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Quality Assurance of Recycled Material: Construction Delay Damage

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Quality-Assurance Considerations in Design of Recycled Asphalt Mixture

THOMAS W. KENNEDY AND FREDDY L. ROBERTS

A procedure that can be used by an engineer to design a recycled mixture by using material salvaged from an existing roadway is described. Special attention is directed toward quality-assurance factors that must be addressed to ensure that variations are kept within limits that will allow production of a mixture that will perform satisfactorily. Some of these quality-assurance factors are commonly overlooked and yet can dramatically affect the field performance of the material. Among these commonly overlooked factors are determining the causes of failures, locating sections with different characteristics, and developing a sampling plan for collecting material for laboratory studies based on the first two factors. After the causes of distress have been determined, the salvaged material is evaluated to determine whether softening agents are needed and whether virgin aggregate and asphalt should be added and, if so, how much. Also included are cautions for preparing candidate mixtures in the laboratory in a manner similar to field material processing. Suggested minimum values of engineering properties are included as well as sample plots to demonstrate areas of concern relative to quality assurance. Concerns for quality assurance in each step of the design process are summarized.

The purpose of this paper is to address the necessary procedures and considerations required to produce quality recycled-asphalt mixtures. In comparison with conventional mixtures, this requires greater care, since the basic materials used in recycled mixtures are salvaged from an existing roadway that has failed. Therefore, a special effort must be made to rejuvenate these salvaged materials. In addition, attention must be directed to detecting variations that occur in the salvaged material as a result of the original design and construction, previous maintenance and rehabilitative activities, and the effects of environment and traffic. In order to ensure the quality of recycled mixtures, these variational aspects must then be considered adequately in the sampling, design, and construction phases of the project.

After recycling has been selected as the most desirable and cost-effective alternative for rehabilitation, a series of steps must be conducted to ensure a satisfactory pavement. First, a sampling plan must be developed and materials secured for the design of the mixture. In addition, a three-phase design must occur that includes general design, preliminary design, and final design (1). General design includes evaluating causes of failure and determining whether the problems are related to mixture or structure. Preliminary design includes a laboratory evaluation to determine the behavior and effects of factors such as softening agents, new aggregates, and antistrip agents, if needed. Final design includes preparing specimens of the actual mixture in various combinations to determine the engineering properties of the mixture and to determine whether the mixture is satisfactory. This includes comparisons of test results for the recycled mixture with the ranges of properties that are expected to provide good field performance.

When construction begins in the field, it may be necessary to modify the final design to provide a mixture that will meet construction requirements; however, these changes should be very carefully recorded and their effect anticipated and monitored.

GENERAL DESIGN

The most common aspects of the general design category are to

1. Determine the nature and cause of distress,
2. Determine the gradation of the recycled aggregate,
3. Determine the residual asphalt content of the recycled mixture,
4. Determine the penetration and viscosity of the recycled asphalt, and
5. Specify the aggregate gradation after pulverization and the addition of new aggregate.

Perhaps the most significant activities in this category of design that affect the quality-assurance issue are related both to item 1 and to establishing the sampling plan for securing materials to be used in items 2, 3, and 4. In fact, the information secured in item 1 is crucial to prevent the engineer from assuming that a rejuvenating (softening) agent always needs to be included in the recycled mixture. The major discussion in this section will then deal with item 1 and the sampling plan.

Determine Causes of Distress

It is essential that the cause of the distress that led to the need for recycling be identified and corrected. Three of the most common causes of distress are (a) aging (brittleness) of the asphalt cement, (b) stripping of the asphalt from the aggregate, and (c) structural inadequacy. Texas experience would suggest that one or more of these causes are involved in most failures that lead to recycling.

A detailed condition survey should be conducted to determine the severity and extent of the distress present on the job for which recycling is being considered. The condition survey should be separate for each section of road that is determined to be different based on considerations of (a) surface thickness or mixture design, (b) presence of heavy maintenance discontinuously along the section, (c) seal or friction coat difference, and (d) half-section skin patching. For each section identified by using the suggestions described above, the types of distress and the severity should be evaluated to determine the primary cause of the distress.

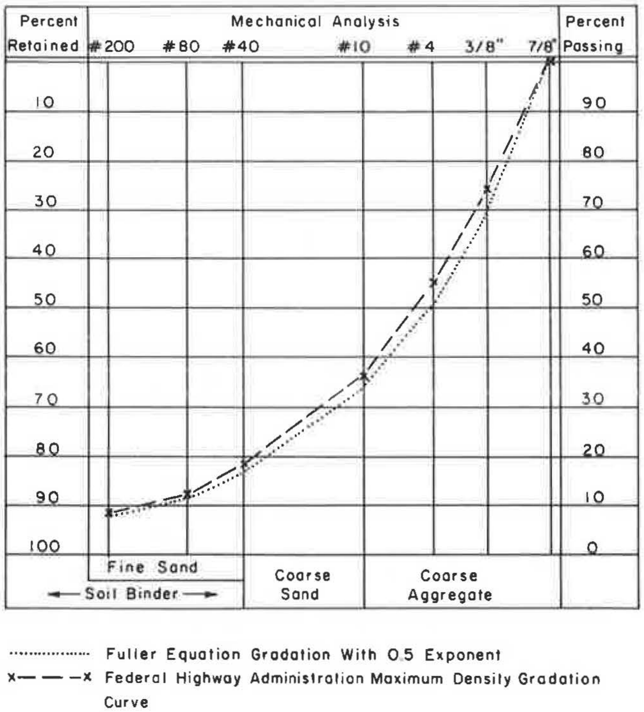
It is most important to identify whether these failures are associated with the characteristics of the mixture to be recycled or with the pavement structure, either locally or in general. In the case of mixture problems the failure can be categorized as either brittle or nonbrittle. An excellent guide to analysis of pavement failure was prepared by Finn and Epps (2).

Mixture Problems

Brittle failures occur when axle loads, thermally induced stresses, or shrinkage of underlying layers combines with aged asphalt cements to produce cracking, e.g., alligator, transverse, block (map), and longitudinal. When such an asphalt mixture is to be recycled, softening agents or soft asphalts typically must be added to restore the salvaged asphalt cement to its original viscosity.

Nonbrittle failures are usually associated with mixtures that are stripping or are exhibiting poor stability. Distresses typical of these conditions

Figure 1. Grading curves for dense-graded asphalt-concrete mixtures.



are rutting, shoving, corrugations, and bleeding. Rutting can also occur as a result of lateral flow of nonbituminous layers. The cause of rutting in each of these three cases is different and the treatment to alleviate the problem must be selected and applied either prior to or during the recycling operation if the recycled pavement is expected to perform adequately.

In the case of the stripping mixture, an appropriate treatment must be applied to the salvaged mixture to alleviate the stripping problem or the mixture must be discarded or used for other purposes such as low-volume road patching or shoulders. Once the stripping problem has been alleviated, the salvaged mixture can be evaluated and a new mixture design developed. Softening agents most often are not required and if included could produce a very soft and unstable mixture that is prone to shoving and rutting.

Poor stability often can be alleviated by adding new aggregate during recycling to improve gradation and introduce more angular aggregate particles. Better gradation may also result in a higher density, which would be beneficial with respect to moisture damage. It is also recommended that serious consideration be given to using approximately equal percentages of recycled material and new material; a recommended maximum is 70 percent recycled material.

Special attention should be given to the final gradation, including new aggregate if added. Grading curves similar to those shown in Figure 1 (3,4) have shown excellent performance. The grading curve should not have humps in the region of the No. 30 to No. 60 sieves nor should there be significant deviations, either coarser or finer, in the regions above the No. 10 sieve. Variations in these regions are especially important for certain types of material-distress combinations. Goode and Lufsey (4) have shown that humps in the region of the No. 30 to No. 60 sieves above the lines shown in Figure 1 produce tender mixes. In addition, these finer mixes can significantly lower stabilities. If the mix is both too coarse (gradations below the lines in Figure 1 for sizes larger than the No. 10 sieve) and made with strip-prone aggregate, the greater porosity of the mix may actually enhance the opportunity for water damage. In fact, some mixes being used by states today have such a small range of combinations of gradation and asphalt content that produce satisfactory mixes that runs only one day apart failed in two different modes, stripping in the open mixture and shoving and rutting in the mixture with slightly higher asphalt content (5). Therefore, the mixture with higher void content stripped, whereas the mixture with lower void content shoved and rutted under traffic.

Structural Problems

Structural deterioration may occur as the result of underdesign, increased traffic volumes and axle loads, decreased support values due to the action of water, and brittleness of the asphalt due to aging, all of which can produce increased stresses and strains. If these increased stresses and strains exceed limiting values, premature fatigue or longitudinal cracking in the surface layer or permanent deformations can occur. This cracking can be localized or can be quite extensive.

An evaluation of the strength conditions of the existing pavement structure can be made by performing and analyzing a Dynaflect survey or other nondestructive test. Such an analysis will help define the extent of soft spots and establish the limits on sections where the underlying support characteristics or layer thicknesses are different or inadequate. Application of these techniques and formulas for estimating moduli for underlying layers have been presented by Lytton and Machalak (6).

Sampling Plan

Each identified subsection should be treated as a separate design, and a representative sample should be secured from each. Sampling sites within each subsection should be selected randomly. The engineer should choose at least six sampling sites for each subsection and secure a minimum of 200 lb of material for subsequent laboratory analysis (7).

The effect of discontinuities or variation of material properties along the length of the pavement or across the width may lead to difficulties in securing representative materials. The effect of large discontinuous areas of patching, the addition of hot mixed overlays or seal coats to surface courses that were originally cold mixed, and many other combinations of different materials may make selection of representative samples to be used for a single mixture design for the entire pavement difficult, if not impossible. In such cases, further subdivision of the subsection may be necessary or perhaps the recycling alternative must be abandoned if only short subsections can be identified.

Of special concern in developing the sampling plan are the causes of failure and variations in asphalt content or gradations of the material to be salvaged. Since brittle and nonbrittle failures require different treatment of the salvaged asphalt cement, it is imperative that the first break in the sampling plan be based on type of failure. The second primary area of concern is that of variations in asphalt-cement content and aggregate gradations down the road. Since seal coats, other surface treatments, and patching, as well as sealing programs, do not necessarily involve the entire roadway, these maintenance operations will affect the selection of relatively homogeneous sections for mixture design considerations. If the materials are to be removed from the site, crushed, sized, and reblended, these problems are minimized but should be considered in developing the sampling plan. If the recycling is to be accomplished in place, careful laboratory studies should be conducted to determine the magnitude of systematic variations in asphalt content and gradations across the roadway and to evaluate the effect of those variations on stability, void contents, density, and strength. If these variations are significant enough to produce instabilities, high void contents, or other problems in portions of the recycled mixture, then the engineer should carefully consider whether the recycling option should be abandoned or whether to proceed but modify the construction sequence to eliminate or minimize these problems. A final decision on these factors could be delayed until more complete information is available on which to evaluate the effect of these variations on mixture properties.

PRELIMINARY DESIGN

The primary objective of the preliminary design is to select the type and amount of additive that can be used to recondition the asphalt or eliminate asphalt aggregate problems in the salvaged mixture, if necessary. If a brittle failure has occurred, this portion of the design involves the selection of an additive that will soften the existing asphalt and return it to its original or desired viscosity. A variety of materials are available, such as soft asphalt, commercially available softening agents, and combinations of these materials. If a nonbrittle failure has occurred, the techniques or type and amount of additive that will minimize distress, such as stripping, must be selected. Materials such as lime and chemical antistrip agents are believed capable of reducing stripping in asphalt-concrete mixtures. Nevertheless, to ensure a successful project, it is imperative that selected antistrip additives be tested to ascertain their effectiveness.

Softening Agents

Often a primary criterion in a preliminary design procedure is to reduce the viscosity or increase the penetration of the asphalt to a value representative of a virgin asphalt cement. The recommended steps usually involved are

1. Extracting and recovering asphalt from the salvaged mixture,
2. Mixing the recovered asphalt with the selected types and amounts of additives,
3. Measuring the viscosity or penetration of the treated asphalt cement,
4. Plotting the relationship between the amount of additive and the viscosity or penetration (Figures 2 and 3),
5. Determining which additives or combinations of additives will produce the desired consistency in the salvaged asphalt cement, and
6. Selecting acceptable additives or combinations of additives that warrant preparation of laboratory mixtures for further evaluation (factors to be considered in this selection are costs, availability, construction considerations, past reliability and experience, etc.).

Generally this portion of the design process is fairly standard. However, careful consideration must be given to the field mixing process and the method of blending the softening agent into virgin or reclaimed asphalt cement. It is conceivable that a particular softening agent could be chosen in this portion of the design that, when applied under field

Figure 2. Typical relationships between penetration and percentage of softening agent for recovered brittle asphalt cement and four softening agents.

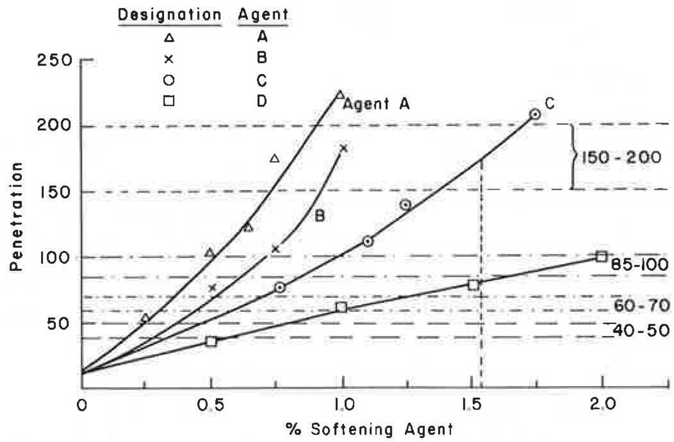
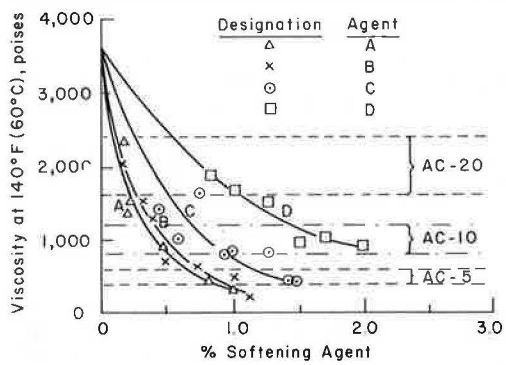


Figure 3. Typical relationships between viscosity at 140°F and percentage of softening agent for recovered brittle asphalt cement and four softening agents.



construction conditions, will not be so effective in rejuvenating the salvaged asphalt content as it was in the laboratory. Therefore, mixture preparations that use the selected softening agents and salvaged materials should closely simulate field conditions, including the method of adding the softening agent, mixing time and temperature, and compaction.

New Aggregate

According to Epps and Holmgreen (7), new aggregate may have to be added to the mixture for one or more of the following reasons:

1. To satisfy gradation requirements;
2. To improve the skid resistance to meet requirements for the new surface course;
3. To meet air-quality regulations associated with hot central plant recycling, typically 30 to 40 percent new aggregate;
4. To meet total pavement thickness requirements;
5. To improve the properties of the mixture, such as stability, durability, and flexibility; and
6. To be able to add enough modifier to restore the salvaged asphalt to meet specification requirements and still maintain required mixture properties.

In addition to these reasons for adding new aggregate to the salvaged mixture, one other factor should be considered--experience in recycled construction. Generally, it is recommended that not more than 50 percent salvaged material be used since the mixture is less forgiving at higher percentages of recycled material. With experience, higher percentages of salvaged material can be used; however, in general, it is recommended that no more than 70 percent salvaged material be included in the mixture.

Antistripping Agents

If it is determined that the action of moisture on the recycled mixtures has resulted in premature failure, the use of an antistripping agent should be considered. Chemical antistrip agents are commonly used. When use of one of these agents is specified, tests should be performed to evaluate the effectiveness of each proposed chemical antistrip agent when combined with the salvaged material. Preliminary results by Lee and Kennedy (9) have indicated that in many cases certain chemical antistrip agents, when combined with certain asphalt-aggregate mixtures, do not alleviate moisture damage and that the treated mixtures are still moisture susceptible. These results have also suggested that lime may be an effective antistrip agent when used properly. Nevertheless it is mandatory that any proposed antistrip additive be tested with the aggregate and preferably the asphalt cement to be used to ascertain their effectiveness. Possible test methods are the Texas freeze-thaw pedestal test, the boiling test, and static and repeated-load indirect tensile test with and without moisture conditioning. Preliminary indications suggest that the Texas freeze-thaw pedestal test may be quite valuable in evaluating potential antistrip additives and in detecting adverse moisture effects on various asphalt-aggregate combinations (10).

FINAL DESIGN

The materials selected in the preliminary design are evaluated to select the final type and amount of additive required to either rejuvenate the asphalt cement or alleviate stripping and the amount of new aggregate to incorporate into the mixture. The final design involves determining whether the engineering properties of the mixtures selected in the preliminary design are acceptable. The steps to be followed are as follows:

1. Prepare duplicate specimens of mixtures containing the approximate amount of selected additives based on weight of recovered asphalt, aggregate, or mixture as determined in the preliminary design and various percentages of new asphalt or other additives. The aggregate gradation, including the salvaged aggregate plus virgin aggregate, should have a gradation curve similar to that shown in Figure 1.
2. Test the prepared specimens according to the standard tests used by the design agency.
3. Compare the results from step 2 with those required in the current specifications for conventional mixtures.
4. Test the prepared specimens by using the static and repeated-load indirect tensile test.
5. Compare the results from step 4 with those obtained for conventional mixtures. Properties recommended for consideration are tensile strength, static modulus of elasticity, and resilient modulus of elasticity. The relationships between the above properties and the amount of additive should be developed by testing recycled mixtures prepared at various additive contents. Sample relationships are shown in Figures 4 and 5. The resulting values should then be compared with desired values even though there is currently a limited amount of data to establish these desired values. Most specifications required minimum values for strength, etc. For recycled asphalt mixtures, the test values on the existing pavement material normally should be specified as a range including a maximum value, since the asphalt in the salvaged mixture is often extremely stiff and brittle.

