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ENGINEERING EXPERIMENT STATION
OREGON STATE COLLEGE
CORVALLIS, OREGON

Proceedings of the 1959 Northwest Conference On Road Building

Sponsored by
OREGON STATE COLLEGE
Civil Engineering Department,
the Oregon State Highway Department,
and the
Student Chapter of the
American Society of Civil Engineers

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Assistant Traffic Engineer
Oregon State Highway Department
Presiding

1:30 Opening Session.
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2:00 PANEL DISCUSSION. UTILIZATION OF AVAILABLE ENGINEERING SERVICES BY COUNTIES AND CITIES.
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Presiding

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4:00 DESIGN PROBLEMS OF THE SEATTLE FREEWAY. W. E. McKibben, District Engineer, Washington Department of Highways.

5:00 SIMPLE SYSTEM OF DETERMINING PRIORITIES OF IMPROVEMENT FOR LOW TYPE ROADS. John A. Anderson, County Engineer, Marion County.

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Assistant Dean of Engineering
Oregon State College
Presiding

9:00 OREGON STATE HIGHWAY DEPARTMENT'S RECENT EXPERIENCE IN DESIGN AND CONSTRUCTION OF PORTLAND CEMENT CONCRETE PAVEMENT. G. W. Harra, Engineer of Materials, Oregon State Highway Department.

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11:00 SOME EFFECTS OF MIXING TIME AND BATCH WEIGHTS ON THE QUALITY OF PAVING CONCRETE. Carl Minor, Materials and Research Engineer, Washington Department of Highways.

WELCOME

Secretary of State Howell Appling, Jr

I am very grateful for this opportunity to be here with you this afternoon and participate in the opening of the Northwest Conference on Road Building.

As I begin to participate in the affairs of our state government and study some of our problems and challenges, it brings home to me once again what a great and continuing need we have not only for your technical talents, but for your resourcefulness and leadership in this and, of course, all other fields of our state endeavors.

In looking over your program, I cannot help but be impressed by the long list of very highly qualified people what are engaged in this conference, and certainly by those who have had the foresight, initiative, and leadership to inaugurate this meeting and to continue it for the past 12 years.

In attending this training session today and for the next several days, you bring with you a responsibility, it seems to me, not only to yourselves, but to your profession to make the fullest application of your abilities. As public employees you certainly carry with you a corresponding responsibility to seek maximum efficiency and economy in our highway construction and maintenance programs.

I am sure you observe election returns as I do, and have received the message the taxpayers sent us concerning economy. That is the reason for my reference to economy in highway construction and maintenance.

Few changes in the American scene have been so dramatic, I think, as the phenomenal increase in what we might refer to as the mobility of the American public. Made possible, of course, by the dedicated efforts of people such as yourselves in years past and, of course, now.

Oregon's centennial observance seems in a large measure a salute to one of the most notable roads in our history. That is, of course, the Oregon Trail. The trailblazer and roadbuilder have always played a unique role in our nation's development. As planked roads replaced wagon ruts and oil surfaces settled the dust of buggies, wagons, etc., so, too, have superhighways become the standards for our generation.

We cannot rest, however, for not only are there new things on the horizon in motor transportation, but we still have many thousands of roads yet to be built or rebuilt. The reminder that primitive roads are still within the memory of many of us, and that substandard routes still hamper the fullest development of our resources, will only to serve to spur our efforts toward finding new approaches to our highway construction problem.

We in state government appreciate the contribution that roads have made in transforming the Northwest from a covered bridge economy, shall we say, to a modern and progressive era. It is equally evident that the contributions that you as engineers and technicians have made and are continuing to make have served as the roadbed for this economic progress.

Just before I left, Governor Hatfield asked that I extend his own warm greetings to you, and to express on his behalf the hope that your work here in the next several days will be both pleasant and profitable.

You know we have the Governor pretty busy these days, what with the Legislature in session and the necessity for working up and putting through a new budget. It is only the pressure of this business that has prevented him from being here with you today.

Again, let me welcome you here to Corvallis.

Panel Discussion

UTILIZATION OF AVAILABLE ENGINEERING SERVICES BY COUNTIES AND CITIES

Edward J. Warmoth, Moderator

O. R. DINSMORE

For purposes of administering Washington's highway department, the State of Washington is divided into seven districts. Each district has a traffic engineer, soils engineer, state aid engineer, maintenance engineer, and other specialists, working under the district engineer. All are available upon request to be of service to cities and counties in an advisory capacity. The district aid engineer, however, is in direct contact with cities and counties, and he often advises officials on matters coming up from day to day.

Probably the most popular division with cities and counties, in terms of requests for advice, is the traffic engineering section where assistance to local officials is given in signing, speed control, pedestrian crosswalks, signalization, and other traffic problems.

Soils and materials testing is constantly being done by the materials division for local governmental units to determine soil characteristics at bridge sites and for road improvement projects. One laboratory employee spends full time on these projects. The equivalent of one field crew, which includes a drill and three men, is occupied approximately four months of the year on these projects. A geologist is often in demand as a consultant on investigation of materials and landslide problems. In addition, the district soils engineer is often called upon to make special studies.

The maintenance division works closely with cities and counties, and a manual of maintenance practices based on state experiences and accepted techniques has, in general, been accepted by the counties and, to a certain extent, by the cities. An indication of the concern over maintaining roadways is expressed by the close working arrangement between state and county officials in working toward uniform weight and clearance restrictions on county roads and state highways.

The bridge department checks all bridge designs on the Federal-Aid secondary system, which includes many miles of county roads. Some counties and cities design their structures, but for the most part consulting engineers are called upon for that purpose, and the state must pass on their design before Federal money is made available.

The planning division keeps a current log on all county roads, together with an inventory indicating their physical characteristics. The inventory is made for all city streets for cities under 5000 population. Origin and destination studies are made with joint participation between state and city or county and, in some instances, the planning division acts only in an advisory capacity. Traffic counting equipment is available at all times for use by other governmental agencies. Each year shows an increase in compilation and use of comprehensive studies by cities and counties. King County for several years made a county-wide traffic count. Their field data are forwarded to the highway department for compilation and processing. This office engineering service is available to all counties that institute a county-wide traffic counting program.

One of the most important services made available to counties by the planning division is map making. Draftsmen in the planning division prepare and process maps for local units of government.

In general, cities and counties have good engineering services. Laws of the State of Washington provide that the county engineer be a registered and licensed professional civil engineer, duly qualified and experienced in highway and road engineering and construction. In addition, over 50 cities employ city engineers. Large cities, of course, have extensive or complete engineering sections, including traffic engineering divisions. A few of the larger counties employ a traffic engineer and, as these counties become more urban in nature, more traffic engineering services will be required.

Federal-Aid secondary funds in Washington are divided 50-50 between state and county systems. Federal-Aid funds are allocated to each county in the same proportion that guides the allocation of gasoline tax revenue. These funds are available for use on approved Federal-Aid secondary roads on a matching basis, where the county puts up 50 percent of the money and the Federal Government puts up the remaining 50 percent. The board of county commissioners selects the projects for improvement, and the county engineer prepares the contract plans. The plans and contract division of the highway department checks all plans, advertises, and awards the contract for the county. The bridge division, as noted before, checks all bridge plans. A resident engineer-inspector is then available to assist the county engineer in the supervision of the construction. This arrangement has long been in effect, and has proved most beneficial to all agencies concerned.

The statutes of Washington require the division of municipal corporations to provide a full-time auditor to assist the counties in arriving at and maintaining a uniform method of accounting in order that all counties will keep their accounts on the same relative basis.

The computer section, which includes IBM equipment and other mechanical means of making machine calculations for our highway program, has been called upon as a consultant to cities and counties desiring to inaugurate this type of program. This service will continue to be used in an advisory capacity as this labor saving method becomes more popular and more easily adaptable to city and county problems.

There is always room for improvement in the utilization of engineering services among governmental agencies. Many important improvements have been made by coordinated effort, such as in uniform traffic controls and in common design criteria. In general, there is a feeling of mutual responsibility among the engineers of the governmental agencies within the State of Washington, and there is no doubt that the close coordination among these agencies will continue to the benefit of all.

W. C. WILLIAMS

The State Highway Department is in a position of aiding cities and counties to a considerable extent in problems of traffic investigation, including control, routings, one-way street couplets, and numerous other factors. This service, however, must be limited in the main to technical advice and the furnishing of information which is already available to the State Highway Department by reason of either individual studies and investigations or the accumulation of information obtained through normal studies and investigations in continuing years. Much of this information can be furnished by the state at little or no additional cost to the State Highway Department. Whereas, if the cities or counties were to obtain this information elsewhere, the cost could run into considerable amounts.

Also, in the matter of bridge design, the state is required by law to design bridges when requested to do so by the counties, and we have in the past and will continue to furnish such bridge designs as are requested by the counties. The counties, however, are required to provide the necessary vicinity map surveys needed to make the bridge design. When personnel was available, the state also has furnished bridge designs to cities if the cities paid the cost thereof. The State Highway Department is not authorized by law to furnish bridge design services to cities without compensation therefor.

The county and city relations division of the State Highway Department is ready and willing to furnish technical assistance and advice insofar as it is capable of doing so. Particularly, the county and city relations division should and can act as a direct liaison between counties and/or cities and the State Highway Department, and the department invites as much usage as possible of this division for the benefit of the cities and counties.

In the matter of actual location and design of city streets, county roads, etc., the department is not in a position to furnish such service except in the case of county roads which are on the Federal-Aid secondary system; in which event the department furnishes the location-survey service to the counties without cost. For other than the Federal-Aid portion of county roads, survey, design, and improvement of county roads (or city streets) should be undertaken by the agency's own engineering force, whether it be city or county engineer, where such is available. In the event that there are counties or cities that do not have an engineer, the aid of consulting engineering firms should be engaged. There are several in the State of Oregon that are extremely capable.

There are some other cooperative projects, such as the $250,000 fund earmarked by legislative action for cities of less than 5000 whose streets are being damaged by reason of increased population or heavy industrial traffic. All investigations and engineering work in connection with such projects are furnished by the Highway Department without cost to the cities.

Similarly, there are a considerable number of cooperative projects between the state and the cities for improvement of city streets over which state highways are routed. In general, in such improvement, when providing for curbs and gutters with widened pavements, the cities usually cooperate toward the cost in the amount of 25 percent. This is based on the fact that such widening and curbing are of major benefit to the abutting properties, and in many instances the benefit is greater than that accruing to the user of the highway surface itself. In such cooperative projects the preliminary engineering is furnished by the state.

B. M. FRENCH

Back in 1954 it was the opinion of the Bureau of Public Roads, and apparently also of Congress, that the secondary program, which represented only 30 percent of the Federal-Aid funds handled by Public Roads, was requiring a disproportionate part of the time of our field personnel. That was the year the big interstate program was first proposed and, after a bitter fight, failed to pass. However, Congress provided in legislation adopted that year that Public Roads turn over to states and counties all responsibility for design and construction of secondary projects.

Under provisions of the 1954 act, states may submit to Public Roads statements of procedures to be followed in administering the secondary roads program. After approval by Public Roads of the state's plans, which include standards to be followed, construction practices, materials testing procedures, maintenance, responsibilities, etc., the state then operates under the Secondary Road Plan as provided by the Highway Act of 1954. All states in Region 8 are now operating under the plan.

When a state is operating in this manner, Public Roads does not review plans or designs, does no concur in awards, nor make periodic inspections of construction, etc. Our area engineers, however, do make a final inspection of the project, which enables payment of the final voucher. Our procedures of approving secondary system changes and additions and preliminary overall planning and programming of projects to absorb available funds remain about as they have always been, but we are a lot further away from the location, design, and construction than we were before.

Apparently the reasoning behind the change provided by the new legislation was a desire on the part of Congress to:

1. Have Public Roads devote a greater portion of its efforts to the more complex problems of interstate, primary, and urban systems.

2. Avoid competition with states for additional engineering talent to handle the anticipated large interstate program by reducing the Public Roads' work load on the least complex of the Federal-Aid programs.

3. Give the states and counties correspondingly greater latitude for independent action, and more responsibility for local road systems.

Most states now have adopted the Secondary Road Plan, and results have been satisfactory to Public Roads. Whether, from the standpoint of states and counties it has been good or bad, is a matter of individual opinion. It has taken considerable work off Bureau engineers, but any slack it might have created has long since been taken up in the accelerated interstate program.

As I stated before, all phases of our work on secondary roads which preceded the design remain substantially unchanged. Public Roads' engineers in both regional and divisional offices will be glad to work closely with state highway departments and counties in studies designed to select an adequate FAS system, or in the classification of a county highway system, regardless of FAS status.

Proper road inventory records and something on the order of a sufficiency rating are recommended for counties as a part of their long-range planning. Public Roads, in cooperation with state highway departments, can offer assistance and advice in organizing and conducting studies of this nature and in compilation and interpretation of the results.

At the programming stage for FAS projects, Public Roads' engineers will meet with county and state people to advise and consult on the adequacy of proposed improvements. Priority studies designed to establish an adequate construction program are important to many counties, and Public Roads also can provide advice on this type of study. Since we are practically out of the picture after the programming stage, we should be consulted before final programming. Our participation after that, except to make a final inspection, is contrary to the way we are supposed to operate.

Public Roads still has the same responsibility as always in determining that counties and states are meeting their maintenance obligations on roads improved with secondary funds. Periodic maintenance inspections are made by area engineers from division offices. Any county seeking advice on maintenance operations could get it upon request from the Bureau and state engineers.

Actually, under the plan, a county should not notice much change. State-county relations and procedures should not be different from those followed before adoption of the plan. Generally speaking, responsibilities relinquished by Public Roads are assumed by state highway departments, and controls and inspections should be about the same as before.

It is true that through the operation of the plan the field personnel of Public Roads will not enjoy as frequent opportunities to meet with county commissioners and engineers as they did formerly. Our engineers, however, will continue to the extent possible to meet with you, especially during final inspections of construction projects and during biennial maintenance inspections. Because our field engineers will maintain close contact with the state highway department engineers, they will be generally informed of the progress of the secondary program. Personnel of our division offices will continue to be available to you at all times to assist in the solution of complex problems connected with the secondary program.

Public Roads, in no sense of the word, is removing itself from the field of secondary roads. By close and continued cooperation with state highway department engineers, we will keep informed of the progress and problems of the secondary programs, and to the extent feasible and permitted within our regulations will work with the state people to offer assistance to the counties.

DONALD B. WEST

We have heard from the three previous speakers of the engineering services that are, or could be, made available to the counties and cities by the Bureau of Public Roads and the state highway departments. Some of us may have expected more, and others of us may be surprised to learn that there are so many. In either event, as a representative of the counties on this panel my comments will be confined to those services mentioned that are useful to the counties and to those services that would be useful if they were available.

It is quite evident that the county engineer is willing and anxious to construct as fine a county road system for his county as financial resources will permit. He is often handicapped by lack of modern engineering tools and specially trained personnel, but I am sure he will take advantage of any engineering services that are made available to him if they will help him to reach his objective.

From my own experience as a county engineer and from comments gathered from colleagues, I have found that there are a number of services available from the highway department that we have taken as a matter of course, and without which we could conceivably get ourselves into difficulties.

The first is in the matter of specifications. Most counties in Washington have adopted the "State of Washington Department of Highways Standard Specifications for Road and Bridge Construction." Obviously, this has saved duplication of effort in specification preparation, developed a standardization of construction, and given counties the advantage of specialized information. Similarly, use of the "Manual on Uniform Traffic Control Devices," and reference to the "Manual of Instructions for Field Engineers," both published by the department and made available to the counties, have smoothed the way for many of us.

Materials testing involves the use of special equipment and trained personnel that very few counties can afford. Services offered by the materials testing lab, both at district and headquarter levels, are utilized extensively by the counties. I am sure that most counties will continue to take advantage of this service as long as it is available.

Few counties have the work load necessary to employ a full-time bridge design engineer, yet bridges are an integral part of the county road system. Use of standard bridge plans as prepared by highway departments, and especially the one prepared by the Bureau of Public Roads entitled "Standard Plans for Highway Bridge Superstructures," relieves counties of much time and expense in preparing designs of their own. Special bridge projects, of course, require special treatment, and the bridge department has been receptive to preparing designs and specifications when there is no conflict with their own state work. They very often have checked the designs of other engineers when they have been employed by the counties. Federal-Aid secondary projects require that this must be done before approval is granted. The burden of the accelerated highway program has swamped bridge departments of the states and the Bureau, and I am sure they would encourage as much as possible the utilization of consulting engineers for structural design. However, their specialized knowledge is welcome assistance to the counties.

County roads as well as state highways are constructed to carry traffic and, again, we welcome the talent and facilities of the traffic engineering departments in the design of intersections, traffic control devices, traffic counting, O&D surveys, and the like. We may not agree with some of their conclusions as they affect county roads, but we are willing on the most part to take advantage of their special training in resolving a difficult intersection problem, or in using their traffic count figures to determine priority of construction.

The use of electronic computing systems has been given much publicity in the last two years. The State of Washington Department of Highways has made its equipment available to counties of that state, and has conducted clinics in several districts to acquaint county engineers and state highway personnel with the proper way to secure necessary field information. My county has not taken advantage of this service as yet, but we feel that we will sooner or later. For smaller counties limited in the number of engineering aids they can employ, it would appear that this device would greatly speed up computation of earthwork quantities and help immeasurably in the rapid preparation of plans. I am sure that as counties become more familiar with the possibilities of electronic computations, they will take fuller advantage of them.

State highway departments and the Bureau of Public Roads have engineers who are specialists in many fields of highway work —all the way from determining the economic feasibility of alternate locations right down to field inspection of materials and construction methods. If the talents of these men are available to counties, it seems to me that in the interest of getting the greatest value from our highway dollar, we should take advantage of their knowledge and experience.

These are a few of the available services that are useful to counties.

What could we use as a service if it was available? One thing in particular comes to my mind, and that is the employment of experienced, qualified right-of-way appraisers and negotiators. Right-of-way acquisition is a frustrating experience at best, and at the county level is generally done by the county engineer or county commissioner, neither of whom knows a lot about what he is doing in offering a fair purchase price for a piece of needed property. Neither does he have the teeth in the eminent domain law that state or Federal agencies are blessed with. If services of state highway department right-of-way men could be made available to counties, backed up by a little more realistic condemnation law, I believe that our county highway program could be carried out more nearly the way it was programmed.

A. M. WESTLING

> It is time to accelerate the evolutionary process of that part of highway planning required to bring into perspective the long-range relationship between cities and urban trafficways.

Opening up new traffic streams to relieve presently flooded arteries requires consideration of the eventual effect on other aspects of the urban community. Past procedures need to be replaced with programs that give more emphasis to community patterns and growth trends and less emphasis to artificial extensions of current traffic origins and destinations.

This need is recognized by the highway profession as well as by those who are concerned with arranging for sound growth and preservation of urban centers. It is significant that some highway literature is beginning to treat urban highway development as a part of an integrated problem of urban planning, rather than as an independent field which can continue to go its own way.

For example, Highway Research Bulletin 190 contains an article by Paul C. Watt, director, National Capital Regional Planning Council, which discusses the planning and research implications of a Washington, D.C. transportation study. It is of interest that this study started, not with traffic volumes and O.D. surveys, but with the development of a general regional plan. This plan initially involved study of the following items:

1. Economic base analysis. (How many people will be working and where.)
2. Existing land use.
3. Land capabilities studies.
4. Regional factors influencing growth.
   a) Water supply and sewage disposal
   b) Airport requirements
   c) Soil conditions
   d) Defense considerations
5. Developmental objectives.
   a) Provision for the seat of government
   b) Provision for population
   c) Provision for employment
   d) Provision for accessibility

e) Provision for open space

6. Population distribution and residential land use.

7. Employment distribution and commercial and industrial land use.

8. Recreation and open space.

9. Circulation.

Finally, after these considerations had been evaluated, there emerged a general development plan. The next step includes determining the transportation requirements.

The purpose in describing this project is not because this kind of planning is new. In general, this is urban planning as it has existed for some years. The point of interest is that this article was published by the Highway Research Board, and the planning study was an intended part of a transportation study.

There are many unanswered questions in the field of urban planning, just as there are in transportation planning. The unanswered questions and mistakes in both fields should be lessened as the working relationship between the professional employes of cities and state highway departments become more intimate. As stated by Mr. Watt at the end of his article on the Washington, D.C. study: "Until all of the professions collaborate on metropolitan problem solving, the present day piecemeal studies approach will continue to be too little and too late."

The need for this working relationship is as important in the city where there is no impending state highway project as it is in a city currently being disrupted by extensive highway construction. Every city cannot have city employes highly versed in the many technicalities of traffic engineering and highway planning. At the same time, "one shot" plans or special studies do not allow for the necessary integration of overall development plans and matters of highways, roads, and streets. The cities need to be able to look to the state highway planning experts for continuous advice and council at an increased activity level.

One of the conclusions reached in the Washington, D.C. study was that it would be only the growth after a decade or two has passed which would be likely to benefit from the influence of a regional plan. If we are ever to reach the stage where our planning for streets is more than a matter of "putting out fires," the long-range kind of plan becomes essential. These long-range plans require use of all the vision and knowledge that can be assembled. Incidentally, by long range I do not necessarily mean just 1957 or 1980.

I believe one of the most beneficial steps that state highway departments can take in contributing to the development of better solutions to urban traffic problems is to open up the availability of their trained staffs to participate directly in community planning studies. I see no reason why the Bureau of Public Roads should not support the expenditure of highway planning funds for this type of program.

The traffic "mess" in cities has been the subject of countless literary efforts, both technical and popular, including federally-sponsored documents. We need to get together on a continuing working relationship basis in each community if we are ever to reach the time when complete working solutions will replace literary commentary. Support for the concept of stronger liaison between highway departments and cities, in a general way, is beginning to become more evident at the national level.

It is now time to quit talking about this good idea and obtain active participation by state and federal experts in this type of specific program at the community level.

DISCUSSION

Mr. Palmer: Can counties secure striping service from the state?

Mr. Williams: We have done considerable of that for cities and counties at their expense, at actual cost. However, we must do that service at such times as our crews are in the vicinity, and it might be a month or two after you wanted it before the crew would be there. We have sufficient striping capacity that we can do a limited amount. In fact, I believe we have many requests now for striping, either on city streets or county roads, at the expense of the county or city.

Mr. Keeley: What formulas do we have for distribution of FAS funds? Does the State Highway Department have anything to do with O&C funds?

Mr. Williams: None whatsoever.

Mr. Keeley: Is the same formula used in Washington on FAS funds that is used here in Oregon?

Mr. Dinsmore: For the counties we have a lesser matching ratio. Ours is about 53.8 percent, I think, whereas you have about 60 percent in Oregon. In the manner of participation, however, we figure about 50-50, because by the time you get down to nonreimburseable items, that's about what it comes out.

Mr. Williams: I might explain very briefly about the O&C funds. These funds go directly into the treasury of the Federal Government, whether they are O&C funds, or whether they are forest highway funds, with the exception of a certain amount that automatically goes to the counties. The road monies are then proportioned by the Federal Government as part of the forest highway system. None of these funds is earmarked. They go into the general fund and Congress then appropriates a certain amount of money known as "forest highway funds." In this the state does have a one-third say, along with the Forest Service and the Bureau of Public Roads, as to where the funds will be expended. You have no control of the O&C funds that go directly to the county.

Mr. McClarty: Where can the county turn if it would like an engineer to come in and evaluate the road department; that is, evaluate the amount of equipment they have, whether it is proper equipment, and whether the organization is the proper size for the county, etc? Where can they get that consulting service?

Mr. Williams: The State Highway Department would be very reluctant to enter into anything like that because we would possibly be interfering with the prerogative of the governing body of the county. We are not dictators, but we might be referees if they asked us if something was right, or which was best. Insofar as the Oregon department is concerned, we would be very reluctant to go in and start tearing apart any county organization.

Mr. Dinsmore: We have in many instances advised counties at their request upon matters of accounting in connection with equipment and setting up equipment funds or equipment ownerships, so they can get rentals properly distributed to certain projects. We have furnished that service to many of the counties in the State of Washington that have adopted a system similar to what we have.

Mr. Westling: Does anybody know if the Automotive Safety Foundation, for example, ever made that kind of survey—whether they might be a source for that sort of evaluation? Of course, the Public Administration Service in Chicago quite often does that sort of thing for governmental organizations, but I don't know about doing this for a road department.

Comment: A recent study in Washington was made by the Institute of Technology for the distribution of the gas tax among counties, although it was never published. The man who made the study had the courage to write a report on each county, which he gave to the county commissioners and county engineers, in which he evaluated the county in those respects. It was something they did not publish, of course.

Comment: Those counties that got a favorable report, put it in the papers; those who did not, put them in file X.

Mr. Williams: I do not know of any instances where the Automotive Safety Foundation has gone into the county level, but I believe we have a gentleman here that could answer that question better than we can; Mr. C. F. Rogers, special assistant research, from the Washington office of the Bureau of Public Roads.

Mr. Rogers: In the literature of the Highway Research Board, you would find a pattern derived from various and sundry counties which might produce some kind of a norm in which a county could appraise its own organization. I doubt that the service is given free. You would have to pay for it. There are a lot of private management concerns who will make studies like this, but they don't come cheap.

Mr. Goodnight: I was very interested in Mark Westling's remarks on planning. My thinking is that we go into arterial or highway development in fringe areas and get plans for it and acquire rights-of-way in advance of the deal. Is that what you were shooting at, Mark?

Mr. Westling: Yes. I think the big problem you have in urban areas is that by the time you get around to deciding you have to build the thing, the right-of-way is not there anymore—it is full of buildings and houses. Therefore, you need to save the right-of-way even though you are not going to build there for a long time. This is one of the long-range planning problems which involves traffic engineering and traffic design. This means the planners end up by having to become traffic engineers in order to do their planning jobs, instead of being able to get the traffic engineers to work with them; to do the traffic engineering job with them.

Mr. Goodnight: How many years in advance of this anticipated construction should you acquire the right-of-way?

Mr. Westling: Twenty or thirty years ago, Multnomah County set up a policy that every mile (or something like that) was going to be a major route. This was their long-range plan. So when the land started being subdivided, they worked it out that way. Now we recognize that from the traffic volumes we are getting, this is inadequate. We need to superimpose upon that something more in the way of a major freeway or expressway design or system, and the need to project that into the future, as well, is involved. This is not anything new. It is only trying to do what has been done in the past, geared to the volumes that we now have to carry, instead of geared to what somebody was thinking about thirty years ago.

Mr. Goodnight: Don't you think we could get somebody to help us look into the future?

Mr. Williams: That is the Utopia which we all hope to reach someday. I think we all realize that you cannot buy rights-of-way ten or fifteen years hence, or we would do nothing but buy rights-of-way, we would build no roads. The Federal Government limits the time to buy rights-of-way to five years. They were supposed to change it to seven years, but I understand it is back to five years in the recodification. So we are limited—we cannot buy rights-of-way on any Federal system more than five years in advance, and five years is about as far as you can tie up your money. I certainly will admit that Mr. Westling has a good point as far as rights-of-way costing more and more each year. Still, there must be a dividing line somewhere.

Mr. McClarty: Have Washington counties been doing some work in trying to interest or keep engineers in the county fields?

Mr. West: We started that program whereby each year each county would take a few engineering graduates and put them to work in the counties. The idea was that we tried to bring them up a little each year, or possibly rotate them. That was when this so-called shortage of engineers was at its peak. Right now, as far as I am concerned, and in most of my neighboring counties, we are not short of engineers. However, many of us have made a practice of retaining those boys year after year, and then they eventually grow up and go to work for the State Highway Department.

DEVELOPMENTS IN HEAVY DUTY ASPHALT PAVEMENT DESIGN AND CONSTRUCTION

Vaughn Smith

My comments will be concentrated upon several aspects of pavement engineering that should be given more intensive consideration in the design and construction of asphalt pavements for heavy duty highways, such as the interstate system or our other modern freeways. These thoughts are offered in light of developments over the years in asphalt paving technology. This discussion will be limited to structural design matters. It will not include such aspects as alignment, traffic handling capacity, interchange design, or the like.

More specifically, the points that will be stressed are:

1. Importance of quality of base-course materials.

2. Advantages of asphalt concrete surfaces of greater thickness than have customarily been used in the western states.

3. Most suitable grades of asphalt binder for heavy duty pavements.

4. Importance of framing design specifications to require construction materials and end results that will assure first class pavement performance, and will essentially eliminate maintenance costs.

The general philosophy of pavement design in the western states until recently has been to build the maximum possible area of reasonably suitable hard-surfaced roads with the funds available. This design philosophy provided us with a system of hard-surfaced roads that otherwise would have been impossible to acquire in view of the large highway mileage needed and the limited highway funds available in this area of low population density.

Stage construction practices were an outgrowth of the approach, which has been so effective in stretching highway dollars in the West. Indeed, many mistakes in judging thickness requirements or in underestimating future traffic intensity have literally "been buried" with an overlay of asphalt mix. When considered in this light, asphalt truly has been a friend to our western states. None of us will argue with this approach to fulfilling our pavement needs in view of the results achieved. However, this philosophy was based upon low traffic intensities and a willingness to compromise on pavement performance (quality) to obtain more miles of paved road (quantity).

In the design of superhighways such as the interstate system, we are confronted with a somewhat different set of circumstances, and our engineering approach should change accordingly. These heavy duty arteries are not just roads; they are highways from the start! Traffic intensity is not an unknown quantity. It will be heavy from the beginning, and is bound to increase further in the coming years. Moreover, the method of financing the interstate system greatly increases the incentives for the states to build maintenance-free roads. Thus, our objective should be to design and construct highways that will serve heavy vehicles and large traffic volumes without these highways developing any structural distress that will result in traffic disruptions or costly repairs. Although we can depend upon stage construction as a last resort (where future traffic intensities are underestimated), we should not rely upon stage construction in the original design of a modern superhighway.

