducting highway research and in analyzing and correlating the results of completed work. The Board's committee roster now includes 1,058 men who fill 1,680 committee assignments. The members are selected on the basis of their abilities to contribute to the studies of subjects under consideration and are appointed from State highway departments, Federal agencies, colleges and universities, counties, cities, industry, and other agencies with qualified personnel.

Associates—Individual Associates are those qualified by interest and technical activity who wish to affiliate with the Board and receive its publications. Associate dues are $15 per year, for which one receives the PROCEEDINGS and HIGHWAY RESEARCH ABSTRACTS. To receive all principal publications of the Board an Associate by an additional fee of $10, $20, or $25, depending on whether he is classified as Academic and Government, General, or Foreign Associate, respectively, may receive all publications except CIRCULARS, ROADSIDE DEVELOPMENT, and SPECIAL REPORTS. Nearly 60 percent of all Individual Associates avail themselves of the special subscription plan. During calendar year 1961 there were 192 new Individual Associates added to the Board's roster. The grand total of affiliates for the year was 1,605, of which 1,121 were domestic and 484 foreign. Because a large percentage of the engineers of State highway departments in this country receive the Board's publications through the Research Correlation Service, the Associate list consists mainly of engineers outside the State highway departments.

Firms and corporations engaged or interested in highway research and transportation may become Associates of the Board. A designated representative and alternate representative receive publications of the Board.

New Staff Personnel—During the calendar year five new members were added to the staff. Three of these had been staff members of the AASHO Road Test: W. N. Carey, Jr., Assistant Director; Paul E. Irick, Research Statistician; and Robert B. Damon, Research Assistant. Adrian G. Clary, formerly with the Wyoming Highway Department, was employed by the Board as Engineer of Maintenance, and Ross D. Netherton, formerly with American Automobile Association, was employed by the Board as Counsel for Legal Research.

Field Service—In response to an expressed need and with the support of the State highway departments, the Board in 1945 expanded its service of stimulating research and of disseminating information through the institution of the Highway Research Correlation Service. An initial and continuing objective of the Service is to find out what highway problems exist that might be solved by research, to assist in the establishment of research project committees or in arranging some other appropriate means to solve the problem, and to convey the findings to all interested persons.

In accordance with the adopted plan for correlation service, the Board has six professional engineers, each of whom is specializing in a branch of highway technology represented by the six departments under which the technical committees operate. As members of the technical staff, these engineers serve their respective departments and committees with technical assistance. A considerable part of their time is spent in making periodic visits to the State highway departments and other agencies engaged in highway research, where they sit down with the administrator and researcher and discuss their problems of operations and research activities, thus acquiring and disseminating first-hand information on developments of a nationwide highway research program amounting to an expenditure of several million dollars annually. Hence, through the field contacts of the technical staff the Correlation Service links each State highway department with other State highway departments, Federal agencies, State colleges, and industrial organizations engaging in highway research.

In addition to the liaison provided by the staff engineers, other special services are provided, such as help in formulating research projects, preparation of special bibliographies, search for specific library information, compilation of regional practices or procedures relating to special problems, preparation of lectures for conferences or schools, and other related services.

During the 1961-2 fiscal year 88 visits were made to State highway departments and 100 to colleges and universities. In addition to these services the staff engineers prepared papers on special topics, conducted seminars, and arranged special conferences as well as interim meetings of committees and departments.

Another example of services performed involved preparation by the library of 51 lists of references; reply to more than 142 inquiries, largely from State highway departments, involving library reference material; and selection of 465 abstracts for use in Highway Research Abstracts.

In order to strengthen, improve and expand the Board's general objectives, the Executive Committee resolved that additional financial resources from industry should be encouraged. To implement this program, a new type of membership, known as "Industrial Members" (annual dues $1,000), was created. In September 1961 a program was held at the National Academy of Sciences to acquaint about 100 industrial representatives with the activities of the Board. The response to this effort was immediate, as 22 industrial organizations have submitted applications for membership. It is expected that other industrial organizations will subscribe to this membership.

The members of the engineering staff are: M. Earl Campbell, Engineer of Economics, Finance and Administration; Ray E. Bollen, Engineer of Materials and Construction; Adrian G. Clary, Engineer of Maintenance; Franklin N. Wray, Engineer of Design; Kenneth G. McWane, Engineer of Traffic and Operations; A. W. Johnson, Engineer of Soils and Foundations.

6. Expressions of Appreciation

Motion: By K. B. Woods, seconded by M. A. Butcher.

That in view of the outstanding contributions made during his term of office, immediate Past Chairman W. A. Bugge be given a special vote of thanks.

That formal appreciation of this body be extended to the Highway Research Board staff for outstanding accomplishments during the past year with an unusually heavy work load.

That formal appreciation be extended to the entire AASHO Road Test staff for outstanding accomplishments during the year, for completion of reports and closing out the work of the project.

That letters of appreciation be sent to those who participated in the program at the 41st Annual Meeting of the Highway Research Board.

Adopted

7. New Business

Chairman R. R. Bartelsmeyer: I want to express my appreciation as Chairman for the very fine reports given by the Department and Special Committee Chairmen.

8. Adjournment

No other business appearing, the meeting was adjourned at 3:30 P.M.

—FRED BURGGRAF, *Director*

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