It can be seen that the effect of softening agents is quite different for materials that experienced brittle failures than for those that experienced nonbrittle failures. For the brittle materials, tensile strength (Figure 4a) decreases rapidly with additional additive, whereas for the nonbrittle material, tensile strength does not (Figure 4b) but generally changes only slightly. The same trend has been observed for static and resilient modulus. However, the stabilities in all cases are reduced dramatically as the percentage of additive increases for both the brittle and nonbrittle salvaged materials.

6. Determine the resistance of the recycled mixture to adverse environmental moisture conditions as previously discussed. The Texas freeze-thaw pedestal test procedure is tentatively recommended for use (10).
7. Evaluate the workability of the mixture by visual inspection and make necessary adjustments in the amount of virgin aggregate and additives to be included in the recycled mixture. However, extreme

Figure 4. Effects of amount of additive on tensile strength of salvaged mixtures (a) with brittle asphalt cement and (b) with nonbrittle asphalt cement.

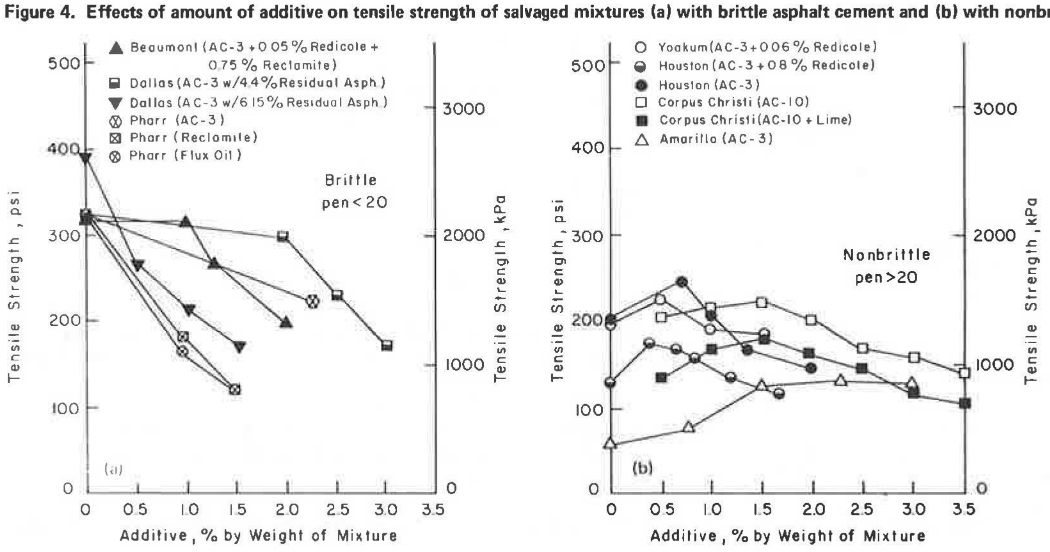
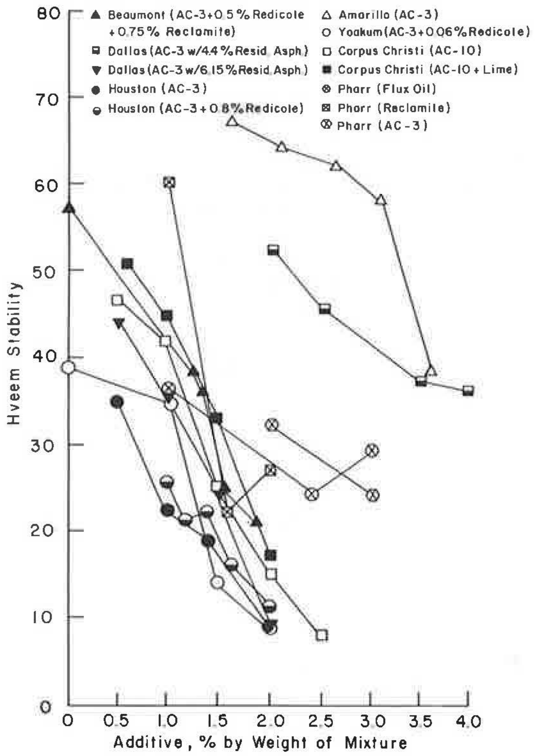


Figure 5. Effects of amount of additive on Hveem stability of brittle and nonbrittle recycled mixtures.



care should be exercised to prevent workability requirements from adjusting gradations and binder content to the point that unstable mixes are produced.

RECOMMENDED INDIRECT TENSILE DESIGN VALUES

Results from previous studies have been used to evaluate the tensile strength, static modulus of elasticity, and resilient modulus of elasticity of both laboratory-prepared and in-service asphalt mixtures. Since these materials are performing satisfactorily in the field, they represent a guide to the level of engineering properties that should provide satisfactory service for recycled mixtures.

Based on the results reported (11-13) for various types of asphalt mixtures, typical values of mixture properties were obtained and are shown below (1 psi = 6.89 kPa):

Property	Design Value (psi)
Tensile strength	73-203
Static modulus of elasticity	$0.10\text{-}0.51 \times 10^6$
Resilient modulus of elasticity	$0.25\text{-}0.94 \times 10^6$

It is recommended that desirable values of engineering properties be determined for the particular location and function of the proposed recycled material.

An example of the use of the desired range of material properties to select the percentage of additive is shown in Figures 6 through 8. Specimens are prepared and tested at various additive contents and the results are plotted as in Figures 6 through 8. At the point where the line of best fit for the test results intersects the middle of the acceptable range of properties, the optimum percentage of additive for the property is obtained. For example, in Figures 6, 7, and 8, these percentages of additives

Figure 6. Determination of percentage of additive from selected range of tensile-strength values.

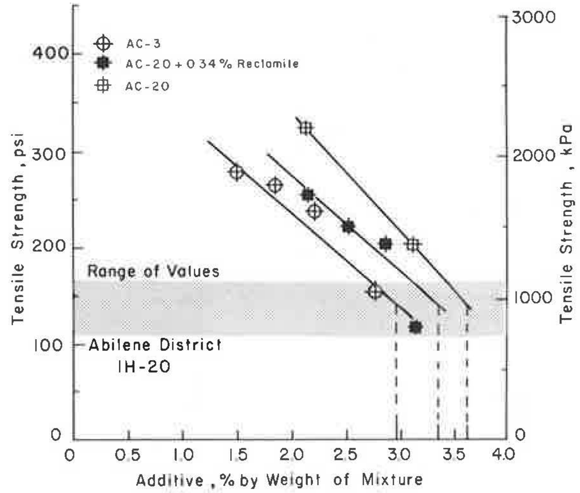
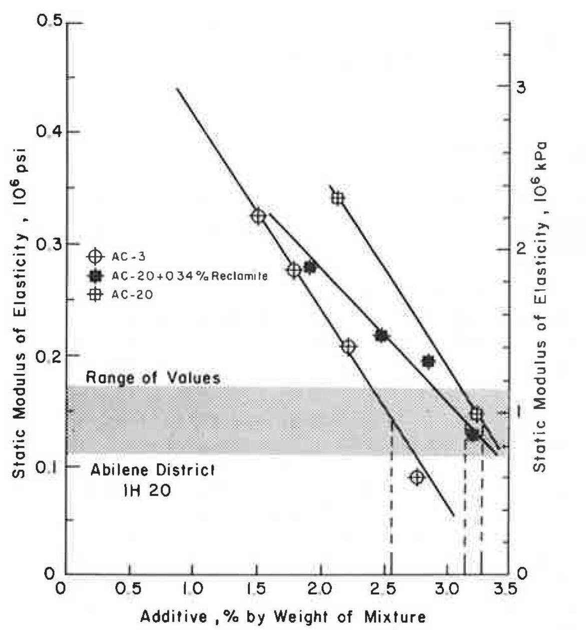


Figure 7. Determination of percentage of additive from selected range of values of static modulus of elasticity.



are shown for each combination of asphalt or asphalt and additive. The individual optimums for the AC-3 are 2.9, 2.6, and 2.7 for tensile strength, static modulus, and resilient modulus of elasticity, respectively. It should be noted that other additives could be investigated and might be acceptable.

QUALITY-ASSURANCE RECOMMENDATIONS

Based on the experience gained to date on designing mixtures for 15 recycling jobs and observing the construction process in the field in the state of Texas, the following recommendations on areas of quality assurance are proposed as the most significant. By paying careful attention to these areas and exercising adequate controls in the field, variations can be kept to an acceptable level. The result will be a reliable product that is expected to perform satisfactorily for its entire design life.

Figure 8. Determination of percentage of additive from selected range of values of resilient modulus of elasticity.

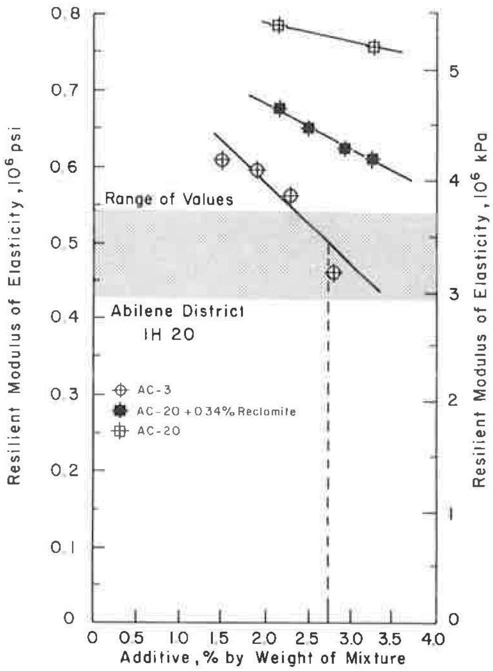
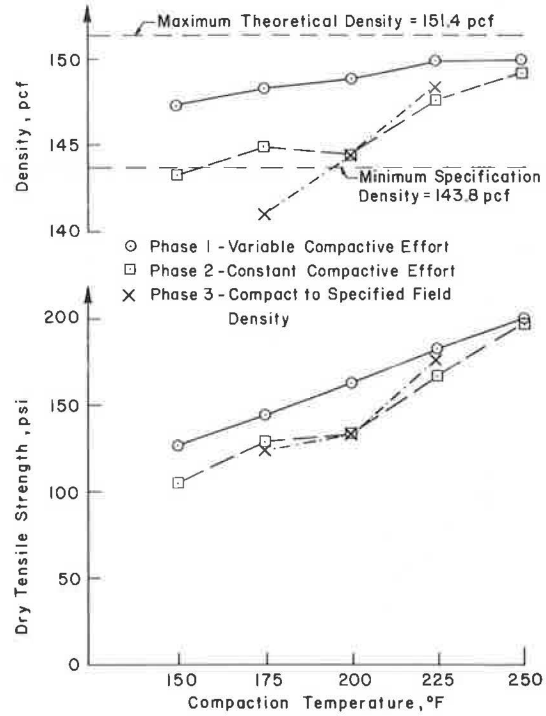


Figure 9. Dry tensile strength for three phases of compaction study of recycled mixtures on IH-10 near Winnie, District 20, Beaumont, Texas.



In General Design

The primary areas of concern for quality assurance lie with the effects produced by variations in the following:

1. Subsection identification and sampling: Use not only design differences but also differences in maintenance and rehabilitation actions as well as type and cause of distress to subdivide for design. Sample within each subsection to secure representative materials so that material variations can be identified and evaluated.
2. Gradations: The designer must know how the material is to be removed, crushed, and blended in order to be able to evaluate variations and their propensity for generating performance problems.
3. Asphalt content: Total variations in extracted-asphalt content along the roadway can be significant. In Texas the construction tolerance on asphalt content is ±0.5 percent and data from dryer drum mixers indicate that as much as 30 percent of the extraction values exceed that tolerance (5). This construction variation plus additional variations produced by maintenance and rehabilitation operations may increase the inherent variation.

In Preliminary Design

The primary areas of concern for quality assurance lie with the effects produced by variations in the following:

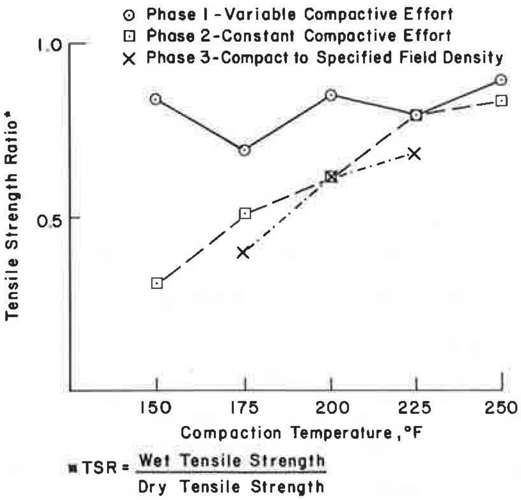
1. Quantity of new material: Strive for a well-graded mixture that produces a smooth grading curve. Avoid humps in the grading curve near the No. 40 sieve that produce a fine mixture that is tender. Mixtures that have 50-70 percent salvaged material seem to be more forgiving to variations in asphalt content, density, etc.
2. Softening agent selected: Ensure that the action of the agent on the salvaged asphalt is the same in the field as it is in the laboratory.
3. Antistrip agent: Test to ensure that it works. Several tests are currently available; however, it is recommended that the Texas freeze-thaw pedestal test be considered.

In Final Design

Of critical importance in this activity is that the designer ensure that the various mixtures to be evaluated be combined under the same conditions in the laboratory as those to be used in the field. For example, the standard hot-mix design procedure often specifies that the mixture ingredients are to be mixed and compacted at a relatively high temperature in the laboratory; however, if a dryer drum plant is used, the mixing and compaction temperature could be significantly less. Thus the recycled mixture should be mixed and compacted at the lower temperatures. This may be of particular importance with recycled mixes since the action of the rejuvenating agent or new asphalt cement may be totally different at the different temperatures and under different mixing conditions. In addition, the amount of water present in the new aggregate as well as the salvaged mixture will almost certainly be different if one set of materials is prepared under standard mix design conditions while the other is run through the dryer drum plant. The combined effect of variations such as these between laboratory procedure and field conditions could be larger than all others, and the mixture produced in the field could have significantly different properties from those produced in the laboratory.

For example, Figure 9 (8) shows the effect of varying the compaction temperature for a laboratory study designed to simulate observed field densities and compaction procedures. The compaction temperature behind the laydown machine and range of field densities observed were used to set the ranges for the study. It can be noted in Figure 9 that the dry tensile strengths vary significantly with laydown

Figure 10. Tensile-strength ratios for three phases of compaction study of recycled mixtures on IH-10 near Winnie, District 20, Beaumont, Texas.



temperatures for all phases of the study. It should also be noted that all but two of the specimens had densities that met the minimum specifications. A set of specimens was also compacted and tested in a wet condition, and the tensile-strength ratios were calculated and plotted in Figure 10 (8). It should be noted that for the specimen compacted at constant compactive effort but at varying temperatures, the tensile-strength ratios are significantly lower at the lower temperatures. This lower ratio reflects the increased water susceptibility for mixes compacted at lower temperatures. However, if the compactive effort is increased as in the phase-1 curve of Figure 10, the tensile-strength ratios are much higher. Also, as the compaction temperature increases, the efficiency of the compactive effort is shown by the converging phase-1 and phase-2 curves. This convergence points out the necessity of maintaining proper compaction temperature, especially when roller patterns are used instead of density control.

In summary, an important point to be emphasized is that in the design of recycled mixtures, special care must be exercised to ensure that the laboratory heating, mixing, and compaction conditions correspond as nearly as possible to those expected in the field. Diligence in applying such control will pay off by having a mixture in the field that reacts to variations in a manner similar to that of variations observed in the laboratory specimens.

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Ensuring Quality in Hot-Mix Recycling

ELTON R. BROWN

The experience of the U.S. Army Engineer Waterways Experiment Station in ensuring quality of hot-mix recycled asphalt concrete is discussed. This experience includes the use of low-viscosity asphalt and recycling agents to modify the existing aged asphalt and to provide the additional needed asphalt. Batch plants and drum mixers have been used to produce the hot-mix recycled asphalt concrete. These recycled mixtures have been satisfactorily used in the construction of binder courses and surface courses.

The state of the art of designing and constructing pavements composed of recycled materials has now advanced to a point where recycling can be considered as an alternative to conventional procedures for most paving jobs. In the past, engineers have been reluctant to consider recycling because (a) it was a new process with unknowns, (b) the technology and equipment needed were not sufficiently developed for recycling, and (c) it was simply not cost-effective for most jobs.

Over a period of years, a change in attitude of pavement engineers has been brought about by several factors. The oil embargo of 1973 stressed the point that there is not an unlimited supply of asphalt materials. Since the embargo, the law of supply and demand had pushed the price of asphalt to $200/ton by 1981. As recently as 1975, the price of asphalt cement was approximately $70/ton.

The amount of high-quality aggregate has become limited in many areas, which has caused the cost of these aggregates to increase substantially. In many locations, economics has forced the use of low-quality aggregates, which has resulted in pavements with reduced life. The use of recycled materials in these areas will provide high-quality materials at lower costs.