While considering the changes in engineering approach made necessary by our modern problems of highway planning, I should like to stress one engineering tenet that is just as true today as ever. It is an engineer's responsibility to build these structures in the most economical manner!

I dare say each of you was told early in your training that, "An engineer is expected to accomplish with one dollar what anyone else can accomplish with a dollar and a half. That is why we have engineers." This principle bears repeating to ourselves now and then. There seems to be a great tendency to forget this basic precept in this seemingly "era of plenty." What appears to be an easy money situation in no way excuses us from using our best engineering tools and judgment to make the most productive use of every available dollar.

Nevertheless, it is apparent that an "easy money" philosophy is warping the judgment of engineers in some organizations. We are finding already that financing of the interstate system of highways will not be nearly as easy as was first anticipated by many. Obtaining adequate funds to finance this superhighway system has become a major problem at both the federal and state levels. The fact is, we do not have an "easy money" situation at the present, and probably never will!

With the foregoing thoughts in mind, I would like to concentrate on the four points mentioned previously.

Importance of Quality of Base-Course Materials

Standards followed at the present time for asphalt pavement thickness design in the Pacific Coast states recognize the importance of providing sufficient foundation thickness to spread the surface loads over the basement soil.

In late 1957, California altered its thickness design method to require substantial increases in thickness for pavements carrying intense traffic. This change served to correct an underestimate of traffic intensity and wheel loads that could not be visualized even as recently as 1951, when the previous thickness design method was adopted in California. With this recent change to greater thicknesses in pavements in California, it appears that structural thicknesses employed in all the Pacific Coast states are adequate to eliminate shear failure of basement soils.

A point that deserves greater stress, however, is the need for much closer attention to quality of materials used for bases and in subbases. Poor quality base materials have been a common cause of pavement distress in the West. For interstate highways or similar heavy duty pavements, this matter deserves first order attention.

Figure 1 depicts a typical pavement cross section. In many instances the majority of attention has been directed to spreading the surface loads over the basement soil (component A), and less consideration given to the quality of materials necessary in components B and C to prevent shear failure (or plastic deformation) in these layers. Shear stresses are high immediately beneath an asphaltic surface, and high shear resistance materials are essential. Moreover, the repetitive déformation and flexing that is harmful to all pavement surfaces also has detrimental effects on the granular base materials, particularly those lying immediately beneath the paved surface.

Cases are still found where "fines" are being added to clean aggregates to impart cohesion and beam strength to base-course layers. More often than not these fines are clayey in nature. They yield a hard, smooth base immediately following construction, but eventually result in a marked reduction in shear strength of an otherwise excellent base-course material. As water accumulates in a clay-bound gravel, the wet clay serves to lubricate the mixture. The initial high degree of cohesion provided by the clay is almost completely lost, and in its place we have a very low shear strength material that causes eventual pavement distress.

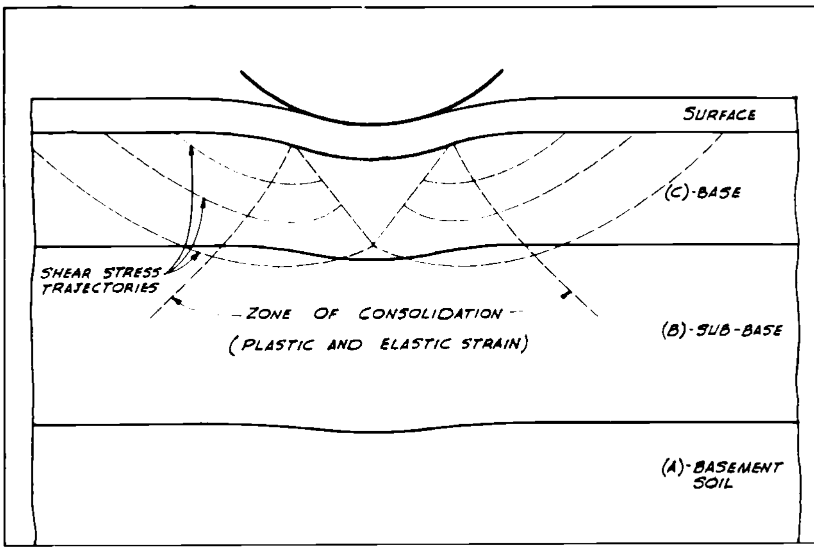

Figure 1. Stress Distribution in Asphalt Pavement

Both experience and structural theory point to the importance of increased beam strength in each of the foundation courses of an asphalt pavement, but this beam strength must be permanent if it is to be relied upon. Beam strength is derived from "keying" action of coarse aggregate particles and from "cementing action" of other materials used to hold the coarse aggregate particles in their initial state of compaction. Clay or soil binders are not suitable cementing substances because they lose their cohesiveness when they take on moisture, which they almost surely will do when placed under a relatively impervious asphaltic surface.

This leads us then to a cementing medium that will not be adversely influenced by moisture. We have two practical choices: asphaltic binders and Portland cement. Both have been used extensively and both have proven their ability to resist the adverse effects that moisture has upon soil binders or clayey cementing materials. Asphaltic binders, however, have characteristics that make them much better suited to achieving a maintenance-free structure.

Dusting of the topmost section of cement-treated bases has been a common type of base-course failure. This powdering or pulverizing action serves to break the bond between surfacing and base, and thus reduces the ability of the surfacing to resist horizontal as well as vertical stresses. This low strength pulverized layer often provides a reservoir for entrapping water because it is bounded on top and bottom by less permeable materials. This problem has been minimized in recent years by employing low fines content aggregates in the cement-treated base, and by guarding against contamination by roadbed dirt during mixing. Probably you have noted the trend toward central plant processing of base-course materials to improve quality and uniformity in base construction.

Use of the present type of cement-treated bases with asphaltic surfaces will always be sources of pavement performance problems because these materials differ radically in physical characteristics, particularly brittleness. "Pattern cracking" of cement-treated bases is common. Invariably the cracks are "reflected" through the asphalt surface. Use of Portland cement in small quantities to correct any plasticity characteristics exhibited by the mineral fines in a base material is undoubtedly worthwhile. However, when sufficient cement is used to give a rigid slab of low tensile strength and high degree of brittleness, modern traffic is bound to crack it.

These remarks probably make it apparent that I am not a proponent of the presently-used type of cement-treated base. It is believed, however, that asphalt base courses will become more important in the future. Surfaces and bases having stress-strain characteristics similar to one another and similar to the underlying foundation materials should overcome problems encountered in combining a resilient, somewhat plastic material (asphalt concrete) with a nonresilient, brittle material (cement-treated base) where the two are being subjected to continual deformation and flexing.

Value of Thicker Asphalt Concrete Surfacings

You are undoubtedly familiar with the conclusion reached on the WASHO road test experiment that a 4-inch asphalt concrete surface is outstandingly superior to a 2-inch surface. This finding confirms many previous observations not as well documented or as carefully compared as the paving sections on the WASHO road. In view of the relatively short time involved in conducting the WASHO test, and because total vehicle passes had to be limited to a reasonable number, it would be unwise to conclude that gross reductions in overall structural thickness could be made by using a 4-inch rather than a 2-inch asphalt concrete surface. But, certainly, we can conclude with confidence that pavements with thicker surfaces perform far superiorly to those with thinner surfaces. This is exceptionally important because, (1) it points the way to maintenance-free, yet economical pavements for the interstate system and other heavy duty roads, and (2) it upsets many long standing beliefs that thin surfaces and mixes made with softer asphalts, being more flexible, should exhibit greater resistance to flexural fatigue.

Further evidence that thicker asphalt surfaces do not result in poorer resistance to flexural fatigue is presented in Table 1. This table lists some of the asphalt pavements that have served virtually maintenance-free for literally generations. Table 2 summarizes the asphalt surface thicknesses employed on our present day turnpikes where cracking would soon become evident if these asphaltic surfaces were not satisfactorily resistant to flexural fatigue.

The demonstrated superiority of relatively thick asphalt surfaces results from:

1. Increased beam strength which is not lost, as is often the case with rigid pavements, due to cracking because of slab warping and brittleness.

2. Ability to conform to slow differential settlements without tensile cracking or otherwise losing integrity.

Theory tells us that thinner surfaces should be more flexible and more resistant to fatigue under flexure of a given amplitude. From the observed superiority of thicker surfaces, we should expect to find that they deform to a much lesser degree than thin surfaces. Yet, deflection measurements have failed to confirm that deflection amplitude is reduced greatly by increasing asphalt surface thickness. As a matter of fact, total deflections of 4-inch thick surfaces under a given load often have been found as great, or nearly so, as total deflection of thinner surfaces having identical total structural thickness and foundation support.

Studies on deflection and flexural fatigue have been directed mainly to total vertical deflection rather than to radius of curvature of the loaded element. Obviously, much more research is needed to develop a better understanding of flexural fatigue in bituminous pavements, but the key to the superior performance of thicker surfaces seems to be related to the radius of curvature, or abruptness of bend, of the pavement surface under loads imposed through pneumatic tires. This is illustrated, for example, by some of the WASHO road test data presented in Figure 2. "Index ratio" term is ratio of total vertical deflection to "radius" of the depressed area. This

Table 1.
Several "Maintenance-Free" Asphalt Pavements

Present location	Year constructed	Total thickness of asphalt surface - inches	Age before maintenance required
Visalia, Calif. - Main St.	1894	6 - 7	Emulsion seal applied after 48 years' service
Los Angeles, Calif. - Wash. St.	1905	8-1/2	Resurfaced after 52 years' service
" - Hope St.	1908	8-1/2	Resurfacing needed after 50 years' service
" - Sixth St.	1909	8-1/2	Resurfaced after 48 years' service
Porterville, Calif. - Main St.	1910	7	Resurfaced after 45 years' service
Bakersfield, Calif. - Eye St.	1913	6	Thin overlay at recent time

Table 2.
Asphalt Surface Thickness of Typical Heavy Duty Pavements

Highway		Total thickness of structure-inches	Total thickness of asphalt surface-inches
Richmond-Petersburg Turnpike	- 35 miles	14	9-1/2
Garden State Parkway	- 143 miles	13-1/2 - 15-1/2	7-1/2
New Hampshire Turnpike	- 15 miles	31-43	7+
New Jersey Turnpike	- 118 miles	18-1/2	6-1/2
Massachusetts Turnpike	- 123 miles	22	5-1/2
Merritt Parkway relocation	-	14-1/2 - 20-1/2	5-1/2
Turner Turnpike	- 86 miles	24	5
Kansas Turnpike	- 181 miles	22	4
Sunshine State Parkway	- 110 miles	10-1/2+*	2-1/2**

* Does not include top layer of basement soil which is "stabilized."
**Eight-inch "limerock" base has appreciable stiffness.

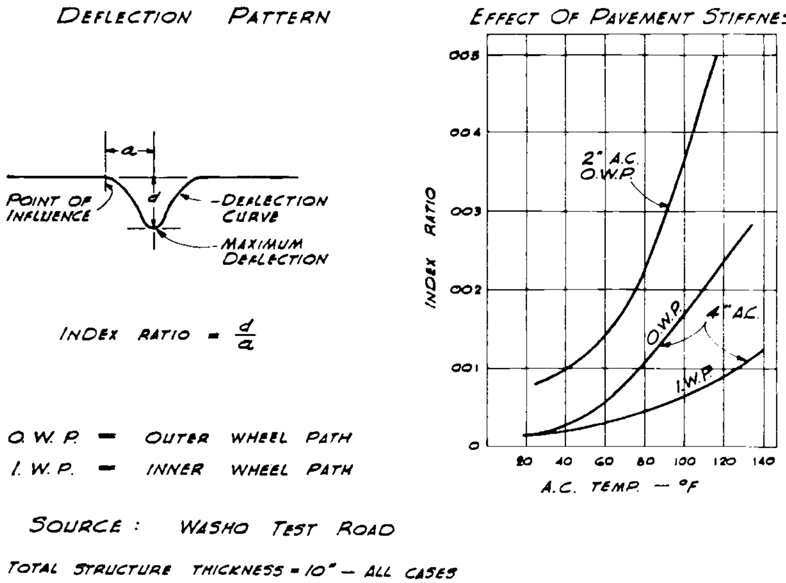

Figure 2. Deflection Characteristics of Asphalt Pavements

term is proportional to radius of curvature of the deformed section of the pavement; a low "index ratio" signifying a large radius of curvature and a high "index ratio" signifying a small radius of curvature or more abrupt bend.

The influence of temperature on beam strength of these pavement surfaces is quite evident, but more significant is the fact that the 4-inch surface exhibits substantially greater beam strength than the 2-inch surface over the entire range of temperatures experienced. Without more extensive data on the profile of the deflected surfaces, it is not possible to determine whether the outer fiber of the 4-inch surface beam is under any greater stress than the outer fiber of the 2-inch surface beam. However, this must not be the case or more cracking should have been observed with the 4-inch surface than with the 2-inch surface.

Stress distribution in a bituminous pavement structure follows the general pattern depicted in Figure 1. The data presented in Figure 2 serve to confirm the importance of the "confining effect" or beam action of the bituminous surfacing. Note the sizeable reduction of "index ratio" in the inner wheel path as compared to the outer wheel path measurements. The outer wheel path load-deflection measurements were made at a distance of approximately 1-1/2 feet from the edge of the pavement, whereas the inner wheel path measurement was approximately 9 feet from the pavement's edge. The confining effect of the 4-inch pavement surrounding the inner wheel path point of loading brought about a sizeable reduction in abruptness of bend in the loaded area, except at near freezing temperatures where pavements had such high beam strength in all cases that there was very little elastic strain at any point near the loaded area.

Findings such as these have highlighted the desirability of paving highway shoulders to increase strength of the traveled way, as well as to protect foundation layers from the harmful effects of surface water.

At the beginning of this discussion on the value of thicker asphalt surfacings, it was contended that the ability of bituminous surfacings to conform to slow differential settlements without appreciable alteration in properties was a major factor accounting for the superior performance of such pavements. We have yet to build a pavement foundation that will not undergo differential settlement during its lifetime. Excellent progress has been made in improving compaction methods, and we know how to provide a uniformly good foundation support for pavement surfaces of any type, but it is a safe bet that complete elimination of differential settlement will forever be beyond our reach. A pavement surface that warps itself away from its underlying support, or one that cannot conform to differential settlement without cracking, cannot support loads of which it was initially capable, nor can it exert a uniform confining effect on base materials to keep them from becoming decompacted, abraded, or otherwise altered.

It has been amply demonstrated that bituminous pavements will conform to gradual differential settlements without loss of their integrity. Moreover, thick asphalt surfacings (7 inches or more) will conform to normal settlements just as readily as do thinner bituminous surfaces. Thus, beam strength and base confinement offered by thick asphalt surfaces are never lost, but always contribute substantially to the strength of pavement structure. However, it is possible at any time to increase these important strength characteristics by the simple expedient of a bituminous overlay. An overlay also can serve to correct undulations that may have developed due to settlement.

In view of experience and results of field experiments made to date, asphalt concrete surfaces for heavy duty roads should never be thinner than 4 inches, and thicknesses of 6 or 7 inches should be chosen if pavement quality is being sought on a "no compromise" basis.

Asphalt Grades for Heavy Duty Pavements

In some areas of the West it has been customary to use the softer grades of paving asphalts, such as the 200/300 penetration grade. This custom is based on the premise that the softer the binder is to begin with, the longer it will last before reaching a point of hardness where pavement cracking or raveling due to embrittlement will result.

I would like to propose to you that 85/100 penetration grade is a better choice for use in heavy duty asphalt pavements than is the 200/300 penetration grade. Reasons for this proposal are:

1. Harder asphalts yield a bituminous mix with greater cohesion. Cohesion contributes substantially to beam strength in the pavement.

2. Due to higher viscosity of harder asphalts, slightly greater binder contents can be used without sacrificing stability. Pavement durability is closely related to binder content.

3. Harder asphalts give better adhesion to mineral aggregate. That is, they provide better resistance to water stripping.

4. Due to their higher viscosity and higher asphaltene content, harder asphalts give more rapid "set" during construction.

5. The new uniform specifications on paving asphalts recently adopted on the Pacific Coast assure products having greater resistance to embrittlement than many asphalts previously marketed in this area.

I note that this subject is to be covered in more detail in Mr. Vallerga's presentation tomorrow. I am sure that his discussion of field experience will point to the need for asphalts harder than the 200/300 penetration grade for heavy duty pavements. The harder grades provide initial toughness and abrasion resistance needed to withstand the type of traffic encountered on such highways as the interstate system or our major freeways.

Importance of Specifications and Inspection Procedures

As a final major point which deserves close consideration if maintenance-free pavements are to be achieved, it should be stressed that a "no compromise" philosophy must be adopted in the drafting of specifications, and the same philosophy must be enforced during construction of the pavement. Two of the most obvious shortcomings of proper inspection and construction control on pavements in the western states are:

1. Inadequate and nonuniform compaction of foundation components (base courses and basement soil).

2. Wide variations in dust (mineral filler) content of paving mixes. (In some instances wide variations are still found over the entire aggregate gradation range.)

Variations in the degree of densification of foundation layers soon show up as variations in settlement in service. This differential settlement makes for a sharply undulating surface which greatly impairs riding qualities at high speeds. With compaction equipment now in use, and with construction control methods already available, there is little excuse for the variations in compaction that show up on some of our modern highways.

Wide variations from the intended dust content of bituminous mixes make for "fat" or "lean" surfaces. When the dust content drops far low without adequate compensation in the way of reduced asphalt content, an overly rich surface sometimes results. This richness impairs both non-skid properties and stability (shear resistance) of the surface.

When dust content is allowed to increase substantially above the "design" percentage, a lean surface is the result. Leanness is undesirable because it makes for a more brittle or less resilient surface, and one that has reduced resistance to oxidation and other aging influences.

These remarks might appear to indicate that it is difficult to control dust content and aggregate gradation so that "leanness" or "richness" are not encountered. Such is not the case! Modern paving plants and practices are easily capable of controlling aggregate gradation and fines content within limits that closely approach the design mix, and with a high degree of uniformity from batch to batch and day to day.

Why, then, aren't we achieving this uniformity in compaction and in mix composition on some projects? I am positive that this is the direct result of leaving <u>too much to personal opinion and too little</u>

to engineering measurement in our inspection and construction control procedures.

New highways are much like newborn babies—to those responsible for them, they all look wonderful at first! Sooner or later, however, they begin to take on a character notably resembling "what they are made of." Poor construction and improper quality control always show up in time.

Elimination of undesirable quality variations in paving mixes and in granular foundation materials is really a straightforward matter. It simple requires, (1) that density and composition specifications be established for the project, (2) that the inspector take frequent samples and test them with accurate testing equipment, and (3) that the project engineer or other authorities in charge give the inspector full backing with respect to approving or disapproving the work at it proceeds.

To accomplish this we must recognize that inspection costs will be higher than have generally been incurred on paving projects. In the end, however, money will be saved through the quality of construction achieved.

Having engineering measurements take the place of personal opinion should be to the liking and benefit of the contracting fraternity, also. With written physical standards to work toward, a contractor can achieve the end results by methods of his own choice. Moreover, his operations need not be upset through whims, opinions, or personal judgments of the inspectors or project engineers.

"End result specifications" have been discussed at several recent meetings of road building people. For example, I refer you to the meeting held in San Francisco in December 1958. End result specifications were strongly endorsed by the AGC members participating in the AASHO-AGC joint cooperative committee. End results are so important to this matter of heavy duty pavement design that a few brief comments on end result specifications would seem in order in connection with my subject. What end results are we seeking? This is something we can answer quite simply. We are looking for:

1. A structure that complies with design grades and cross sections.

2. A smooth riding surface (and one that remains smooth).

3. A structurally stable pavement.

4. A durable pavement structure.

Also, in the interest of the public and of the contractor who bid the job and lost, the engineer is morally obligated to ascertain whether the qualities and quantities of materials specified were used on the project.

All of the foregoing objectives of our present day pavement construction specifications can be measured quite simply; some by direct and some by indirect methods. There seems to be little need or justification for specifications which outline for the contractor "how to do" each construction step. Moreover, there is absolutely no justification for leaving the matter of specification compliance to the judgment or personal opinion of the job engineer, inspector, or someone else, when methods are available for measuring all end results of importance.

Initial smoothness, thickness of each component, and compliance with design grades and cross sections can be measured by elementary methods. Certainly there should be no inspection and control problems here. Personal opinions are not needed at all!

To arrive at a suitable means for measurement of the other important performance qualities listed above, consider the various ways a pavement surface can lose its original smooth riding qualities:

1. Through plastic shear deformation (inadequate stability).

2. Through differential settlement.

3. Through differential expansion (frost action or uneven swell pressures).

4. Through deterioration of the surface (e.g., raveling or cracking).

These important aspects of structural stability and durability of a pavement structure are closely related to the quality of materials employed, to the precision with which the materials were properly combined and placed (workmanship), and to the extent to which deleterious materials have been eliminated during construction.

All of the foregoing qualities and materials characteristics can be measured quite simply:

1. Field density determinations on each structural component, from bottom to top, will quickly show the extent to which

differential consolidation is likely to occur, and where it will occur. Density determinations are simple to make, but they must be made at frequent intervals as construction proceeds if they are to be of any help in achieving densities that will minimize differential consolidation and thus preserve initial smooth riding qualities.

2. Structural stability (shear resistance and compressive strength) of each pavement component can be measured directly on specimens procured immediately following construction of each component. Again, straightforward methods are available for such measurements.

3. Durability of an asphalt pavement component cannot be measured directly. Nevertheless, by uncomplicated indirect methods we can ascertain whether any portions of a contractor's work are likely to give poor service performance. This merely requires a laboratory extraction of field samples of the completed paving mix to, (1) determine whether the amount and quality of materials specified were used, (2) determine whether any deleterious components were added, or (3) whether improper construction practices unduly harmed high quality materials originally supplied to the construction forces.

This extraction and analysis can easily be made on the same sample that was previously used, both for a density determination and for a stability determination. A similar analysis can be made on unasphalted base-course or subgrade materials without requiring any sort of extraction.

The important thing I wish to emphasize is that the techniques we already have available for construction control, for insuring satisfactory pavement performance in all cases, and for protecting the general public's investment, are of no value whatever unless we follow the practice of frequent field sampling throughout every construction day, followed by immediate analyses to determine whether these field samples measure up to specification standards.

Two things are missing from many present day construction specifications:

1. Density requirements on each pavement component to give the contractor an end result to shoot at.

2. A program of field sampling for inspection and control purposes.

These are the yardsticks that both contractors and inspectors must have if we are to be assured of uniformly good construction and if the matter of specification compliance is to be decided upon a businesslike basis, rather than being left to personal opinion (which is never the same from person to person, and seldom is completely correct). With yardsticks such as these, our present typical construction specifications can be condensed to a fraction of their present length by eliminating most, if not all, the "how to" requirements.

To achieve this goal, specifications must be written around what is needed—not around materials that are most simply or most conveniently available. Practicality of achieving desired end results must be considered in design and drafting of specifications, but design and specifications should not be subordinated to the convenience of construction. Compromising specifications to permit use of materials that are less than top quality means compromising on performance of the completed pavement. Admittedly, costs will increase when quality is not compromised, but in most instances the increase will be small on a relative basis.

The usual objections to "end result" control of the type I have outlined are:

1. Inspection costs will increase.

2. High plant production and rapid earth moving capacity of modern equipment makes it difficult to get test results fast enough to control construction operations.

3. Some desirable construction steps cannot be described adequately in the form of end result requirements.

Consider these objections one at a time.

1. Increased inspection costs. It is certainly true that inspection costs will be increased—perhaps doubled or quadrupled! Indeed, this is what I am arguing in favor of! Even so, inspection costs will still comprise only a small fraction of total job cost. Increased costs will be well justified by results achieved.

2. Problems of rapid construction. To get test results in the hands of the project engineer in time to be of real assistance, there must be enough inspection help and equipment <u>on the project site</u> to run the necessary tests. The common practice of sending samples to a central laboratory is too slow in obtaining helpful

results.

In accepting end result specifications, the contractor is obligated to remove and replace any work not meeting end result specifications. In some instances, certain previously agreed to penalties might be assessed for work not fully meeting the specified standards. Undoubtedly, the contracting profession is mindful of these problems. They have recommended end result specifications irrespective of such problems.

3. Construction steps not amenable to end result specifications. It must be admitted that we do not know yet how to specify desired end results than can be achieved by certain construction steps that will benefit the final pavement. The beneficial effect achieved through rubber-tired rolling of an asphalt surface is perhaps the best example of a desirable end result for which we do not have a direct measurement. Until we can express the desired end results by numbers, construction specifications must be on a "how to do it" basis.

"How to" specifications are always built around methods used on certain successful projects. They have the undesirable effect of restricting progress on construction methods that will yield the same or better results. They restrict the development of more economical ways of achieving the same results.

The points I should like to emphasize are that very few "how to do it" items are necessary in asphalt pavement construction specifications. We are not taking full advantage of our present ability to specify desired end results! We are allowing "how to do it" specifications take the place of good inspection practices! Actual measurement is a much surer way of judging a final result than is a recipe for achieving this result.

In conclusion, I would like to stress again that with asphalt pavements it is readily possible to "design out" the factors that cause maintenance costs. Present practices in designing and constructing asphalt concrete pavements will achieve this objective provided quality of materials and quality of workmanship are not compromised. For heavy duty highways of the interstate type, asphalt concrete surfaces should be at least 4 inches in thickness, and thicknesses up to 7 inches should be provided if complete elimination of maintenance costs is the prime objective. For these heavy duty pavements, an asphalt cement of 85/100 penetration or harder is recommended, but certainly nothing softer than 120/150 penetration should be used. The harder grades will give toughness, abrasion resistance, and cohesive strength without sacrificing durability if the pavements are designed and constructed without relaxing on quality.

MINIMUM LABORATORY REQUIREMENTS FOR THE PROPER CONTROL OF MATERIALS

Herbert W. Humphres

The title assigned to this paper is indefinite in scope in that it makes no reference to the specifications that are to be enforced. To reduce the subject to an area that can be discussed properly within the time allotted, I will exercise a speaker's prerogative and restrict the subject matter to control testing during the contract phase of highway construction as required to meet State of Washington specifications. The majority of control tests that I will discuss will be those performed on the job in the field laboratory.

Why do we have control tests? To answer this we must go back to the planning and design stage of highway building. As with any engineered structure, the highway designer must utilize materials to economic advantage. He cannot do this unless he knows what strength values or other physical and chemical property values he can assign to any given material. Materials research and application experience are used to establish these values and the conditions which must be met before these values can be relied upon. Material specifications are established, usually in the form of minimum allowable values, and the designer uses these values in his design.

To ensure a successful structure, the builder must use materials which meet the requirements assumed by the designer. For example, if the designer of a bridge based his design on concrete having a compressive strength of 3600 lb/sq in. in 28 days, the builder would be flirting with trouble should he use 3000 lb concrete. Satisfactory construction depends upon the builder using materials meeting the specifications selected by the designer.

Obviously, many of the laboratory tests necessary to determine properties of materials are much too complex, time consuming, and expensive to be of practical use to the builder for quality control during construction. To fill the need of the builder, control tests have been developed which, in the majority of cases, are indicator tests rather than actual physical property tests. In other words, these tests measure some characteristics of the material which can be correlated to its physical properties. Control tests are designed to utilize a minimum of time and equipment, and are intended to furnish the builder with positive indication that a material either will or will not meet the requirements assumed by the designer.

To illustrate the function of a control or indicator test, let us consider the simple compaction control, or field density test, used for soils in highway work. The designer of the roadway section must assume a certain strength value for the subgrade soil before he can design the proper surfacing and pavement section. The laboratory tests the subgrade soil and determines that the soil will have a certain strength value if compacted to a given density. This strength value is used by the designer, and the required density is specified for control during construction. The simple field density test will indicate the strength and stability of the subgrade soil.

Minimum requirements for proper control of materials can be stated very simply. Control testing should be performed sufficient to establish with reasonable certainty that all materials used in the structure meet design specifications. Degree of specification refinement will have a direct bearing on the kind and amount of control testing necessary. One should not specify requirements beyond his ability to control. This is a very important axiom from the standpoint of economic as well as structural design. Contract prices are based on specification requirements, and we must have some method for determining that we are getting what we are paying for.

When control facilities are adequate, we can revert to the ideal "end product" type of specification for much of our work. However, where control facilities are limited, as is the case with most county and city organizations, we must rely on the "methods and procedure" type of specification. Proper consideration during the planning and design stage can eliminate the need for a large share of field control work. For example, if we know from experience that four coverages with a 10-ton pneumatic roller of a 6-ton lift of a given soil at the proper moisture will produce the desired density, then specifications can be written requiring this amount of rolling, and our control requirements will be reduced to controlling moisture content only. The combined control through specifications and field control procedures should insure that design requirements are fulfilled.

The following outline covers the minimum control testing requirements necessary for constructing highways under State of Washington specifications. For clarity, control requirements are listed for each of the major divisions of work involved; i.e., grading, aggregate production, and paving. Purpose and application of each test is given, but details of procedure are omitted as these are available elsewhere. Test methods used by other agencies are mentioned in some instances, but this coverage is not to be considered complete nor to imply specific preference of one method over another.

Control Testing Requirements

<u>During Grading Operations</u>

1. Compaction control (establishing required density).
   a) Fine-grained soils (less than 30% retained on 1/4-inch sieve).

      Standard moisture-density test - ASTM D-698. This test is used to establish maximum density required and optimum moisture condition. Modifications of this test, which yield higher density values, may be used.

   b) Coarse-grained soils (more than 30% retained on 1/4-inch sieve).
      1) Washington establishes a density versus gradation curve in the central laboratory, using the vibrated spring load test (HRB Bulletin No. 159). This allows establishing required density prior to construction, and eliminates need for standard tests in field. Has proven satisfactory for granular soils having maximum-sized particles up to 3 inches in diameter.

      2) Other agencies use such tests as the California impact test, modification of ASTM D-698, or special vibratory compaction tests. A wide variation of procedures exist, and most are difficult to perform with reasonable speed and accuracy in the field.

2. Field density control.
   a) Fine-grained soils.
      1) Washington uses the Washington densometer method (HRB Bulletin No. 93, and ASTM suggested method, "Procedures for Testing Soils," April 1958). This method has proven very satisfactory for all soils and test hole sizes up to 1/2 cubic foot in volume. Very rapid and accurate relative to other methods.

2) Other rubber balloon apparatus is available which is restricted to small test holes up to about 1/20 cubic foot.

3) Small sand cone method is satisfactory for hole sizes up to 1/30 cubic foot.

4) Other: drive cylinders, heavy oil, etc.

b) Coarse-grained soils.

1) Washington densometer (see above).

2) Large sand cone method is principal other apparatus capable of measuring hole sizes suitable for coarse material. This method is cumbersome and relatively slow.

3. Moisture control.

a) Alcohol burning method (Washington test method). Satisfactory for field use with all soils except fat clay. Very rapid, and has accuracy of $\pm$ 1/2%.

b) Hot plate or stove method. Satisfactory for all soils, but care must be taken to avoid overheating. Slower and more cumbersome than alcohol method.

c) Other: pycnometer, carbide-pressure, etc. These methods should be evaluated for the intended use.

<u>During Aggregate Production Operations</u>

1. Cement concrete aggregates.

a) Gradation control. Use standard screens and sieves.

b) Cleanliness control. Test consists of washing material through a No. 200 wire mesh sieve in accordance with ASTM C-117.

c) Organic matter content control. Use colorimetric test in accordance with ASTM C-40.

d) Soft rock and foreign matter control. Use visual inspection. If material is suspected, a laboratory check of specific gravity will be necessary.

2. Mineral aggregate for bituminous concrete.

a) Gradation control (see above).

b) Fracture control. Visual count of fractured particles in each specified size above No. 10 wire mesh sieve.

c) Cleanliness control.
    1) Liquid limit test - ASTM D-423. (Not suited for field use.)

    2) Plastic index test - ASTM D-424. (Not suited for field use.)

    3) Sand equivalent tests. ASTM suggested test method, "Procedures for Testing Soils," April 1958. This is an excellent rapid field indicator test for the presence of plastic fines.

d) Soft rock and foreign matter control (see above).

3. Surfacing aggregate control.
    a) Require same control as outlined for mineral aggregate (see above).

    b) If produced for use as cement-treated base material, organic content must be controlled (see 1-c above).

<u>During Paving Operations</u>

1. Cement concrete pavement.
    a) Proportioning of mix. (Depends on method.) Pycnometer test used for rapid determination of percent moisture in raw aggregate.

    b) Entrained air in fresh concrete. Use commercial air meter.

    c) Consistency control. Slump test using standard slump cone test ASTM C-143, or Kelly ball test.

    d) Strength control.
        1) Flexural strength test ASTM C-78. This test is a simple field test in which a field-cast and cured concrete beam is broken. Test serves as a basis for acceptance rather than for actual production control.

        2) Compressive strength test ASTM C-39. This test is usually performed on field-cast cylinders in a central laboratory. Test results are used as a basis for acceptance and quality record rather than for actual production control.

2. Bituminous concrete pavement.
    a) Control of temperatures of asphalt and mix. Use armored thermometer.

    b) Control of asphalt cement content.
        1) Use field extraction test (Washington test method),

        2) Centrifuge extraction, re-flux extraction, etc.

    c) Control of aggregate gradation. Use standard screens and sieves.

    d) Mix design and control of mix.
        1) California stabilometer and cohesiometer test.

        2) Marshal test, Hubbard field test, centrifuge kerosene equivalent test. All of these tests are used for design and as a basis for acceptance rather than for actual production control. With possible exception of the last two, these tests are not suitable for field use.

3. Cement-treated base construction.
    a) Compaction control (see 1-b and 2-b above).

    b) Gradation control. Use standard screens and sieves.

    c) Cement content control.
        1) Electrical conductivity test (Washington test method.

        2) Titration test (California test method).

    d) Strength control. Compressive test on cylinders fabricated in field using California CTB cylinder fabrication equipment. Test results are used as a basis for acceptance and for quality record.

ASPHALT STABILIZED BASES FOR LIGHT AND HEAVY DUTY PAVEMENTS

Carl S. Larson

Asphalt stabilization of bases, in one form or another has been used by highway engineers for many, many years. Countless articles have been prepared and papers presented and published on various types and methods of base stabilization with asphalt, yet this seems to be one of the least understood fields in highway engineering today.

One of the main reasons for this is that during 1920 and 1930, emphasis was directed to the pavement surface as the answer to highway needs; a so-called "permanent pavement," which was to solve all problems. Failures occurred which too often were blamed on excess traffic, deficiency in pavement surfaces, etc. In some cases this may have been true, but the basic fact remains that a road surface is no better than its foundations, which consist of bases, subbases, subgrades, or anything that supports the pavement itself.

Another reason might be due to the numerous types of asphalt stabilized bases that are available. As an asphalt pavement surface may be many things, so is it true with asphalt bases. They may consist of combinations of materials ranging from in-place soils to closely controlled manufactured aggregates combined with an asphaltic binder, which may be selected from about a dozen different grades, then mixed, and placed in about as many different ways.

Treating of bases with asphalt began over fifty years ago when heavy petroleum crudes were sprayed directly on the earth to produce an improved condition. Some of the earliest experiments were conducted in the Pacific Coast states. Many miles were thus improved in the next few years, but results were quite varied due to little control over type, quality, or quantity of aggregate and oil used. Also, exact methods of soil appraisal were all but unknown. As I mentioned before, more attention was beginning to be directed to the so-called higher types of paving with untreated aggregate bases. Consequently, asphalt base potentialities were somewhat delayed until recent years.

Two points stressed in Mr. Vaughn Smith's presentation were: (1) importance of quality of base course materials, and (2) advantage of asphalt concrete surfaces of greater thickness than have been customarily used in the western states. My discussion will deal with how to achieve these results with the use of asphalt as the binder for base materials.

Surface Versus Base Thickness

The first question that comes to mind is, "What is the difference between increasing the asphalt surface course thickness and using an asphalt base course?" The answer would have to be that there is little or no difference. My reasoning is that asphalt bases may be considered to be a part or portion of an asphalt pavement. One way to look at this is to consider that a pavement surface is that portion which comes into contact with a vehicle. In other words, anything that supports this surface could be designated as a base. Generally though, in highway engineering terms, the top course of a pavement section is designated as the surface or wearing course, and the underlying courses as bases. By this reasoning, advocating thicker asphalt concrete surfacings and increasing the use of asphalt bases mean one and the same thing. Use of asphalt bases is only the means of achieving this result.

There are a number of reasons why asphalt bases should be preferred over specifying a thicker surfacing using the same materials and proportions normally used in a surface course. Some of these are:

1. Advantage of using larger maximum-sized aggregates that are more economical to produce.

2. Advantage of utilizing soils and aggregates which, without a cementing agent for stabilization, would not be suitable.

3. Gradation limits need not be as exacting as for surface courses.

4. Advantage of many available construction methods adaptable for asphalt stabilization. (In most cases equipment used for asphalt surfacing can be the same for asphalt bases.)

Extent of Use

Asphalt base stabilization projects are again increasing at a rapid rate. This is shown by the many projects that have been completed the past few years in various states. There also are numerous projects under construction, in the design stage, or planned for the immediate future. Let us examine just a few of these to determine what the national trend is and what it might indicate as to future design and construction. (See Figure 1.)

1. Oregon awarded a contract in 1958 on Highway 101 between Bullards bridge and Bandon. The present contract calls for 1-1/4 inch, Type 0-11, oil mat surfacing over a 6-inch asphalt

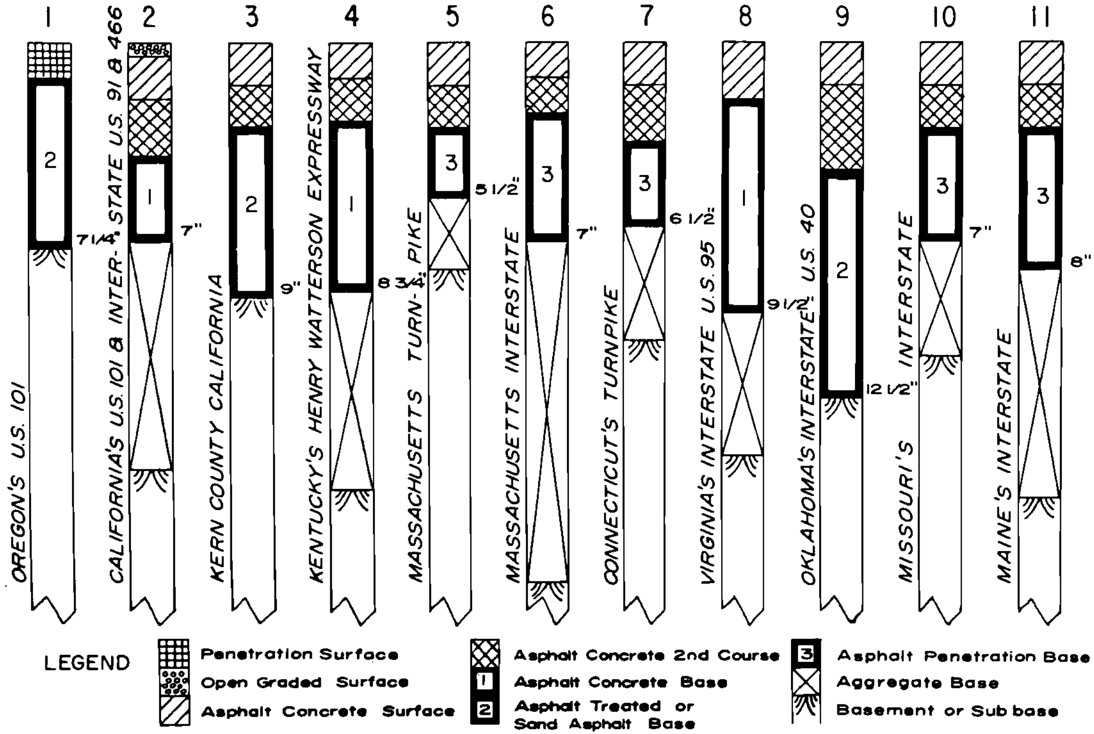

Figure 1. Various Structural Sections of Asphalt Concrete Base Designs

treated base, which will be plant-mixed using local natural aggregates, and 200-300 penetration paving grade asphalt. Plans indicate an additional 3-1/2 inch asphalt concrete surfacing to be placed as traffic warrants. Total asphalt thickness in the first stage is 7-1/4 inches, and future, 10-3/4 inches.

2. California has announced plans for two projects for 1959. One is on U.S. 101 near Soledad, and the other is on Interstate U.S. 91-466 from Baker to Valley Wells. This latter project will be about 25 miles of 4-lane divided highway. The same general design will be used on both projects and will consist of:

a) One-half inch open-graded plant-mixed surfacing
b) One and one-half inch asphalt concrete surface
c) Two-inch asphalt concrete leveling course
d) Three-inch asphalt concrete base
e) Eight-inch untreated rock base plus imported subbase materials as required
f) Total asphalt thickness 7 inches

Advertising of these projects is expected sometime in March of 1959.

3. Kern County, California, completed an experimental FAS project in May 1957 on Bear Mountain Boulevard south of Bakersfield, consisting of:

a) One and one-half inch asphalt concrete surface
b) One and one-half inch concrete leveling course
c) Six-inch sand-asphalt base using road-mixed methods with liquid asphalts and asphaltic emulsion
d) Six- to 12-inch selected subbase
e) Total asphalt thickness 9 inches

Original report prepared by The Asphalt Institute, dated July 29, 1957, is available. A 2-year report is to be published soon.

4. Kentucky's Henry Watterson Expressway has:

a) One and one-quarter inch asphalt concrete surface
b) One and one-half inch asphalt concrete binder
c) Six-inch asphalt concrete base
d) Seven-inch waterbound macadam subbase
e) Total asphalt thickness 8-3/4 inches

Kentucky's interstate design is the same except that the binder course is 1-3/4 inches, making a total asphalt thickness of 9 inches.

5. Massachusetts' Turnpike is:

   a) One and one-half inch asphalt concrete surface
   b) One and one-half inch asphalt concrete base
   c) Two and one-half inch asphalt penetration macadam base
   d) Two and one-half inch drybound macadam base plus frost-free gravel subbase
   e) Total asphalt thickness 5-1/2 inches

6. Massachusetts' interstate design is:
   a) One and one-quarter inch asphalt concrete surface
   b) One and one-quarter inch asphalt concrete base
   c) Four and one-inch asphalt penetration macadam base
   d) Twelve-inch gravel base
   e) Total asphalt thickness 7 inches

7. Connecticut's Turnpike:
   a) One and one-half inch asphalt concrete surface
   b) Two-inch asphalt concrete base
   c) Three-inch asphalt penetration macadam base
   d) Four-inch dry macadam base
   e) Eleven- to 23-inch general subbase
   f) Total asphalt thickness 6-1/2 inches

Connecticut's interstate design is essentially the same as the Turnpike.

8. Virginia's interstate 95 (Richmond-Petersburg Turnpike):
   a) Two-inch asphalt concrete surface
   b) Seven and one-half inch asphalt concrete base
   c) Five-inch soil aggregate base
   d) Total asphalt thickness 9-1/2 inches

9. Oklahoma's interstate 40 (formerly U.S. 66):
   a) One and one-half inch concrete surface
   b) Three-inch asphalt concrete base
   c) Eight-inch hot-plant-mix sand asphalt base
   d) Ten-inch subbase
   e) Total asphalt thickness 12-1/2 inches

10. Missouri's interstate project near Joplin:
    a) One and one-half inch asphalt concrete surface
    b) One and one-half inch asphalt concrete base
    c) Four-inch asphalt penetration macadam base
    d) Four-inch Joplin chat base

e) Nine-inch select gravel subbase
f) Total asphalt thickness 7 inches

11. Maine's interstate design:
    a) One and one-half inch asphalt concrete surface
    b) One and one-half inch concrete base
    c) Five-inch asphalt penetration macadam base
    d) Six- to 8-inch crushed gravel
    e) Eighteen-inch select bank run gravel
    f) Total asphalt thickness 8 inches

From this we can see that combined thicknesses of asphalt surfaces and bases vary from state to state and from job to job. The average asphalt thickness is approximately 8 inches, with a minimum of 5-1/2 and a maximum of 12-1/2 inches. These thickness differences are accounted for by conditions occurring in each location that could be traced to bearing capacity of foundations, quantity and quality of available aggregates, climatic conditions, construction methods, and others.

Types of Asphalt Bases

It also should be noted that various types of asphalt bases are being used. These can be categorized into three main groups, as follows: asphalt concrete, asphalt-treated, and asphalt penetration macadam. Essentially, all these types have one common characteristic—they are combinations of mineral aggregates and asphalt. From this point on, each has its own individual requirements, which we will take up later.

All types may be used for most traffic conditions, depending on overall structural design, materials, construction methods, etc. In general, asphalt concrete bases are used for heavy to very heavy traffic conditions. Asphalt-treated bases are usually confined to light or medium traffic, although some higher quality aggregate materials when treated with asphalt can support heavy traffic. Penetration macadam bases can be designed to fit most traffic requirements.

I will not go into traffic definitions because I believe that most of you are familiar, or should be familiar, with these terms and the methods of design based on traffic conditions.

To discuss in detail each type of asphalt base, each class of mineral material, each grade of asphalt, and all the various methods of mixing would, as you know, require much more time than allotted here today. I would like to discuss the more important points dealing with two of these bases—asphalt concrete and asphalt-treated.

Asphalt Concrete Bases

First, because it is first in quality, let us discuss asphalt concrete bases. They are foundations consisting of a mixture of well-graded mineral aggregates and a paving grade asphalt mixed in a central mixing plant at elevated temperatures and spread and compacted while still hot (similar to asphalt concrete surfacing).

Aggregates should be of the highest quality, either crushed rock, crushed gravel, or natural gravel, having high strength characteristics and reasonably clean and free from deleterious substances. Maximum aggregate sizes up to approximately 2 inches may be used, and particle sizes should be well distributed from top to bottom. Material passing the No. 200 sieve should be limited to a maximum of about 10 percent, which should be essentially free of clay minerals.

Paving grade asphalts ranging in penetration from 60-70 to 120-150 may be used. Selection of grade will depend somewhat on local conditions; i.e., severity of traffic, character of aggregate used, weather and roadbed temperatures, etc. Lower penetration grades result in faster setting and better cohesive qualities. A word of caution here. If a hard grade of asphalt is used, care must be exercised to achieve initial compaction or density before cooling or setting of the mix takes place. If this initial compaction is not achieved, later compaction by traffic may cause surface distortion that could result in increased maintenance costs.

Mixing may be accomplished by any of the numerous batch or continuous type hot-mix plants now available. Maximum mixing temperatures should be limited to about 325°F. Exact temperature of mix would depend on workability of mix, length of haul, climatic conditions, roadbed temperatures, etc. Completed mix temperatures of 260°F to 300°F are usually sufficient. Moisture content should be a maximum of about 1/2 percent.

There is one word which, to me, is the secret or key to all phases of highway construction, and that is "uniformity." It is especially important in hot-plant operations. This means uniformity of aggregate feeding, aggregate temperatures, aggregate gradations, and proportioning of materials. Just allow one of these to become a variable and the completed base results will not be of the highest quality.

Spreading the mixture on the roadbed should be done with a self-propelled mechanical spreader. Thickness of spreads should be limited to 3-inch maximum lifts, or about 1-1/2 times the thickness of the maximum-sized aggregate being used. Rolling should be started immediately so that all desired compaction is attained before the mix cools.

Compaction should be accomplished in much the same manner as with asphalt surfacing. Usually the initial or breakdown rolling is done with a steel-wheel roller followed by a pneumatic-tired roller. Uniformity of compaction will depend to a large degree upon uniformity of completed mix temperature. Hotter areas will compact more readily than cooler areas. Mixes rolled too hot tend to shove or move under the roller and if they are too cold, proper compaction will not be achieved. Final rolling need not be done with steel, as any small indentations from the pneumatic tires will aid in bonding the next course to the base.

Generally speaking, anything that applies to construction of an asphalt concrete surface may well apply to an asphalt concrete base.

Asphalt-Treated Bases

Asphalt-treated bases are mixtures of various combinations of aggregates (i.e., sands, gravels, crusher run, pit run, etc.) and, generally, a liquid or an emulsified asphalt. Mixing may be accomplished in several different ways—central plant, travel plant, or road mixing machines and blade mixing. Most bases in the category may be spread and compacted at ambient temperatures. However, heating and drying of aggregates may be required when central plant mixing is used. In this latter case, higher viscosity liquid asphalts or even paving grade asphalts may be used, and the resulting mixture placed and compacted before cooling.

As with asphalt concrete bases, the higher quality aggregates usually result in higher quality bases, but nearly all classes and combinations of aggregates, if properly designed and processed, will result in good bases. Each must be considered as a special case and handled accordingly. Material of borderline stability, when untreated, is especially adaptable to asphalt treatment.

Many factors must be considered in choosing type and grade of asphalt best suited for each job. Some are gradation of aggregates, amount of fines, type of mixing equipment available, climatic conditions, etc.

Here are a few general rules that can be used. Rapid-curing asphalts will work best with sandy or gravelly materials containing a minimum amount of fines. Medium-curing asphalts are recommended for sandy soils containing more fines (say about 5 to 15 percent passing No. 200 sieve), and for soils with more than this, slow-curing asphalts should be used. Of course mixing equipment, other than central plant mixing, is a big factor in deciding the grade of asphalt. Lighter grades of asphalts containing larger amount of cutbacks should be used for methods which require longer mixing periods. Climate also helps determine the grade. Heavier grades of asphalt may be used during warmer weather.

Emulsified asphalts of the mixing type (SS-1, SS-1h) also can be used effectively with sands, gravels, and crusher-run and pit-run materials. In general, the SS-1h grade is used with materials low in fines, because it contains a harder base asphalt (40-90 pen) than the SS-1 (100-200 pen).

Amount of asphalt to be used in the different types of bases must be determined for each individual case. Normally, the range will be between 3 and 7 percent of the dry weight of the aggregate.

Mixing may be accomplished with a central mixing plant, travel plant, or blade mixing. Mixing with a blade grader will be considered first because it is one machine that can completely prepare, mix, and place material without the use of other equipment. It is a machine normally found on all road construction jobs.

Mixing is done by cutting and turning of windrow material to which asphalt has been applied with an asphalt distributor. A word of caution at this point. Do not attempt to apply the total amount of asphalt in one application; rather, add it at about 1/3 increments and partially mix between applications. This helps to prevent asphalt from running to low areas and causing fat spots. It also speeds up mixing time and results in a more uniform completed product. The blade grader can be used for aerating and drying the mix prior to placing.

There are many other variations of mixing and combinations of equipment that have been used for in-place mixing—rototillers, multiple-blade drags, disk harrows, and springtooth harrows. All these types of equipment require that asphalt be applied with a distributor in advance of mixing.

Next we come to travel plants. These operate directly over the areas to be treated. Some types require that the aggregate be placed in windrows in advance of the machine so, as it moves ahead, aggregate is picked up and passed through a mixing chamber where asphalt is proportioned volumetrically. The completed mix is deposited on the roadbed (usually in another windrow ready to be aerated), then leveled and compacted.

Other types of machines scarify material-in-place to required depth, pick it up, proportion components, mix, and deposit on the roadbed. With some types, aggregates may be hauled to the job site and dumped directly into the hopper which feeds the mixer. Most travel plants are capable of completely processing as much as 2 cubic yards per minute.

Central plant mixing is done in much the same manner as for asphalt concrete bases. Temperatures of materials at time of mixing must be controlled according to type and grade of asphalt used.

When liquid asphalts and road-mix methods are used, moisture content of aggregates at time of mixing should not exceed about 3 percent. This moisture content should be reduced to 1 percent or less before spreading and compacting. When asphalt emulsions are used, moisture in each layer should be reduced to less than about 4 percent of the dry weight before placing the next layer.

Spreading road-mixed materials is usually done by blade graders. Central plant mixed material may be spread with a mechanical spreading device or finisher. This is the preferred method as material is placed directly into position for rolling.

Compaction of asphalt-treated bases is achieved in much the same manner as with asphalt concrete bases. Under certain conditions, segmented rollers have been used with good results. With asphalt-treated sands it is generally advantageous to compact the material in thin lifts of 1 to 2 inches in thickness with pneumatic-tired rollers.

Sand-asphalt bases and bases utilizing borderline materials should be considered as an asphalt-treated base, even though a paving grade asphalt and hot-mix methods might be used.

Conclusion

One important factor to consider when planning any type of asphalt construction is that work done during the warmer and drier seasons of the year usually gives the best results.

The following are a few reasons why an asphalt-stabilized base is considered better suited to support an asphalt surface:

1. Provides a base with same general characteristics as the asphalt surfacing.

2. It is a base that can adjust itself to ground movements, such as those caused by settlement of subgrade or differential expansion, without cracking.

3. Gives greater protection against surface moistures penetrating into the subgrade or subterranean moistures from working up into the base and surface.

4. Increases cohesion of base aggregates. Consequently, better load-bearing capacities per inch of thickness.

5. Uniform base without expansion joints or excessive cracking due to shrinkage or expansion.

6. No long curing periods required with hot-laid asphalt concrete bases. No delay in placing of next course or in allowing traffic usage.

7. Any type of asphalt wearing surface may be used, depending on traffic needs.

8. Permits use of local borderline materials not normally suited for base construction. This is especially advantageous where high-quality aggregates are not plentiful and cost of importing them would be prohibitive.

9. No special equipment necessary other than used in normal asphalt construction.

10. Economical advantage of stage construction on roads with low traffic may be utilized.

As probably you have gathered by now, I am a firm believer in the ability of asphalt bases to help answer the need for highways that will satisfy the traffic demands of today and tomorrow. Moreover, with proper design, adequate specifications, sufficient inspection, and good construction practices on all portions of the structural section of a highway, we will not be plagued with high maintenance costs and with highways that do not measure up to the quality of the materials used in their construction.

In a way, constructing a highway pavement is like making a cake. The cook who uses a good recipe, proportions the ingredients exactly, mixes and bakes as specified, usually wins the blue ribbon. Let us strive to win more blue ribbons in future highway construction.

FIELD EXPERIENCE WITH ASPHALTS MEETING THE UNIFORM ASPHALT SPECIFICATIONS OF THE PACIFIC COAST

B. A. Vallerga

On February 4-5, 1957, at a producer-consumer conference held in San Francisco, an agreement was reached among representatives of the six Pacific Coast states and the Asphalt Industry of the Pacific Coast on a new uniform asphalt specification developed with a view toward improving the durability and uniformity characteristics of the asphalt itself. With the adoption of this new specification by the states of Arizona, California, Idaho, Nevada, Oregon, and Washington, with the concurrence of the Bureau of Public Roads whose representatives also participated in the deliberations, the Pacific Coast refineries began to produce this new specification material and to supply it to all users of asphalt. This was possible because material manufactured to meet the new specification also easily met all other existing asphalt specifications in the same geographical region.

Background

The change in asphalt specifications was first promulgated by the California Division of Highways in early 1954. Because of the far-reaching effects of any change in asphalt specifications, industry on the Pacific Coast suggested that any contemplated change be given careful study by all state and federal agencies in the natural marketing area of the Pacific Coast refineries. At two producer-consumer conferences held in San Francisco in June 1956, and February 1957, agreement was reached on a uniform Pacific Coast asphalt specification. Table 1 is a copy of the new asphalt specification as finalized.

In suggesting this new specification it was the intent of the California Division of Highways primarily to obtain asphalts having greater durability. According to the state, tests which were used in existing asphalt specifications left much to be desired in the way of accurately measuring asphalt durability. However, since control testing by both consumer and producer requires test methods that can be performed in a reasonable length of time with apparatus that is readily available and not too complex, a schedule of tests and specifications limits was prepared which, it was felt, would provide for greater durability yet would utilize as far as possible test methods covered by AASHO standards and, in most cases, would use apparatus common to any laboratory currently engaged in testing of

Table 1.
Uniform Specification for Paving Grade Asphalts
(As agreed upon at Second Pacific Coast Conference on Asphalt Specification, February 4-5, 1957)

Specification designation	AASHO test method	Grade				
		40-50	60-70	85-100	120-150	200-300
Flash point, PMCC °F min	T 73	460	450	440	425	400
Penetration of original sample at 77°F	T 49	40-50	60-70	85-100	120-150	200-300
Penetration ratio, min $\frac{\text{Pen } 39.2°\text{F} - 200\text{ gm} - 1\text{ min}}{\text{Pen } 77°\text{F} \quad -100\text{ gm} - 5\text{ sec}} \times 100$	T 49	25	25	25	25	25
Furol viscosity at 275°F	T 72	120-430	100-325	85-260	70-210	50-150
Solubility in CCl_4, % min*	T 45	99	99	99	99	99
Heptane xylene equivalent** %, not more than	T 102 (modified)	35	35	35	35	35
Thin film oven test	Test method No. 337-A as published by Calif Div of Hwys					
Loss in weight, % max		0.75	0.80	0.85	1.00	1.50
Tests on residue:						
Penetration, % of original, min		52	50	47	44	40
Ductility at 77°F, cm, min		50	50	75	75	75

* Ethylene dichloride, dichloroethylene or trichloroethylene may be used as substitute solvents, but carbon tetrachloride is solvent of specification.

** Repeating normal spot test and/or glass plate test at end of 24-hour period does not apply.

asphalt.

Asphalt producers of the Pacific Coast reacted favorably to the objective of the state and offered to cooperate fully in development of a suitable and realistic set of asphalt specifications, provided it be done on a regional basis rather than in California alone. This was agreeable to California, and after a period of time during which an extensive program of cooperative testing was carried on to determine the degree of reproducibility of the test methods proposed and to accumulate test data on which to base the realistic setting of specification limits, several changes and revisions in the specifications, as originally proposed by California, were made. The asphalt industry offered no further objections.

Main Features of New Specifications

Following is a brief discussion of some of the main features of the new specifications:

1. Method of performing the flash test was changed to the Pensky-Martens closed tester from the Cleveland open cup unit in order to detect the presence of silicon compounds used as anti-foam agents. Their presence in an asphalt, even in minute amounts, will materially raise the flash point if test material is not stirred. Pensky-Martens apparatus has a stirrer.

2. Penetration ratio was adopted to provide a relative measure of asphalt temperature susceptibility, especially at low temperature. A minimum ratio of penetration at 39.2°F to 77°F was set to avoid pavement brittleness at low temperature.

3. A high temperature viscosity range (at 275°F) was established to prevent the addition of any agent to the asphalt which might improve the flash, but which would raise the mixing temperature to a point where damage to the asphalt might result during the mixing process.