During the last few years, technology developed to the point that recycling is no longer in the experimental stage. Equipment has been developed that can properly remove the old pavement materials and mix these reclaimed materials with virgin aggregates, asphalt, and a recycling agent to produce a satisfactory recycled asphalt concrete. There still exist problems that are peculiar to recycling; however, the number and complexity of these problems have been reduced significantly in recent years.

This paper discusses procedures used by the U.S. Army Engineer Waterways Experiment Station (WES) to minimize problems during design and construction of recycled mixtures and thus to ensure quality in the hot-mix recycled asphalt concrete.

MATERIAL EVALUATION

Existing Materials

Existing reclaimed materials for hot-mix recycling generally consist of a mixture of asphalt cement and aggregates. It is essential to evaluate the properties of these materials to determine the aggregate and asphalt type that must be used to modify them to meet specification requirements. This initial evaluation is necessary to estimate the properties of materials such as the asphalt cement and recycling agent needed for the job. If possible, this information should be obtained before the project is advertised and be made a part of the bidding documents for information to the prospective contractors.

The analysis of the existing materials consists of extracting the asphalt binder from the mixture and recovering this asphalt from the asphalt-solvent solution. After the asphalt and aggregate have been recovered, tests should be conducted on each of these materials. In order to perform the mixture design for the recycled mixture, it is necessary to determine the apparent specific gravity, water absorption, and gradation of the aggregate and the specific gravity and asphalt penetration of the asphalt. Other aggregate properties that are generally not evaluated but may need to be in cases where the aggregate quality appears to be a problem include the Los Angeles (L. A.) abrasion, percentage of crushed faces, soundness, and amount of rounded natural sand in the mixture. The aggregate requirements should be the same as those for the aggregate to be used in a virgin mixture. The analysis of the asphalt binder should include as a minimum the determination of penetration and specific gravity.

New Materials

The new materials to be added to a recycled mixture generally include the aggregate, asphalt cement, and recycling agent. Depending on the gradation of the aggregate in the existing pavement, the new aggregate may or may not consist of fine and coarse sizes. The new aggregates and reclaimed aggregates when blended should meet the specification requirements for the gradation of total aggregate. If there is a limit on the amount of natural sand that is allowed to be used in a mixture, this limit should apply to the new aggregate to be added to the recycled mixture since the existing aggregate will more than likely already contain natural sand.

The new asphalt cement added to a recycled mixture provides the additional asphalt binder needed and in many cases modifies the properties of the existing asphalt binder. An AC-2.5 asphalt cement can often modify the existing asphalt binder to an acceptable level. To be acceptable, asphalt recovered from a recycled mixture should initially have an asphalt penetration between 40 and 70 for most climatic locations. Acceptable modification of existing asphalt binder depends on the properties of this binder, properties of the new binder, and amount of reclaimed asphalt concrete to be used in the recycled mixture. For instance, the asphalt binder from an existing mix with penetration in the range of 10-15 can generally be modified to satisfactory properties with an AC-2.5 when the amount of reclaimed asphalt concrete to be used in the mixture is 40-50 percent.

When the penetration of the existing asphalt binder is below 10 and/or when the amount of reclaimed mixture to be used in a recycled mixture is more than 50 percent, it is generally necessary to use a recycling agent to properly modify the existing asphalt binder. A small amount of recycling agent can generally modify the existing asphalt binder without satisfying the desired binder content. If additional binder is needed after the asphalt has been modified, this should be accomplished by the addition of an asphalt cement. When AC-2.5 asphalt is used for the additional binder, the amount of recycling agent needed is less than that required when a higher-viscosity asphalt such as an AC-10 is used because the AC-2.5 modifies the properties of the recovered asphalt more than the higher-viscosity asphalts.

At the present time, there is no widely accepted standard for specifying recycling agents. However, it is obvious that the recycling agent must be able to modify the properties of the asphalt binder to the desired characteristics (40-70 pen). The recycling agent must also be resistant to heat so that the properties will not be adversely affected during production of the recycled mixture at the asphalt plant. There are a number of recycling agents on the market with widely differing properties; therefore, caution should be used in specifying and using these recycling agents.

MIXTURE DESIGN

After the properties of the reclaimed materials and new materials have been determined, the mix design should be performed. The mix design establishes the percentage of each of the various materials to be used in the mixture to ensure that the combined aggregate properties, asphalt properties, and mixture properties are satisfactory. These properties should be evaluated in a similar manner to that for virgin materials and mixtures. The properties of the combined asphalt must be determined from asphalt recovered from the recycled mixture. Material and mixture properties used to evaluate and control asphalt mixtures are tabulated below:

Aggregate	Asphalt	Mixture
Specific gravity	Specific gravity	Stability
Absorption	Penetration	Density
L.A. abrasion	Ductility	Voids total mix
Soundness	Viscosity	Voids filled with asphalt
Percentage of crushed faces	Flash point	Flow
Flat and elongated particles	Thin-film oven test	Immersion compression
	Solubility	

The viscosity, ductility, flash point, solubility, and thin-film oven test are used to evaluate the properties of the virgin asphalt binder only. Penetration and specific gravity are used to evaluate the properties of the combined recovered asphalt.

The first step in the mixture design is to determine what percentage of each new aggregate and reclaimed asphalt concrete should be used. The amount of reclaimed asphalt concrete used in the mixture is usually limited to 70 percent when a drum mixer is used to ensure that a satisfactory mixture is obtained and pollution requirements are satisfied. When a modified batch plant is used to produce the recycled mixture, the amount of reclaimed material used in the mixture is generally restricted to a maximum of 50-60 percent. The availability of reclaimed material, economic considerations, pollution control requirements, and practical considerations of the type of plant to be used are usually considered in selecting the amount of reclaimed material to be used in the mixture. After the percentage of reclaimed material has been selected, the percentage of each virgin aggregate to be used in the mixture can be selected to provide a satisfactory blended gradation.

The second step is to determine the type of binder and/or recycling agent to be used in the mixture. For most areas within the United States, it is desirable that the penetration of the asphalt binder recovered from the recycled mixture be 40-70. These criteria can generally be met with the use of a low-viscosity asphalt (such as AC-2.5) when the amount of reclaimed material used in the mixture is 50 percent or less and when the penetration of the existing asphalt binder is 10 or more. If a low-viscosity asphalt cement can be used to provide additional asphalt and modify the existing asphalt binder to an acceptable range, a recycling agent should not be used.

If it is necessary to use a recycling agent, the smallest amount that can be used to properly modify the existing asphalt should be selected. An excessive amount of a recycling agent will reduce the viscosity of the asphalt binder excessively, thus causing insufficient strength in the mixture. Too much recycling agent can also cause an oily film to form on the aggregate that will prevent satisfactory adhesion of the asphalt to the aggregate. The additional asphalt needed should be provided by the use of an asphalt cement. Since there are no widely accepted criteria for specifying recycling agents, the use of recycling agents that have shown satisfactory performance in the past is recommended.

The third step in developing a job-mix formula is to select the percentage of asphalt cement and/or recycling agent to be used in the mixture and to ensure that the mixture properties are satisfactory. The percentage of binder to be added is selected in a manner similar to that for virgin mixtures. Typical designs for a binder course that uses AC-2.5 asphalt and various amounts of reclaimed materials are shown in Figures 1-3. The optimum mixture properties for these designs are indicated in Table 1. When 40 percent reclaimed material was used, the modified asphalt did not provide the desired stability properties (modified asphalt binder did not possess satisfactory viscosity). When 50 or 60 percent reclaimed material was used, the mixture did possess satisfactory properties.

Potentially, the recycled mixture may consist of four aggregates (reclaimed material, coarse aggregate, fine aggregate, and natural sand) and three binder materials (reclaimed asphalt binder, new asphalt cement, and recycling agent). It is difficult in the laboratory to properly mix this large number of materials; therefore, a high variation in test results is expected. These materials can be properly mixed in large quantities at the asphalt plant, and consistent test results should be obtained. Full-scale plant production of recycled materials should never begin until a satisfactory field mixture design has been developed. This field mix design may be no more than a verification of the laboratory mix design, but it is necessary to determine that the mixture produced at the plant is satisfactory before full-scale production.

The mix design is normally performed on samples of material obtained from the existing pavement before milling or removing and crushing. When the existing pavement is milled or removed and crushed, there is generally more material passing the No. 200 sieve (dust) than that indicated by the original sample. This additional dust is caused by abrasion effects of the milling machine or crusher. Dust is also manufactured when producing recycled hot mix at an asphalt plant. In order to ensure that the amount of material passing the No. 200 sieve is not excessive, the original mix design should be well below the maximum limits for material passing the No. 200 sieve.

REMOVAL OF EXISTING MATERIALS

The quality of the in-place materials cannot be controlled; therefore, these materials must be used with consideration given to modifying the existing quality if necessary. When properties of the existing materials do not meet the specification requirements for the recycled mixture, the virgin materials (asphalt, recycling agent, and aggregate) when added must be able to modify these materials to meet these requirements. Although the quality of the existing materials cannot be controlled, it is imperative that the existing material be milled and stockpiled in such a way that properties of the materials are consistent throughout the stockpile. When the properties are consistent, the material can be properly modified; however, if the stockpile properties are highly variable, an undesirable product will be produced.

The existing asphalt concrete must be uniformly removed to the desired grade without damage to the underlying material. When damage does occur to the base, it should be scarified, moistened if necessary, and compacted. Generally, the asphalt concrete is removed with a milling machine, but occasionally a ripper is used to remove the existing asphalt. After being ripped from the pavement, the material is transported to a crusher for further processing. When a milling machine is used, further crushing is not normally needed.

The use of a ripper generally requires that the existing asphalt concrete be removed full depth to the surface of the base course. When the existing asphalt mixture is ripped, particles from the surface of the base course may tend to adhere to the asphalt concrete being removed. A small amount of the base course in the asphalt mixture will not cause any problem so long as the amount adhering to the asphalt mix does not vary significantly between adjacent areas being removed.

The milling machine can remove the existing asphalt concrete to any desired depth. Generally,

Figure 1. Recycled asphaltic-concrete mix design: AC-2.5 asphalt binder and 40 percent reclaimed asphalt concrete.

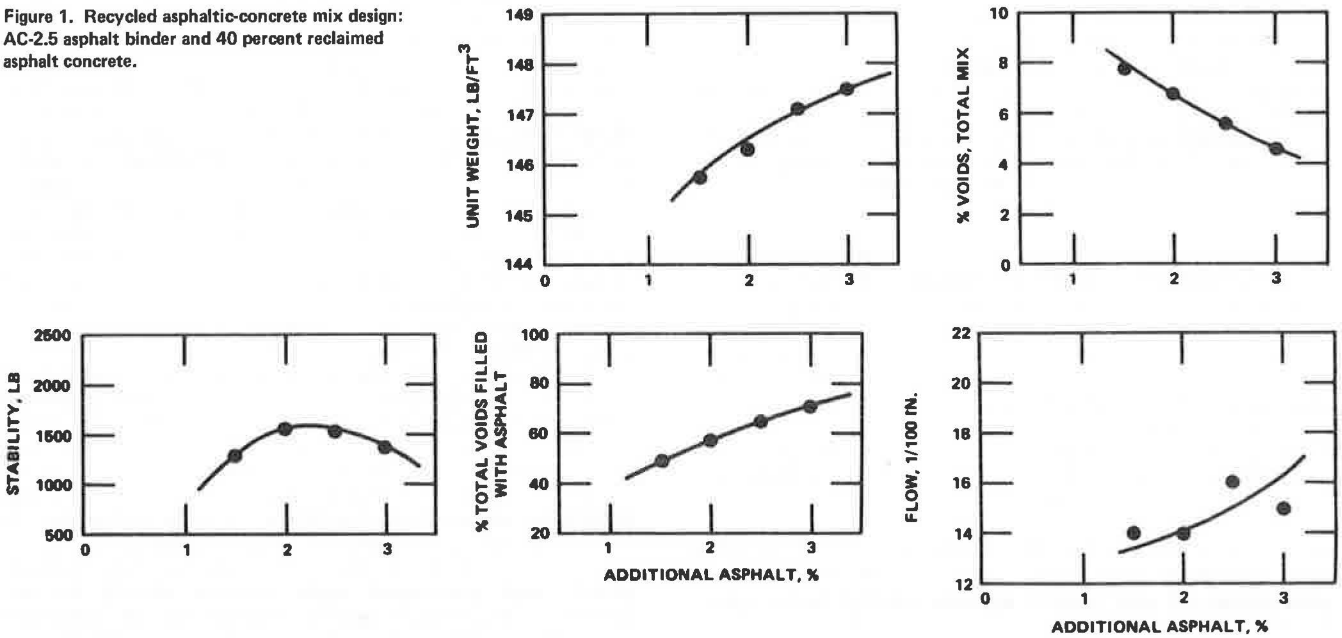


Figure 2. Recycled asphaltic-concrete mix design: AC-2.5 asphalt binder and 50 percent reclaimed asphalt concrete.

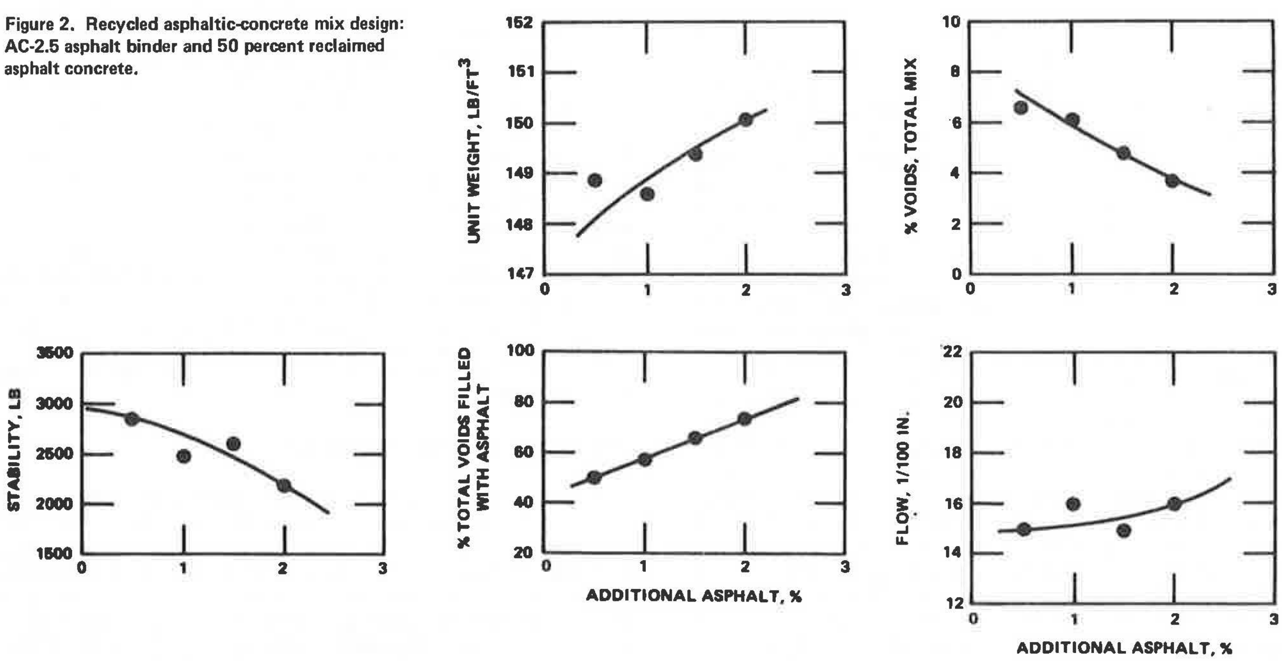


Figure 3. Recycled asphaltic-concrete mix design: AC-2.5 asphalt binder and 60 percent reclaimed asphalt concrete.

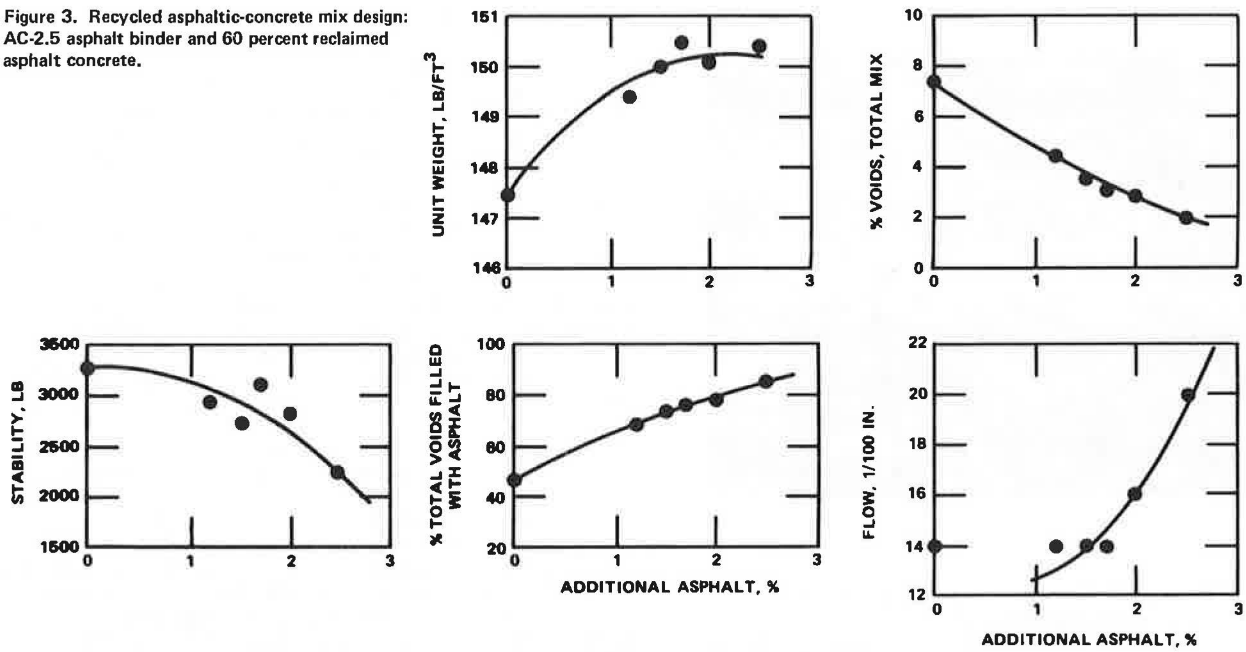


Table 1. Mix designs for recycled asphalt concrete at Pope Air Force Base.