4. The thin film oven test, a development of the Bureau of Public Roads, with a slight modification in procedure, was substituted for the loss on heat test. This test was considered to offer a control on hot-mix hardening in the pugmill. Test limits were set on percent loss of volatiles, percent retention of original penetration and ductility after exposure for 5 hours at 325°F.

5. Finally, the balance of the new requirements are based on tests which were used in previous specifications.

At this point it should be noted that the sliding scale principle was used in establishing test limits in recognition of the fact that makeup of the various grades differs somewhat. For instance, it is just as severe a requirement for a 200-300 penetration material to meet 40 percent retention of pen after loss as the 52 percent requirement is for 40-50 penetration asphalt.

Further details regarding these specifications are outside the scope of this paper. It suffices to say that asphalts produced on the Pacific Coast today are considered to be more durable than those previously supplied, both in respect to heating in the pugmill and to aging on the road. Moreover, added requirements and general tightening of these specifications have tended to reduce variations in asphalts from different suppliers, with the result that they now behave in a more similar manner.

Behavior of Asphalts Meeting Uniform Asphalt Specifications

Since August 1, 1954, a great deal of asphalt meeting the new specifications has been produced and, in most cases, no problems have been encountered. However, there has been some difficulty with so-called "slow setting" of mixes on some paving projects where new asphalts have been used.

Initial reports of difficulties with asphalts meeting the new specifications were in Shasta County, California, on two county road projects at a period when ambient air temperature exceeded 90°F. Both pavements were made with 120-150 penetration asphalt, and were "tender" and "scuffing" under construction equipment. Although the freshly-placed asphalt surface was susceptible to some marking when scratched with a knife blade or dug into with the heel of a shoe (which caused a great deal of concern on the part of the street superintendents), within three weeks' time the contractor purposely skidded a loaded 10-wheel truck on the surface with no ill effects.

Samples were taken of asphaltic concrete surfacing and the asphalt recovered. It was found that the particular new specification asphalt used had an original penetration of 125 and a recovered penetration of 115, or a percentage drop in penetration of less than 10 percent. This was interpreted as a definite indication of a more durable asphalt product, as the normal drop for California asphalts going through the pugmill mixer has generally been around 40 to 50 percent. Additional tests have indicated that asphalts meeting the new California specifications have a drop in penetration of about 10-25 percent in the pugmill. It seemed logical to conclude, therefore, that harder initial grades of asphalt meeting the new California asphalt specifications should be used in order that the setting characteristics (i.e., final consistency of asphalt cement in pavement on the road) of the new specification material would be similar to that meeting previous specifications. The difficulty is not one of stability, as the pavements involved have been able to support the heaviest of traffic loading.

Difficulties of a similar nature also were encountered on a road job in Tehama County, California, and in several other localities on various paving jobs. However, it is interesting to note that California, which has been the largest single consumer of the new asphalt material, has had no problems with slow setting or tender mixes. Moreover, on an experimental project on U.S. Highway 101 near Santa Barbara (California), 17 asphalts were used with the same aggregate grading to prepare an asphaltic concrete. Thirteen of these were asphalts meeting new California specifications. Although penetration range of all asphalt used on this experimental project was 200-300, no difficulty at all was encountered with slow setting of the mix with any of the 13 new specification asphalts used.

The above-referenced project is known as the Zaca-Wigmore experimental test road, and was constructed during the fall of 1954 and spring of 1955 for the purpose of evaluating field performance of the new specification asphalt. Now over 3-1/2 years old, a progress report is in preparation which will be presented as part of a symposium on the Zaca-Wigmore project scheduled for the Third Pacific Area National Meeting of ASTM to be held in San Francisco the week of October 11, 1959. This report will cover design, construction, and performance of various test sections to date.

From an analysis of the situation, it appears that any slow setting characteristics of freshly laid asphalt pavement is probably due to a combination of factors, such as kind and amount (insufficient) of fines, and overly sanded aggregate grading, high ambient temperatures, and type and amount of compaction; all combined with the admittedly slower rate of hardening of the new specification asphalt. Mix segregation, due to poor workmanship, also is known to be a contributing factor. Of significance in this matter also is the fact that the recently instituted "Sand Equivalent Test" in State of California specifications, which most contractors are now meeting, has resulted in a definite limitation on the character of the passing 200-mesh material to fines that are granular in nature.

On the basis of experience with asphalts meeting the new specifications on highway pavement, the following alternate procedures are recommended in order to overcome any situation where scuffing or tenderness of the freshly laid asphalt paving is observed:

1. Use a harder grade of new specification asphalt so that penetration value of asphalt in pavement will end up the same as that usually obtained with previous asphalt. A drop of one grade usually is sufficient. For example, from 200-300 to 120-150.

2. Redesign mix to include more fines as a means of compensating for lower viscosity of new specification asphalt after going through the pugmill, and to avoid an excess of aggregate material in the sand sizes.

3. Use pneumatic-tired rolling equipment to develop a traffic seal, which considerably improves resistance to scuffing. Preferably, this should be done prior to final rolling while mix is still warm enough to iron out any marks by rubber-tired rolling, although rolling with pneumatic equipment is effective and beneficial even one or two days after mix has been placed and compacted.

4. Allow pavement to cure with time and traffic. In one to two weeks the pavement will have sufficient set to resist scuffing or marking under any severe abrasive actions.

5. Proceed with any combination of above four measures.

Conclusions

Considering service behavior of asphalts meeting the new California asphalt specifications, it is concluded that:

1. All reported problems with freshly laid asphalt pavement in not setting up and scuffing and marking under construction equipment are confined to interior valley areas during periods when ambient temperatures exceed 90°F.

2. Although most, if not all, of the asphalt tonnage marketed in the Pacific Coast states meets the new uniform asphalt specification, the difficulty with slow setting asphalt mixes has been confined to a very limited number of jobs, and the 1958 construction season was free of complaints on this score.

3. Identical asphalt used in a mix reported to be slow setting has been used in numerous other mixes where no difficulty at all has been reported.

4. The problem of slow setting is due to a combination of several factors which, individually, may cause no problem, but when combined may result in a mix lacking in sufficient tensile strength to

resist abrasive or sharp turning actions. Some of these factors are:

a) Too soft an asphalt in pavement mixture after going through mixing cycle

b) An overly sanded aggregate grading

c) Insufficient fines in mix

d) High ambient temperatures

e) Physical characteristics of aggregate particles, including fines

5. On highway pavements where extremely high densities are generally not required and traffic is generally of the through or continuous type, the problem of slow setting is not a critical one. The corrective measures given in previous paragraphs have been very effective. Moreover, even if no immediate corrective measures are taken, within a short period, under the beneficial effects of time and the kneading action of traffic, all evidence of slow setting disappears.

6. On other pavements where initial density requirements may be critical, the matter of slow setting is of greater significance. Large areas of paving will never be touched by rubber-tired traffic, where other pavement may be subjected early after placing to severe abrasive action or turning with power steering. Every effort should be made, therefore, to take into account all factors that influence the tensile strength of the mix immediately after placing.

It is the firm conviction of the asphalt industry on the Pacific Coast that once user agencies become familiar with the characteristics of these asphalts meeting the new California specifications and how to work with them, the resulting asphalt pavements will be more durable, and their service life will have been materially increased.

RESTORING NONSKID PROPERTIES OF FLUSHED BITUMINOUS PAVEMENTS AND OILED ROADS

I. A. DeFrance

During fall and winter of 1957, there were a number of widely scattered complaints about slick highway pavements. There were several accidents and some fatalities that undoubtedly were contributed to by vehicles skidding on slick pavement. The complaints and accidents were not exactly new. We recall placing signs 30 years ago that warned, "PAVEMENT SLIPPERY WHEN WET OR FROSTY." In those days, however, the driver was expected to drive so that his vehicle was under control. Probably he is supposed to do so now, too. Nevertheless, these complaints and accidents served to make us keenly aware of the growing seriousness of the slick pavement hazard under present fast driving and heavy traffic conditions.

We urged our men in the field to pay particular attention to the problem, but we found them fully aware of the slick pavement hazard. We were sanding, had organized special shifts for round-the-clock sanding, and in some cases were using salt and calcium chloride, as well. We found we were doing such nonskidding as our available equipment would permit.

In addition, we were doing a stopgap type of burning that was very effective. This consisted of burning transverse strips across the slick highway with patch-kettle torches. We found that some of our annual re-oil schedule was being performed on nonskid slick surfaces, even though it was agreed the nonskidding effect did not last because the newly applied rock sank into excess asphalt that had been covered. It was proposed that we resurface with AC to nonskid the slick surface. We found several sections where this had been tried, and it was obvious that the excess asphalt that had been covered was beginning to flush to the surface and that again we would have our slick pavement.

As a long-range solution to the slick pavement problem, re-oiling and resurfacing with AC seemed to rule themselves out on two counts. First, they were both unable to bury the excess asphalt for long and, second, the tremendous cost. Where would we find funds to re-oil or repave several hundred miles of slick highway?

We were asked why we have so many slick highways. A brief review of the development of the oiled road in Oregon may help us to understand.

Use of fuel oil as a dust palliative was started in 1923, but the actual advent of the oiled road began in 1925 when R. H. Baldock was appointed maintenance engineer. There followed a multitude of experimental applications of various types of road oil with various rates of application of oil and rock. There were primes, roadside cover job, road mixes, oil mats, and bituminous macadams. We wound up a few years later with a set of standard specifications for single and double shot oil mats and light and heavy bituminous macadam; all designed with a ratio of asphalt and rock calculated to produce a stable wearing surface. We have continued to use these specifications, and today have many miles of excellent oil mats and bituminous macadams.

We find a great variation in texture and composition, however, with many sections of slick surface where these is considerable fat or excess asphalt. This is not surprising when we remember the many variables that enter into the oiling.

1. There was a considerable latitude in the amount of asphalt and rock specified, and we had engineers and oil foreman who had their own ideas as to what was the proper spread. Oiling equipment had been greatly improved during the years and there were, and still are, vast differences in the road oil or asphalt used. In the early days much 95 road oil was used. Later we used 200-300, and today we use 120-150. At the same time there were then, as now, the SC, MC, and RC's and many different combinations of them tried out, and their location shortly forgotten.

2. There were many different types of rock used. Some areas used pea gravel and sand. Other areas used part fractured; others all fractured. Some rock was produced on round screens; some on square. Some was not screened; some was almost hidden in fines. Some was plain dirty. Some rock was found to be hydrophilic; some hydrophobic.

3. There was a wide range in weather conditions. Some oil was laid in the sun, some in the rain, and some late in the season, with the result that much cover rock was lost and we had fat surfaces.

The same variables affected our pavement patching. There was the same latitude in the mix, kind and quantity of asphalt, kinds and sizes of rock, and the same variation in weather conditions. We patched for many years with open flame on the mix, without pugmill mixers. We used every type of road oil and asphalt and every kind of rock.

Not all of the slick highways are oldtimers, however. Some of the asphaltic pavements and bituminous macadams being constructed today have shown a tendency to become slick under traffic. Perfection is difficult to obtain, and even under our rigid construction specifications there still is enough latitude to permit a wide range in penetration and quantity of asphalt, size and gradation of rock, and conditions of laying and compaction—all of which lead to a certain degree of variation in bituminous macadams and AC mixes.

Degree of subgrade compaction obtained varies for grading construction performed under different weather conditions and whether compacted early or late in the season. Our rapid fire construction schedule does not always allow the desired time for optimum results. Complete compaction of grade and subgrade is difficult to obtain, and delayed compaction of the subgrade has a serious effect on asphaltic concretes and bituminous macadams.

Another factor that contributes to the problem is delayed compaction of the pavement or oil mat, itself. Asphaltic concrete or bituminous macadam laid in hot weather receives greater compaction than that laid in cold weather. The effect may be slight, but delayed compaction of bituminous macadam or AC surface usually results in some flushing of asphalt.

The smooth textured design of our newly constructed highways also may contribute to the problem. Possibly we should finish to a coarser texture. We recall that a few years ago there was some oiling done that had a coarse texture. Some users immediately complained of a rumbling noise in their cars. We had to change back to the smooth textured oiling. We have often thought that many skidding accidents might have been prevented had we retained the coarse textured specifications.

The problem is one of great complexity due to the many variables involved. Is it any wonder, then, that some of our highways are slick?

To evaluate the problem and consider methods of correction, we investigated and tabulated the sections of slick pavement throughout the state that needed nonskidding. We found a surprisingly large mileage in every part of the state, and that there was an obvious need for some alternate solution to the problem.

We reviewed the equipment we had or that was otherwise available for nonskidding slick surfaces, and found that such equipment was limited in kind, capacity, speed, and effectiveness. Over and above re-oiling and paving equipment, we had and were using four types of equipment for nonskidding:

1. Patch-kettle torches used to burn transverse strips. This was an effective operation with lasting effect because it actually burned part of the excess asphalt.

2. A converted old-type pavement heater. Several torches had been added and converted to propane. This equipment was towed at slow speed to thoroughly heat the surface so that rock spread ahead of the heat could be rolled in. Many miles of slick surface were treated in this manner, with fair results. Results were short-lived, however, as the new rock either sank into the excess asphalt or shelled out under traffic. Apparently heating the excess asphalt and rock by direct flame coated both with smoke and soot so that the bond was not lasting.

3. Clarkmore pavement heater planer. This equipment has fuel-oil torches which heat the surface so that planer knives can cut off high spots. It was designed for planing and leveling, and for these jobs it is effective. For nonskidding it is slow and expensive.

4. Propane "weed burner." Our engineers at Pendleton and LaGrande developed the plans for this equipment. A propane company in Idaho built it, and it was used with success for burning roadside weeds and grass for firebreaks. It was used later for burning excess asphalt from slick pavements. It was successful also in this work, but in a limited way for two reasons. The tractor that towed it could not go slow enough and left a jerky burn pattern, and its three torches were not capable of burning all of the excess asphalt in a traffic lane.

With the help of the entire maintenance division staff we reviewed the work of the four types of equipment and made special test runs with each. We rented a propane weed burner in Portland, but it was not as effective at burning pavement as the Pendleton weed burner. However, it did help to confirm our conclusions.

We all agreed that excess asphalt was our enemy, and that our problem was how to dispose of it. For years here had been a feeling that whatever we did we must not burn the asphalt. Instructions to operators of pavement heaters and planers had been "heat but do not burn" the asphalt. After observing results of the patch-kettle strip burning and the burning by the propane weed burners, we questioned the value of excess asphalt, or that it was essential to the pavement or the oil mat below. We concluded it was just so much excess material, and could be done away with.

We burned many sections of pavement and oil to observe results and to search for damaging effects. We learned that when we applied torches long enough to burn surplus asphalt, the asphalt and rock mixture below was protected from burning by its rock content. As long as the burning period was short, no damage was done.

We then began the design of specialized equipment for the express purpose of burning excess asphalt and affording positive speed control to avoid damage to the highway surface. We adopted the best points of existing equipment, aiming for a unit that would be effective, fast, and economical. Our final design included the following:

1. A flatbed truck geared down by the addition of two standard 4-speed transmissions, one behind the other, to permit a smooth operation at speeds of from 6 feet per minute to 45 miles per hour. Two tanks were mounted on the truck—a 400-gallon one for water and a 500-gallon one for propane.

2. A hood 8 feet wide mounting five 8-inch propane torches placed vertically to provide a powerful downdraft that would actually burn to cinders the excess asphalt as it passed over it at speeds varying from 10 to 36 feet per minute. There were individual valves and flexible fuel lines for each torch.

3. A hoist and boom on the truck to raise hood from pavement to truck bed for traveling to and from the job.

Operation of the unit requires four men (truck driver, burner operator, and two flagmen). Surprisingly, the same number required for strip burning. We now have five of the new pavement burners, each costing an average of $3500, exclusive of truck. All five units are operating to good advantage, and averaging a mile of 2-lane highway a day.

We have had considerable difficulty with the hood. Heat of burning asphalt, added to that of burning propane, burned holes in the steel hood and required replacement hoods at irregular intervals. We lengthened the original hood, raised the torches 4 inches above the hood, and left the back open to facilitate escape of the flames. These changes doubled the life of the hood and decreased fuel consumption. We are experimenting with other methods to further extend the life of the hood.

We have pushed the burning, however, due to urgency of the slick pavement hazard, and have not permitted our experiments to interrupt the program of burning. Field reports indicate that from August 1, 1958, to February 20, 1959, with five burners operating throughout the state, we have burned 233 miles of 2-lane highways. This mileage has been intermittent to include the slickest pavement, curves, intersections, hills, etc., with the worst accident records.

We anticipate we will have burning to do each year, and plan to operate continuously, except possibly during the early fall fire season. Our cost has averaged $230 per mile of 2-lane highway. We may be able to reduce this slightly during the summer months, when burning speed will increase over winter burning speed. We also hope to reduce our cost when we are successful in extending the service life of burner hoods. Cost of burning, however, will be offset, at least partially, by a saving from the reduced amount of sanding required. We also hope to reduce the number of midnight emergency calls for sanding, and hope to reduce the number of round-the-clock patrols that we have had to put on.

Another source of saving we are examining is the possibility that some re-oiling (formerly scheduled to nonskid a slick surface, and some plant patching performed for the same purpose) may be eliminated or postponed. We plan to make certain that highways to be re-oiled, repaved, or patched each year are carefully gone over with the burner so we will not be covering excess asphalt that will flush through and ruin the nonskid texture of our good re-oil or patching jobs.

Burning presents a certain hazard to traffic, but the 8-foot hood, with the flame confined by long side panels, does not seem to bother traffic. On some narrow roads we raise the hood and shut off the torches while gas trucks or school buses pass. It would reduce our costs if we could have a wider hood so we could burn 10- to 12-foot lanes, but a hood this wide on 2-lane roads would crowd traffic, and we have not considered it.

BRIEF HISTORY OF PAVEMENT BURNER OPERATION

Victor H. Coffey
(Assistant Maintenance Engineer)

On July 21, 1958, we made our first trial run with the propane pavement burner. The burner combination consisted of a truck and burner hood, or "boot," suspended at the rear. The flatbed truck on which the burner was mounted was modified by installing two additional standard 4-speed transmissions, one behind the other. This enabled the truck to tow the burner at a smooth uniform speed ranging from 6 feet per minute to 45 miles per hour. Some minor cab alterations were necessary. Also, a waterproof cap had to be constructed for the third transmission, as it rests behind the cab and had to be protected from the weather. A motor tachometer was installed to control the towing speed accurately.

The burner hood was enclosed on all four sides and mounted five 8-inch burners. Each truck had a 500-gallon propane tank equipped with regulators, valves, and pressure gages. Each truck was also equipped with two 15-pound CO_2 fire extinguishers and one pyrene extinguisher. A 400-gallon water tank was installed on the truck. The water was used to wet the pavement back of the flame with a water spray designed to cause excess asphalt to boil up and be burned.

Our first test was with 110 pounds pressure per cubic inch of liquid gas. The burner was run at various speeds. Best results, however, were obtained at 10 feet per minute.

The water spray hit the pavement too far back of the flame, and it was decided to try to shoot it into the flame to get better results. It also was noticed that the rear of the truck was subjected to too much heat and would need a protective shield. After burning about a 500-foot stretch, the burner was taken back to the shops for the suggested corrections.

On July 24, the burner was sent to Klamath Falls, where another test was made. The correction made on the water spray did not prove satisfactory as it set up a curtain in back of the boot, shutting off burner exhaust. The spray was again adjusted on the job. We were burning on extremely fat pavement and the burning asphalt generated so much heat that the flexible fuel lines caught fire and burned. The burner was taken to the Klamath Falls' shops and protective shields were constructed across the top of the boot, with vent stacks on the front.

Those corrections improved operation of the burner, but it still was not performing satisfactorily because it was not getting enough draft. After burning 1/2 mile of 2-lane pavement, it was decided to send the burner back to the Salem shops for further improvements. Better protection was constructed for the rear of the truck and for the flexible fuel lines to the burners. We also opened the back of the burner boot to give the smoke and fumes a better chance to escape. Other improvements were made and test runs again started on the Salem bypass on July 30.

Those tests proved satisfactory, and it was decided to construct five pavement burners in order to provide one for each highway division.

Numerous other improvements have been made and difficulties overcome. Under normal conditions, we are now able to burn up to speeds of 36 feet per minute with only one-half the original fuel consumption.

We no longer use the water boom, and have found through experience that the water spray is more of a detriment than a benefit. The water tank has been retained, however, and fitted with a fire hose. Water is kept under pressure for use in fighting fires.

We have raised the burners approximately 4 inches above the boot and have finally solved our burner oxygen supply problem. We have found that by cutting down the pressure of the liquid fuel according to the amount of excess asphalt to be burned there is more burner efficiency with less fuel.

Wind blows the flame and smoke badly, and we find it necessary to operate according to the direction of the wind so that the flame and smoke is blown away from the passing traffic lane. In many instances the burner is raised and the torches shut off while certain vehicles such as gas trucks and school buses are passing the operation.

One serious difficulty was the burning of the burner boot itself, but we have increased its size and lengthened its life by about 200 percent or more. We are studying other revisions of the burner boot in an attempt to extend its service life. We now are attempting to line the inside of the boot with diatomaceous silica blocks as insulation against the heat, but so far have had little success. We also plan to try stainless steel lining at those areas where the heat is most destructive.

We have made many experiments on the job trying out different speeds, pressures, boot designs, heights, and shapes. These experiments have had considerable influence on the design of burners now being used.

We also tried spreading crushed rock ahead of and behind the burner, but with unsatisfactory results. The charred and sooty surface would not hold rock.

We encourage field engineers to study methods of improving efficiency of burner operations, but have cautioned against conducting experiments on their own until contact has been made with the maintenance office. Many of the experiments proposed have been tried, and there is a serious hazard potential in the operation under the best of circumstances.

Records are kept when experiments are conducted. Results are analyzed, and when successful are incorporated into future burner operations in all areas of the state. One district engineer suggested adding narrow side plates to the boot, to be attached with chain links to ride upon the pavement and keep the flames from coming under the sides of the boot. This eliminated the large scallops from burn design formerly left in the pavement. The plates also further protected passing traffic. This suggestion was cleared through the Salem office, tried with great success, and is now being used on all five burners.

We now have five burners in operation and have our operation costs down to $230 per mile of 2-lane pavement on a state-wide average. A total of 233.34 miles have been burned as of Feburary 20, 1959.

HIGHLIGHTS OF THE AASHO ROAD TEST

C. F. Rogers

Historical Review

The AASHO road test now under way in Illinois is essentially an engineering study of the behavior of concrete and bituminous road pavements of different thicknesses and of highway bridges of varied design when subjected to traffic of controlled weights applied at uniform rates. Sponsored by the American Association of State Highway Officials, the test is being administered by the Highway Research Board of the National Academy of Sciences. Just how did it come about?

Early in 1950, finding it impossible to attribute to any particular magnitude of loading the damage observed on roads subjected to a normal composition of traffic, the AASHO formulated plans for a series of controlled traffic load tests on highways. The first purpose was to determine the effects of specific axle loads applied in measured frequency on representative pavements.

Road Test One-MD, involving an existing concrete pavement, was the first in a contemplated series of regional tests of both concrete and asphaltic pavement types. The test was conducted under the direction of the Highway Research Board as a cooperative undertaking of eleven eastern states, District of Columbia, Bureau of Public Roads, and allied industries. Results of tests have been presented in Special Reports 4 and 14 issued by the Board.

The Western Association of State Highway Officials developed the idea of a road constructed for test purposes both to measure physical damage for policy determination of legal load limits and to develop information necessary for rational pavement design and utilization. The WASHO road test, second in the proposed series, involved varied designs of bituminous pavement subjected to a similar range of axle loading. The test, also under direction of the Highway Research Board, involved Public Roads and industry cooperation with eleven western states. Parts 1 and 2 of the final report are contained in Special Reports 18 and 22 issued by the Board.

A third test was planned to involve cooperation with member states of the Mississippi Valley Conference of AASHO, which group requested advice of the Highway Research Board as to the desirable scope and extent of a test project. The report of the Board indicated the feasibility of a substantial undertaking subsequently determined to be beyond the ability of the regional conference group to finance. The project was referred back to AASHO, which established a working committee representative of all regions to plan the enlarged research.

In July 1952, the working committee submitted a project statement containing the basic concepts of the AASHO road test, and recommended the selection of a site for the project near Ottawa, Illinois. A more complete program was submitted to the AASHO executive committee in March 1953. In November 1954, the AASHO approved the project, and on February 22, 1955, at the request of the association, the Highway Research Board accepted responsibility for administering the project.

Design and Layout

From its inception the AASHO road test has undergone many changes in concept, scope, and extent. It can be said that the final research now under way is the product of many minds, frequent compromise, and a gradual process of evolution. But many of the basic principles established by the AASHO working committee have been retained in the program.

The test site selected lies between Ottawa and La Salle, Illinois, about 80 miles southwest of Chicago, in an area conforming to predetermined requirements as to climate, precipitation, frost penetration, and soil conditions, established to insure widespread application of the seasonal test results to particular conditions prevailing throughout the country. The fine-grained soils which predominate are susceptible to pumping, and typical of large areas in other regions. Average annual rainfall is 32 inches, average depth of frost is 28 inches, and mean summer temperature is 76°F. Mean winter temperature is 27°F, although Chairman Ken Woods of the National Advisory Committee recently stated that the current winter in the Midwest has been the coldest since 1870.

The test road, on a relocation of U.S. Route 6, was constructed as a Federal-Aid interstate project, with financing from apportioned Federal-Aid and Illinois Division of Highways funds to the extent of the cost of the normal 4-lane divided highway to be ultimately provided. The 8-mile section traverses flat to gently rolling topography adapted to a virtually level and tangent roadway embankment, assuring uniform conditions for operation of the test vehicles.

In the tradition of the state which had been host to the Bates road test of the early 1920's, the assurance of full cooperation and the willingness of the Illinois Division of Highways to preempt the use of the road facility for research purposes was an added factor in selection of the test site.

The 4-lane divided roadways are connected by turnarounds to provide the four major test loops designated 3(A), 4(B), 5(C), and 6(D), with tangents approximately 6800 feet in length. An additional loop 2(E) for comparative testing of lighter axle loads was incorporated in the test to provide information which would assist the Bureau of Public Roads in reporting to the Congress with respect to maximum desirable weights and dimensions of vehicles and the equitable allocation of highway cost, pursuant to Sections 108(k) and 210 of Public Law 627. Loop 2(E), with 4400-foot tangents, is located alongside and parallel to loop 5(C). Loop 1(F), with 2200-foot tangents, placed in line with loop 2(E), carries only strain test traffic. It will be used for special tests and to evaluate and isolate the effects of weather from the effects of loading. Each loop provides two test lanes; concrete pavement on one side of the median and bituminous pavement on the other. The varied sections of pavement structure are identical in the paired lanes of each pavement type. In each loop, one turnaround is paved with concrete and the other with bituminous.

In each of the four major loops the controlled axle loadings are applied by 3-axle and 5-axle truck-tractor semitrailer combinations. Each passage of a test vehicle combination, therefore, accounts for two applications of the controlled axle loading. Six vehicles are operated in each test lane, applying single-axle loadings on the inner lanes of each loop and tandem-axle loadings on the outer lanes. Respectively, in loops 3(A), 4(B), 5(C), and 6(D), single-axle loadings of 12,000, 18,000, 22,400, and 30,000 pounds and tandem-axle loadings of 24,000, 32,000, 40,000, and 48,000 pounds are applied. In the inner lane of loop 2(E), four 2-axle vehicles are operated with 2000-pound loads on each axle, and in the outer lane eight vehicles operate with 6000-pound loads on the single rear axle.

The wide research range of controlled axle loadings is accompanied by an equally wide range of pavement structure depths and layer components. Progressively, the test-section designs in each loop are stepped up in relation to the axle loadings, from the lightest to the heaviest. In each test lane, the structures range well below and above the generalized design which would be regarded as adequate for the load. It has been construed that for each pavement type, some two-thirds to three-fourths of the test sections under each loading are less than adequate in varying degrees. Accordingly, varying degrees of distress or damage may be expected with repeated load application, adding up to a number of different points on the performance-thickness curve.

Loop 1(F), with only special-test traffic, contains 56 test sections of both plain and reinforced concrete pavement of 15-foot and 40-foot panels, respectively. Slab thicknesses of 2.5, 5.0, 9.5, and 12.5 inches are combined with subbase thicknesses of 0 and 6 inches. On the bituminous concrete tangent are 64 test sections, each of 25-foot length. Surface thicknesses of 1, 3, and 5 inches are combined with base thicknesses of 0 and 6 inches, and with subbase thicknesses of 0, 8, and 16 inches.

In the remaining loops under controlled loadings, the rigid pavement test sections of reinforced concrete consist of six 40-foot panels, and those of plain concrete consist of eight 15-foot panels. The regular flexible pavement sections have a length of 100 feet, and the special study sections a length of 160 feet. The latter are included to study the effects of paved shoulders, and wedge sections have been included to study base type variations, inclusive of crushed-stone gravel, cement-stabilized, and bituminous-stabilized bases.

Loop 2(E) contains 40 rigid pavement sections having slab thicknesses of 2.5, 3.5, and 5.0 inches in combination with subbases of 0, 3, and 6 inches. The 68 flexible sections include thin surface treatments and surface thicknesses of 1, 2, and 3 inches combined with bases of 0, 3, and 6 inches, and subbases of 0 and 4 inches.