Mixture Property	Binder-Course Criterion	Reclaimed Asphalt Concrete in Mix (%)		
		40	50	60
Optimum additional asphalt content (%)	-	2.3	1.1	0.6
Density (pcf)	-	146.6	149.2	148.4
Stability (lb)	1800 min	1440	2600	3200
Flow (0.01 in)	16 max	15	15	14
Voids total mix (%)	5-7	6.0	5.6	5.6
Voids filled with asphalt (%)	50-70	60.0	58.0	58.0

3-4 in can be removed in one pass. Grade-control devices can be used when close control of the grade is required. When the full depth of asphalt concrete is to be removed, approximately 0.5 in of asphalt mixture is generally left over the base course to prevent damage from the milling machine and to prevent water from entering and damaging the base course.

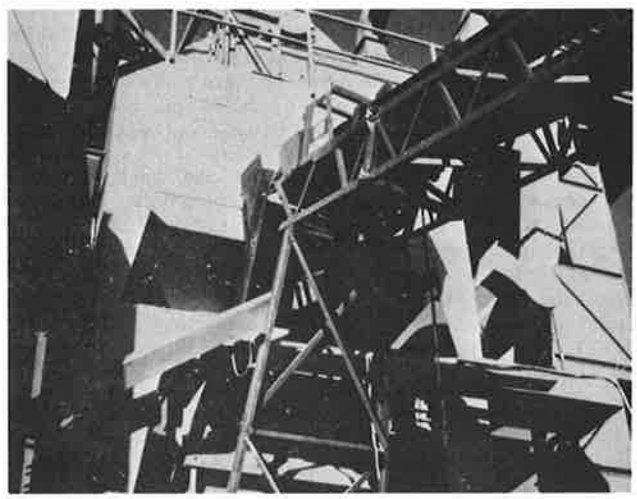
When the asphalt mixture is removed down to the base course, steps should be taken to protect the base course from water intrusion. These steps may involve the application of a prime coat or the placement of the bottom course of recycled asphalt mixture within a short time.

The cutting teeth of the milling machine must be replaced periodically. When the teeth become dull, oversized chunks of the asphalt mixture are produced. Milling with dull teeth near the bottom of the asphalt-concrete mixture can cause the mix to shear between the base course and asphalt mixture, which produces large chunks of asphalt concrete that will have to be removed from the mixture or broken down further before the mixture can be fed through the asphalt plant.

QUALITY CONTROL OF HOT-MIX RECYCLING JOBS

WES has been involved in the quality control for a number of hot-mix recycling jobs. These jobs have included recycling at Pope Air Force Base, North Carolina; Reese Air Force Base, Texas; and Lajes Air Force Base, Azores Islands, Portugal. These three jobs have provided experience for a range of material and equipment types.

Figure 4. Modified batch plant used at Pope Air Force Base.



The asphalt concrete on the runway at Pope Air Force Base was recycled in the summer of 1980 (1). A modified batch plant (Figure 4) was used to produce the recycled hot mixture, which consisted initially of 50 percent reclaimed asphalt-concrete materials and 50 percent new materials. A recycling agent was not needed for this recycled mixture. An AC-2.5 asphalt cement was added to modify the existing asphalt cement and to provide the additional asphalt binder.

The modification to the plant consisted of adding a hopper for the reclaimed asphalt mixture and adding a conveyor belt to carry this reclaimed asphalt mixture from the hopper to the scales. The virgin aggregate was fed through the dryer and heated to approximately 600°F. When the virgin aggregate, reclaimed asphalt concrete (approximately 50 percent of total mixture), and AC-2.5 asphalt

Figure 5. Drum mixer used at Lajes Air Force Base.

cement were mixed in the pug mill, the resulting temperature was approximately 275°F.

A 2-in screen was placed over the reclaimed asphalt-concrete storage bin to remove all large chunks while the bin was being loaded. The removal of the large chunks allowed the remaining reclaimed asphalt concrete to break down in the pug mill and properly mix with the virgin aggregate and AC-2.5 asphalt cement. A few chunks were noticed during the laydown operation, but these chunks were soft and were compacted under the rollers with no noticeable problems. The recycled mixture was used in the leveling course, whereas the virgin mix was used for the surface course.

The material that was removed had been designed to satisfy surface-course requirements. The recycled mixture designed to satisfy binder-course requirements resulted in the addition of only 1.3 percent new asphalt binder to the recycled mixture. This low percentage of new asphalt cement produced a combined asphalt binder with an asphalt penetration of 27. When the amount of new asphalt was increased to 1.5 percent, the asphalt penetration rose to 37. Subsequently, when the amount of new asphalt was increased to 1.8 percent and the amount of reclaimed asphalt concrete decreased to 45 percent, the resulting mixture met the surface-course requirements with a recovered asphalt penetration near 50. This final mixture was considered to contain the proper asphalt content and asphalt quality.

The asphalt concrete on the runway at Reese Air Force Base was recycled in 1981 to produce a binder course before overlaying with a new asphalt-concrete mixture. A drum mixer that had to be modified to produce recycled asphalt concrete was used on this job. The modification provided for the addition of the reclaimed asphalt-concrete materials to the drum mixer. The mixture design required that 50 percent reclaimed asphalt-concrete materials be used in the recycled mixture. It was necessary to add 0.4 percent recycling agent and 2.5 percent AC-5 to the recycled mixture to provide the additional asphalt needed and to properly modify the asphalt binder. The penetration of the resulting asphalt binder was approximately 50.

The asphalt concrete on several taxiways and parking aprons at Lajes Air Force Base was recycled in 1981, and the asphalt concrete from additional areas is scheduled to be recycled in 1982 to produce material for binder courses and surface courses for these areas. A drum mixer that was designed and constructed to produce recycled mixtures or new mixtures was used to produce the recycled asphalt concrete (Figure 5). The mixture design required that 50-60 percent reclaimed asphalt-concrete materials be used in the recycled mixture. The addition of approximately 0.7 percent recycling agent and 3.1 percent AC-2.5 to the recycled mixture resulted in an average penetration of recovered asphalt binder of 48. A screen was placed over the reclaimed asphalt-concrete storage bin to prevent oversized material (primarily chunks of asphalt concrete) from getting into the recycled mixture.

During the plant operation, a number of tests on the materials and mixture must be conducted to ensure quality in the recycled asphalt-concrete mixture. Basically, these tests are the same as those for conventional mixtures, but a few additional tests are needed. Tests on conventional mixtures include extraction of the asphalt binder from the mixture, which allows the gradation of the aggregate and the asphalt content to be determined. When recycled mixtures are tested, it is necessary to recover the asphalt from the extract and conduct penetration tests on the asphalt binder to ensure proper asphalt consistency. Recovery of the asphalt should be done in such a way that the amount of mineral filler in the recovered asphalt is minimized. The recovery procedure requires that the reflux extraction be used to extract the asphalt binder or that some method such as the high-speed centrifuge be used to remove the mineral filler from the asphalt-solvent solution if the rotorex method is used for extraction. Other than the recovery of the asphalt binder and the penetration test, all other tests are the same as that for conventional mixtures. A summary of the tests required in the field laboratory for conventional mixtures and recycled mixtures is shown below:

Both Mixtures	Recycled-Asphalt Mixtures
Marshall compaction and test: stability, flow, density, voids total mix, and voids filled with asphalt	Asphalt recovery
Aggregate gradation	Asphalt penetration
Asphalt extraction	
Temperature	
Density: laboratory and field cores	

The variability of aggregate gradation and asphalt content is important to the performance of asphalt-concrete mixtures. Due to the small amount of elapsed time since the beginning of construction of recycled asphalt concrete, very little has been published on the variability of properties of recycled mixtures. This information on variability has been published for new asphalt-concrete mixtures (2).

The average and standard deviation can be used to conveniently summarize a large amount of data and yet describe the variability of that data. An analysis of the data for the three recycled asphalt-concrete jobs and a comparison with the variability of new mixtures are given in Table 2.

As given in Table 2, the variability of the aggregate gradation and asphalt content for recycled mixtures is higher than that for new mixtures. This higher variation may be caused by a number of factors. First, the data for new mixtures were obtained primarily from batch plants, whereas two of the three recycled jobs were produced with drum mixers. Since a batch plant rescreens the aggregate and weighs the aggregate fractions and asphalt in each batch, the variation of asphalt concrete produced in a batch plant should be less than that

Table 2. Variability of recycled asphalt-concrete materials.

		Standard Deviation			
			Recycle at:		
Property	Avg	New Mixtures	Pope Air Force Base	Reese Air Force Base	Lajes Air Force Base
Aggregate gradation (percent passing)	95	2.0	2.5	2.0	2.5
	90	2.5	3.0	4.0	3.5
	80	2.5	3.5	4.5	4.0
	70	2.5	4.0	5.0	4.5
	60	2.5	4.0	5.0	4.5
	50	2.5	4.0	4.0	4.0
	40	2.5	4.0	3.5	3.5
	30	2.0	3.5	2.5	2.5
	20	1.5	2.5	2.0	2.0
	10	1.0	2.0	2.0	1.0
	5	1.0	1.0	-	1.0
Asphalt content	-	0.20	0.32	-	0.58
Stability (%)	-	10	10	13	13
Flow	-	-	0.8	2.0	1.4

produced in a drum mixer. Hence, the differences in variation of the gradation and asphalt content for new mixtures and recycled mixtures may be partly caused by the type of plant used in the analysis.

Second, very little can be done to control the variation of the reclaimed asphalt-concrete materials. In other words, the more variability that exists in the aggregate gradation and asphalt content of the reclaimed materials, the more variation that will occur in the aggregate gradation and asphalt content of the recycled mixture. This variation in properties of the recycled mixture can be minimized by proper handling but cannot be controlled as closely as that for new asphalt-concrete mixtures.

Third, two asphalt products are often added to the recycled mixture. These two products are generally an asphalt cement and a recycling agent. Adding two liquid materials to the recycled mixture provides more chance for error and thus causes a higher variability in asphalt content with recycled mixtures than with new mixtures.

The high variation in aggregate gradation and asphalt content for recycled materials is undesirable, but the variation is not so large that unacceptable material is necessarily obtained. The variation in stability and flow indicates that the variation in asphalt content and gradation did not excessively affect the properties of the mixture. This high variation does require continuous monitoring of the production quality so that adjustments in the mixture design can be made as needed.

CONCLUSIONS AND RECOMMENDATIONS

Recycled asphalt mixtures should be designed and controlled by using the same techniques as those for conventional mixtures.

Additional testing is necessary during the mix design of recycled mixtures to extract the asphalt from the reclaimed asphalt concrete so that the asphalt content and asphalt properties, such as penetration and specific gravity, can be determined. The amount of new asphalt and/or recycling agent to be used in the mixture must be selected so that the mixture properties as well as the properties of the combined asphalt are satisfactory.

Additional tests are required during plant production to ensure that the recycled asphalt-concrete mixture is acceptable. These tests include recovery of the extracted asphalt and a measurement of the penetration of this recovered asphalt.

With the exception of additional tests on recycled asphalt mixtures to evaluate quality of the combined asphalt binder, recycled-asphalt mixtures should be designed, produced, and placed by using the same techniques as those for conventional mixtures. Based on the analysis of three hot-mix recycled jobs, it appears that the variation in aggregate gradation and asphalt content for recycled mixtures is larger than that for new mixtures. This increase in variability requires that recycled asphalt-concrete construction jobs be continuously monitored so that mix adjustments can be made as needed.

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Ensuring Quality of Recycled Asphalt Concrete

CHARLES F. POTTS

Specifications related to the control of the quality of hot-mix recycled asphalt-concrete pavements have become increasingly important in recent years. The specifications developed and used by the Florida Department of Transportation to ensure that a high level of quality is maintained during the design and construction process are discussed. The need for control of the uniformity of the mix is stressed with corresponding recommended specifications for the measurement of viscosity levels of the reprocessed binder material. Performance as well as economic and energy considerations are discussed. Recommendations are also given for using strength equivalencies equal to those of the conventional paving mixtures.

When recycling of asphalt-concrete pavements is considered as a design alternative, all too often the initial thought of those individuals involved in the construction process is that an inferior product must be accepted. In many cases, this has resulted in the use of specifications that do little, if anything, to ensure the level of quality of asphalt-concrete mixtures and the flexible pavement structure.

Hot-mixed recycled asphalt-concrete mixtures can be produced from a variety of materials obtained from many sources. The handling and processing techniques permitted by specifying agencies will ultimately dictate the level of quality in the final product. In many cases, hot-mixed recycled asphalt pavements have been viewed as a means of making use of waste materials. This attitude is changing, but there still seems to be a general hesitancy to expect the same standards of quality required of conventional paving mixtures.

In the case of the Florida Department of Transportation (FDOT), some of these same fears existed in the initial stages of specification and procedural development. A great amount of latitude was given to the contractor to permit wide variations in asphalt content and gradations. In addition, little effort was made to actually specify the physical properties of the reprocessed asphalt cement in the final mixtures.

It was the attitude of FDOT that initial projects should be constructed and evaluated very carefully so that realistic specifications could be developed that would ensure performance levels equal to or exceeding those of conventional paving projects.

Other research efforts had confirmed the fact that proper specification controls would yield quality paving mixtures. Little and Epps (1) have reported insignificant differences between properties of recycled and conventional asphalt mixtures or pavements. There are also indications that recycled asphalt cements may not harden (increase in viscosity) as rapidly as the original asphalt (ASTM STP 662).

Portions of the data obtained from the hot-mix recycled pavements constructed in Florida have been published previously (2-4). In June 1980, guidelines (5) were published and distributed throughout the state to FDOT's materials, construction, and design personnel for use in selecting and evaluating pavements as potential candidates for recycling. Although minor modifications are required to meet the needs of individual projects, basically a standard set of specifications is used for all recycled pavements. These specifications include most of the same control restrictions of any conventional paving mixture, and laboratory and field test results have confirmed that these control levels can be maintained. Furthermore, it has become evident that violations of the basic principles of good quality control and acceptance limits result in the same poor performance of pavements that would be expected in any other construction phase.

PRELIMINARY EVALUATIONS

Ensuring the quality of recycled asphalt pavements is very similar to ensuring the quality of any other conventional mixture. The primary problem exists in the fact that we have been using conventional procedures for so long that we have lost touch with the evolutionary development of all of the quality control measures employed in association with the component parts of the final mixture. Many engineers have failed to view the salvaged asphalt and aggregate combination as another commercial component that must be monitored and controlled in a fashion similar to that for any of the other materials composing the final asphalt-concrete paving mixture.

It is for this reason that I would like to devote a portion of this paper to some of the more important aspects of the control of the salvaged asphalt-concrete mixture.

As is the case with conventional asphalt paving, the quality control measures adopted for the final mixture are worth very little if there is no assurance that the component parts are produced under similar standards.

The procedures presented here for the development of a final mix design for a recycled-asphalt project are what I believe to be a preferred sequence of events. The steps taken may vary depending on (a) how the salvaged material is obtained (milled or processed through a crusher) and (b) the percentage of salvaged material proposed for incorporation into the final mixture.

Regardless of the above conditions, the major elements in the process are (a) materials characterization of salvaged asphalt-concrete mixture, (b) preliminary mix design, and (c) final mix design. The purpose of the preliminary and final mix designs is to establish the estimated design asphalt content and to determine the final job-mix design that conforms to the requirements of the standard specifications.

Characterization of Salvaged Asphalt-Concrete Mixture

A sufficient quantity of salvaged asphalt-concrete mix should be obtained in order to determine asphalt content and gradation and to perform preliminary mix design tests. In Florida, the Marshall design method is used to establish the standard mix design. Sampling of these materials should be based on consideration of the following:

1. If material is obtained from an existing stockpile or from a crushing process, the same frequencies and sampling locations should be used as would be required in conjunction with any commercially produced aggregate. Special care must be taken to identify quantities of materials that have widely varying viscosity levels. Normally, if average viscosity values can be established and the variance of these quantities does not exceed 10-15 percent of the mean, the variations can be easily handled in the field.
2. If the material is to be obtained from exist-

ing pavement by the cold-milling process, variations in layer thicknesses and type of asphalt-concrete mixtures, according to data from prior sampling and original construction plans, must be established. Care must be taken to identify changes in materials that result from having the recycling project encompass sections of existing pavement constructed under more than one original construction contract.

3. Pavement being removed by cold milling and reprocessing must also be separated by viscosity level. Variations in the degree or type of cracking may provide indications where additional samples should be taken for recovery and characterization of the existing asphalt cement.

The standard procedure established for use in Florida when an existing pavement is characterized requires a minimum of two samples per lane mile or five representative samples per project to be tested to determine asphalt content and aggregate gradation and to obtain recovered asphalt for testing.

The bitumen is extracted from the asphalt cement by using the trichloroethylene reflux procedure (AASHTO T-164, Method B) and recovered by the Abson Method (AASHTO T-170). Samples should be cut to approximately the same thickness as anticipated for the milling depth. Sufficient quantity of asphalt cement must be recovered, regardless of whether the mix is from an existing roadway or previously developed stockpile, to conduct the following tests:

1. Absolute viscosity at 140°F (60°C) (ASTM D2171);
2. Cannon constant stress rheometer: (a) viscosity and shear susceptibility at 77°F (25°C) and (b) viscosity, shear susceptibility, and shear modulus at 41°F (5°C);
3. Kinematic viscosity at 275°F (135°C) (ASTM D2170); and
4. Penetration at 77°F (ASTM D5).

On standard hot-mixed recycling projects bid in Florida where portions of the existing pavement are removed and permitted to be incorporated into the new mix, a summary of the characterization data is provided in the bid document. An example of such a summary is shown below (average values based on test results from top 3 in of roadway):

Extracted gradation:

Sieve Size	Percent Passing
3/4 in	100
1/2 in	98
3/4 in	95
No. 4	70
No. 10	48
No. 40	36
No. 80	16
No. 200	6.8

Asphalt content: 6.0 percent
Viscosity at 140°F, 102 907 poises
Penetration at 77°F, 17

This provides the prospective bidders with information that can be used when potential material combinations are developed for bid purposes.

Preliminary Design

This aspect of controlling the quality of recycled asphalt mixtures is often minimized, but it is believed to be a key in arriving at the proper combination of salvaged asphalt concrete, new aggregate,

Table 1. Preliminary design blend.

Sieve Size	Percent Passing: Salvaged Material Extracted Gradation (65%)	Crushed Coarse Aggregate (20%)	Crushed Stone Screenings (15%)	Job-Mix Formula	Specification Range (FDOT Type S-1)
3/4 in	100	100	100	100	100
1/2 in	98	80	100	95	88-100
3/8 in	95	45	100	86	75-93
No. 4	70	7	100	62	47-75
No. 10	48	7	85	45	31-53
No. 40	36	6	58	33	19-35
No. 80	16	4	33	16	7-21
No. 200	6.8	2.8	7.2	6.1	2-7

Table 2. Marshall properties: preliminary design.

Asphalt Content (%)	Air Voids (%)	Voids Mineral Aggregate (%)	Stability (lb)
5.5	5.0	15.0	2080
6.0	3.9	15.0	2018
6.5	2.8	15.0	2030
7.0	2.1	15.4	1825

Note: Optimum asphalt content, 6.0 percent; asphalt cement using 65 percent salvaged material at 6.0 percent, 3.9 percent; new asphalt residual required, 2.1 percent.

and asphalt rejuvenator. This same procedure is used in the design of conventional mixtures except that the percentage of asphalt cement is not affected by asphalt existing in one aggregate material component. The following general procedure should be used:

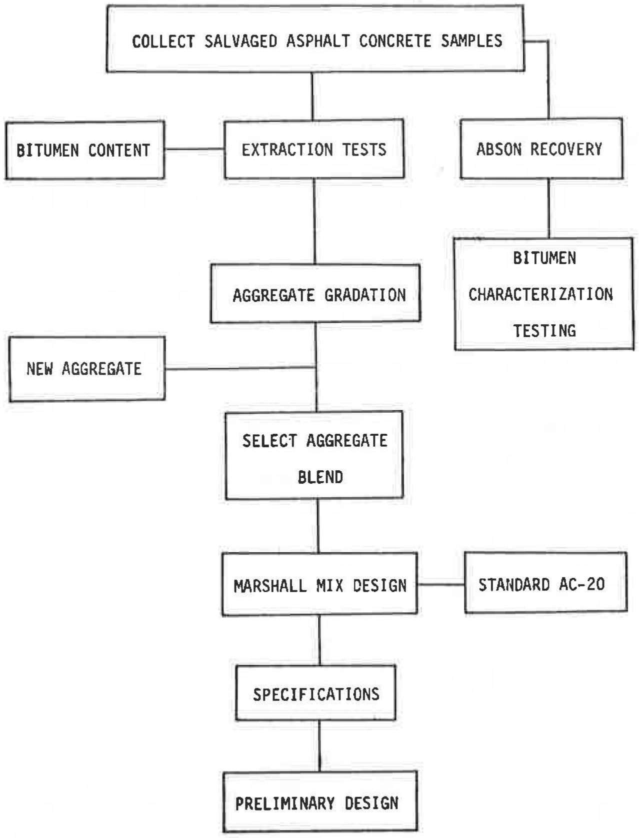
1. The mean value of the extracted gradations of the salvaged asphalt should be combined with new aggregate materials to obtain a final gradation that will comply with standard gradation requirements.
2. A Marshall mix design evaluation should then be performed in order to establish the design asphalt content. The Florida procedure requires the blending of aggregate from the extraction of salvaged mix with new aggregates. The hot mix is prepared by using a standard AC 20 material to prepare the Marshall test specimen. The Marshall properties obtained for the selected mix design should conform to FDOT standard specifications (6).

Examples of the preliminary design blend and Marshall design properties are presented in Tables 1 and 2. The flow diagram shown in Figure 1 illustrates graphically the steps involved in arriving at the preliminary design. The information obtained in the preliminary process becomes the basis of developing bid estimates by the contractors and at the same time provides the materials engineer a beginning point in evaluating the final design.

Final Design

After a contractor has been awarded a project, the proposed aggregate combinations as well as the rejuvenating agent for use in the final mixture must be submitted. Current Florida specification requirements for rejuvenating agents are given below. The asphalt rejuvenator should be a soft asphalt cement or asphalt cement blended with a softening agent or flux oil conforming to the requirements shown below. It should contain an approved anti-

Figure 1. Preliminary mix design.



stripping agent [t°F = (t°C ÷ 0.55) + 32]:

Characteristic	Requirement
Absolute viscosity after thin-film oven test	3:1 ratio minimum
Smoke point	260°F minimum
Flash point	400°F minimum
Solubility	97.5 percent

Residue from the asphaltic emulsion rejuvenator should meet the requirements shown above. The asphaltic emulsion rejuvenator should contain an approved antistripping agent. It should meet the requirements shown below:

Characteristic	Requirement (%)
Storage stability, 24 h	1.0 maximum
Sieve test	0.1 maximum
Residue by evaporation	65.0 minimum

The specified properties of the rejuvenating agents are primarily to assure that the material will be suitable from the standpoint of construction, safety, operation, and handling without excessive alteration of the absolute viscosity (V_{60}). The selection of the rejuvenating agent is a most important factor in controlling the quality of the mix since the standard specifications require that the bitumen recovered from laboratory specimens as well as the plant-produced mix meet the 140°F viscosity requirements of 4500 poises ± 1500 poises.

During the laboratory evaluation of the final mix design, the following areas are evaluated:

1. Are the Marshall mix design requirements in compliance with standard specification requirements? The bitumen demand for the job-mix formula is confirmed, and this in turn permits calculation of the amount of rejuvenating agent required for the recycled mix.
2. The bitumen recovered from the test specimens at optimum asphalt content from the Marshall design procedures must meet the 140°F viscosity requirements of 4500 poises ± 1500 poises. In addition, data are collected to provide for "straddle" design formulations to allow for adjustments of the job-mix formula and/or formulation of the asphalt rejuvenator to achieve the desired end result recovered viscosity.
3. Field adjustments of aggregate gradations obtained after extraction of the bitumen are evaluated as required, if necessary, to maintain specification compliance.

CURRENT SPECIFICATION REQUIREMENTS

The current specifications contain a number of key elements that past experience has indicated play a major role in providing realistic bids as well as ensure a high-quality flexible pavement layer.

Bid Document

Current bid documents provide for no minimum amount of salvaged asphalt-concrete mix for incorporation in the final job-mix formula; however, an upper limit of 70 percent has been included. This provides the materials engineer with latitude to adjust gradations, asphalt contents, and viscosities of the recycled mix. There have been no special gradations developed for recycled mixtures; rather, current standard gradation and Marshall design requirements are specified for these as well as for conventional mixtures.

The asphalt paving mixtures are bid by the ton; the asphalt cement is included as a part of the ton price, regardless of the percentage of salvaged material used by the contractor in the approved job-mix formula. This has expanded the bid competition and has provided the possibility for modified batch plants to compete with drum mixers.

FDOT has, for a number of years, provided an escalation clause in its standard specifications for conventional asphalt paving mixtures. In the past year, an adjustment clause has been included as a part of the supplemental special provisions. The provision is as follows:

> In addition to the pay adjustment for varying asphalt content in the job mix formula as issued, pay adjustments for the quantity of recycling agent included in the payment for Recycled Asphaltic Concrete will be made based on the Asphalt Price Index as specified in Amendment 009 of the Specifications Package. Asphalt Rejuvenator will be based on the Asphalt Cement Index and the Asphalt Emulsion Rejuvenator will be based on the Emulsion Asphalt Index. As an exception, the total adjustment will be made on the final estimate. The adjustment will be made on the actual amount of recycling agent used as determined by field measurement, excluding the quantity required for the adjustment to the six percent asphalt content in the job mix formula.

The price adjustment applies only to the price of the bituminous material, free on board the manufacturer's asphalt terminal, and does not reflect variations in the cost of transportation from the terminal to the job site. Implementation of the adjustment on projects that use recycled asphalt concrete has had the effect of stabilizing the bids and has removed the apprehension surrounding the use of asphalt rejuvenator when there is competition with conventional asphalt-concrete mixtures.

The 6 percent asphalt content level referred to in the adjustment clause is varied based on past history of optimum design levels of FDOT's standard design mixtures.

Construction Control Specifications

After construction has begun, the quality control and acceptance testing required are very similar to those for conventional paving mixtures. Gradation analyses of aggregate component stockpiles are monitored daily along with extractions of the salvaged asphalt-concrete stockpiles. These tests are performed by the contractor's quality-control technician as a part of the plant control program. The department's technician performs extraction tests at a minimum rate of one sample per 1000 tons, or a maximum of one per day, for acceptance purposes. Any variations exceeding the tolerance limits of the established job-mix formula should be corrected immediately.

A major addition to the standard specifications concerned the control of the consistency of the asphalt cement in the final recycled asphalt-concrete mixture. Samples of recycled mix are taken at a minimum frequency of one per 2000 tons, and the asphalt cement is recovered by ASTM D1856. The viscosity of the recovered asphalt is measured at 140°F and must be 4500 ± 1500 poises. This level was established as a result of previous age-hardening studies performed by FDOT (7).

It had been established that recovered samples of asphalt cement taken from conventional paving mixtures produced with AC 20 were in the same viscosity range at the time of placement on the roadway. It is believed that this is one of the single most important quality-control measures, since it ensures the uniformity of the mix consistency during placement and compaction. Should a mix fail to meet this specification requirement, the contractor must adjust the mix immediately.

The corrective action may be accomplished in a number of ways, depending on the degree of nonconformance. The contractor can refer to the straddle design developed during the final design phase to decide whether the percentage of salvaged material should be adjusted or the grade of rejuvenating agent being used should be changed.

The established mix temperature at the time of discharge at the asphalt plant must be in the range of 240-300°F. Our experience has shown that this operating range can be uniformly maintained, and as long as the viscosity level is controlled within specification limits, the mix can be handled in the field in the same manner as any other conventional paving mixture.

There have been no revisions made to FDOT's standard placement and compaction specifications that relate specifically to recycled asphalt-concrete mixtures. We have made every effort to comply with the same standards of quality expected in conventional mixtures. If this is accomplished, it follows that the placement and compaction specifications should be no different than would be used with any similar paving mixture.

DISCUSSION OF PERFORMANCE

Performance data to date are limited since hot-mixed recycled asphalt-concrete mixtures have only been used in Florida for five years. However, the performance has been excellent. These pavements, in all cases, have equaled or exceeded the performance of similar roadway sections constructed by conventional processes.

There are strong indications from data collected that the in-service hardening rate of the recycled asphalts is somewhat less than that of comparable standard asphalt cements. At the present time, firm conclusions cannot be made, but the trends will be monitored until an analysis can be made to confirm the trends that have developed.

In areas where the pavement is in an advanced stage of cracking and the asphalt-concrete layer is removed entirely for reprocessing, the performance has far exceeded the conventional leveling and resurfacing approach. To date, this has been the most effective rehabilitation used by the state, and a major reason has been the elimination of the reflective cracking potential.

ECONOMIC AND ENERGY CONSIDERATIONS

The economic savings and energy reductions have been previously documented and reported by FDOT (2-5,8). Recently completed projects have confirmed the fact that generally there is a 15-30 percent reduction in total cost of the recycled project as compared with conventional paving methods. Depending on the location within the state and location of commercial aggregate sources as related to the recycling project, energy savings have ranged from 25 to 45 percent when compared with the conventional alternative.

SUMMARY

The specifications used by FDOT for control of recycled asphalt-concrete mixtures are still in the development stage, but our experience to date has shown that a high-quality pavement can be constructed incorporating salvaged asphalt materials. The following appear to be key points related to this type of construction process:

1. The same general gradation requirements and design properties should be used when recycled asphalt-concrete mixtures are specified.
2. Design strength equivalencies used in the pavement design process should be the same as those that would be assigned to the same standard mix produced by conventional processes.
3. Recoveries of the asphalt cement from recycled asphalt-concrete mix should be made at regular intervals during the production process. This material should be obtained at the asphalt plant during normal production operations. Absolute viscosity measurements should be performed at 140°F (ASTM D2171) on the recovered-asphalt cements, and the viscosity level should be maintained at 4500 ± 1500 poises.
4. Placement and compaction requirements should not deviate from standard construction requirements used in conjunction with normal paving projects.
5. No lower limit should be placed on the percentage of salvaged material incorporated into the final design; however, an upper limit of 70 percent is suggested. This limitation permits more design latitude and uniformity of the mix during production.

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Method to Establish Pay Schedules for Rigid Pavement

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An equation is derived to compute the appropriate pay factor for any quality level of rigid pavement. The measure of quality used in this development is the estimated load-bearing capacity of the pavement although the results may be applied to specifications based on other quality measures. The appropriate pay adjustment is considered to be the present worth of any expense or savings expected to occur in the future as the result of a departure from the specified level of quality and may be positive or negative. Sensitivity tests demonstrate that the method is reliable provided the input variables are determined with reasonable accuracy. By using input values typical of a relatively urbanized area, this procedure indicates that a minimum pay factor of about 60 percent is appropriate for the poorest-quality work and a maximum pay factor of about 115 percent is justified for work of truly superior quality. Additional factors are cited that, although unquantified, would tend to lower the minimum pay factor and raise the maximum pay factor. Finally, pay schedules are developed, the operating-characteristic curves of which closely approximate the theoretically derived relationship.

Statistical end-result specifications are now in widespread use and one of the reasons for their popularity among specification writers is that they provide a practical way to deal with work that is only slightly deficient. A construction item that falls just short of the specified quality level does not warrant rejection but neither does it deserve 100 percent payment. Accordingly, statistical specifications usually employ some form of adjusted pay schedule to award payment in proportion to the level of quality actually achieved.

Throughout the nearly 20 years that specifications of this type have been evolving, several methods (1-3) have been proposed to establish the level of payment appropriate for different levels of quality. In those cases for which there is little or no information relating quality measures to performance, this is an especially difficult task and the methods have necessarily been quite arbitrary. However, there are a few cases for which the quality-performance relationship is well established and these, at least, provide the opportunity to develop a rational and logical procedure for determining appropriate pay factors.

One type of construction for which there are ample data relating performance to various quality characteristics is rigid (portland cement concrete) pavement. The design guide (4) of the American Association of State Highway and Transportation Officials (AASHTO) has just been updated and now provides an equation that gives the expected number of equivalent 18-kip load applications that a rigid pavement can sustain as a function of several common quality characteristics. The details of the manner in which this equation can be used are presented in a separate paper by Weed in this Record. For the purposes of this paper, it is simply desired to establish that the technology required to design a pavement can also be used to assess the quality of a pavement, the as-built characteristics of which differ from the intended design values.

BASIS FOR PAY ADJUSTMENTS

Ordinarily, a pavement is designed to sustain a specified number of load applications before major repair (overlaying with bituminous concrete) is required. If, due to construction deficiencies, the pavement is not capable of withstanding the design loading, it will fail prematurely. The necessity of repairing this pavement at an earlier date results in an additional expense that, since it usually occurs long after any contractual obligations have expired, must be borne by the highway agency. It is the purpose of the adjusted pay schedule to withhold sufficient payment at the time of construction to cover the extra cost anticipated in the future as the result of deficient-quality work.

Based on the procedure used to arrive at the original design parameters of the pavement, the as-built parameters can be used to estimate the fraction of design loadings the pavement will actually be able to sustain. For practical purposes, it is reasonable to assume that the yearly traffic volume is constant so that this fraction can be multiplied by the design life to obtain the expected life. Then, based on current construction costs and projected interest and inflation rates, it is possible to compute both future and present-worth values for credits and debits resulting from the rescheduling of the several generations of overlays that are required after the useful service life of the original pavement has been exhausted. The appropriate pay adjustment is the present worth of the sum of these credits and debits and, depending on the estimated life of the original pavement, this adjustment may be either positive or negative. As a result, the corresponding pay factors obtained by this method are not limited to a maximum of 100 percent.