Each of the four major loops contains 68 rigid pavement sections and 84 flexible pavement sections, with component thicknesses in inches tabulated as follows:

Loop	3(A)	4(B)	5(C)	6(D)
PCC slab	3.5-5.0	5.0-6.5	6.5- 8.0	8.0- 9.5
	6.5-8.0	8.0-9.5	9.5-11.0	11.0-12.5
Subbase	0-3-6-9	0-3-6-9	0-3-6-9	0-3-6-9
BC surface	2-3-4	3-4- 5	3-4- 5	4- 5- 6
Base	0-3-6	0-3- 6	3-6- 9	3- 6- 9
Subbase	0-4-8	4-8-12	4-8-12	8-12-16

In dimension and arrangement, the test section variations conform to established methods of statistical experiment design. In the layer components of the pavement structures, each design variation occurs in combination with all other variations. The factorial design improves the analytical basis for evaluation of test section behavior. A second feature of the statistical experiment involves randomization both in the chance order of alinement of the test loops and in the order in which the test sections are arrayed, as by lottery, within the construction blocks of each test loop.

The 50-cent word for the third feature of the statistical method is "replication," literally an echo or reproduction. Some of the test sections are repeated—inserted again at random locations in the tangents. There are two kinds of replication. Like sections subjected to the same loading are direct replications. Like sections subjected to different loadings are hidden replications. Both randomization and replication are calculated to minimize bias, such as from the vagaries of construction and testing environment, and to contribute to a determination of reliability in the test findings.

Sixteen test bridges of varied design and composition are included in the four test lanes of the two loops subjected to the heavier single- and tandem-axle loadings. Each is of 50-foot span and single-lane width, and all have been purposely designed such that the particular controlled loads carried induce an unusually high order of overstress.

Eight of the bridges consist of steel I-beams with concrete decks, involving design stress variations of 27,000 and 35,000 pounds per square inch, and designs with and without composite action and cover plates. All of the steel structures are located on the rigid pavement tangents.

Four of the eight concrete bridges are of conventional reinforced concrete design, and four are of prestressed concrete design, with variations of desired stress level in concrete and reinforcing steel. All of the concrete bridges are located on the flexible pavement tangents.

Unlike the statistical experimental design of the pavement test sections, the bridge design variations constitute a series of individual case studies. These are calculated to contribute specific information necessary to evaluate the effects of repeated overstress, dynamic impact reactions, use of cover plates, and the development of composite action.

Locations of the test structures by designated numbers are as follows:

Test bridge no.	Type	Loop	Axle load-lb	Stress-psi
1-A	Steel	5(C)	22,400 S	27,000
1-B	Steel	5(C)	22,400 S	35,000
6-A	Prestressed	5(C)	22,400 S	800
6-B	Prestressed	5(C)	22,400 S	300
2-A	Steel	5(C)	40,000 T	35,000
2-B	Steel	5(C)	40,000 T	35,000
5-A	Prestressed	5(C)	40,000 T	800
5-B	Prestressed	5(C)	40,000 T	300
3-A	Steel	6(D)	30,000 S	27,000
3-B	Steel	6(D)	30,000 S	27,000
7-A	Reinf. concr.	6(D)	30,000 S	40,000
7-B	Reinf. concr.	6(D)	30,000 S	40,000
4-A	Steel	6(D)	48,000 T	35,000
4-B	Steel	6(D)	48,000 T	35,000
8-A	Reinforced	6(D)	48,000 T	30,000
8-B	Reinf. concr.	6(D)	48,000 T	30,000

Cost and Cooperative Financing

When the Highway Research Board undertook the administration of the AASHO road test, it was recognized that the ultimate scope of the research had not materialized, full impact of its cost had not been realized, and the requirements for its cooperative financing had not been wholly met. There was scant precedent in the Board's previous administration of the regional tests to indicate what might be expected. Road Test One-MD had cost approximately $250,000, and the WASHO road test about $900,000.

The project statement of the working committee in July 1952, envisioned a test with four loops, eight lanes, three vehicles per lane, and one year of operation at an estimated cost of $4,000,000. When the project program was published in January 1953, the idea of two years of operation had taken hold and the cost was estimated at $8,300,000. In July 1954, provision was made for the operation of six vehicles per lane, and the estimate of $11,836,000 was the figure in effect when the Board was given administration in 1955.

During 1956, additional loops were added to facilitate the congressional studies, decision was reached to adopt the statistical experimental design, and other refinements due to the altered scope and purpose of the research were added. Department of Defense support services made necessary the provision of driver housing facilities, the Federal-Aid primary route containing the test road section was designated an interstate route, increasing both standards and cost, wage and price inflation continued, and the award of the grading contract confirmed the increased costs inherent in the unprecedented construction processes and controls required by research considerations.

By March 1953, the cost estimate stood at $19,111,300, and it was apparent that additional financing would be essential. In May 1957, it was necessary to reject the paving bid because of inadequate financing. Efforts to obtain additional financing were successful during the summer, and the readvertised paving contract was awarded in August. It was not until fall that a final estimate was made of the total project cost and total financing was assured.

In the following table the items of expense expected to be incurred, or already committed, are broadly classified. Probably some changes of classification will be found necessary and, undoubtedly underruns and overruns will occasion some transfers of funds from one classification to another.

Item		Cost estimate
Buildings & grounds		$ 918,200
Construction		7,350,800
Maintenance		730,000
Rehabilitation		633,600
Operations		2,427,000
Research & testing		3,221,600
Project supervision		1,001,800
Contingencies		793,200
Indirect expense		726,500
Reserve		500,000
	Subtotal	18,302,700
Less salvage		641,400
Test features		17,661,300
Federal-Aid		4,042,000
	Total	21,703,300

This final total cost is now firmly financed by the highway departments of all states, the District of Columbia, territories of Hawaii and Puerto Rico, Bureau of Public Roads, Automobile Manufacturers Association, American Petroleum Institute, and the Department of Defense. All of the contributions are regarded as being made to a common fund available for the reimbursement of all items of expense. Sources and amounts, or value of services contributed, are as follows:

Source	Amount
Joint-state funds	$ 7,399,200
Contributions of industry:	
Automobile Manufacturers Assn.	1,300,000
American Petroleum Inst.	875,000
Department of Defense	700,000
Other agencies	81,400
Bureau of Public Roads:	
Administrative grants	6,365,100
Research services	940,600
Total test features	17,661,300
Total Federal-Aid facility	4,042,000
Total	21,703,300

Monetary contributions of industry were made in lieu of provision of vehicles, fuel, etc., in order that the project staff might have complete freedom in the specification, selection, and operation of the test vehicles.

Support of the Department of Defense is in the form of services provided by a special unit from the U.S. Army Transportation Corps stationed at the site. Test vehicles are operated by the personnel of this unit.

The Bureau of Public Roads, in addition to direct financial assistance and Federal-Aid participation, is providing technical research personnel, advisory services, equipment, and shop work and materials in the development of instrumentation. The Bureau is cooperating in all phases of the work, including authorization for and expense of the base station and mobile radio communication system, and the making of a motion picture record of the project.

Under an agreement with the National Academy of Sciences, each state highway department, and the Illinois Division of Highways, the Bureau acts as fiscal agent in proportioning reimbursement of construction costs from apportioned Federal-Aid funds, Bureau administrative funds, and from the joint-state funds contributed by the several states from the 1-1/2 percent highway planning survey funds. The covering project agreement, executed in May 1958, includes costs for both the Federal-Aid facility, for research construction, and for all costs for both pre-test and post-test construction at an estimated total cost of $12,206,415.13. Early in 1958, the Bureau, at the request of the Academy, undertook and completed procurement of the 70 test vehicles now in operation on the project.

Much credit is due the Illinois Division of Highways for the major role performed throughout the construction phase—service far beyond the requirements of any normal construction project. The Division was responsible for planning, design details, surveys, purchase of rights-of-way, contractual arrangements, and supervision of construction, started in March 1955 and completed in December 1958. Total costs for provision of the complete test facility as of December 31, 1958, are tabulated below.

Type of work	Estimated total cost
Rights-of-way	$ 785,498
Grading & drainage	3,000,330
Grade separations	338,440
Test bridges	171,766
Concrete paving	3,076,615
Bituminous paving	3,128,692
Total	10,501,341

Deduction of this amount from the project agreement total leaves a balance of $1,705,000 for post-test construction, maintenance and rehabilitation of the test road. The financial report of the project as of January 1, 1959, indicated a total of $15,754,036 committed or expended, leaving a balance of $5,949,264 available for further financing. Both of these balances are regarded by the Board as adequate for completion of the project.

Project Organization

The spirit of cooperation manifest in the planning and financing of the road test is exemplified in the project organization. The executive committee of the Highway Research Board administers the project through a national advisory committee, regional advisory committees, and a project staff. The Board, desiring to secure representation of all contributing highway departments, requested the AASHO to nominate a qualified person from each state of the four AASHO regions to constitute regional advisory committees. The regional committees designated three members each to represent the region on the national advisory committee. In addition to the twelve regional members, AASHO is represented by the president, executive secretary, and chairmen of the AASHO Committees on Highway Transport and Design.

The National Advisory Committee has a total membership of 36, with 8 alternates. It includes representation of the Board, AASHO, 3 colleges or universities, 10 industrial associations, foundations, or institutes, Department of Defense, and Bureau of Public Roads. The national group meets three or four times annually, and has a designated 11-man steering committee to resolve matters of policy requiring interim decision.

The project staff consists of a project director, appointed by the Board in April 1956, a chief engineer for research, and 11 special branches headed by supervisors. The functional staff organization includes the construction branch, 3 primary research branches (rigid pavement, flexible pavement, and bridges), operations branch, maintenance branch, instrumentation branch, data processing and analysis branch, materials branch, special assignments, and public information. The National Academy of Sciences maintains a business office at the site.

Throughout the construction phase the project staff has been materially aided by a permanent task force of the Illinois Division of Highways, headed by an engineer of physical research and consisting of a road engineer and assistant, 2 resident engineers, an office engineer, 3 field engineers, a chief of party, and at times up to 65 employees.

In addition to the professional personnel recruited by the Board, the staff has been further augmented by engineers assigned to the project by the states of Ohio, Wisconsin, Iowa, Missouri, Kansas, and Oklahoma, and by the Bureau of Public Roads. Also, skills of numerous specialists have been made available to the project on both short- and long-term bases, including industrial observers.

Further aid has been provided by the Board in the form of special advisory panels made up of recognized experts and outstanding specialists recruited on a voluntary basis from universities, highway departments, and industrial sources. The advisory panels, functionally like the staff branches, are statistical, soils, instrumentation, public information, materials and construction, maintenance, vehicles, bridges, performance rating, and economic data.

The United States Army Transportation Corps Road Test Support Activity arrived at the project during summer and fall of 1958. The group consists of 2 medium truck companies under a colonel in command, with 9 officers, 28 noncommissioned officers, and about 300 men. The operations branch maintains liaison with the Army support group in scheduling, operating, and maintaining test vehicles.

Test Road Construction

The Illinois Division of Highways assumed responsibility for construction of the test road on March 25, 1955, and the following year was devoted principally to location work, preliminary plan preparation, soil surveys, and testing.

Acquisition of rights-of-way started early in 1956, steel fabrication contracts for bridges and grade separations were awarded in May, and the principal contract for grading, drainage and substructures on July 19.

The grading contract involved construction of approximately 1-1/4 million cubic yards of embankment from selected borrow pits. The requirement for uniformity was paramount, specifications were very rigid, and close tolerances were assured by rigid inspection and assembly line testing procedures.

More than 200 pieces of construction equipment were used in embankment construction; required to proceed simultaneously in all loops. Each loop required 4 to 10 large earth movers, 5 rotary speed mixers, 3 rubber-tired roller compactors, 3 water trucks, and miscellaneous service equipment. Grading equipment was permitted to turn or cross over the embankments only in the transition areas between construction blocks.

Fine-grained clay soil, type A-6, from the borrow pits was spread in 6-inch loose layers, compacted to 4-inch lifts, and the entire 3-foot soil embankment built up in construction blocks 500 to 800 feet in length. All lifts were required to have a standard Proctor density between 95 and 100, accomplished by careful control of water added to the rotary speed mixers operated in tandem across the width of the lift. Moisture content was controlled within plus or minus 2 percent of optimum. Some 800 moisture, density, and uniformity control tests were made each day, or on the average of a test for each 30 cubic yards of embankment.

About 98 percent of the grading contract was complete at the end of the construction season of 1956, and the entire test road embankment had been covered with sand-gravel mulch material which composed the first lift of the subbase under both the concrete and bituminous pavement types. During the winter, the Illinois task force prepared plans and specifications for paving operations.

Paving contracts, advertised for award in the spring of 1957, produced only a single bid, which was rejected because of inadequate funds. Subsequently, changes were made in specifications without impairment of research requirements, and paving contracts were readvertised and awarded in August—too late for completion that year. The remainder of the construction season of 1957 was spent in production and stockpiling of additional sand-gravel subbase material, crushed-stone base, and materials for the paving mixes. Drainage structures, overpasses, and test bridges, except for deck slabs, were substantially completed. When work shut down in November, some of the paving on turnarounds was in place and the service and connecting roads were partially completed.

The winter shutdown was terminated in April, with resumption of paving on turnarounds, test tangents, and deck slabs of test bridges. The same close tolerances and controls required in embankment construction were applied to each component layer of pavement structures. Side forms were set for placement of each structural layer, and finishing was by mechanical subgrader to a tolerance of plus or minus 1/8 inch. All hauling and placing equipment operated from the shoulder and only compacting and finishing equipment was permitted on the grade. Frequent moisture and density tests were made on each layer of pavement structures, and the uniformity of each was controlled by field CBR tests and Benkelman beam deflections tests made at regular intervals.

Portland cement concrete paving was resumed April 16, 1958, and completed on July 10, 1958. Spring and early summer was dry but, later, wet weather and difficulties experienced in developing construction procedures hampered bituminous paving, which started on May 13, but was not completed until October 4. At the time of the dedication ceremony inaugurating the start of controlled-load traffic on October 15, 1958, only a few incidental items of work remained to be finished. All construction work was complete on December 3, 1958.

Precision construction necessary to insure maximum possible uniformity in the layer components of the test sections would hardly be feasible on a normal construction project. It is essential in this research in order that behavior of test sections can be related directly to their structural depth and layer composition. Size of the work force is an indication of the magnitude and complexity of the research requirements, the unusual construction features, and the assembly line controls and testing procedures. At the peak of the construction activity, some 213 engineers and technicians were associated directly with the project, exclusive of contractor's personnel. In all, more than 800 persons participated in construction of the 8-mile test road.

Operations

Construction was only one of a number of related activities at the AASHO road test during 1958. During the summer, test vehicles purchased in the spring were delivered to the project. Total purchase included 70 conventional commercial vehicles ranging from light pickup trucks to heavy truck-tractor semitrailers.

Criteria for selection of test vehicles included low bid price, availability of parts and service facilities near the test site, and different makes broadly representative of normal highway operation. As a result of limiting the number of vehicles permitted any one manufacturer, 10 different makes of trucks and tractors are being used, the latter in combination with 7 different makes of semitrailers.

Required test loads were fabricated on the project in units made up of concrete blocks strap-bound on wooden pallets and solid concrete 1000-pound blocks. Final loading to the desired axle weights was such that no vehicle load was more than one tier high. More than 1150 tons of test loads were required.

Vehicles operate at a constant speed of 30 miles per hour in the direction of normal traffic on a 4-lane divided highway. In each of the 10 test lanes the lateral placement is varied in a pattern established to simulate the transverse distribution of wheel paths of normal traffic on 12-foot wide lanes. The 200-foot radii turnarounds on the major loops have 2 lanes of 14-foot width. The inner lane has a superelevation of 0.1 foot per foot of width, and the outer lane 0.2 foot. The outer lane is used alternately by the vehicles in the 2 test lanes of each loop, and 30-mph tangent speed is maintained on the turnarounds. The inner turnaround lane is used in inclement weather when use of outer lane would be hazardous.

Test vehicles in each lane are operated in two 9-hour shifts daily, 6 days per week, applying axle loadings to all test sections at the same rate. Three loading schedules are operated at intermittent periods to approximate around-the-clock operation on normal highways. Each vehicle on the longer loops makes from 8 to 9 round trips per hour over the roughly 3-mile circuit. Monotony of the operation is broken by driver rest periods of 10 minutes in each hour.

A loaded standby vehicle of each weight is available for replacement of a disabled vehicle in any lane. In general, vehicle maintenance is performed in nonoperating periods, but vehicle breakdowns can delay operations. To the extent possible, maintenance of test sections also is accomplished during down time. However, since much of the 14 miles of 2-lane highway under test is underdesigned for the loads, it is necessary to stop traffic on occasions to maintain distressed sections. Failed sections are excluded from consideration as test sections, upon failure, but these are reconstructed to carry test traffic, and are kept under observation.

Based on experience thus far, about 50,000 load applications, it appears as a practical matter that about three-fourths of the theoretically possible rate of load application can be realized. Thus, the surviving test sections should be subjected to about 3/4 of a million load applications in the 2-year period of testing.

Following completion of controlled-load tests, an extensive program of special studies will be conducted with special vehicles, including some of the largest military vehicles, in order to determine design requirements for their accommodation.

Test Section Behavior

Probably the most important, and certainly the most difficult aspect of the AASHO road test, is the requirement to appraise and evaluate the behavior of the test pavements and bridges, and to relate cause and effect in interpretation of their performance. To this end the very substantial measurements program will utilize all applicable means of record and analysis.

Certain types of observation and measurement will be made and recorded by trained engineers. Examples are physical manifestations of behavior such as condition surveys, recording of cracks, pumping, and the like; record of type and extent of effort and materials required to keep test sections in operable condition; and studies of moisture, density, and gradation in failed sections. There will always be a place for the engineer and his notebook.

A subjective method of evaluating pavement test-section behavior consists of a statistical correlation of opinion rating with measured phenomena. Prior to the start of test traffic, the performance rating panel evaluated a large number of pavement sections on existing roads in Illinois and Minnesota. Sections of varied age, traffic usage, and condition were individually rated by the panel members. A composite numerical rating of current adequacy was established for each section. On each section record was made of certain physical data such as profile, cracking, etc., such as will be a matter of record on the road test sections. Measured phenomena are expressed in the form of analytical equations, utilizing suitable coefficients, designed to reproduce the composite panel rating of the sections. Applied to road test sections, the equations containing factors for the measured data can be made to reproduce subjective ratings of adequacy. Equations developed for each pavement type can be verified or the coefficients modified with experience on the test road to obtain periodic serviceability indices of the test sections representative of their progressive behavior under load.

Additionally, more than $1,000,000 worth of complex electronic and mechanical instruments, measuring devices, and data collecting and processing equipment is in use at the project. Much of the instrumentation was developed expressly for the road test and, in itself, represents a significant contribution to highway research.

The measurements program includes longitudinal and transverse profiles, static and dynamic strains, deflections, and deformations at various places on slabs and bridges, static and dynamic deflections and curvature on flexible pavements, changes in thickness of pavement components, vehicle load-shift and load-forces transmitted, tire pressure and slab-subbase pressure distributions, environmental studies and various special studies applicable to each pavement type, to the test bridges, and to the no-traffic loop. To assist in conducting this program, 7000 measuring devices have been installed in or on the test pavements and bridges during or after construction.

In the bituminous pavements there have been installed 2000 settlement rods to record changes in thickness of layer components, 200 curvature strips to measure surface bending under load, 400 rods to be used with linear variable differential transformers to measure transient deflection, 20 soil pressure cells, 700 thermocouples, 40 frost depth indicators, and 10 vertical volume change devices.

Installed in the concrete pavements are 1122 strain gages, 552 rods for measurement of transient deflection, 528 rods to record subbase thickness changes, 36 frost depth indicators, 20 slab-subbase contact indicators, 903 thermocouples, 24 moisture cells, 44 Carlson pressure cells, and 384 reference points for warping studies.

Instrumentation assists in measurement of various effects of traffic and in recording variations in environmental conditions. It permits some measurements not otherwise possible, speeds acquisition of test data, and makes possible automatic reduction and rapid engineering and statistical analysis. Many of the installed devices that change measured phenomena into electrical potential have high sensitivity to small changes, can be inserted at inaccessible places, can be made to record changes automatically, and permit observations that occur too rapidly for visual recording.

One of the more impressive developments is the longitudinal profilometer. This is an electromechanical device mounted in a trailing carriage towed by a recording van. Changes in slope of pavement in wheel paths are registered in the motion of the trailer wheels and measured in relation to a constant plane of reference. The reference plane is a spinning disc, the ball bearings of which are lubricated by gas jets and virtually frictionless. Referenced motion from each wheel path is captured by an ingenious electrical contrivance and recorded on 2-trace oscillograph tape as the carriage is towed along the pavement lane at about 8 miles per hour.

Oscillograph tapes are reduced to digital form at intervals corresponding to 1 foot of roadway; equal to about 7 readings per inch of tape. Measurements are repeated periodically to register profile changes under repeated load application, and it is estimated that in excess of 40 actual miles of tape will be produced during the test period. To keep the processing of tape recordings current, an automatic chart reader is being utilized.

The Benkelman beam developed on the WASHO road test has been made automatic in the shops of the Bureau of Public Roads. Mounted in a specially adapted semitrailer, the device measures surface deflections in flexible pavement at regular intervals as the vehicle moves along the road at about 3 miles per hour. Loads on the trailer may be varied, and resulting deflections are measured and automatically recorded at 11 points near the rear wheels.

Also in use is a device for the nondestructive determination of pavement density. The instrument involves an application of nuclear physics, utilizing the principle that attentuation of gamma radiation increases with increase in density.

Other devices include transverse profilometers permitting automatic recording of periodic changes in transverse profiles and depth of rut measurements, equipment for recording transient strains, and the Bureau of Public Roads' bridge testing trailer containing 48 channels of oscillograph recording equipment being used on the test bridges.

The voluminous flow of interacting data from the forty-odd different measuring systems requires elaborate installations of equipment for data reduction, processing, and analysis. A fairly complete IBM system has been installed, and automatic equipment is used to convert punched tape records for punched card analysis. A Bendix G15D digital computer at the project has been used an average of 12 hours daily for the past 40 weeks. More elaborate programs are scheduled for use of the Datatron computer at Purdue University. Use of the automatic computer equipment will permit the analysis of engineering data developed by the research to be kept reasonably current, and the research findings to be made known at the earliest possible date.

Reporting

Other than the numerous papers, largely descriptive of the AASHO road test, presented at various times and places, and occasional news releases of specific happenings on the project, there have been no official reports issued concerning the findings of the research. Upon recommendation of the National Advisory Committee, the Highway Research Board has established the policy that all information and publicity shall be cleared through the Public Information Branch.

The Advisory Committee is kept informed of developments by means of bimonthly reports prepared by the project staff. The reports contain factual data on all phases of the research, but are confidential and no release is permitted nor any analysis or interpretation of the data. The Committee is advised also of official releases of information. It is news that one of the test vehicles was overturned the first day of traffic and that, subsequently, the height of vehicle loads was lowered. It is news that, "As expected, some of the thinner pavement sections and several of the overstressed bridges have shown distress. In fact, four of the bridge spans have failed and are out of the test."

On a project where more than 4000 visitors have signed the guest book (and some have not), it is obvious that news also can be made from direct observation of events on the project. It is problematical to what extent the release of unofficial observations can be restrained. The extent to which the facts recorded from time to time will warrant official interim findings has not yet been determined.

However, the urgent need for the use of official findings in the studies requested by Congress will exert pressures for their early release.

Plans for the final reporting on the AASHO road test are under study. It is probable that a series of reports, each dealing with a completed phase, will be prepared by the staff and reviewed by special review panels prior to release. These reports, of course, would become public information as they are completed. Similarly, the motion picture record of the project will probably consist of separate films on different phases, available for showing upon completion.

Objectives and Implications

The formally stated objectives of the AASHO road test have been reserved intentionally in this paper to this point in order to relate them more intimately to some of the more important implications of the research. These objectives are as follows:

1. To determine significant relationships between number of repetitions of specified axle loads of different magnitude and arrangement and the performance of different thicknesses of uniformly designed and constructed asphaltic concrete, plain Portland cement concrete, and reinforced Portland cement concrete surfaces on different thicknesses of bases and subbases when on a basement soil of known characteristics.

2. To determine significant effects of specified vehicle axle loads and gross vehicle loads when applied at known frequency on bridges of known design and characteristics.

3. To make special studies dealing with such subjects as paved shoulders, base types, pavement fatigue, tire size and pressures, and heavy military vehicles, and to correlate the findings of these special studies with the results of the basic research.

4. To provide a record of type and extent of effort and materials required to keep each of the test sections or portions thereof in a satisfactory condition until discontinued for test purposes.

5. To develop instrumentation, test procedures, data, charts, graphs, and formulas, which will reflect the capabilities of the various test sections, and which will be helpful in future highway design, in evaluation of load-carrying capabilities of existing highways, and in determining the most promising areas for further highway research.

The implication of design is strong in the first objective. The AASHO design committee is concerned with the development of improved bases for structural design of both rigid and flexible pavements. The committee has undertaken a complete review of past design theory and practice and of the design variables essential of consideration in rational design procedures. Road test data will be of material assistance to the committee through the evaluation of vehicular variables such as load magnitude, dynamic impact reaction, and frequency of load application, and of the road variables required for essential supporting ability, under conditions existing at the test site. Design requirements established for conditions in the area of the test road can be made applicable to conditions in other areas by comparative testing and research. The opportunity exists for any state to construct facsimile road test sections in line of traffic on normal roads or on parallel turnouts to which normal traffic can be diverted for test purposes. Such test sections can be compared with other sections built of local materials. Translation of the AASHO road test results to local conditions would be facilitated also by use of instrumentation to obtain measurements comparable to those made on the road test.

Objective 5, above, carries the implication of vehicle size and weight regulation. AASHO road test findings will have immediate application to recommendations to be made to the Congress by the Secretary of Commerce pursuant to Section 108(k) of Public Law 627, with respect to maximum desirable weights and dimensions of vehicles using the Federal-Aid highway systems. During the last session of Congress, the final reporting date for this study was advanced from May 1, 1959 to January 3, 1961.

Public Roads is working closely with states in developing these recommendations, and the AASHO executive committee has directed the Committee on Highway Transport to review the April 1, 1946 policy concerning weights and dimensions of vehicles, and upon completion of the road test to prepare revised recommendations. Structural aspects of these policy determinations will be based on the engineering data derived from the road test. The transport committee also is cooperating with the design committee and the committee on bridges and structures in a joint effort to improve the relation between design standards and vehicle regulation. Effects of varied degrees of overstress of the road test bridges will be evaluated in relation to the design standards of existing bridges in order to ascertain maximum loads which can be authorized without detriment to existing bridges of varied standard. Additional data bearing on the geometric aspects of regulatory policy are being developed by studies being conducted with AASHO and the Highway Research Board.

Road test objectives embrace two other areas of research to which the findings will contribute. The Highway Research Board Project Committee No. 5, Economics of Motor Vehicle Sizes and Weights, has the long-range objective to establish optimum economy in highway transport considering both costs of vehicle operation and highway provision. Certain vehicular economies inherent in the operation of heavier vehicles will, at some level, be offset by the increased highway cost of accommodating such vehicles. Data developed by the committee on the range of cargo densities transported and the cost of operation of vehicles of varied size and weight will be supplemented by information available from the road test to assist in determination of economic balance in overall highway transportation costs.

The other research is the Highway Cost Allocation study being conducted by Public Roads pursuant to Section 210 of the Highway Revenue Act of 1956, concerned with equitable allocation of highway cost responsibility among highway users and nonuser beneficiaries. For both studies engineering data provided by the road test will need to be given an economic interpretation to provide rationalized estimates of the highway cost of providing for, and maintenance costs of accommodation of, the full range of axle loadings under investigation.

Public Roads has long been interested in vehicle impact reactions and their effects upon road and bridge structures. Early studies of truck and bus impact established the beneficial effects of pneumatic over solid tires. Later studies of vehicle impact on bridges were concerned with design factors and regulation. When the electronic scale for weighing vehicles in motion was being developed, difficulty was experienced in checking against static loads of the same vehicles, attributed to dynamic variations. Instrumentation of a test vehicle at the WASHO road test, and oscillograph recording of its operation established substantial dynamic variation from static wheel-load magnitudes. Public Roads' further work in recording dynamic tire bulge and tire pressure variation was undertaken as background in the effort to get some of these impact reaction measurements introduced into the AASHO road test.

In pursuit of these studies, Public Roads encountered other interests concerned with vehicle vibrations and response to road roughness, associated with design of suspension systems, tire equipment, etc., and came to the conclusion that there would be mutual advantages to all in coordination of research effort, whether the concern is driver comfort, cargo damage, vehicle maintenance, or road damage. Discussion of the problem with representatives of industry, AASHO, the road test staff, university personnel, and others, developed the overall system concept in which all aspects of the interacting behavior of the road and the vehicle are considered as separate elements of the same problem. To initiate further study along these lines, Public Roads has entered into research contracts with Purdue University and with the Cornell Aeronautical Laboratory of Cornell University.

Work at Purdue on the subject of highway characteristics related to vehicle performance is a merger of a statistical and engineering approach to the problem which grew out of road test staff discussion as to the preferred means to relate test road parameters and vehicle responses. The Cornell project, referred to as road loading mechanics, utilizes the overall system approach. A framework in theory, mathematically arrived at and laboratory tested, will be expressed in dynamic equations, to be subsequently verified in full-scale experiments. This appears to be a very promising area of research activity, but completion of the studies in a reasonable period of time undoubtedly will require more extensive cooperation and support. Carried to a successful conclusion, such research will broaden the usefulness of the road test findings in application to other highways having different ride characteristics, and to other vehicles with different suspension characteristics.

These are some of the more presently apparent implications of the road test, indicative of the broad application of the findings to decisions required for solution of many problems important to the future of highway transportation.