In essence, pay schedules derived by this method comprise both a liquidated damages clause and a bonus provision. For those agencies not desiring to apply a bonus provision, the pay factors can be limited to a maximum of 100 percent. Alternatively, a crediting concept (5) can be used that allows pay factors greater than 100 percent to offset other pay factors lower than 100 percent while the overall average pay factor is still limited to a maximum of 100 percent.

BASIC FORMULAS

Certain basic engineering economics formulas (6) will be found to be useful in the development of the pay-factor equation. The compound-interest formula can be modified slightly to compute the projected future cost of an item as follows:

$$C_n = C_o(1 + R_{INF}/100)^n \quad (1)$$

where

C_n = future cost after n years,
C_o = present cost, and
R_{INF} = inflation rate (percent per year).

The present worth of this future cost is given by

$$W_o = C_n/(1 + R_{INT}/100)^n \quad (2)$$

where

W_o = present worth,
C_n = future cost after n years, and
R_{INT} = interest rate (percent per year).

Then, by defining the ratio

$$R = (1 + R_{INF}/100)/(1 + R_{INT}/100) \quad (3)$$

and substituting Equation 1 into Equation 2, the expression for the present worth of a future cost can be simplified to

$$W_o = C_oR^n \quad (4)$$

which provides an effective means to estimate the present economic impact of the decision to make (or cancel) a future expenditure.

Finally, since the interest rate is often stated as compounded on some periodic basis of less than a year, it will be useful to have an expression to convert it to an equivalent annual interest rate of the form used in Equations 2 and 3. This can be accomplished by the following equation:

$$R_{INT} = 100[(1 + R_{COMP}/100m)^m - 1] \quad (5)$$

where

R_{INT} = equivalent annual interest rate (percent per year),
R_{COMP} = compound interest rate (percent), and
m = annual frequency of compounding.

NUMERICAL EXAMPLE

Before the general expression for the appropriate pay factor is derived, it will be instructive to work out a numerical example by using data representative of a moderately urbanized area. A typical in-place bid price for concrete pavement is approximately $30/yd². Because the pay factors to be developed will be based on the economic effect of rescheduling the successive overlays that will eventually be installed, the overlay costs must account for all operations normally included in a resurfacing contract. A review of construction costs for several projects suggests that $8/yd² and $7/yd² are typical costs for the first and subsequent overlays, respectively. It will be assumed that the design life of the pavement is 20 years, its expected life based on as-built measurements is 16 years, and the expected life of all overlays is 10 years. The annual interest and inflation rates are assumed to be 15 percent and 10 percent, respectively.

Based on this information, it is required to determine the economic impact on the highway agency of the expected premature failure of the original pavement. This will include not only the effect of installing the first overlay four years sooner than planned but in addition the effects of installing all subsequent overlays an equal amount of time ahead of schedule. The credits and debits resulting from the rescheduling of the overlays are computed by means of Equation 4 by using R = 1.10/1.15 = 0.9565. The computations for the first three overlay generations are tabulated as follows:

Year	Overlay Status	Present Worth	Credit or Debit
n = 16	Scheduled	$\$8 \times R^{16} = \$3.93$	Debit
n = 20	Cancelled	$\$8 \times R^{20} = \$3.29$	Credit

(The net effect of rescheduling the first overlay is a debit of $3.93 - $3.29 = $0.64.)

n = 26	Scheduled	$\$7 \times R^{26} = \$2.20$	Debit
n = 30	Cancelled	$\$7 \times R^{30} = \$1.84$	Credit

(The net effect of rescheduling the second overlay is a debit of $2.20 - $1.84 = $0.36.)

n = 36	Scheduled	$\$7 \times R^{36} = \$1.41$	Debit
n = 40	Cancelled	$\$7 \times R^{40} = \$1.18$	Credit

(The net effect of rescheduling the third overlay is a debit of $1.41 - $1.18 = $0.23.)

The total effect of rescheduling the successive generations of overlays is the sum of the individual effects. Unless it is known that the pavement is planned to be phased out of existence at some specific time in the future, it is appropriate to continue this computational procedure until the terms become so small that they contribute nothing further to the total. The continuation of this procedure produced the following results:

Overlay	Debit ($)
1	0.640
2	0.359
3	0.230
4	0.147
5	0.095
6	0.061
7	0.039
8	0.025
9	0.016
10	0.010
11	0.007
12	0.004
13	0.003
14	0.002
15	0.001
16	0.001
Total	1.640

The total unit debit resulting from the early scheduling of the successive generations of overlays is $1.64/yd². Since the unit cost of this pavement is $30/yd², the appropriate pay factor for this particular example can be expressed in decimal form as follows:

$$F = (30 - 1.64)/30 = 0.945 \quad (6)$$

This will be used as a check value for the general expression that is to be derived.

It is interesting to note at this point that a pavement, the expected life of which is 80 percent of its design life, is deemed worthy of 94.5 percent of the contract price. More will be said about this later.

DERIVATION OF PAY-FACTOR EQUATION

Like the numerical example, the pay-factor equation will be derived as a function of the following variables:

C_P = present unit cost of original pavement ($/yd²),
C_{01} = present unit cost of first overlay ($/yd²),
C_{02} = present unit cost of subsequent overlays ($/yd²),
L_{PD} = design life of original pavement (years),
L_{PE} = expected life of original pavement (years),
L_{OE} = expected life of overlays (years),
R_{INT} = annual interest rate (percent),
R_{INF} = annual inflation rate (percent), and
$R = (1 + R_{INF}/100)/(1 + R_{INT}/100)$.

The variable C_P represents the bid price of the original pavement and is included as a reference on which the pay factors will be based. The variables C_{01} and C_{02} represent the total costs of the resurfacing projects that must be moved forward or backward in time depending on the expected life of the pavement. Two overlay costs are included because the first resurfacing often includes items of work not required for subsequent overlays.

By inspecting the computations for the numerical example, for which R = 1.10/1.15 = 0.9565, it can be seen that the portion of the pay adjustment resulting from the early scheduling of the first overlay is the following:

$$A_1 = C_{01}(R^{L_{PD}} - R^{L_{PE}}) = \$8 \times (R^{20} - R^{16}) = -\$0.64 \quad (7)$$

The negative sign indicates that this represents an expense to the highway agency. If the expected life of the pavement (L_{PE}) had been greater than the design life (L_{PD}), this value would have been positive, which represents a savings. In a similar manner, the equations for the adjustments resulting from the rescheduling of the next two overlays are written as follows:

$$A_2 = C_{02}(R^{L_{PD}+L_{OE}} - R^{L_{PE}+L_{OE}}) = \$7 \times (R^{30} - R^{26}) = -\$0.36 \quad (8)$$

$$A_3 = C_{02}(R^{L_{PD}+2L_{OE}} - R^{L_{PE}+2L_{OE}}) = \$7 \times (R^{40} - R^{36}) = -\$0.23 \quad (9)$$

By inspection of Equations 7 through 9, it is now possible to write the equation for the infinite series that gives the sum of all the individual pay adjustments:

$$\begin{aligned}\Sigma A_i = {} & C_{01}(R^{L_{PD}} - R^{L_{PE}}) + C_{02}[(R^{L_{PD}+L_{OE}} - R^{L_{PE}+L_{OE}}) \\ & + (R^{L_{PD}+2L_{OE}} - R^{L_{PE}+2L_{OE}}) \\ & + (R^{L_{PD}+3L_{OE}} - R^{L_{PE}+3L_{OE}}) + \ldots] \end{aligned} \quad (10)$$

By factoring and combining terms, this can be simplified as follows:

$$\Sigma A_i = (R^{L_{PD}} - R^{L_{PE}})[C_{01} + C_{02}(R^{L_{OE}} + R^{2L_{OE}} + R^{3L_{OE}} + \ldots)] \quad (11)$$

In this form, the last expression in the parentheses on the right is recognizable as a geometric progression (6, pp. 5-11), which, since R is less than unity, sums to $R^{L_{OE}}/(1 - R^{L_{OE}})$. Substituting this result into Equation 11 yields

$$\Sigma A_i = (R^{L_{PD}} - R^{L_{PE}})[C_{01} + C_{02}R^{L_{OE}}/(1 - R^{L_{OE}})] \quad (12)$$

as the sum of all pay adjustments. The final step is to combine this with the initial cost of the pavement (C_P) to write the equation for the appropriate pay factor:

$$F = \{C_P + (R^{L_{PD}} - R^{L_{PE}})[C_{01} + C_{02}R^{L_{OE}}/(1 - R^{L_{OE}})]\}/C_P \quad (13)$$

As a check, the values used in the numerical example will be substituted into this equation. With R = 1.10/1.15 = 0.9565, this yields

$$F = \{30 + (R^{20} - R^{16})[8 + 7R^{10}/(1 - R^{10})]\}/30 = 0.945 \quad (14)$$

which checks exactly with the result obtained earlier in Equation 6.

TYPICAL RESULTS

Before discussion of some additional factors that are involved, it will be of interest to use Equation 13 with the input parameters from the numerical example to compute the appropriate pay factor for various levels of the ratio of expected life to design life (L_{PE}/L_{PD}) as follows:

L_{PE}/L_{PD}	Appropriate Pay Factor
0.0	0.597
0.2	0.709
0.4	0.802
0.6	0.880
0.8	0.945
1.0	1.000
1.2	1.046
1.4	1.084
1.6	1.116
1.8	1.143
2.0	1.165

There are two very interesting observations to be made from the values in this table. First, a pavement of such poor quality that its expected life is zero warrants a relatively high pay factor of about 60 percent. Second, a pavement of such exceptional quality that its expected life is double the design life warrants a bonus pay factor of approximately 116 percent, less than might have been anticipated.

The first observation can readily be explained. Although many of the early attempts to establish appropriate pay factors tended to relate payment directly to performance, the justification for such an approach is rather dubious. Unless unusually drastic repairs are required, a pavement capable of providing essentially zero performance still has considerable value as the subsystem on which the first generation of overlay will be placed. In the sense of liquidated damages, the highway agency has been damaged only to the extent of the present worth of the cost to restore the serviceability of the pavement throughout its intended design life. This is the basis for the pay factors computed by Equation 13.

The second observation was more of a surprise. Apparently, based on the typical input values that were used, the highway agency benefits only marginally from an extended service life.

UNQUANTIFIED FACTORS

In actual practice, there are additional factors that must be taken into consideration:

1. There will be administrative costs involved in preparing for the premature repair of poor-quality pavement;
2. There will be costs to the motoring public for the earlier disruption of traffic to make the necessary repairs;
3. For practical reasons, a small section of poor-quality pavement may make it necessary to overlay a larger section of pavement; and
4. Premature failures, if many should occur, could severely restrict the priority-setting capabilities of a highway agency.

Because these factors are extremely difficult to quantify, they will be dealt with only in a qualitative manner. Since all four represent additional expenses that may occur when the quality is substandard, they provide a valid argument for a lowering of pay factors that are less than F = 1.0. Conversely, it can be argued that these same factors will result in a saving to the highway agency when the quality is superior. This would justify an increase in pay factors greater than F = 1.0. The net effect of these unquantified factors, therefore, would be a slight broadening of the range of possible pay factors. In the previous example, a minimum pay factor somewhat lower than F = 0.597 might be appropriate and a maximum pay factor slightly greater than F = 1.165 might be justified. This is a decision that would have to be made by each highway agency based on its assessment of the importance of the unquantified factors. One way in which this might be handled will be discussed in the section on the development of pay schedules.

SENSITIVITY TESTS

When Equation 13 is applied, some of the input variables will be well determined whereas others will be known with less certainty. Of particular concern are the interest and inflation rates (R_{INT}, R_{INF}), since these values must be projected many years into the future. Another important variable is the design life (L_{PD}) of the original pavement. Although it might seem that this variable would be known exactly, it is strongly dependent on the accuracy of the forecast of traffic volume. The design loading of the pavement may be reached several years ahead of schedule if the traffic volume is substantially underestimated. The remaining variables--the present cost of the original pavement (C_P), the present costs of the bituminous overlays (C_{O1}, C_{O2}), and the expected life of an overlay (L_{OE})--would be known quite accurately and will not be treated as variables in this first test. The pay factors in the following table were computed with Equation 13 by using C_P = 30, C_{O1} = 8, C_{O2} = 7, and L_{OE} = 10:

	Appropriate Pay Factor			
L_{PE}/L_{PD}	L_{PD}=20 R_{INT}=15 R_{INF}=10	L_{PD}= 20 R_{INT}=12 R_{INF}=7	L_{PD}=20 R_{INT}=13 R_{INF}=10	L_{PD}=16 R_{INT}=15 R_{INF}=10
0.0	0.597	0.599	0.575	0.652
0.2	0.709	0.711	0.679	0.743
0.4	0.802	0.804	0.773	0.821
0.6	0.880	0.882	0.857	0.890
0.8	0.945	0.946	0.932	0.949
1.0	1.000	1.000	1.000	1.000
1.2	1.046	1.045	1.061	1.045
1.4	1.084	1.082	1.116	1.083
1.6	1.116	1.113	1.165	1.117
1.8	1.143	1.139	1.209	1.146
2.0	1.165	1.161	1.248	1.171

It can be seen from the pay factors in this table that Equation 13 is quite stable over a wide range of input values. By comparing the second and third columns, it is observed that changing the interest and inflation rates from R_{INT} = 15 and R_{INF} = 10 to R_{INT} = 12 and R_{INF} = 7 produces virtually no change in the pay factors that are obtained. Therefore, the method is not sensitive to the actual values of interest and inflation but to their difference, a parameter that is somewhat easier for a highway agency to predict.

By comparing the second and fourth columns, it can be seen that a substantial decrease in this difference from $R_{INT} - R_{INF}$ = 15 - 10 = 5.0 to $R_{INT} - R_{INF}$ = 13 - 10 = 3.0 has very little effect on the pay factors below F = 1.0 but an increasingly noticeable effect on the pay factors above F = 1.0. The minimum pay factor is reduced from F = 0.597 to F = 0.575, whereas the maximum pay factor is increased from F = 1.165 to F = 1.248.

Finally, by comparing the second and fifth columns, a decrease in design life from L_{PD} = 20 to L_{PD} = 16 is observed to have a moderate effect. The minimum pay factor is raised from F = 0.597 to F = 0.652, whereas all other pay factors are affected to a lesser degree.

Another set of computations will be of interest. For a variety of reasons, the unit cost of the first resurfacing may occasionally be greater than the value of C_{O1} = \$8/yd² assumed in the examples thus far. To test the effect of this parameter, the pay factors in the following table were computed with Equation 13 by using C_P = 30, C_{O2} = 7, L_{PD} = 20, L_{OE} = 10, R_{INT} = 15, and R_{INF} = 10:

	Appropriate Pay Factor			
L_{PE}/L_{PD}	C_{O1} = 8	C_{O1} = 9	C_{O1} = 10	C_{O1} = 11
0.0	0.597	0.578	0.558	0.539
0.2	0.709	0.695	0.680	0.666
0.4	0.802	0.792	0.783	0.773
0.6	0.880	0.874	0.868	0.862
0.8	0.945	0.943	0.940	0.937
1.0	1.000	1.000	1.000	1.000
1.2	1.046	1.048	1.050	1.052
1.4	1.084	1.088	1.092	1.096
1.6	1.116	1.122	1.127	1.133
1.8	1.143	1.150	1.157	1.164
2.0	1.165	1.174	1.182	1.190

It can be seen from the values in this table that substantial changes in the cost of the first resurfacing have a noticeable but moderate effect. As this variable increases from C_{O1} = 8 to C_{O1} = 11, the minimum pay factor decreases from F = 0.597 to F = 0.539 and the maximum pay factor increases from F = 1.165 to F = 1.190.

In all the preceding tests, the value of C_{O1} has been assumed to remain constant for all values of the ratio L_{PE}/L_{PD}. If the as-constructed quality of the pavement were so poor that immediate repair would be necessary, the first overlay might have to be thicker than usual. If so, it would be justifiable to increase the value of C_{01} when computing the appropriate pay factors at or near the zero performance level. However, the only effect of such a change would be a slight decrease at the extreme lower end of the pay schedule. It will be discussed in the section on the development of pay schedules why this refinement is believed to be unnecessary.

Still another set of computations will be useful. Although the present cost of the original pavement (C_P) and the present costs of the bituminous overlays (C_{01}, C_{02}) would be reasonably well known, there is no question that these prices will escalate and fluctuate with time. Furthermore, the experience of some highway agencies may suggest that the expected life of an overlay is different from the value of $L_{OE} = 10$ years used in the preceding examples. To test the effect of these variables, the pay factors in the following table were computed with Equation 13 by using $L_{PD} = 20$, $R_{INT} = 15$, and $R_{INF} = 10$:

	Appropriate Pay Factor			
L_{PE}/L_{PD}	$C_P = 30$, $C_{01} = 8$, $C_{02} = 7$, $L_{OE} = 10$	$C_P = 60$, $C_{01} = 16$, $C_{02} = 14$, $L_{OE} = 10$	$C_P = 27$, $C_{01} = 8.8$, $C_{02} = 7.7$, $L_{OE} = 10$	$C_P = 30$, $C_{01} = 8$, $C_{02} = 7$, $L_{OE} = 9$
0.0	0.597	0.597	0.508	0.564
0.2	0.709	0.709	0.644	0.684
0.4	0.802	0.802	0.758	0.785
0.6	0.880	0.880	0.853	0.870
0.8	0.945	0.945	0.933	0.941
1.0	1.000	1.000	1.000	1.000
1.2	1.046	1.046	1.056	1.050
1.4	1.084	1.084	1.103	1.091
1.6	1.116	1.116	1.142	1.126
1.8	1.143	1.143	1.175	1.155
2.0	1.165	1.165	1.202	1.179

The effects of both parallel and opposite movements of pavement and overlay costs can be judged from the second, third, and fourth columns in this table. The values in the second and third columns demonstrate that a uniform escalation of all prices produces no change in the pay factors that are computed. The values in the second and fourth columns show that an opposite movement of prices does have a noticeable effect. A decrease in pavement cost of 10 percent coupled with a 10 percent increase in overlay costs reduces the lowest pay factor from F = 0.597 to F = 0.508 while raising the largest pay factor from F = 1.165 to F = 1.202. Although this effect is not drastic, it suggests that the costs entered into Equation 13 should be composite values, averaged over a period of time, to minimize the influence of short-term price fluctuations.