Conclusion

This, then, is the word picture of the AASHO road test, a majestic research undertaking made possible by the unprecedented cooperation of numerous participating groups and persons. All are in agreement that the test has been soundly conceived and carefully planned, and confident that the conclusions will be objectively reached. A highly competent staff organization has been assembled, the test road has been well constructed, and research is now under way. To those of you who have not seen the project in operation, it is recommended that sometime during the period of test traffic application, you create the opportunity for a visit to the site. To paraphrase an old proverb, one visit is worth a thousand pictures.

DESIGN PROBLEMS OF THE SEATTLE FREEWAY

W. E. McKibben

In the city of Seattle we are designing and, in fact, have started construction on a section of highway that will contain more lanes in one facility than ever has been attempted. Many of the features of this improvement are new to engineers of the Washington State Highway Department. In fact, several facets of this design have never been attempted any place in the United States. The main purpose of this paper is to point out some of these unusual features, explain the reasons for their existence, and our proposed method of solving the problems. For those who are unfamiliar with the Seattle area, I will attempt to preface my remarks with a brief descriptive background to show the need for such improvement.

Seattle, like most large cities, is suffering from acute traffic congestion. While the problem is much the same in urban areas throughout the nation, the traffic problem in Seattle has some unique features not found in other major cities.

The urban area is made up of a series of hills sandwiched between Elliott Bay of Puget Sound on the west and Lake Washington on the east. The elongated and hourglass waistline formed by these two bodies of water is only about two miles across. Within this narrow strip of land is located the central business district. To the south are Seattle's principal industries, including the Boeing plant with its 70,000 employees. A large percentage of the city's population resides north of the central business district. This means, of course, that these people must pass through the narrow corridor to reach the business district or their places of work.

A series of high hills and ridges extending through the area in a northern and southern direction also limits the possible routes for northern and southern vehicular travel. Further aggravating the traffic snarl is a large ship canal running east and west which connects Lake Washington with Puget Sound. All but one of the five existing bridges over this canal have insufficient vertical clearance, and must be opened many times a day to permit passage of water traffic.

Existing U.S. Highway 99 follows a tortuous route through the heart of the city by means of a surface arterial varying from four to six lanes in width. It is augmented by several city arterials, including the double-decked Alaskan Way viaduct which parallels the waterfront, and which is scheduled for completion in 1960.

In 1947, state and city engineers, realizing that something must be done to expedite northern and southern traffic, collaborated in an origin and destination survey. Among other things the survey revealed was that 90 percent of traffic entering the city from the north wanted to make at least one stop in the central business district or the industrial area.

Early planning stages included a facility that would handle anticipated 1965 traffic. Design standards dictated a minimum of six and a maximum of ten lanes. From a study of the desired lines of travel and by preliminary surveys of terrain and other topographic conditions, a feasible and practical route for a major highway was soon selected.

The problem of financing such a facility, however, remained unsolved until a study in 1954 revealed that it was economically feasible to amortize the necessary bonds by toll collection. Accordingly, the 1955 session of the State Legislature passed an enabling act to make this possible. In a subsequent test case before the Supreme Court, however, this act was declared unconstitutional. In the meantime, the Federal Aid Highway Act of 1956 was passed. Since this section of highway is on the interstate system, it appeared feasible to construct the improvement as a free facility.

In April of 1957, a new highway district was created by the State Highway Commission for the purpose of designing and constructing the freeway through urban Seattle.

So much for the historical background; now a brief resume of some of the principal features that are being incorporated into the present planning of the Seattle freeway.

The first job, of course, was to make a completely new traffic analysis and to project it for 1975 volumes in compliance with the standards incorporated in the Federal-Aid Highway Act. Comfortable capacity assignments were made to all possible parallel arterials. The residue was then assigned to the freeway. These volumes varied from 50,000 vehicles a day at the extremities of the urban section to 160,000 vehicles a day in the area immediately north of the central business district. Unfortunately, the portion which will carry the 160,000 vehicles a day has an unbalanced directional distribution of about 4 to 1 during peak hours. This results in a peak-hour volume assignment of nearly 15,000 vehicles in the major direction of flow.

It was never expected that any single highway facility would handle such volumes as this. Normally, other parallel routes would be developed to handle a greater portion of local traffic; probably at a lesser cost per vehicle served. Due to local conditions in Seattle, this was not practical.

It is a basic concept that any highway improvement should be designed to serve traffic demand in an economical manner. To apply this concept, the decision was made to incorporate reversible roadways into the design of the Seattle freeway.

Use of reversible lanes to handle unbalanced traffic flow is not new. It has been used in many instances on surface streets to permit a greater number of lanes in the major direction of travel. The usual regulation is accomplished by signals over the appropriate lanes, or by portable stanchions or barricades. On Aurora Avenue bridge in Seattle, reversible lanes have been used for several years. Traffic is reversed on 9-1/2 foot lanes by means of portable barricades. Volumes in excess of 1500 vehicles per lane per hour are being handled daily, with an accident rate that is comparable to other arterial streets. Operating conditions, however, are entirely unsatisfactory and cost of policing is very high. To my knowledge, not more than two reversible lanes have ever been incorporated into the design of a high-type facility.

During early stages of the Seattle freeway design, it was proposed to improve on this type of operation by actually constructing an independent reversible roadway. This had never been done on a freeway, and the proposal stimulated a great deal of interest in the staff of the Bureau of Public Roads. Conferences were held with the Bureau and a tentative plan was adopted in which there would be two 3-lane unidirectional outer roadways and two 2-lane reversible roadways in the center. This plan required transfer points between the reversible roadways and the outer roadways. Because of the close spacing of the interchange points, the weaving movements required by this plan were an objectionable feature.

The comprehensive traffic analysis made necessary by the change in design criteria indicated that the maximum section could be handled by three 4-lane roadways. By providing a variable cross section with regard to number of through lanes, some of which begin and end at major ramps and some in reversible plan, a close approach was made to a tailored "design-to-fit-traffic."

A major consideration in the plan was width of right-of-way to be taken. The route, as proposed, skirted the eastern edge of the central business district, but required the taking of very few major buildings. The original proposal required a full one-block width in this area for right-of-way, leaving intact the surface streets on either side. Any increase in right-of-way requirements because of change in design would raise the cost of acquisition out of proportion to the benefits gained.

The present design provides additional traffic lanes only where needed and when needed. Traffic analysis indicates that for 16 of the 24 hours in the day four lanes in each direction will adequately handle anticipated traffic volumes in 1975. Throughout a distance of approximately 7-1/2 miles, however, a variable number of additional lanes are required for the other 8 hours. By reversing the flow on these additional lanes, they will carry maximum capacity during both morning and afternoon peak periods. Connections from the unidirectional roadways are provided only at the north and south extremities of the two longest reversible lanes. The other reversibles are terminated at arterial streets as dictated by demand. Direct connections to the reversible roadway from some of the major interchanges also are planned to provide maximum use of the facility.

In effect, this amounts to superimposing an additional freeway in the same right-of-way, and permits connections to a completely different set of streets. By alternating ramp connections between unidirectional lanes and reversible lanes, a much greater use is made of the street system for collection and dispersion of traffic.

Right-of-way in the central business district is costly. This fact, coupled with the need of preserving existing surface streets, dictated the present design wherein reversible lanes are carried under one of the unidirectional roadways. In addition to the savings in right-of-way width, this plan also makes it possible to provide better connections to the streets from the reversible lanes. It has been a basic principle in design of this freeway that ramps and connections to the street system be developed to operate efficiently and smoothly under the assigned traffic load.

One of the major problems has been to provide adequate capacity at the ramp terminals and to determine whether the local street system is capable of handling the assigned volumes beyond the ramp terminals. In this determination a very detailed and complete study of street capacities was made for several blocks in all directions from each interchange. Fortunately, we have found that the present street system is adequate, or can be made so at a nominal cost. Where increased street capacity is demanded, it is possible to provide it by elimination of parking, rephasing of traffic signals, or the introduction of one-way streets.

It has been our goal to maintain the highest possible standards in the design of the Seattle freeway. In general, grades are limited to 3 percent upgrades and 4 percent downgrades, with a maximum plus grade of 3-1/2 percent occurring only at one location south of the central business district. Reversible lanes, of course, will have a maximum plus grade of 4 percent.

Horizontal curvature on the freeway is limited to 3-1/2 degrees. A 60-mile-per-hour design speed is used except for a short section in the downtown area, where vertical curves limit design speed to 50 miles per hour. Medians between separate roadways are laid out to provide 10-foot shoulders on both the right and left, with a positive barrier of the guardrail type in the middle. This will enable all disabled vehicles to clear the travel lanes as rapidly as possible, and thus to cause the least amount of interference to through traffic.

A minimum design speed of 40 miles per hour has been attained for vertical curves on ramps entering or leaving the freeway. The horizontal curvature permits a 35-mile-per-hour design speed except in a few critical locations where it is necessary to reduce to 25 miles per hour. Deceleration lanes are of sufficient length, however, to permit ample time for reduction in speed, so there should be no adverse effect on freeway traffic.

As I have stated previously, use of reversible lanes in freeway design is not commonplace. To my knowledge, only three other states have incorporated them in their design for a particular urban area, and the maximum number of such lanes has been two. Even they have been faced with the problem of controlling on and off movements at the terminal points. Needless to say, these problems are greatly magnified in any plan that incorporates four reversible lanes.

We found in the Seattle design that we encountered three distinctive types of problems. One occurs at the north and south termini of the reversible lanes where one of the unidirectional lanes is transferred to the reversible section. A second type of control problem is the situation that exists where a reversible lane terminates in a couplet to the one-way street system. Still another occurs where the connection is made to a two-way street. The latter is proving to be the most difficult for which to provide a completely satisfactory solution.

We still have a lot of details to be worked out. In most cases it is planned to provide positive control by opening and closing gates to force traffic into the proper pattern. This control will be supplemented by use of stop signals and changeable message signs. Remote control of all gates, changeable message signs, and signals will be possible from a central operational control office. It will be supplemented by manual control at the site to take care of any emergencies that might arise.

I have attempted in this paper to enumerate for you only the unusual features of the Seattle freeway. I can assure you, however, that they do not constitute all of the problems to be solved. The magnitude of the improvement is still somewhat frightening, even to those of us who have been closely associated with it for several years. A list of statistical facts and figures is very boresome, and I certainly am not going to burden you with them here. However, the following highlights will serve to emphasize the problems that we encountered.

To accommodate the 20 miles of freeway in urban Seattle, over 3000 parcels of property, comprising approximately 750 acres, will be acquired. Total estimated cost of the right-of-way is $47,000,000. Seventy-five bridges, estimated to cost $61,000,000, will be required for separation of cross traffic and to span the ship canal. Twelve million cubic yards of earth will have been moved, and 1,600,000 square yards of pavement laid before traffic will use the completed facility. Needless to say, the design and construction of a project of this magnitude, located as it is in a densely populated area, presents a great many problems—problems that probably would be insurmountable if a great deal of time and care had not gone into the planning.

Despite all the care that can be exercised in the design of the freeway and its connections, eventually there will come a day when it becomes inadequate and congested. This may be brought about by changes in driving habits, changes in population centers that could not be anticipated, variances in traffic forecasting, or the accumulation of normal travel increase over a period of years. This has been the history with almost all highways constructed in the past. We have no reason to believe that it will not be the history of freeways at some time in the future. Because of the tremendous sums of money that have been and will be spent to provide for such high-type facilities, it becomes more and more important to see that they will permanently carry out their intended purpose.

To do this means that someday traffic on the freeway at the various ramps must be metered so freeway traffic will operate smoothly and efficiently at optimum volumes. Equipment that will count and control traffic in this manner is being developed and will be in general use before long. It certainly can be expected that such control will be required by the year 1957 on the Seattle freeway, even with construction of 12 lanes.

There is no need, however, to include provisions for such devices in the plan at present as no doubt there will be many improvements and changes in the years to come. In the meantime, city and state engineers are certain that features of this highway, as now being designed, are workable and will provide the best traffic service at the lowest cost for the vehicles served.

SIMPLE SYSTEM OF DETERMINING PRIORITIES OF IMPROVEMENT FOR LOW-TYPE ROADS

John A. Anderson

(The road priority rating method outlined in the following report was adopted by the Marion County Commission. Examples of priority determination and assignment are tabulated to illustrate the method presented.)

April 8, 1958

The Honorable County Court
of Marion County
Courthouse
Salem, Oregon

Gentlemen:

Pursuant to the authorization given this office by the Honorable County Court, I am herewith presenting a report which represents a "Road Rating Program for Marion County for Graveled County Roads."

As part of the background for this report, I should like to state the following:

1. Roads scheduled for improvement during 1958 are a part of this report as to priority listing on Table III. However, most of the roads on the 1958 program have not been analyzed under Table II.

2. Traffic counts have been taken as rapidly as possible, but at the present we have not been able to take a count of all our graveled county roads.

3. Short dead-end county roads were not investigated as a part of this report.

In accordance with the letter submitted to the County Court on April 9, 1957, the following point system has been used in this study:

1.	Average daily traffic count	50 points maximum
2.	Average houses per mile	6 points maximum
3.	Special traffic served	6 points maximum
4.	Farm products hauled	6 points maximum
5.	Connecting road use	6 points maximum
6.	Average land value	5 points maximum
7.	Industry served	5 points maximum
8.	Maximum distance to a paved road	5 points maximum
9.	Cost of maintenance	5 points maximum
10.	Recreational use	5 points maximum
	Total	100 points maximum

As can readily be seen from the above, the most important factor that influences the priority of a graveled road is the average daily traffic count. This, I believe, is quite logical, and it appears that at least 50 percent of the reason for paving any of our graveled county roads should be based upon traffic usage.

To further explain the attached system, each of the above factors has been further reduced to its component parts. For example, in the case of Item 1 the maximum weight of 50 points is allowed for those roads carrying an average of 150 vehicles per day or more. The fewer the number of vehicles using the road, the lower the point value. However, it is quite possible for a relatively "low traffic-count" road to receive a fairly high priority rating if it meets the maximum points allowed for the other factors.

In addition to establishing the basic point system for the priority improvement of our graveled roads, the system has been compiled and studied on an area basis. The following five areas of Marion County have been established in connection with this report. Only in this manner can we properly distribute our improvement funds throughout the entire County.

A. Salem area
B. Woodburn - St. Paul area
C. Silverton area
D. Ankeny - Talbot area
E. Stayton area

Attached hereto is the following report:

Table I.	Detailed breakdown of point distribution system
Table II.	Detailed road ratings based upon actual study
Table III.	Recapitulation of top priority graveled roads for the next 5-year period, 1958 to 1962, inclusive
Figure 1.	County map showing the 5-year priority roads

This Road Rating Report is quite revealing. In the first place, it has been made as simple as possible so that it should be readily understandable by all persons. Secondly, it was done by our own road department personnel (the major credit going to Mr. W. R. Massey), since it was felt that these men were able to better understand our own local road needs than any outside persons.

The really important feature of this study is that it contains the best information that Marion County has with which to program its road improvement work. While some changes may be desirable to meet changing conditions, it is believed this program should be followed as closely as possible. Undoubtedly, portions of this report may be challenged, with cause and without cause, by those residents who will not benefit by it in the near future. I would like to request that the County Court study this report and point out any changes they feel should be made before final release to the public.

In the meantime, I want to assure the County Court that this office will continue to take traffic counts on all of our county roads, so that adjustments can be made to meet changing traffic patterns.

Very truly yours,

John A. Anderson
County Engineer

JAA:ms
Encl.

Table I.
Detailed Breakdown of Point Distribution System
Road Rating Priority - Graveled Roads

Item	Description	Point distrib	Max points
1	Average daily traffic		50
	a) Vehicles per day 0- 49	5	
	b) Vehicles per day 50- 74	10	
	c) Vehicles per day 75- 99	30	
	d) Vehicles per day 100-149	40	
	e) Vehicles per day 150 & over	50	
2	Average number of houses		6
	a) Houses per mile 0- 2	1	
	b) Houses per mile 3- 5	2	
	c) Houses per mile 6-10	4	
	d) Houses per mile 11 & over	6	
3	Special traffic served		6
	a) School bus, mail, or logging	2	
	b) School bus & mail route	4	
	c) School bus, mail, & logging	6	
4	Farm products (tons/mile/year)		6
	a) 0 - 200	1	
	b) 200 - 400	2	
	c) 400 - 800	4	
	d) 1000 - over	6	
5	Connecting road use		6
	a) County road connection	2	
	b) Market road connection	4	
	c) State highway connection	6	
6	Average land value per acre		6
	a) $ 0 - 200	1	
	b) 200 - 300	2	
	c) 300 - 400	4	
	d) 400 - over	6	
7	Industry served		5
	a) 1 - 25 employees	2	
	b) 25 - 100 employees	4	
	c) 100 & over	5	
8	Max distance to paved roads		5
	a) 1/2 mile	1	
	b) 1 mile	2	
	c) 1-1/2 miles	3	
	d) 2 miles	5	

Table I (continued)

Item	Description	Point distrib	Max points
9	Cost of gravel maintenance		5
	a) Easily maintained	2	
	b) Moderately hard to maintain	3	
	c) Difficult to maintain	5	
10	Recreational benefit		5
	a) Minor recreational use	2	
	b) Major recreational use	5	
	Total points allowable		100

Table II.
Area D

Co. road no.	From To	Traffic rating	Houses per mile	Special traffic served	Farm produce hauled	Connecting roads used	Average land value	Industry served	Max distance from paved road	Cost of gravel maintenance	Recreational use	TOTAL	PRIORITY
		50 max	6 max	6 max	6 max	6 max	6 max	5 max	5 max	5 max	5 max		
MR 83	MR 22 - 956	5*	1	4	2	4	2	0	2	3	0	23*	16
904	MR 39 - 907	10*	4	4	2	2	2	0	1	3	0	28*	6
906	MR 53 - 504	10*	2	6	4	4	2	0	5	3	0	36*	7
909	MR 39 - MR 54	10	1	0	4	4	2	0	1	3	0	20*	8
917	MR 41 - MR 25	5	4	4	1	4	1	0	3	3	0	25*	11
919	Wipper Hill	5*	2	4	1	2	1	0	5	5	0	25*	12
920	Old 99E-929	5*	4	4	2	2	2	0	1	3	0	23*	17
921	922 - 924	5*	1	4	2	2	2	0	1	3	0	20*	19
922	MR 35 - 920	10*	2	2	2	2	2	0	2	2	0	24*	14
924	MR 35 - 937	10*	4	4	2	4	2	0	3	3	0	32*	15
924	MR 41 - 843	10*	2	4	1	2	1	0	2	3	0	25*	13
927	929 - Turner	5*	1	2	1	2	1	0	3	3	0	18*	20
928	MR 3 - MR 28	40	4	4	2	4	1	0	2	5	2	64	3
930	924 - 929	5*	4	4	1	2	1	0	2	3	0	22*	18
938	MR 40-67-87	5*	4	4	1	4	1	0	2	3	0	24*	14
940	924 - 3 - 938	5*	1	2	1	4	1	0	2	3	0	19*	9
942	MR 87 - MR 3	5*	4	4	2	4	2	0	2	3	0	26*	10

Table III.
Top Priority Graveled Roads for 5-Year Period, 1958-1962

Road	Priority	From	To	1958	1959	1960	1961	1962
924	1	927	940	1.5				
945	2	Loop road		1.5				
		Total for 1958		3.0				
928	3	MR 3	MR 28		2.5			
944	4	MR 35	E 0.5		0.5			
			Total for 1959		3.0			
924	5	940	921			2.7		
			Total for 1960			2.7		
904	6	MR 39	907				1.0	
906	7	MR 54	W 2.0				2.0	
			Total for 1961				3.0	
909	8	MR 39	MR 54					1.0
940	9	MR 3	924					2.0
			Total for 1962					3.0

OREGON STATE HIGHWAY DEPARTMENT'S RECENT EXPERIENCE IN DESIGN AND CONSTRUCTION OF PORTLAND CEMENT CONCRETE PAVEMENT

G. W. Harra

A contract for placing 9.12 miles of reinforced Portland cement was completed during the 1958 construction season. This section of highway is described as the "North Jefferson Junction - North Albany Junction Unit of the Pacific Highway Interregional." Excluding several structures, some 133,000 square yards of pavement were placed. This pavement parallels the old existing concrete slab which was placed in 1947 and, incidentally, until this time was the last concrete job of any size contracted by the Oregon State Highway Department. The new pavement is now functioning as the southbound lane of the divided 4-lane modernization of this section.

Plans and special provisions for this concrete paving contract called for using 8-inch steel forms and placing in two 12-foot lanes with a tongue-and-groove type of longitudinal joint. Transverse 5/8-inch tie bars 36 inches in length and placed at 40-inch intervals in the joint were specified. The design called for reinforcement with 1/4-inch diameter welded wire fabric, with a wire spacing of 6 inches by 12 inches and an overall mat size of 11 feet 6 inches by 16 feet. Four mats were to be placed in each panel and lapped for 12 inches, making a total reinforcement length of 61 feet. Spacing between each 4 such placed mats was 6 inches, making a total panel length of 61-1/2 feet. Cracking was to be controlled in this 6-inch area by sawing.

This method of making contraction joints was decided upon as being most desirable from the standpoint of smoothness, so it was specified that a cut of 2-inch depth be made between panels. Mats were to be placed 2-1/2 inches from the pavement surface, and with the contractor's available equipment, this necessitated placing the concrete in 2 lifts of 5-1/2 inches and 2-1/2 inches. Twelve 1-inch load transfer dowels of 18-inch lengths were to be placed at each 61-1/2 foot panel spacing at the end of each day's pour.

To function properly in load transfer, it is very important that the dowels be maintained in both a true horizontal and longitudinal plane. This was accomplished by use of wire chair-type holders. No dowel sleeves were used, as a coating of light grease on the dowels was considered adequate. The panel length of 61-1/2 feet with load transfer dowels was considered a balanced design.

However, longer lengths with heavier wires, and thus fewer joints and total number of dowels, or shorter lengths with lighter wire and more dowels could have been employed.

Another type of reinforcement that is being used experimentally in some states is that of continuous reinforcement. This calls for the use of much more steel—normally from 5 to 7 percent of steel relative to the cross-sectional area of the slab. Amount of steel determines width and frequency of cracks that will form in the concrete. It will be interesting in the future to appraise the results of these experiments.

Further provisions of the contract called for 3/4-inch preformed expansion joints at all structures, and 20 feet of bar mat reinforcement adjacent to all structures. These sections of pavement were to be increased to 12 inches in thickness. All surface was to be transversely broomed and covered with a membrane-type curing compound.

Grading this section was let previously under two separate grading contracts. One section (North Jefferson Junction - SPRR Overcrossing) called for placing 18 inches of so-called subgrade reinforcement prior to placing the crushed gravel base. Material for the 18-inch subgrade reinforcement was bar-run Santiam river sand and gravel, except at the north end of the section where quarry-run rock was placed. In some sections fine sand was blended with the sand and gravel to improve stability. This was well compacted and surface treated with a grid roller as a means of breaking down excessively large aggregate. A densely compacted mat was obtained that compared favorably with many of our regular base courses. This became a part of the design because of the readily available material and, at the time of contracting, it was anticipated that the surfacing pavement would be 4 inches of the flexible type.

The selected borrow or subgrade reinforcement was topped with 9 inches of crushed gravel base rock, which was included as a part of the paving contract. Eleven inches had been considered originally, but because of the quality of the 18 inches of borrow, it was cut to 9 inches. The other grading contract (SPRR Overcrossing - North Albany Junction) did not incorporate the use of any selected borrow. However, the crushed gravel base was increased to 12 inches in thickness for this section.

The crushed base-course surfacing was contracted under optional provisions as to maximum size. Either 2-1/2 to 0 inches, 2 to 0 inches, or 1-1/2 to 0 inches could be produced at the option of the contractor. This provision enabled the contractor to produce a size that would fit in most economically in the simultaneous production of the several other sizes of materials required in the contract.

The contractor elected to produce 1-1/2 to 0 inches of crushed base material. During its production it was possible to produce at the same time three separate sizes of coarse aggregate and one of fine aggregate for the Portland cement concrete pavement, and two sizes of aggregate for 43,000 tons of asphaltic concrete required on the job. If asphaltic concrete aggregate was not being produced, the two sizes were combined to make 3/4 to 0 inch for shoulder dressing and oiling aggregate. This simultaneous production of aggregates resulted in a very efficient operation, and it was not uncommon when 500 cubic yards of total material were produced in an hour.

Compacting the base course was accomplished by use of both pneumatic-tired and steel-wheeled rollers, with a final densification by two tractor-towed vibratory compactors. An excellent, firm, and well compacted granular mat was the result.

Because of the use of 1-1/2 to 0 inch base material, rather than a coarser size, a very small amount of 3/4- to 0-inch material was required for surfacing chinking, or to bring the final surface to grade.

A cushion course of 3/8- to 0-inch material was placed, dampened, and compacted between base course and concrete pavement. Both natural 3/8- to 0-inch sand and 3/8- to 0-inch crushed material were used in this phase of the work; each proving to be satisfactory.

Two methods of application of cushion material were tried. One was spreading and compacting material before placing the forms, and the other was spreading and compacting material subsequent to placing the forms. The latter method was used almost entirely throughout the job as less material was required and it was found that larger crushed-base material compacted better beneath the forms than the finer cushion material.

Only a bare minimum of material for complete coverage and true grade was used. Proper grade was accomplished by the use of a form-mounted strikeoff blade equipped with sharp-pointed scratch or depth gages. The strikeoff blade was towed by trucks used in hauling and placing the cushion material.

Since tongue-and-groove longitudinal joints were specified, it was necessary to bolt 1-inch thick wood strips of the proper width, and bevel them to the inside of the center steel form. Thus, the slab having the horizontal groove had to be placed first. The forms were fastened with steel pins driven into the base rock and aligned with steel wedges for surface regularity. Pins holding the forms in place were driven by a pneumatic hammer operated by a tractor-mounted compressor. The forms were brought to final grade, and of sufficient rigidity to support the concreting equipment, by a form-mounted, gasoline-operated, horizontal tamping device. This machine was equipped with an exhaust-accentuated oil sprayer for coating the forms. This phase of the operation was soon terminated, however, as more oil was applied to the operator's shoes than to the forms. A more prosaic method of applying the oil coating was made by the use of a hand-operated, garden-type, pressure sprayer.

Actual proportioning of the aggregates comprising the concrete mixture was based on a standard specification of the Highway Department. Because of the availability of aggregates, and the historical record of the concrete job placed in 1947, it was decided to specify an aggregate maximum size of 2-1/2 inches. The Oregon specification for 2-1/2 inch maximum size concrete aggregate requires separation of the coarse aggregate into three sizes. This separation was made into sizes of 2-1/2 to 1-1/2 inches, 1-1/2 to 3/4 inches, and 3/4- to 1/4-inch natural gravel, with the exception that about 20 percent of the 2-1/2 to 1-1/2 inch was crushed gravel. The fine aggregate was 1/4- to 0-inch natural sand, meeting our specified grading requirement.

The four sizes of aggregates were separately stockpiled and later transported by two front-end loaders to two receiving bins for transfer to the batching plant during concreting operations. Two belt conveyors carried the four sizes of material to the plant from these ground level bins. Swinging, trough-shaped gates at the upper end of the conveyors directed discharge of aggregates into the plant bin for that particular size of material. This operation was controlled by a workman stationed in a crow's-nest at the top of the batching plant. Signals from the workman indicated to the loader operators the size of aggregate to transport to the conveyor bins as needed to keep the plant batching bins sufficiently filled.

Several field gradation analyses, as well as central laboratory tests were made daily during production of the aggregates to determine the best proportioning for a well graded and uniform concrete mixture. Test records of the 1947 job and results of laboratory compression tests of concrete specimens made from the aggregate being produced, indicated that 1.4 barrels or 5.6 sacks per cubic yard of concrete would be sufficient for a compressive strength requirement of 3300 pounds per square inch at 28 days. The bid basis for cement content was based on the use of 1.45 barrels per cubic yard of concrete. By using only 1.40 barrels in the designed mix, a job saving of approximately 1500 barrels was made by the department.

Actual job averages of compression tests made on 6- by 12-inch cylinders were 2574 pounds per square inch at 7 days, and 3592 pounds at 28 days. A set of 4 cylinders was cast for each 1000 linear feet of 12-foot concrete placed.

The Johnson batching plant was equipped to weigh simultaneously 4 sizes of aggregate and the required amount of cement per batch. The operation was fully automatic, thus resulting in a high degree of uniformity from batch to batch. Also, consistent uniformity of various aggregate sizes was such that no change was made in the proportioning ratio or batch weights, other than the necessary correction for variable moisture content or when a richer mix was required for early opening to traffic at temporary crossings.

By weighing 5 materials simultaneously, another decided benefit was the premixed characteristics of each batch deposited in the trucks. Three batches were hauled by each truck, and the separating batch gates were of sufficient height that no spillage occurred from one compartment to another when dumping in the mixer skip.

The batching setup was quite centrally located jobwise, and placement of the concrete paving was made in either direction from this site. All machines making up the concreting equipment train were new and, consequently, in excellent condition. All except the mixer were self-propelled and traveled on the forms. First in line was a dual drum Rex 34E paver. Batch sizes for the paver were started at a volume of 37.5 cubic feet. It was found that this quantity could be increased to 1-1/2 cubic yards and still maintain a uniform consistency within the 60-second mixing time specified. However, when mixing this greater quantity was first tried, some difficulty was encountered in maintaining the same slump or consistency. It was noted by observation that the drier batches of concrete were larger than the wetter ones, so it was concluded that the first drum was not always completely discharging into the second drum. This condition was readily corrected by increasing the batch transfer or penalty time from 11 seconds, the original time setting, to 14 seconds.

A solution of an air entraining agent was automatically dispensed into each batch by a device attached to the mixer. Quantity of mixing water also was automatically measured at the mixer. Quantity of air entrainment added was such that would produce from 3 to 4 percent entrained air in the concrete mixture. Amount of mixing water normally used was sufficient to produce a slump from 1-1/2 to 2 inches. This approximated 5-1/4 gallons per sack of cement, or about a water-cement ratio of 0.7 by volume. Control of these quantities was made by a department inspector stationed at the mixer.

Mixer and batch trucks traveled on the shoulder area during placement of the first lane, and on the first-laid lane during placement of the second one. Protection for the sawed joints was made by covering them with pieces of used conveyor belting. Pouring of the second lane began 21 days after starting placement of the first lane.

The mixer was followed by a Blaw Knox spreader. The oscillating spreading device operated between the forms and was adjustable for varying the height of spread. A vibrating bar mounted on the rear end and adjustable to height was a part of this equipment. Concrete was placed in 2 lifts; the first being at a depth of 5-1/2 inches. The specified wire mesh was then placed and covered with the second lift of 2-1/2 inches. Distance of travel before placing the second lift was usually 2 or 3, 61-1/2 foot panels.

Following the Blaw Knox spreader was a Rex finisher equipped with 2 horizontal strikeoff screeds. Screeding action brought to the pavement surface sufficient mortar for the proper operation of the Johnson float, which followed.

The Johnson machine was a long wheel-based final finisher equipped with a series of diagonally mounted mahogany shod floats. This finishing machine was fully reversible in operation, and as many passes as necessary were made to produce a proper surface smoothness. This is considered an important piece of equipment as the final surface smoothness depends upon its proper operation.

Following the Johnson finisher was a Rex traveling broom and curing membrane spreader. Two wire brooms were mounted on a continuous transverse belt at the front of the machine, and the spraying device was operated transversely at the rear. Because of the forward travel of the machine, the broom marks produced were not at a right angle to the pavement edge, but on a slight diagonal.

Because this was the first job placed under the previously mentioned specifications and the first concrete pavement job by the contractor in several years, it was expected that various problems would occur.

The first problem was that of training men in the operation of equipment assigned to them. None of the men had operated any of the particular pieces of equipment required on the job. All, however, were regular employees of the contractor, who had satisfactorily operated other pieces of equipment. His judgment in selecting his crew was good. By the second day of operation they were familiar with necessary mechanical adjustments, and they continued to operate in an expert manner throughout the job. Normally, placement of concrete approximated 300 linear feet per hour.

When forms were stripped from the first day's run, a great deal of spalling occurred in the upper section of the grooved edge. The weather was hot and dry, and the upper portion of the keyway adjacent to the steel form had quickly lost moisture, resulting in a very poor curing characteristic. It was noted also that concrete in this area was low in mortar content and high in coarse aggregate. By observing operation of the Blaw Knox spreader, it was found that coarse aggregate of about 1-1/2 inch size was being brought above the surface of the first lift at each end of the vibrating bar. This produced a very harsh concrete in this limited area when the second lift was placed. This condition was corrected by having the laborers who placed the steel mesh shovel the segregated gravel into the center of the slab. This correction greatly improved the spalling condition, but the poor curing condition next to the hot metal form resulted in sufficient spalling to discontinue use of the grooved joint. In the balance and greater portion of the job a plain butt joint was employed. In this section, spacing of tie bars was reduced from 40 inches to 20 inches.

The most perplexing problem we encountered was in sawing joints at the weakened or nonreinforced areas. Preliminary studies indicated that other states and governmental agencies had been successful in this method of controlled cracking, but we immediately experienced trouble.

The weather was unseasonably hot during most of the daytime working period, which accelerated setting of the concrete. Temperature change from hot daytime to cool evenings and nights frequently was 40 degrees or more. Hardening of the concrete continued during the night, and was accompanied by a lowering of the temperature. These two conditions acted to produce a tension stress sufficient to produce premature cracking in the weakened section. This cracking sometimes took place before sawing operations began, but more often it occurred in the unsawed portion of a partially sawed slab.

A means of correcting this undesirable cracking seemed to be that of earlier sawing, but because of the very hard quality of the aggregate, its displacement took place if the concrete had not quite set firmly. This resulted in excessive spalling along the saw cut, and also extensively damaged the saw blades. Sawing was discontinued, and 2-inch preformed contraction joint material was used as a substitute.

Facilitation in placing these strips was accomplished by use of a shop-made vibrating blade as a means of displacing the least possible amount of aggregate in order to aid in inserting and maintaining them in a vertical position. Placing was prior to the final pass of the Johnson finisher.

A solution of the sawing problem could have been made by paving at night and sawing during the day. However, this was not deemed entirely practicable under the conditions prevailing.

A minor problem occurred in the brooming operation. The brooming device, as received, came equipped with soft fiber brooms that did not sufficiently groove the pavement surface. Wire brooms were substituted, and their immediate effect was to produce a gouged-out section about 6 inches in width next to the outside form. This was corrected by lengthening the distance of horizontal travel so that the upward motion started only after a complete pass across the full width of the panel was made.

Another problem of a minor nature was that occasionally a float of the Johnson finisher would catch the end of a piece of wire fabric and drag it to the surface. The thought was that the large sized aggregate was being dragged by the finishing operation to the extent that the mats were being displaced. An attempt was made to correct this by reducing the thickness of the first lift of concrete from 5-1/2 inches to 5 inches, thus placing the steel 3 inches below the surface. This did not materially change the condition, so a crimping tool was devised to fasten together the ends of the mats. This eliminated the trouble and the mesh reinforcements were again placed according to plan.

One other thing to consider as quite important in placing concrete pavement is the control of concrete quantity relative to the form height. Obviously, an insufficient quantity would result in delay while additional material was being placed. If the concrete is carried too high, some will be wasted by being carried over the forms by the screeding equipment. This did not become an engineering problem on this job because the contractor reacted properly and immediately to this situation.

Even with the problems encountered and one condition not resolved properly, we believe that a creditable section of concrete pavement has been placed as a valuable addition to the Oregon highway system.

CONSTRUCTION PROBLEMS AT THE PORT OF SEATTLE

Robert J. Laughlin
Supervising Construction Engineer

You are probably wondering what in the world an engineer of the Port of Seattle is doing at a highway engineers' conference. One of the reasons I am here is because we have major construction problems that are identical to those you have in highway construction.

Some of your problems are probably due to the overloading of a big auto freighter, or maybe an oil tanker has a little too much gasoline in it, etc. If you stop to realize, we have a problem a little more critical than that at the Seattle-Tacoma Airport. When one of the big DC-6's or DC-7's drops in out of the sky and lands on our pavement, we have almost as much of a problem as when some of your auto transports put on the brakes or try to stop in a hurry when overloaded.

One problem is that of sawing construction joints to get a nice smooth joint. We usually did get a very smooth joint, but we had the same difficulty in preliminary cracking that Mr. Harra mentioned. I made some inquiries as to whether anybody else was having that problem, and found that the Washington State Highway Department was having somewhat the same trouble. The difficulty was in timing the sawing of the joints. Just when should you saw the joints?

The aggregate that we used was from Steilacoom, and it is a very, very hard aggregate. If we got in too early and started to saw the joints, we had spalling. As a result, the joints instead of being the required width sometimes became two or three times this width. This meant we had to put in more asphalt and, in addition to more asphalt, there was the possibility of making that joint a little rough.

Another difficulty we had was with the problem of blades that Mr. Harra mentioned. We were using a fabulous number of blades in sawing the joints, especially if we started sawing a little bit early. However, if we waited until later, we would start sawing and have everything going fine when all of a sudden we would get cracks in locations other than in those we wanted, or we would have the same problem Mr. Harra mentioned. We would start sawing a joint and it would pop across in front of us. Instead of a nice straight joint, we would have one that looked like a lightning streak, and which could not be completely sawed and filled with asphalt.

There is another problem that I would like to have discussed. We used wooden forms, so we did not have a particular problem with them and joint sawing, but inasmuch as we do have some more of this work coming up, and some of the contractors prefer steel forms, I was wondering about this. When you saw the joints and come over to the edge of the strip, do you stop before you get to the edge, or just what do you do in regard to this steel joint?

Answer. You stay back a little way from the steel joint.

Mr. Laughlin. I understand you have had this same problem in Oregon in regard to the sawing of joints, and also that the Corps of Engineers has encountered it. I have one thought, and that is perhaps if we would use a Kelly ball to determine hardness of concrete and to predict speed of setting, it might be of some aid. That is about the only practical solution I have had.

Another of our problems is that on some days our production is not as great as on other days. At other times we are producing the same amount of concrete, but it is very hot, so we have to get in and start sawing joints earlier. Other days, for no reason we can predict, it seems to start cracking. One time we produced concrete continuously for two weeks without a single crack, then all of a sudden something happened and it changed. We could not find out why.

Do we have some engineers here who have had experience with sawed joints, and who have arrived at a solution as to when to saw them? Even though we knew in some cases when they should be sawed, we still were confronted with the problem of having only one multiple gangsaw there, and could saw only a certain number of joints. In some cases it appeared that perhaps we had produced too much concrete.

Question. Did you consider the Army Engineers' practice of putting in preformed joint material, and then sawing it out afterward?

Mr. Laughlin. No, we didn't, because we felt that by using preformed material there is a tendency to build a little ridge on either side. It may be that there are some types of preformed joints that do not do this. For instance, I understand that now there is a vibrating form that will vibrate the joint area, and later you can come back in and saw without cutting aggregate particles.

I think you all realize that our problem is much more acute in a runway than it would be in a highway, because it is very important that we get a very level smooth surface. When those planes come in, they need something pretty smooth to land on.

Question. Your concrete was reinforced, wasn't it?

Mr. Laughlin. In some cases. About the only place we used reinforcing was where we were crossing an existing utility line (a water line or electrical conduits, something of that nature), but in general it was a 12-inch slab unreinforced.

Question. Twenty-four foot width?

Mr, Laughlin. Twenty-five foot panels were used generally. We have 6 lanes 25 feet wide. Our taxiways were a little different width than the runways, but out biggest problem was in the runways.

MODERN DEVELOPMENTS AND EQUIPMENT FOR CONCRETE HIGHWAY PAVING

Gordon K. Ray

Modern concrete technology is a glamorous topic. You probably are envisioning thin, prestressed slabs or new superstrength, quick-hardening concrete. I am afraid I will have to disappoint you somewhat, as I want to emphasize many of the same basic fundamentals; i.e., good specifications, diligent inspection, and careful construction.

We are being advised constantly by engineers, contractors, equipment manufacturers, and materials producers, of new developments in construction of concrete pavement, some of which are quite revolutionary. Not all new developments necessarily represent progress, however. We have developed our own yardstick for evaluating new construction developments reported in the field. If these new developments result in a better, smoother, or more economical pavement, then we consider them as a positive addition to modern concrete technology. New gimmicks, machines, or techniques that do not result in a better or more economical pavement are not rated as progress.

The big news in paving today is mechanization. In an effort to build a smoother riding, more durable pavement of uniform high-quality concrete, and in order to increase the rate of pavement production to accomplish greater economy, contractors have resorted to more extensive use of machines for pavement construction. Replacement of men by machines, and the constant reduction in number of machines required to produce a finished pavement, generally result in a smoother riding, more economical pavement of uniform high quality.

Requirements for semiautomatic or full automatic batch plans and improved methods of materials handling are resulting in more uniform batches of generally higher quality concrete. With this type of equipment, the specified amount of fine aggregate, coarse aggregate, and cement for each batch is weighed and recorded at the push of one button. Some of the larger aggregate producers are now using newly developed aggregate beneficiation equipment to remove unsound particles of coarse aggregate. Others are using electronic methods for making continual observations of moisture content of fine aggregate. Some contractors are using semiautomatic batch plants that discharge as many as four batches into batch trucks at the push of a button by the batch-truck driver.

On large highway and airfield projects, many paving contractors are now employing central mix plants with two large mixers to provide a continual flow of concrete to agitating or nonagitating type trucks.

One equipment manufacturer this past summer produced a special new spreader for handling concrete transported by bathtub type trucks from the central mix plant. This spreader resulted in a more uniform distribution of concrete than had been possible in the past from central plant mix concrete deposited on the grade by conventional chutes from trucks outside of the forms.

Construction of subgrade and subbase also has made rapid progress through the development of vibratory equipment for the compaction of granular or nonplastic type materials. The increasing use of large supercompactors for proof-rolling subgrade and subbases is resulting in the location and subsequent correction of local soft spots, which otherwise would affect pavement performance at these locations. Large steel-tired vibratory compactors and the small multiple-plate vibrators have proved to be very satisfactory for compaction of granular subbase for concrete pavement.

Other contractors are combining two types of rollers to accomplish better compaction with fewer passes. One manufacturer is experimenting with combinations of two types of rollers on one machine.

Another new "wrinkle" in subbase construction has been the increasing use of pugmill mixers for controlling gradation and adding proper moisture control to granular subbase materials. Spread by a spreader box, these subbases have resulted in reduced costs through reduction in equipment for mixing, watering, and compacting on the grade.

In California, where extensive use was first made of cement-treated subbases under concrete, it has been the practice to build concrete in 12-foot lanes in order to permit cement stabilization with the use of one pass machine which rode on the side forms. This past summer, one contractor used a new machine which scarified and windrowed subbase in one pass for a 24-foot pavement. This was followed by two passes of a conventional 12-foot, one-pass stabilizer.

Construction of smoother riding pavements, and elimination of one piece of equipment from the paving train have been accomplished by two different equipment manufacturers who have produced long wheelbase finishing machines that perform finishing functions previously handled by transverse finishing machines and the mechanical longitudinal float.

One of these machines consists of two elements: a special leveling device which is attached to the rear of the conventional spreader to accomplish the first screeding operation, and a second element which is a 30-foot long wheelbase machine with two screeds, one centrally located and suspended so that it operates independently of the side forms. This is followed by a 16-foot long, chevron-shaped, longitudinal float that removes any irregularities left in the surface by the transverse oscillating screeds.

The other machine, which has a 16-foot wheelbase, employs two conventional transverse screeds and a 36-inch wide float that also is supported from the machine in such a manner that it is not affected by any vertical deviations in the side forms. This float is pulled forward over the slab with no transverse or vibratory motion.

Both of these machines have been widely acclaimed by contractors and engineers for the excellent riding characterisitics of the surface and close adherence to specified surface tolerances.

Construction of pavement to more strict surface tolerances also has been made possible by the use of rolling straight edges that can be used on the forms prior to paving to check the vertical alignment and, later on, to check the finished slab to make certain that the concrete as constructed does comply with specified tolerances.

Compliance with surface tolerances is being more rigidly enforced by engineers of both airfield and highway construction agencies because of the requirement that deviations in excess of the specified limit must be corrected by removing the slab or grinding the high spots.

The long wheelbase, diamond bump cutter developed by a West Coast saw manufacturer has proved to be an excellent machine for removing bumps from pavements and bridge decks. This machine can be set to check the finished surface, and also to cut any bumps that exceed the specification limits. Surface left by this bump cutter has adequate texture to prevent skidding, and gives the appearance of a uniform surface texture.

Control of the concrete itself to insure uniform high quality from batch to batch has been improved through development and increased use of the Kelly ball, which permits more frequent and rapid testing of concrete consistency. Many engineers find this to be a big improvement over the old familiar slump cone.

The Chace air indicator, a pocket device for determining air content of mortar in concrete, has permitted inspectors and engineers to make more frequent quick checks to determine whether or not air content is remaining within the specified limits. While not as accurate as a large pressure meter, the Chace air indicator does provide for more frequent tests and, therefore, more uniform control of air-entrained concrete. Large variations in the percentage of entrained air can be readily and quickly determined if frequent checks are made by the engineer.

The joints in concrete have always been one of the main sources of trouble, both during construction and after the pavement is open to traffic. The increasing use of sawed joints and improvements in concrete saws and saw blades have simplified construction operations and resulted in smoother riding, more durable joints if the concrete is sawed at the proper time.

In some areas where especially hard aggregates are used, sawing of transverse contraction joints is quite expensive due to excessive blade wear. Difficulty sometimes is experienced in sawing concrete with such hard aggregates before cracking develops. Use of special insert type joints that can be removed later by sawing is spreading rapidly where hard, coarse aggregates are prevalent.

These inserts, generally, have been premolded, bituminous, impregnated fiber boards or paraffin treated corrugated cardboard. Strips of these materials, about 1/4 inch wide and 2 to 2-1/2 inches in depth, are inserted in the plastic concrete groove by a vibrating T-bar at the proper location and after the passage of the last mechanical finishing machine. The inserts, which act as positive crack control, must be carefully aligned in order to permit sawing. Surface of the slab over the joint must be finished with a long float or scraping straightedge to eliminate any bumps or depressions created during insertion of the vibrating cutter.

These materials simplify curing of the joint, and since they will control cracking at the joint location, it is not necessary to saw out the insert until the contractor is ready to seal the joint. If a bituminous, impregnated fiber is used, usually only the top 1 to 1-1/2 inches is removed for sealing. By using one or two saw blades with a total width slightly greater than the width of insert, the contractor makes certain that all paper is removed from sides of joints.

In some areas where plain concrete without mesh or dowels is used, engineers are now specifying diagonal joints to reduce noise and impact. With joints angled 2 feet in a 12-foot lane, no two wheels cross the joint at the same time. In Washington, where pavements are built one lane at a time, they use the chevron-shaped pattern.

The dream of all contractors and engineers has long been that of a single machine that would mix, place, finish, and cure the concrete in one pass. While such a machine is probably still several years in the future, development of the slip-form paving machine, which has been widely acclaimed by many engineers, has been a step in the right direction. This machine, which recently has been used for the construction of many miles of highway pavement in Iowa, Colorado, and other midwestern states, shows promise of greatly reducing pavement costs and building a smoother pavement, with less manpower and without the use of side forms.

I have touched on just a few of the many new developments in concrete paving technology that are taking place in the construction field. Many of these may be dropped by the wayside, but I am sure that others will turn out to be improvements that will benefit all contractors and engineers. We are continually looking for new ideas, new techniques, and new equipment to speed up and improve paving methods. We welcome your ideas and comments, and hope that all engineers and contractors will continually seek better, faster, and more economical methods for building durable, safe, smooth riding concrete pavement.

SOME EFFECTS OF MIXING TIME AND BATCH WEIGHTS ON THE QUALITY OF PAVING CONCRETE

Carl E. Minor

Background

Cost of concrete paving is dependent to a large degree on productivity of the paver. Paver productivity, in turn, is a function of mixing time, batch size, time required to back up the batch truck to the skip, time required to dump batch into the skip and get the truck out, and the interval required for the paver operator to react after the batch truck clears the skip. The first two items are governed entirely by the engineer's specifications, while the last three are controlled by the efficiency of the contractor's crew and equipment.

Quantity of Concrete Produced with Various Mixing Times and Batch Weights

Dual drum pavers can produce a batch of concrete mixed 60 seconds in about 40 seconds. The mechanism of a dual drum paver is such that reducing the required mixing time reduces the paver cycle by only 1/2 of the amount by which the mixing time is lowered. Thus, a 50-second batch can be mixed in about 35 seconds. During the paving cycle, the skip must be raised, emptied, lowered, and reloaded. An efficient crew can accomplish these tasks in 35 seconds.

Specifications for mixing time for dual drum pavers range presently from as high as 75 seconds to as low as 50 seconds. Allowable batch sizes range from 0 percent overload to 20 percent overload, or from 34 cubic feet to 40.8 cubic feet for a 34E+10% paver. Assuming that the above paver cycles are maintained uniformly, quantity of concrete that can be produced in 8 hours in a 34E+10% paver with different batch weights and mixing times varies as follows:

Mixing time Seconds	Overload %	Cubic yards	Sq yd of 9-in. pavement	Footage of 12 ft × 9 in. pavement
60	10	1000	4000	3000
60	20	1085	4340	3250
50	10	1140	4560	3420
50	20	1250	5000	3750

Thus, a reduction in mixing time from 60 seconds to 50 seconds with the same batch weights increases production 14 percent, while a companion increase in batch size from 37.4 to 40.8 cubic feet will mean a 25 percent increase in production.

Put in another light, assuming a contract price of $4 per square yard for pavement in place, the gross daily return to the contractor would range from a low of $16,000 to a high of $20,000 at the above hypothetical production rates. While no paving operation will be 100 percent efficient, the above figures indicate that mixing time or batch weight specifications that are unnecessarily restrictive will cost the owner important money.

Washington's Test Project

Realizing that mixing time and batch weight specifications were the vital factors in production of concrete paving, we decided, at the request of the Bureau of Public Roads, to participate in a cooperative study on an actual paving project.

Our studies were conducted on the inside or passing lanes of a new 4-lane urban freeway through the city of Olympia. The outer lanes had been paved previously and were open to construction traffic. The pavement was 9 inches thick, nonreinforced, with contraction joints at 15-foot centers and expansion joints only at the ends of structures.

DESCRIPTION OF TESTS

Scope of Tests

Since our standard specifications require a 60-second total mixing period and allow a 10 percent overload of the mixer, and since we had no desire to either increase mixing time or reduce allowable overload, it was determined that the study would be limited to mixing times of 45 and 60 seconds and overloads of 10 and 20 percent. A minimum mixing period of 45 seconds was used instead of the 30-second period suggested by the Bureau of Public Roads, because it was feared that 30 seconds was a dangerously low period, particularly since the concrete pavement was being placed on a portion of a new urban freeway in the interstate highway system.

Variables Studied

Tests were performed on concrete mixed according to the following schedules:

Series 1 - 60-second mixing time - 10 percent overload

Series 2 - 45-second mixing time - 10 percent overload

Series 3 - 60-second mixing time - 20 percent overload

Series 4 - 45-second mixing time - 20 percent overload

Six batches from approximately 100 batches of concrete mixed in accordance with each of the above schedules were sampled.

Sampling and Testing Schedule

Three samples weighing approximately 150 pounds each and representing roughly the first, middle, and last third of the batch were obtained from batches 10, 20, 40, 50, 70, and 80 of each of the 100-batch test runs.

The following tests were performed on each of the four series:

Batch & sample no.	1A, 4A, 7A	1B, 4B, 7B	1C, 4C, 7C	Total
Slump cone test	1 1 1	1 1 1	1 1 1	9
6×12 cylinders for compressive strength test	3 3 3	3 3 3	3 3 3	27
Chace air test	Remainder of 3 samples combined & 1 test made	Remainder of 3 samples combined & 1 test made	Remainder of 3 samples combined & 1 test made	
Pressure air test	Remainder of 3 samples combined & 1 test made	Remainder of 3 samples combined & 1 test made	Remainder of 3 samples combined & 1 test made	3
Batch & sample no.	**2A, 5A, 8A**	**2B, 5B, 8B**	**2C, 5C, 8C**	**Total**
Kelly ball consistency	1 1 1	1 1 1	1 1 1	9
Fresh unit wt test	1 1 1	1 1 1	1 1 1	9
*6×6×36 in. beams for flexural strength	- 2 -	- 2 -	- 2 -	6

*Two 6×6×36 inch beams for flexural strength tests were prepared from three samples from each of batches 20, 50, and 80. Since the central laboratory was only a 10-minute drive from the job site, beams were fabricated there because it simplified transportation, fabrication, and storage problems. Fabrication was completed within 45 minutes of obtaining samples.

CONTRACTOR'S EQUIPMENT AND CONSTRUCTION METHODS

The contractor used two 34E Koehring dual drum mixes; one traveling on the prepared subgrade, and the other on the adjacent cured slab. Test batches all were mixed in the machine that traveled on prepared subgrades. The mixer was a modern machine in good repair. The contractor stated that the operator was reasonably experienced and reliable. All of contractor's personnel were extremely cooperative throughout the test period.

Concrete was batched approximately 3 miles from the site of the work and hauled in 3- and 4-compartment batch trucks. Only the 3-batch trucks could be used for 40.8-cubic foot batches.

CONCRETE MIX SPECIFICATIONS

A 5.6-sack concrete mix with a slump of 1-1/2 to 2 inches was specified for the project. Type IIA cement was used throughout except that on the last 1-1/2 days of paving, while the tests were being performed, type II without air entrainment was substituted. Consequently, only series 1 of the test (10 percent overload and 60-second mixing time) had air-entrained cement in the concrete. Since air content of the concrete in that series was not high, it was believed that the absence of air-entrained cement in series 2, 3, and 4 did not seriously affect test results.

DISCUSSION OF TEST RESULTS

Test results are tabulated in Tables 1 through 13 at the end of the report. Tests were designed to measure uniformity of the concrete within a single batch, from batch to batch within each series, and between each of the four series.

Uniformity of Fresh Concrete

Uniformity of fresh concrete was measured by means of consistency tests with both slump cone and Kelly ball, unit weight tests, and air content tests with both pressure and Chace air meters.

1. Slump cone tests. Consistency as measured by the slump cone ranged from a high of 6-1/2 inches to a low of 1-1/2

inches. Outside of batch 10 of series 1, maximum range of slump values within a single batch was 1/2 inch and, again neglecting batch 10 of series 1, the maximum range of slump value in 33 slump tests on the other 11 batches was 2 inches. Seventy percent of the 33 individual slump tests on the last 11 batches tested were within a range of 1-3/4 to 2-1/2 inches.

Batch 10 of series 1 was sampled early in the morning before production had leveled out to normal uniformity, and variations noted within that batch were not considered typical of the general run of concrete produced.

2. Kelly ball tests. The 36 individual consistency tests as measured by the Kelly ball varied over a wider range than the slump cone tests, probably indicating that the ball test is somewhat more sensitive than the slump cone. Maximum range was from 1.6 to 4.0 inches, and 75 percent of the individual tests were between 1.5 and 3.0 inches.

The ball tests, on the average, compared favorably with the cone tests.

	Average	
Series	Slump, in.	Ball, in.
1	2.08	2.51
2	2.50	2.91
3	2.75	2.67
4	2.50	2.20
Overall average	2.49	2.58

Unit Weight

Unit weight of fresh concrete was measured in a calibrated 1/2-cubic foot bucket.

The 36 individual tests ranged from 149.75 to 155.25 pounds per cubic foot. Since only series 1 used air-entrained concrete, a comparison of unit weights of series 2, 3, and 4 would be more significant. Here we find the range to be from 152.25 to 155.25 pounds per cubic foot, or a difference of less than 2 percent between the lowest and highest values. Approximately 75 percent of the 27 individual unit weight tests in series 2, 3, and 4 were between 152.75 and 154.25 pounds per cubic foot, or within ± 1/2 percent of 153.5 pounds per cubic foot.

Air Content Tests

Air content tests on the fresh concrete were made with both pressure and Chace air meters. As stated before, only concrete in series 1 had air-entrained cement. The pressure meter showed fairly consistent results within each of the series. The Chace meter also showed reasonably consistent results within each of the series, but actual results were significantly higher than those with the pressure meter.

The pressure meter was calibrated before the tests started and rechecked at completion of series 4. It was our conclusion that air contents indicated by the pressure meter were substantially accurate, and those by the Chace meter were, on the average, too high.

Uniformity of Hardened Concrete

Uniformity of the hardened concrete was measured by compressive and flexural strength tests at 28 days.

The 6- by 12-inch cylinders were fabricated in paper molds with metal bottoms. They were covered with damp burlap sacks after molding and cured at the job site for approximately 20 hours, after which they were moved to the moist room at the central laboratory and cured at 70°F and 100 percent relative humidity until tested.

The 6- by 6- by 36-inch beams were fabricated in rigid wooden forms in the central laboratory and then covered with damp burlap for 48 hours. After the initial 48-hour curing period, the beams were cured in the laboratory moist room until tested.

1. Compressive strength tests. The arithmetic mean and the coefficient of variation of the 9 compressive strength tests from each batch and the same values for the 27 tests in each series are shown in Tables 1 through 4. Coefficient of variation within the individual groups of 9 tests from a single batch ranged from 4.4 to 10.6 percent, and the coefficient of variation of the 27 tests within each of the 4 series ranged from 6.6 to 8.0 percent. We feel that these values represent surprisingly good uniformity of test results, and a high degree of confidence can be placed in the various averages. Considering average compressive strength results of the individual series, for example, the highest coefficient of variation of 8.0 percent on series 4 means that only 3 tests out of 1000 in that series would have fallen below a compressive strength of 3900 psi at 28 days.

2. Flexural strength tests. Two flexural strength tests were made on each of the 6- by 6- by 36-inch beams by breaking them on an 18-inch span with third-point loading. Thus, the 24 beams fabricated furnished 48 individual tests, or a total of 12 tests for each series.

Arithmetic mean and coefficient of variation for the 12 tests within each of the 4 series are shown in Tables 5 through 8. Average coefficient of variation is somewhat higher for the flexural than for the compressive tests, although not as much higher as had been expected. The difficulty of reproducing flexural strength results within a relatively narrow range of values has been noted by many investigators. We feel that uniformity of the beam tests in this study is unusually good.

CONCLUSIONS

Reliability of Test Methods

Considering first the results obtained by different methods of measuring the same property of the concrete, the following conclusions can be drawn on the basis of tests reported herein:

1. The Chace air meter is not a reliable tool for measuring air content of fresh concrete. It should be used only as an auxiliary method, and only then if measurements with a pressure meter are made at frequent intervals where quality control of concrete is required.

2. The Kelly ball is a reliable tool for measuring consistency of concrete, although it appears to be somewhat more sensitive than the slump cone. The reading in inches in the Kelly ball appears to correlate directly with the slump in inches as measured by the slump cone.