The values in the second and fifth columns illustrate the effect of a 10 percent decrease in expected overlay life from $L_{OE} = 10$ to $L_{OE} = 9$. The lowest pay factor is reduced from F = 0.597 to F = 0.564 and the largest pay factor is increased from F = 1.165 to F = 1.179. Since a highway agency would have a reasonably accurate knowledge of the average life of an overlay, any error in the determination of this variable will not have a great effect on the resultant pay schedule.

Because the establishment of the minimum pay factor is an important step in developing any pay schedule, it will be of value to have a graph illustrating how this critical value is affected by uncertainty in the independent variables of Equation 13. The relative importance of each variable can be judged from the steepness of the curves in Figure 1. Of particular interest are the extremely shallow slopes and opposite inclination of the curves for interest and inflation rates, which indicates that a substantial degree of uncertainty in these variables can be tolerated. At the other extreme, the variable that has the steepest slope in Figure 1 is C_P, the cost of the original pavement. Since this variable has a strong influence on the resulting pay relationship, it is especially important that it be well determined. Fortunately, this can easily be accomplished by averaging the bid prices from several recent contracts.

Figure 1 can also be used as a guide to perform an especially severe test of the reliability of Equation 13. Although it would be very unlikely for any errors in the independent variables to all act in the same direction, the values used in the next test will be chosen to demonstrate the effect of such an improbable event. All variables will be incremented by 10 percent from their nominal values and, to produce the maximum effect, the variables C_P, L_{OE}, and R_{INT} will move in a direction opposite the variables C_{01}, C_{02}, L_{PD}, and R_{INF}. The results are presented in the following table:

	Appropriate Pay Factor		
L_{PE}/L_{PD}	$C_P = 30$, $C_{01} = 8$, $C_{02} = 7$, $L_{PD} = 20$, $L_{OE} = 10$, $R_{INT} = 15$, $R_{INF} = 10$	$C_P = 27$, $C_{01} = 8.8$, $C_{02} = 7.7$, $L_{PD} = 22$, $L_{OE} = 9$, $R_{INT} = 13.5$, $R_{INF} = 11$	$C_P = 33$, $C_{01} = 7.2$, $C_{02} = 6.3$, $L_{PD} = 18$, $L_{OE} = 11$, $R_{INT} = 16.5$, $R_{INF} = 9$
0.0	0.597	0.376	0.724
0.2	0.709	0.526	0.808
0.4	0.802	0.663	0.875
0.6	0.880	0.786	0.927
0.8	0.945	0.898	0.968
1.0	1.000	1.000	1.000
1.2	1.046	1.092	1.025
1.4	1.084	1.176	1.045
1.6	1.116	1.251	1.061
1.8	1.143	1.320	1.074
2.0	1.165	1.382	1.083

As expected with the extremely unfavorable conditions assumed for this test, the resultant pay schedules are substantially affected. The greatest effect occurs at the minimum pay factor, which moves from F = 0.597 down to F = 0.376 with one combination of independent variables and up to F = 0.724 with the other. As with all of the tests, the pay factors closer to F = 1.0 are affected to a lesser degree.

Although this test was designed to produce a worst-case result, it emphasizes that the accuracy of the resultant pay schedule is dependent on the accuracy with which the independent variables have been determined. This suggests that pay schedules developed by use of Equation 13 should be reviewed periodically to verify that they are still appropriate, particularly if an unexpected change in any of the independent variables has occurred. However, as long as the input variables are average values that tend to be quite stable, a modification of the pay schedule should seldom be necessary in actual practice.

Collectively, the tests in this section demonstrate that the values computed by Equation 13 are relatively insensitive to minor fluctuations of the independent variables. This indicates that this equation can be relied on to establish appropriate pay factors provided the input values are reasonably accurate.

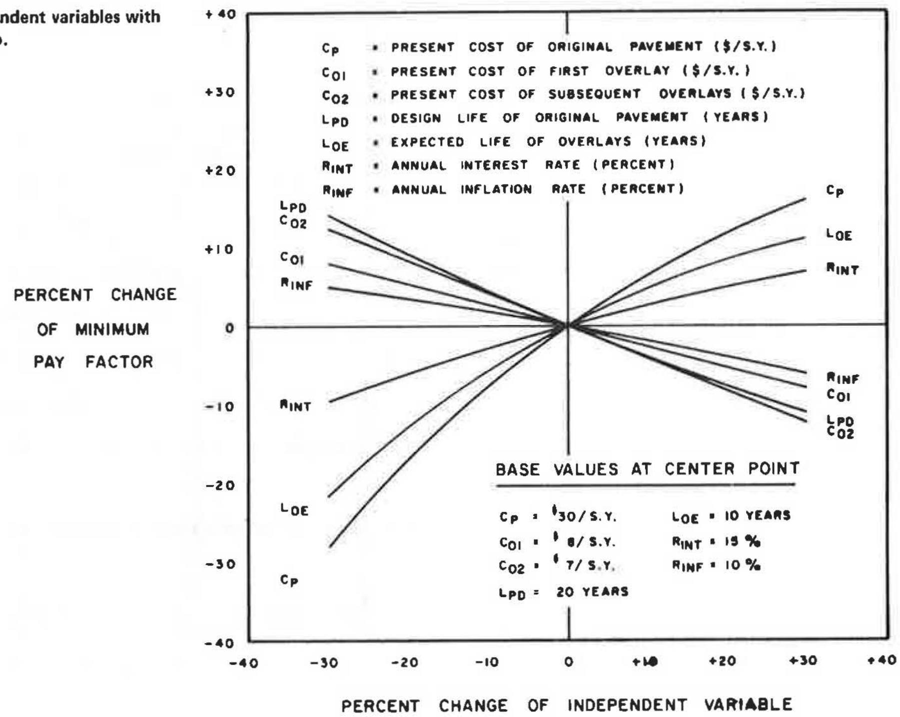


Figure 1. Sensitivity analysis of all independent variables with expected pavement life (L_{PE}) fixed at zero.

DEVELOPMENT OF PAY SCHEDULES

There are three ways in which Equation 13 can be used to establish appropriate pay schedules. First, by defining discrete intervals for the ratio of expected life to design life (or its equivalent, the ratio of expected load-bearing capacity to the design loading) and then computing the pay factor associated with the midpoint of each interval, it is possible to construct a stepped pay schedule such as the following:

Load Ratio	Pay Factor
<0.50	0.60
0.50-0.69	0.88
0.70-0.89	0.94
0.90-1.09	1.00
1.10-1.29	1.04
1.30-1.49	1.08
1.50-1.69	1.12
1.70-1.89	1.14
≥1.90	1.16

The pay factors in this table have been computed with Equation 13 by using the input parameters from the numerical example. The decision has been made in this particular case that below a load ratio of 0.50, the unquantified factors previously discussed take precedence and the pay factor is arbitrarily set at the minimum level of F = 0.60.

However, when the operating-characteristic curve for a pay schedule such as this is checked, it will usually be found that the provisions for minimum and maximum pay factors have biased it away from the desired curve. This effect is more pronounced with small sample sizes, and if it is considered large enough to require correction, either the sample size must be increased or some of the pay factors in the pay schedule must be raised. A typical refinement of this pay schedule, designed to achieve a close match with the desired curve between load ratios of 0.50 and 1.50, might be as follows:

Load Ratio	Pay Factor
<0.50	0.60
0.50-0.69	0.90
0.70-0.89	0.95
0.90-1.09	1.00
1.10-1.29	1.05
1.30-1.49	1.10
≥1.50	1.12

Although stepped pay schedules are in common use, they do have a minor disadvantage. Unless the discrete intervals are quite small, the difference in pay between two successive steps may be fairly substantial. Whenever the true population quality happens to fall close to one of the boundaries in a stepped pay schedule, it is almost entirely a matter of chance whether the larger or smaller pay factor will be assigned. Although this tends to balance out in the long run, it can work to the disadvantage of either the highway agency or the contractor on a project with relatively few lots.

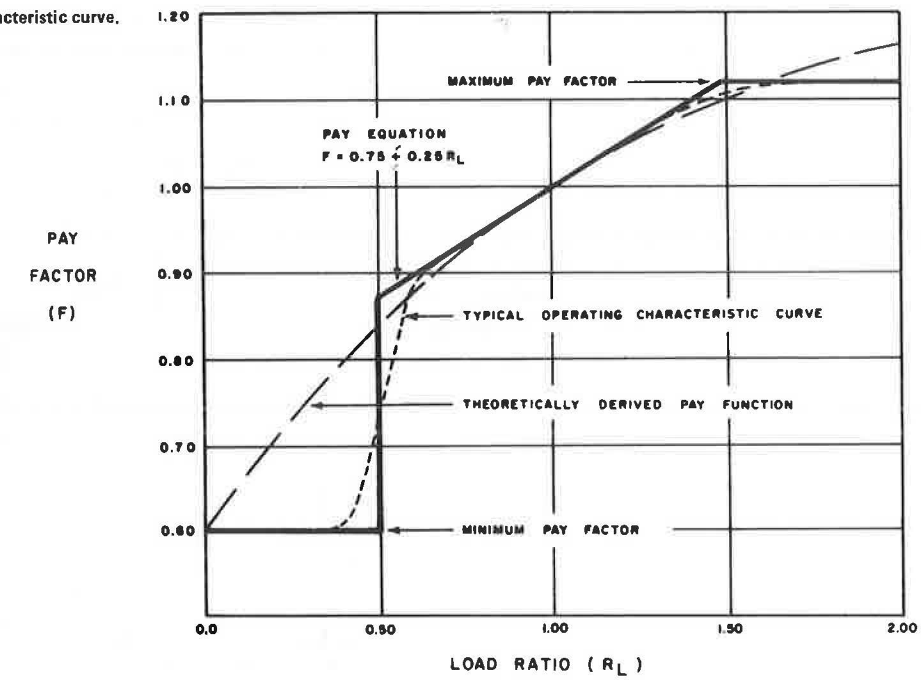
A second approach avoids this problem by expressing the pay schedule in the form of a continuous equation. When the pay factors computed previously are plotted as a function of the load ratio, they are seen to lie on a gentle curve as shown in Figure 2. Although it is possible to derive an equation that fits this curve, this turns out to be unnecessary. A simple linear pay equation can be found that will produce an operating-characteristic curve that closely approximates the desired curve.

One such equation is

$$F = 0.75 + 0.25R_L \qquad (15)$$

in which R_L is the load ratio. Two constraints that must be imposed on this equation are a maximum allowable pay factor of F = 1.12 and the restriction that if R_L is less than 0.50, the pay factor will be set equal to the minimum value of F = 0.60. With these modifications, the equation-type pay schedule is essentially equivalent to the stepped pay schedule previously discussed.

Figure 2. Typical pay equation and operating-characteristic curve.



As can be seen in Figure 2, the operating-characteristic curve can be made to match the desired curve very closely between load ratios of R_L = 0.60 and R_L = 1.60. Although it begins to fall below the desired curve above R_L = 1.60, this is not considered to be a serious drawback because it is believed that a pavement would seldom exceed this level of quality in actual practice. Below R_L = 0.60, the operating-characteristic curve drops rapidly to the minimum pay factor of F = 0.60, a result considered justifiable because of the unquantified factors previously cited. It is because of this rapid drop, which provides considerable incentive to avoid extremely low levels of quality, that it is believed unnecessary to account for a possible increase in cost for the first resurfacing (C_{01}) at the zero performance level.

The third manner in which Equation 13 can be used to develop a pay schedule is less precise but still useful. If the acceptance procedure were based on some quality characteristic other than load ratio (such as the percent defective of some construction parameter), Equation 13 would not be directly applicable. However, it can be used to determine what the minimum and maximum pay factors should be. This would establish two extreme points, and an additional known point would be a pay factor of F = 1.0 at the acceptable quality level. Either a stepped or a continuous pay schedule could then be developed that would produce an operating-characteristic curve that passes through these points.

SUMMARY AND CONCLUSIONS

The same technology used to design a rigid pavement to have a specified service life can be used to estimate the expected life of a pavement whose as-built characteristics differ from the design values. Then, by the use of basic engineering economics methods, it is possible to compute the expense or savings that result from the rescheduling of the successive generations of overlays that eventually must be installed. On the assumption that it is justifiable to adjust the contract price by the amount of this expense or savings, an equation was developed to compute the appropriate pay factor for any quality level of rigid pavement as a function of input information readily available within most highway agencies. Additional factors were cited that should be taken into consideration and sensitivity tests were performed to show that the procedure is reliable provided the input values are determined with reasonable accuracy. Various methods of using the equation to establish pay schedules were then discussed.

The nominal input values used in the examples in this paper were obtained from recent construction cost records and other sources representative of a relatively urbanized area. Use of these values in Equation 13 plus consideration of the effect of several unquantified factors resulted in a pay schedule with pay factors that range from a minimum of F = 0.60 to a maximum of F = 1.12. The real importance of Equation 13, however, is that it provides a reliable and extremely easy method to develop pay schedules by using whatever input values a highway agency considers appropriate.

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Development of Multicharacteristic Acceptance Procedures for Rigid Pavement

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The manner in which the design method of the American Association of State Highway and Transportation Officials (AASHTO) can be used to develop multicharacteristic acceptance procedures for rigid pavement is outlined. The AASHTO equation is used to compute both the expected load-bearing capacity based on the as-built characteristics of the pavement and the desired load-bearing capacity based on the design parameters. The ratio of these two values is then used to determine the appropriate pay adjustment, which may be either positive or negative. Sensitivity tests are performed to verify the reliability of this approach and computer simulation is used to demonstrate the effectiveness of several different acceptance procedures of this type. A secondary study is conducted to determine how the procedure based on the AASHTO equation compares with several other methods of treating multiple pay factors to obtain a single overall pay factor. Under the assumption that the AASHTO method is the fundamentally correct approach, the method of multiplying individual pay factors together is shown to be among the best of the other methods that were tested.

Statistical end-result specifications with adjusted pay schedules are now used by many highway agencies and are often based on more than a single quality characteristic. A statistical specification for rigid pavement, for example, may have separate acceptance procedures for compressive strength and thickness or a combination of these and other parameters. A pay factor is computed for each characteristic, and in order to arrive at an overall pay factor, a variety of methods have been used. Perhaps the most common method defines the overall pay factor as the product of the individual pay factors. Another approach is to use the smallest of the individual pay factors. Still another method is a cumulative one in which the individual pay adjustments are summed to obtain the total adjustment. The average of the individual pay factors has also been used.

Although all of these methods may be effective from a practical standpoint, the present state of the art is such that none of them has been conclusively demonstrated to be correct. Almost certainly, there is no single method that would be appropriate for all types of construction situations since this would be a function of the true quality-performance relationship and the degree of association among the various quality characteristics. However, in the case of rigid pavement, this problem can be avoided by working directly with the design equation of the American Association of State Highway and Transportation Officials (AASHTO) (1), which gives the number of equivalent 18-kip load applications that can be sustained as a function of several common quality characteristics. In this manner, the multiple effects of the individual characteristics can be reduced to a single fundamental effect--the resultant load-bearing capacity computed from the as-built characteristics of the pavement. By comparing this with the desired load-bearing capacity computed from the design parameters, a ratio is obtained that can be used to form the basis for a rational adjusted pay schedule.

It is the objective of this paper to outline how these concepts can be used to develop acceptance procedures based on the AASHTO design equation. Included are discussions of several factors that must be taken into consideration when using the AASHTO equation in this manner, sensitivity tests to confirm the soundness of certain assumptions, examples of several variations of this approach, and a series of computer simulation tests to demonstrate the effectiveness of the resultant acceptance procedures. In essence, this work constitutes a feasibility study that will serve as a useful guide for any highway agency desiring to develop a specification of this type.

AVERAGES VERSUS DISTRIBUTIONS

Many existing statistical specifications are based on the estimate of percent defective (2) of some construction parameter. These specifications recognize that the characteristic of interest is stochastically distributed and typically allow a small percent of the population to fall outside some critical limit (or pair of limits). Lot sizes are usually defined to be small enough so that, within each lot, the quality level is believed to be fairly uniform. To assure that a reasonable degree of uniformity has been achieved, a secondary requirement is often imposed that specifies additional limits within which the individual test values must fall.

The existence of a small amount of variability within a lot is not necessarily detrimental. In the case of a pavement lot the average quality of which is exactly at the desired quality level, this means that approximately half the pavement will require a little more than the normal amount of routine maintenance throughout its design life, whereas the other half will require a little less. These two conditions will tend to balance out provided the poorer-quality half is not so defective that it requires something more than routine maintenance. An additional limit on individual test values protects against this possibility.

An acceptance procedure based on the AASHTO design equation can be developed that will perform in much the same way. Each sample provides an independent estimate of the load-bearing capacity of the pavement lot and these can be combined to determine an average load ratio from which the appropriate pay factor is determined. As a safeguard against isolated sections of poor quality within a lot, a lower limit on the individual estimates can be imposed. Whenever an individual value falls below the lower limit, coring or other procedures can be employed to more precisely determine the quality of that particular section.