Desirability of Allowing Mixing Time Less Than 60 Seconds

Data on uniformity of the fresh concrete as measured by consistency, unit weight, and air content, either within the same batch or from different batches, show no advantage for the 60-second mixing time over the 45-second time.

In comparing series 1 to series 2, and series 3 to series 4, wherein the variable between each pair of series was mixing time, both compressive strength and flexural strength varied inversely with mixing time. Not only were both compressive and flexural strengths higher at 45 seconds than at 60 seconds for the same batch size, but, in addition, the coefficient of variation of the 12 individual tests in the case of the flexural strength tests was much better at 45 seconds than at 60 seconds. For compressive strength tests, however, individual results on the 60-second mix were slightly more uniform than results on the 45-second mix.

All of the data indicate without exception that the concrete was not harmed to the extent that quality was measured by the above tests when the mixing time was reduced from 60 seconds to 45 seconds. While the data indicate a definite advantage strengthwise to the 45-second mix, we are not prepared to accept that conclusion without further tests. We do feel, however, that the data justify the conclusion that a mixing time of 50 seconds is sufficient to provide uniformly high quality concrete provided that the mixer is in good repair and an experienced operator is in charge.

It was our observation that a mixing time of 50 seconds was a practical minimum. The operator had difficulty in consistently reducing the time to 45 seconds, and several batches to be mixed only 45 seconds were not sampled because the time ran up to 50 or 51 seconds before the batch could be discharged. There would be no advantage in allowing the contractor a shorter mixing time than he could achieve without disrupting the smoothness of his overall operation.

Desirability of Allowing a 20 Percent Overload in 34E+10% Mixers

The effect of the batch weight variable can be studied by comparing series 1 with series 3, and series 2 with series 4.

Here, again, uniformity of the fresh concrete as measured by consistency, unit weight, and air content showed no definite advantage for the 10 percent overload over the 20 percent overload.

Effect of overload on uniformity and quality of the hardened concrete, as measured by compressive and flexural tests, was not consistent. For example, on compressive strengths, series 1 averaged about 5 percent higher than series 3 but, conversely, on the 45-second mixes series 4 with a 20 percent overload was about 7.5 percent higher than series 2 with a 10 percent overload.

On the other hand, on flexural strengths the 10 percent overload at 45 seconds was about 8.5 percent higher than the 20 percent overload, and at 60 seconds the 20 percent overload was about 2 percent higher than the 10 percent overload.

FINAL RECOMMENDATIONS

Following is a summary of compressive and flexural strength results on each of the four series:

Series no.	Overload %	Mixing time seconds	Strength in psi at 28 days Compressive	Flexural
1	10	60	4418	807
2	10	45	4841	926
3	20	60	4197	824
4	20	45	5124	848

Series 1 was mixed according to our present specifications, and results of that series were considered the norm.

Although all of the other series except for the compressive strength in series 3 had higher test values than series 1, we must remember that the concrete in series 1 had more air than in the other series and, therefore, would be expected to have somewhat lower strength values at 28 days.

We feel the tests were sufficiently conclusive to justify the following statements:

1. A 50-second mixing period was sufficient to insure uniformly well mixed concrete in the paver used for the tests.

2. A 40.8-cubic foot batch can be allowed in a 34E+10% mixer provided equipment is in good condition, batch trucks are of sufficient capacity to retain each batch without spillage, discharge bucket has sufficient capacity, and paver is not operating on a vertical grade in excess of 3 percent.

The limitation of 20 percent overload to a grade below 3 percent is necessary, in our opinion, until actual tests can be made on grades in excess of that figure. All tests in this study were taken with the machine on a maximum 1.5 percent grade.

Table 1.
Compressive Strength Tests
Series 1 - 10% overload, 60-second mix

Batch no.	From station*	Sample no.	Cylinder no.	Weight lb/cu ft	28-day compressive strength - psi	Arithmetic average	Coefficient of variation	Remarks
			1-1-1	150	4139			
1	1098+60	1-1-1	1-1-1	150	4510			
			1-1-1	149	4351			
			1-1-2	150	4669			
		1-1-2	1-1-2	152	4236	4312	7.0	
			1-1-2	151	4537			
			1-1-3	152	4301			
		1-1-3	1-1-3	151	3572			Arithmetic average of compressive strength for batches 1,4,& 7, Series 1, is 4418. Coefficient of variation is 6.9.
			1-1-3	150	4492			
			1-4-1	151	5005			
4	1003+50	1-4-1	1-4-1	150	4227			
			1-4-1	152	4230			
			1-4-2	149	4935			
		1-4-2	1-4-2	151	4360	4576	5.9	
			1-4-2	151	4793			
			1-4-3	152	4554			
		1-4-3	1-4-3	151	4598			
			1-4-3	152	4483			
			1-7-1	150	4289			
7	1007+00	1-7-1	1-7-1	153	4186			
			1-7-1	150	4390			

Table 1 (continued)

Batch no.	From station*	Sample no.	Cylinder no.	Weight lb/cu ft	28-day compressive strength - psi	Arithmetic average	Coefficient of variation	Remarks
			1-7-2	153	4656			
		1-7-2	1-7-2	151	4952	4367	6.3	
			1-7-2	150	4500			
			1-7-3	151	4085			
		1-7-3	1-7-3	150	4121			
			1-7-3	150	4121			

*All samples taken from south-bound inside lane on October 10, 1958.

Table 2.
Compressive Strength Tests
Series 2 - 10% overload, 45-second mix

Batch no.	From station*	Sample no.	Cylinder no.	Weight lb/cu ft	28-day compressive strength - psi	Arithmetic average	Coefficient of variation	Remarks
			2-1-1	153	4351			
1	1017+50	2-1-1	2-1-1	154	5660			
			2-1-1	156	4728			
			2-1-2	156	5368			Arithmetic
		2-1-2	2-1-2	153	5403	4944	10.0	average of
			2-1-2	153	5103			compressive
			2-1-3	154	3997			strength for
		2-1-3	2-1-3	154	4855			batches 1, 4, &
			2-1-3	153	5032			7, Series 2,
			2-4-1	153	4316			is 4841. Co-
4	1024+00	2-4-1	2-4-1	153	4890			efficient of variation is
			2-4-1	154	4802			7.9.

Table 2 (continued)

Batch no.	From station*	Sample no.	Cylinder no.	Weight lb/cu ft	28-day compressive strength - psi	Arithmetic average	Coefficient of variation	Remarks
			2-4-2	154	4375			
		2-4-2	2-4-2	153	4678	4662	4.4	
			2-4-2	155	4924			
			2-4-3	153	4510			
		2-4-3	2-4-3	153	4775			
			2-4-3	152	4687			
			2-7-1	154	4293			
7	1028+80	2-7-1	2-7-1	152	4704			
			2-7-1	153	5191			
			2-7-2	156	4961			
		2-7-2	2-7-2	153	4935	4916	6.4	
			2-7-2	154	5306			
			2-7-3	154	5297			
		2-7-3	2-7-3	154	4952			
			2-7-3	153	4607			

*All samples taken from south-bound inside lane on October 10, 1958.

Table 3.
Compressive Strength Tests
Series 3 - 20% overload, 60-second mix

Batch no.	From station*	Sample no.	Cylinder no.	Weight lb/cu ft	28-day compressive strength - psi	Arithmetic average	Coefficient of variation	Remarks
			3-1-1	152	4041			
1	1018+80	3-1-1	3-1-1	152	4333			
			3-1-1	153	4209			
			3-1-2	153	3999			Arithmetic average of compressive strength for batches 1, 4, & 7, Series 3, is 4197. Coefficient of variation is 6.6.
		3-1-2	3-1-2	153	3979	4078	5.0	
			3-1-2	153	4324			
			3-1-3	153	4262			
		3-1-3	3-1-3	153	3733			
			3-1-3	154	3822			
			3-4-1	152	4068			
4	1009+40	3-4-1	3-4-1	152	4943			
			3-4-1	152	4351			
			3-4-2	152	4174			
		3-4-2	3-4-2	151	4050	4263	6.5	
			3-4-2	153	4165			
			3-4-3	151	4192			
		3-4-3	3-4-3	151	4466			
			3-4-3	153	3955			
			3-7-1	151	4059			
7	1007+25	3-7-1	3-7-1	151	3891			
			3-7-1	153	4192			

Table 3 (continued)

Batch no.	From station*	Sample no.	Cylinder no.	Weight lb/cu ft	28-day compressive strength - psi	Arithmetic average	Coefficient of variation	Remarks
			3-7-2	151	3918			
		3-7-2	3-7-2	150	4433	4256	7.0	
			3-7-2	151	4764			
			3-7-3	151	4623			
		3-7-3	3-7-3	152	4171			
			3-7-3	152	2582**			**Large rock in mold

*All samples taken from north-bound inside lane on October 14, 1958.

Table 4.
Compressive Strength Tests
Series 4 - 20% overload, 45-second mix

Batch no.	From station*	Sample no.	Cylinder no.	Weight lb/cu ft	28-day compressive strength - psi	Arithmetic average	Coefficient of variation	Remarks
			4-1-1	154	5094			
1	1032+25	4-1-1	4-1-1	154	4926			
			4-1-1	154	5050			
			4-1-2	154	4696			
		4-1-2	4-1-2	156	5775	5171	6.4	Arithmetic average of compressive strength for batches 1, 4, & 7, Series 4 is 5124. Coefficient of variation is 8.0.
			4-1-2	155	5058			
			4-1-3	155	5359			
		4-1-3	4-1-3	153	4935			
			4-1-3	154	5642			

Table 4 (continued)

Batch no.	From station*	Sample no.	Cylinder no.	Weight lb/cu ft	28-day compressive strength - psi	Arithmetic average	Coefficient of variation	Remarks
			4-4-1	154	4324			
4	1035+72	4-4-1	4-4-1	155	5536			
			4-4-1	152	5147			
			4-4-2	154	4528			
		4-4-2	4-4-2	153	5076	5087	10.6	
			4-4-2	154	4941			
			4-4-3	154	5757			
		4-4-3	4-4-3	154	4528			
			4-4-3	154	5946			
			4-7-1	152	5271			
7	1037+62	4-7-1	4-7-1	153	5341			
			4-7-1	154	4979			
			4-7-2	153	4855			
		4-7-2	4-7-2	153	5218	5115	6.3	
			4-7-2	154	4360			
			4-7-3	152	5235			
		4-7-3	4-7-3	152	5518			
			4-7-3	153	5262			

*All samples taken from south-bound inside lane on October 10, 1958

Table 5.
Flexural Strength Tests
Series 1 - 10% overload, 60-second mix

Batch no.	From station*	Sample no.	Beam no.	Modulus of rupture "R"	Average	Remarks
2	1001+00	1-2-1	1-2-1-A	826	770	Arithmetic average of flexural strength for batches 2, 5, & 8, Series 1, is 807 "R." Coefficient of variation is 10.0.
		1-2-2	1-2-1-B	713		
		1-2-3	1-2-2-A	716	697	
			1-2-2-B	678		
5	1006+50	1-5-1	1-5-1-A	763	756	
		1-5-2	1-5-1-B	748		
		1-5-3	1-5-2-A	804	814	
			1-5-2-B	824		
8	1013+00	1-8-1	1-8-1-A	993	949	
		1-8-2	1-8-1-B	904		
		1-8-3	1-8-2-A	857	856	
			1-8-2-B	855		

*All samples taken from south-bound inside lane on October 10, 1958.

Table 6.
Flexural Strength Tests
Series 2 - 10% overload, 45-second mix

Batch no.	From station*	Sample no.	Beam no.	Modulus of rupture "R"	Average	Remarks
2	1022+75	2-2-1	2-2-1-A	954	935	Arithmetic average of flexural strength for batches 2, 5, & 8, Series 2, is 926 "R." Coefficient of variation is 5.0.
		2-2-2	2-2-1-B	916		
		2-2-3	2-2-2-A	860	920	
			2-2-2-B	980		
5	1027+30	2-5-1	2-5-1-A	880	873	
		2-5-2	2-5-1-B	866		
		2-5-3	2-5-2-A	920	891	
			2-5-2-B	862		
8	1029+20	2-8-1	2-8-1-A	992	972	
		2-8-2	2-8-1-B	951		
		2-8-3	2-8-2-A	998	966	
			2-8-2-B	933		

*All samples taken from south-bound inside lane on October 10, 1958.

Table 7.
Flexural Strength Tests
Series 3 - 20% overload, 60-second mix

Batch no.	From station*	Sample no.	Beam no.	Modulus of rupture "R"	Average	Remarks
2	1018+20	3-2-1	3-2-1-A	944	882	
		3-2-2	3-2-1-B	819		
		3-2-3	3-2-2-A	878	878	Arithmetic average of flexural strength for batches 2, 5, & 8, Series 3, is 824 "R." Coefficient of variation is 11.0.
			3-2-2-B	878		
5	1009+10	3-5-1	3-5-1-A	897	872	
		3-5-2	3-5-1-B	846		
		3-5-3	3-5-2-A	738	843	
			3-5-2-B	747		
8	1007+00	3-8-1	3-8-1-A	963	876	
		3-8-2	3-8-1-B	788		
		3-8-3	3-8-2-A	769	692	
			3-8-2-B	615		

*All samples taken from north-bound inside lane on October 14, 1958.

Table 8.
Flexural Strength Tests
Series 4 - 20% overload, 45-second mix

Batch no.	From station*	Sample no.	Beam no.	Modulus of rupture "R"	Average	Remarks
2	1032+85	4-2-1	4-2-1-A	861	866	Arithmetic average of flexural strength for batches 2, 5, & 8, Series 4, is 848 "R." Coefficient of variation is 6.0.
		4-2-2	4-2-1-B	870		
		4-2-3	4-2-2-A	822	882	
			4-2-2-B	941		
5	1037+00	4-5-1	4-5-1-A	774	776	
		4-5-2	4-5-1-B	778		
		4-5-3	4-5-2-A	882	806	
			4-5-2-B	730		
8	1022+00	4-8-1	4-8-1-A	868	876	
		4-8-2	4-8-1-B	884		
		4-8-3	4-8-2-A	878	883	
			4-8-2-B	888		

*Samples of batches 2 and 5 taken from south-bound inside lane on October 10, 1958.
Samples of batch 8 taken from north-bound inside lane on October 14, 1958.

Table 9.
Consistency Tests

Series 1 10% overload, 60-sec mix				Series 2 10% overload, 45-sec mix				Series 3 20% overload, 60-sec mix				Series 4 20% overload, 45-sec mix			
Batch no.	Sple no.	Slump value in.	Kelly ball penetration in.	Batch no.	Sple no.	Slump value in.	Kelly ball penetration in.	Batch no.	Sple no.	Slump value in.	Kelly ball penetration in.	Batch no.	Sple no.	Slump value in.	Kelly ball penetration in.
	1-1-1	6.5			2-1-1	3.0			3-1-1	1.75			4-1-1	2.5	
1	1-1-2	4.25		1	2-1-2	3.0		1	3-1-2	2.0		1	4-1-2	2.25	
	1-1-3	3.5			2-1-3	2.75			3-1-3	2.0			4-1-3	2.5	
	1-2-1		1.8		2-2-1		2.6		3-2-1		2.4		4-2-1		1-6
2	1-2-2		2.0	2	2-2-2		2.4	2	3-2-2		2.0	2	4-2-2		2.0
	1-2-3		1.8		2-2-3		2.6		3-2-3		2.8		4-2-3		1.6
	1-4-1	1.75			2-4-1	2.5			3-4-1	3.0			4-4-1	2.25	
4	1-4-2	2.0		4	2-4-2	2.5		4	3-4-2	3.0		4	4-4-2	2.5	
	1-4-3	1.5			2-4-3	2.5			3-4-3	2.5			4-4-3	2.25	
	1-5-1		2.6		2-5-1		3.4		3-5-1		3.4		4-5-1		2.2
5	1-5-2		2.4	5	2-5-2		4.0	5	3-5-2		3.0	5	4-5-2		1.6
	1-5-3		2.6		2-5-3		3.6		3-5-3		3.2		4-5-3		1.6
	1-7-1	2.0			2-7-1	2.0			3-7-1	3.5			4-7-1	2.75	
7	1-7-2	2.5		7	2-7-2	2.25		7	3-7-2	3.5		7	4-7-2	2.5	
	1-7-3	2.5			2-7-3	2.0			3-7-3	3.5			4-7-3	3.0	
	1-8-1		2.8		2-8-1		2.0		3-8-1		2.2		4-8-1		2.8
8	1-8-2		3.4	8	2-8-2		3.2	8	3-8-2		2.4	8	4-8-2		3.4
	1-8-3		3.2		2-8-3		2.4		3-8-3		2.6		4-8-3		3.0

Table 10.
Unit Weight of Fresh Concrete

Series 1 10% overload, 60-sec mix			Series 2 10% overload, 45-sec mix			Series 3 20% overload, 60-sec mix			Series 4 20% overload, 45-sec mix		
Batch no.	Sample no.	Weight lb/cu ft	Batch no.	Sample no.	Weight lb/cu ft	Batch no.	Sample no.	Weight lb/cu ft	Batch no.	Sample no.	Weight lb/cu ft
	1-2-1	151.0		2-2-1	153.5		3-2-1	154.5		4-2-1	153.75
2	1-2-2	150.25	2	2-2-2	153.5	2	3-2-2	153.5	2	4-2-2	152.75
	1-2-3	149.75		2-2-3	153.5		3-2-3	154.0		4-2-3	152.75
	1-5-1	151.0		2-5-1	154.25		3-5-1	155.25		4-5-1	154.25
5	1-5-2	151.0	5	2-5-2	154.25	5	3-5-2	153.5	5	4-5-2	155.0
	1-5-3	151.25		2-5-3	154.25		3-5-3	154.0		4-5-3	155.0
	1-8-1	153.0		2-8-1	154.25		3-8-1	153.0		4-8-1	153.75
8	1-8-2	152.5	8	2-8-2	152.25	8	3-8-2	152.5	8	4-8-2	153.75
	1-8-3	153.75		2-8-3	153.5		3-8-3	153.25		4-8-3	154.5
Average		151.5			153.70			153.72			153.94
Maximum		153.75			154.25			155.25			155.0
Minimum		149.75			152.25			152.5			152.75

Table 11.
Air Content of Fresh Concrete

Batch no.	Series 1 10% - 60 sec Air content Pres. %	Chace %	Series 2 10% - 45 sec Air content Pres. %	Chace %	Series 3 20% - 60 sec Air content Pres. %	Chace %	Series 4 20% - 45 sec Air content Pres. %	Chace %
1	2.8	3.04	1.2	3.2	1.5	4.0	1.2	4.0
4	2.4	2.4	1.1	3.2	1.0	4.0	2.0	2.8
7	2.6	-	1.0	-	1.5	4.0	1.0	4.0
Avg	2.6	2.7	1.1	3.2	1.3	4.0	1.4	3.6

Table 12.
Effect of Mixing Time on Various Results

	10% overload		20% overload	
	Series 1 60 sec	Series 2 45 sec	Series 3 60 sec	Series 4 45 sec
Compressive strength	4418 psi	4841 psi	4197 psi	5124 psi
Flexural strength	807 psi	924 psi	824 psi	848 psi
Fresh unit weight	151.50* lb/cu ft	153.70 lb/cu ft	153.72 lb/cu ft	153.94 lb/cu ft
Slump	2.08 in.	2.51 in.	2.75 in	2.50 in.
Kelly ball	2.51 in.	2.91 in.	2.67 in.	2.20 in.
Air content by pressure meter	2.6%*	1.1%	1.3%	1.4%

*Type IIA cement used.

Table 13.
Effect of Batch Weight on Various Results

	60-sec mixing time		45-sec mixing time	
	Series 1 10%	Series 3 20%	Series 2 10%	Series 4 20%
Compressive strength	4418 psi	4197 psi	4841 psi	5124 psi
Flexural strength	807 psi	824 psi	924 psi	848 psi
Fresh unit weight	151.50* lb/cu ft	153.72 lb/cu ft	153.70 lb/cu ft	153.94 lb/cu ft
Slump	2.08 in.	2.75 in.	2.51 in.	2.50 in.
Kelly ball	2.51 in.	2.67 in.	2.91 in.	2.20 in.
Air content by pressure meter	2.6%*	1.3%	1.0%	1.0%

*Type IIA cement used.

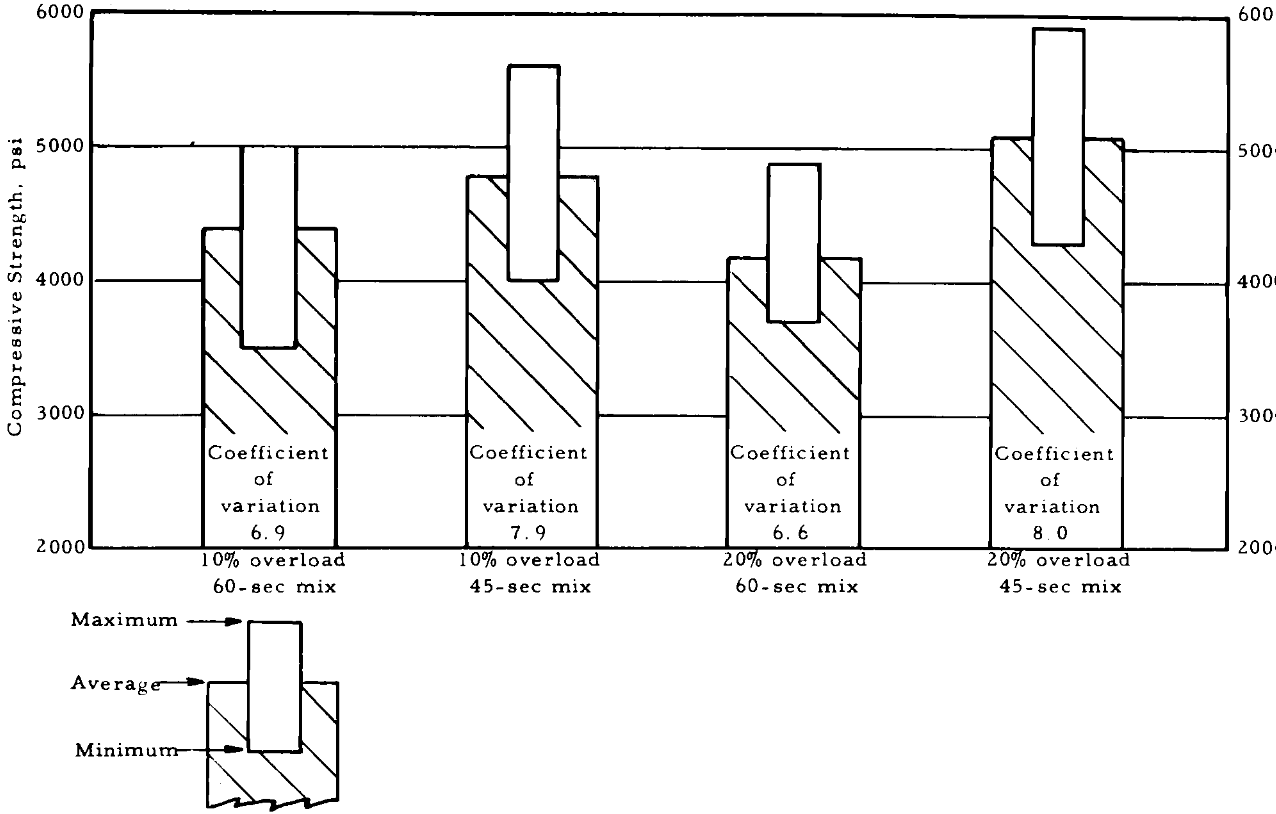

Figure 1. Comparison of Compressive Strength Results

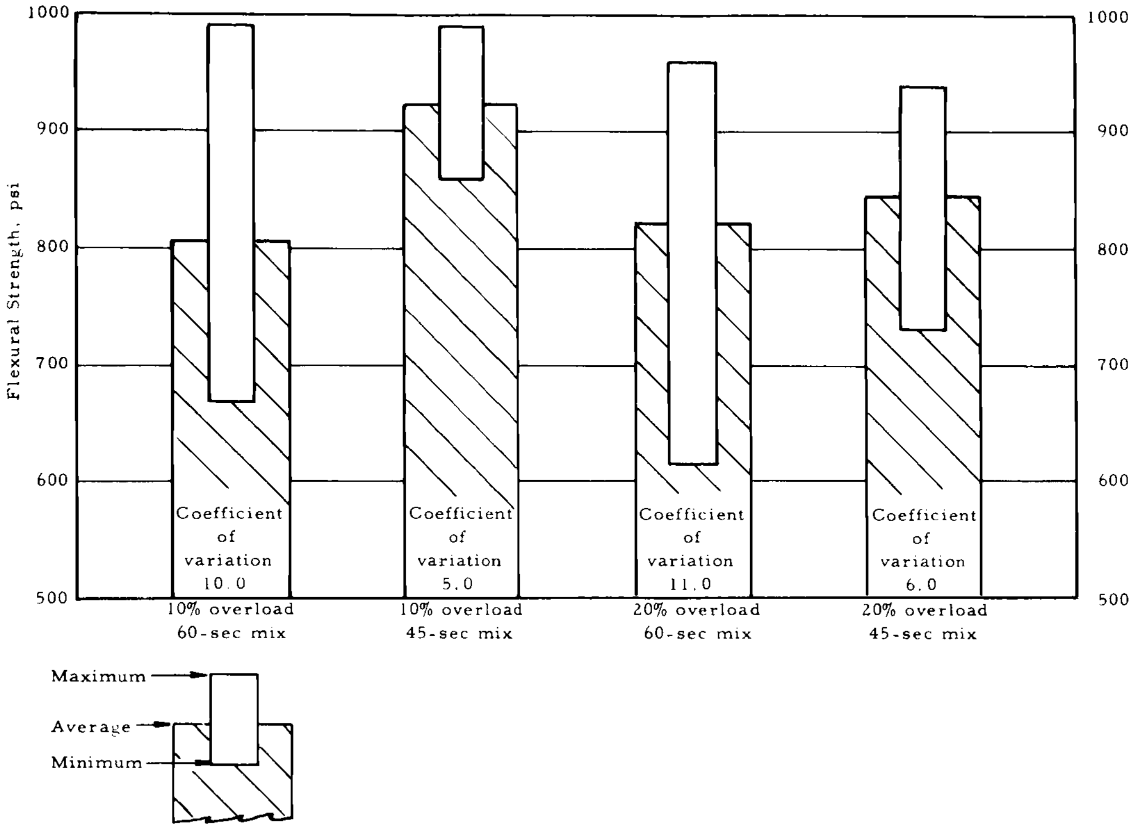

Figure 2. Comparison of Flexural Strength Results

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Blum, William M.	Asst City Engineer	Springfield	City Hall
Clancy, J.P.	Field Engineer	Olympia	6550 Martin Way
Culver, Howard R.	City Superintendent	Hermiston	City Hall
Curran, Daniel E.	Engineer IV	Portland	2602 NE 44th Ave
Davis, Donald A.	City Engineer	Milton-Freewater	1004 Davis
Eddleman, Charles A.	Asst City Engineer	Pendleton	City Hall
Eison, L.T.	City Engineer	Springfield	City Hall
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McKinstry, Edward N.	City Engineer	North Bend	3634 Sherman
Manes, Jim	Public Works Supt	The Dalles	City Hall
Mitchell, Arthur F.	Road Maintenance	Richland	1108 Williams
Monahan, William T.	Office Engineer	Portland	City Hall
Newell, William A.	Asst City Engineer	Eugene	410 River Ave
Roller, Merle L.	Street Superintendent	Richland	722 Building
Taylor, Dale	Street Foreman Public Works	The Dalles	City Hall
Thoreson, Ervin M.	Jr Civil Engineer	Portland	City Hall
Verbick, Bob	Inspector	Medford	City Hall

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Wilson, George E.	Street Supt	Madras	Box 771
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Minor, Carl E.	Materials & Res Engr	State Highway Dept	Olympia

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	UNIVERSITY OF WASHINGTON		
Ekse, Martin	Prof of Civil Engr	University of Wash.	Seattle
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Bureau of Public Roads

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School of Agriculture (B.S., B.Agr., M.S., M.Agr., Ph.D. degrees)

School of Business and Technology (B.A., B.S. degrees)

School of Education (B.A., B.S., Ed.B., M.A., M.S., Ed.M., Ed.D. degrees)

School of Engineering and Industrial Arts (B.A., B.S., M.A., M.S., A.E., Ch.E., C.E., E.E., I.E., M.E., Min.E., Ph.D. degrees)

School of Forestry (B.S., B.F., M.S., M.F. degrees)

School of Home Economics (B.A., B.S., M.A., M.S., M.H.Ec., Ph.D. degrees)

School of Pharmacy (B.A., B.S., M.A., M.S., Ph.D. degrees)

Graduate School

(M.A., M.S., Ed.M., M.F., M.Agr., M.H.Ec., A.E., Ch.E., C.E., E.E., I.E., M.E., Min.E., Ed.D., Ph.D. degrees)

Summer Sessions

Short Courses, Institutes, Workshops

RESEARCH AND EXPERIMENTATION

General Research

Science Research Institute

Agricultural Experiment Station—

Central Station, Corvallis

Branch Stations at Astoria, Aurora, Burns, Hermiston, Hood River and The Dalles, Klamath Falls, Medford, Moro, Ontario, Oregon City, Pendleton, and Union.

Experimental Area at Redmond.

Engineering Experiment Station

Oregon Forest Research Center

EXTENSION

Federal Cooperative Extension (Agriculture and Home Economics)

General Extension Division