AASHTO EQUATION FOR RIGID PAVEMENT

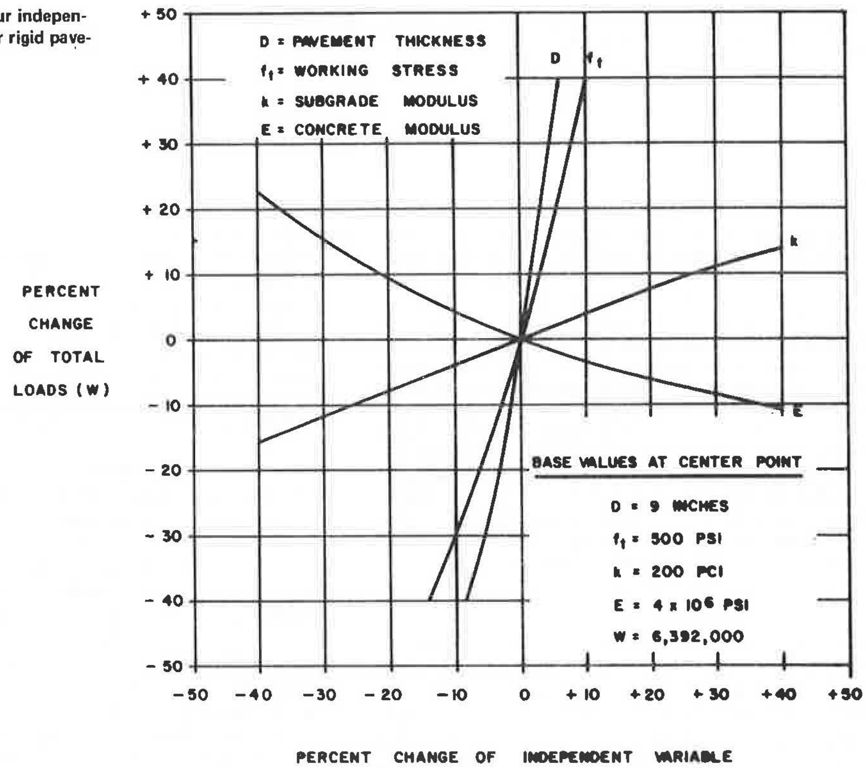
The equation for rigid pavement is presented in the AASHTO design guide (1) as follows:

$$\log W = 7.35 \log(D+1) - 0.06 - 0.1761/[1 + 1.624 \times 10^7/(D+1)^{8.46}] + 3.42 \log \{(f_t/690)(D^{0.75} - 1.132)/[D^{0.75} - 18.42/(E/k)^{0.25}]\} \quad (1)$$

where

- W = number of equivalent 18-kip load applications,
- D = pavement thickness (in),
- f_t = working stress in concrete (psi),
- E = concrete modulus of elasticity (psi), and
- k = modulus of subgrade reaction (pci).

Figure 1. Sensitivity analysis of the four independent variables in AASHTO equation for rigid pavement.



Because this equation is somewhat awkward to work with, the various tests described in this paper were performed with the aid of a computer. The following FORTRAN subroutine was written to execute the computations indicated in Equation 1:

```
SUBROUTINE AASHTO(THICK,WRKSTR,SUBMOD,CONMOD,LOADS)
TERM1=7.35*ALOG10(THICK+1.)-0.06
TERM2=0.1761/(1.+1.624E7/(THICK+1.)**8.46)
ROOT1=THICK**0.75
ROOT2=(CONMOD/SUBMOD)**0.25
TERM3=3.42*ALOG10((WRKSTR/690.)*(ROOT1-1.132)/(ROOT1-
  18.42/ROOT2))
LOADS=IFIX(10.**(TERM1-TERM2+TERM3)+0.5)
RETURN
END
```

where

THICK = pavement thickness (in),
WRKSTR = working stress in concrete (psi),
SUBMOD = modulus of subgrade reaction (pci),
CONMOD = concrete modulus of elasticity (psi), and
LOADS = number of equivalent 18-kip load applications computed and returned by subroutine.

The AASHTO equation gives the number of loads a pavement can sustain as a function of thickness, working stress, subgrade modulus, and concrete modulus. The relative importance of these variables can be judged from the steepness of the curves in Figure 1. Thickness (D) and working stress (f_t) are clearly the more important variables since a small change in either one produces a large change in the load-bearing capacity of the pavement. The subgrade modulus (k) and the concrete modulus (E), on the other hand, are less important since their curves are less steeply inclined. For this reason, plus the fact that compaction deficiencies can be corrected before the pavement is placed, the acceptance procedures developed in this paper are based on the two primary variables, thickness and working stress. Once the method has been established, a similar approach can be used to develop acceptance procedures based on all (or any subset) of the four AASHTO variables, if desired.

Although the acceptance procedure could be based on flexure tests (ASTM C78-75) that relate directly to the working stress, it will be more desirable from the standpoint of most highway agencies to base the procedure on standard compression tests (ASTM C39-80). The Portland Cement Association has published the following relationship (3, p. 57):

$$MR = K(f_c')^{1/2} \tag{2}$$

where

MR = modulus of rupture (psi),
f_c' = compressive strength of concrete (psi), and
K = a constant, usually between 8 and 10.

Then, since the working stress is defined by AASHTO as 75 percent of the modulus of rupture, Equation 2 can be rewritten as

$$f_t = 0.75K(f_c')^{1/2} \tag{3}$$

in which f_t is the working stress to be entered into Equation 1. It will be demonstrated in the next section that for purposes of the acceptance procedure, the value of K is not critical. If a midrange value of K = 9 is assumed for the working-stress constant, Equation 3 becomes

$$f_t = 6.75(f_c')^{1/2} \tag{4}$$

which makes it possible to base the AASHTO equation on concrete compressive strength.

ADDITIONAL SENSITIVITY TESTS

In the acceptance procedure to be developed, Equations 1 and 4 will be used to determine both the expected number of loads computed from the as-built measurements of thickness and compressive strength of the pavement and the desired number of loads computed from the design values for these same parameters. The ratio of these two values will then be used to determine the appropriate pay factor. It is the purpose of this section to show that this ratio is essentially independent of the nominal values assumed for subgrade modulus, concrete modulus, and the working-stress constant.

The first of these three variables to be tested is the subgrade modulus. The load ratios in the following table were computed by using a design average thickness of D = 9 in, a design average compressive strength of f_c' = 4000 psi, and nominal values for the concrete modulus and working-stress constant of E = 4 x 10^6 psi and K = 9, respectively.

As-Built Values		Load Ratio		
D (in)	f_c' (psi)	k = 100 pci	k = 200 pci	k = 300 pci
8.5	3000	0.428	0.433	0.438
8.5	4000	0.699	0.709	0.716
8.5	5000	1.024	1.038	1.049
9.0	3000	0.611	0.611	0.611
9.0	4000	1.000	1.000	1.000
9.0	5000	1.465	1.465	1.465
9.5	3000	0.863	0.852	0.845
9.5	4000	1.411	1.394	1.381
9.5	5000	2.066	2.041	2.023

There are two interesting observations to be made from the values in this table. First, increasing the subgrade modulus by a factor of 3 from k = 100 pci to k = 300 pci produces only small changes in the resultant load ratios. Since a highway agency would know the nominal value of the subgrade modulus much more precisely than this, this variable will have essentially no effect on the load ratio that is computed as part of the acceptance procedure. The second observation to be made is that this type of acceptance procedure recognizes that, within reasonable limits, an excess in one quality characteristic can offset a deficiency in another. For example, although the assumed design values are D = 9 in and f_c' = 4000 psi, the third line in the table indicates that a pavement constructed with D = 8.5 in and f_c' = 5000 psi will actually have a slightly greater load-bearing capacity since the load ratios are greater than unity.

The next variable to be tested is the concrete modulus. The load ratios in the following table were computed by using the same design parameters of D = 9 in and f_c' = 4000 psi along with nominal values for the subgrade modulus and working-stress constant of k = 200 pci and K = 9, respectively:

As-Built Values		Load Ratio		
D (in)	f_c' (psi)	E = 3x10^6 psi	E = 4x10^6 psi	E = 5x10^6 psi
8.5	3000	0.437	0.433	0.431
8.5	4000	0.714	0.709	0.705
8.5	5000	1.046	1.038	1.033
9.0	3000	0.611	0.611	0.611
9.0	4000	1.000	1.000	1.000
9.0	5000	1.465	1.465	1.465
9.5	3000	0.847	0.852	0.856
9.5	4000	1.385	1.394	1.400
9.5	5000	2.029	2.041	2.050

Precisely the same effects are observed in this table. The load ratios are very stable in spite of the large changes in concrete modulus, and the two as-built quality characteristics have the same offsetting property already noted. Like the subgrade modulus, the concrete modulus would be known sufficiently precisely that it would have no appreciable effect on the acceptance procedure.

The last variable to be tested is the working-stress constant. The load ratios in the following table were again computed by using the same design values of D = 9 in and f_c' = 4000 psi and nominal values for subgrade modulus and concrete modulus of k = 200 pci and E = 4x10^6 psi, respectively:

As-Built Values		Load Ratio		
D (in)	f_c' (psi)	K = 8	K = 9	K = 10
8.5	3000	0.433	0.433	0.433
8.5	4000	0.709	0.709	0.709
8.5	5000	1.038	1.038	1.038
9.0	3000	0.611	0.611	0.611
9.0	4000	1.000	1.000	1.000
9.0	5000	1.465	1.465	1.465
9.5	3000	0.852	0.852	0.852
9.5	4000	1.394	1.394	1.394
9.5	5000	2.041	2.041	2.041

These results are even more consistent than those in the previous two tables. The working-stress constant has no effect on the load ratios that are computed.

Taken together, these tests indicate that it is not necessary to have an exact knowledge of the subgrade modulus, concrete modulus, or the working-stress constant for this application. With the substitution of nominal values for these parameters, the AASHTO equation can be used to obtain reliable estimates of the load ratio of the pavement based on the as-built measurements of thickness and compressive strength. This clears the way for the development of the remainder of the acceptance procedure.

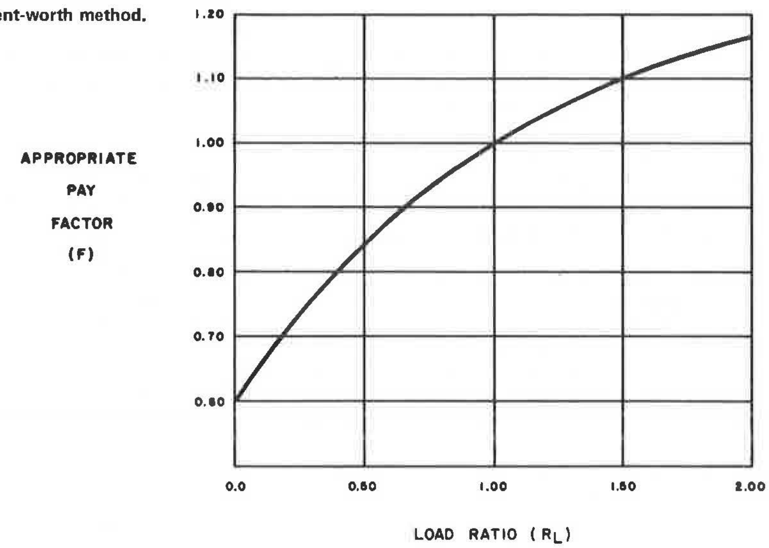
DEVELOPMENT OF PAY SCHEDULES

The derivation of pay schedules based on the concept of load ratio is developed in a companion paper by Weed in this Record and will be summarized only briefly here. If, due to construction deficiencies, the pavement is not capable of withstanding the design loading, it will fail prematurely. The necessity of repairing this pavement at an earlier date will result in an additional expense to the highway agency. Conversely, a pavement of superior quality that lasts longer than the intended design life will result in a savings. The appropriate pay adjustment is considered to be the present worth of any expense or savings expected to occur in the future as the result of a departure from the specified level of quality and may be positive or negative. In essence, a pay schedule based on this premise constitutes both a liquidated-damages clause and a bonus provision.

If a highway agency elects not to apply a bonus provision, the pay factors are limited to a maximum of 100 percent. However, this restriction tends to bias the higher pay factors downward and, in certain cases, this can create serious problems for both the contractor and the highway agency. This is discussed further in Example 7 in this paper and was explained in detail in an earlier paper (4).

Figure 2 illustrates a typical pay function that might result when bonus payments are permitted. The appropriate pay factor (F) is expressed as a decimal and is plotted as a function of the load ratio (R_L). For load ratios between R_L = 0.0 and R_L

Figure 2. Typical pay function derived by present-worth method.



= 2.0, the curve is concave downward and is seen to rise smoothly from a minimum of about F = 0.60 to a maximum of approximately F = 1.16.

Although it would be possible to develop a pay schedule that follows this curve, this turns out to be unnecessary. In actual practice, most pay schedules have various arbitrary conditions imposed on them, such as a minimum pay factor, a maximum pay factor, a requirement for retesting, and so forth, and these conditions invariably cause the operating-characteristic curve to depart from whatever pay function is used. This fact can be used to advantage as long as it is recognized and taken into account. The objective, of course, is to make the operating-characteristic curve, not the pay schedule itself, conform to the desired pay function.

In general, pay schedules may be of two types, stepped or continuous. Stepped pay schedules define discrete intervals for the quality characteristic and assign a specific pay factor for each interval. Continuous pay schedules employ an equation to compute the appropriate pay factor for any given quality level. The companion paper by Weed in this Record presents two pay schedules for use with the pay function shown in Figure 2. The first is a stepped pay schedule as follows:

Load Ratio	Pay Factor
<0.50	0.60
0.50-0.69	0.90
0.70-0.89	0.95
0.90-1.09	1.00
1.10-1.29	1.05
1.30-1.49	1.10
≥1.50	1.12

The second is given by the equation

$$F = 0.75 + 0.25R_L \quad (5)$$

with the added constraints that the maximum allowable pay factor is F = 1.12 and if the load ratio is less than R_L = 0.50, the pay factor is set at the minimum value of F = 0.60. Although this is a linear equation, it will be shown in a later section that its operating-characteristic curve conforms closely to the desired pay function within the primary region of interest.

Although stepped pay schedules are still more prevalent, continuous pay schedules are rapidly gaining acceptance and the reasons are quite obvious. Not only are they concise and easy to apply, they provide a more precise determination of the appropriate pay factor, a result beneficial to all parties. Because of these desirable features, plus the fact that the two types are essentially equivalent in the long run, equation-type pay schedules will be used for the remaining developments in this paper.

RETESTING PROVISIONS

Since it is common practice to require retesting when the first test indicates an unusually low level of quality, it will be worthwhile to consider the manner in which such a provision might be applied. There are two distinctly different ways in which the retest values can be processed and there are advantages and disadvantages associated with each. The first method combines the retest values with the original values and reevaluates the lot or sublot on the basis of the enlarged sample. The second method discards the original sample and evaluates the lot or sublot on the basis of the second sample only.

An advantage of the first method is that it uses all the available information. Advocates of this method argue that there is a cost associated with each sample and that it is wasteful to discard any valid information. An opposing viewpoint would question whether the original sample is truly valid. If the low quality level is the result of some malfunction of the testing process, then it would be more appropriate to discard the contaminated data.

This is a question of philosophy that each highway agency must answer for itself. However, there is another theoretical argument that can be offered in favor of the second method. When the retest values are combined with the original values, the probabilities of passing both the first test and the retest are correlated to some unknown degree. This lack of independence precludes the direct computation of the overall probability of acceptance. As a result, the operating-characteristic curve for the procedure must be determined somewhat imprecisely by a boundary method (5,6) or else obtained empirically by computer simulation. Consequently, if the test results are relatively inexpensive and easy to obtain, it may be more practical to use the second method and discard the original sample. Both methods will be illustrated in the examples that follow.

DEVELOPMENT OF ACCEPTANCE PROCEDURES

There are many ways in which the concepts outlined thus far can be applied, and it will be necessary to investigate several of them to understand the effects of minor procedural differences. For example, since the desired load-bearing capacity of a pavement is computed from the design average thickness and design average compressive strength, it might be thought necessary to compute the as-built average thickness and average compressive strength before entering these values into the AASHTO design equation to compute the as-built load capacity. Although this may turn out to be desirable, other procedures may be equally effective. One possible alternative would be to use the measured thickness and compressive strength from each sublot to determine the load ratios for all sublots. These would then be averaged to obtain the load ratio for the lot. An advantage of this approach is that the individual load ratios can be used to guard against isolated sections of poor quality within a single lot. As will be demonstrated in the following examples, these and still other variations can all be made to be extremely effective.

Example 1

For this example, it is desired to determine how well the pay schedule given by Equation 5 fits the desired pay function shown in Figure 2. The constraints imposed on this equation are a maximum pay factor of F = 1.12 and the stipulation that, for load ratios less than R_L = 0.50, the pay factor will be set at the minimum value of F = 0.60. There are no provisions for retesting in this case and the lot load ratio will be computed by using Equations 4 and 1 after first computing the average as-built thickness and compressive strength for each lot. A stratified random sampling plan is assumed with a single thickness and compressive strength determination made from each of a total of N = 5 sublots.

The only practical means to test this pay schedule is by computer simulation with the use of basic techniques described in a recent publication (7). For each of many different combinations of pavement thickness and compressive strength, a large number of random values of these parameters were generated and then processed in accordance with the requirements of the acceptance procedure. Figures 3 and 4 show the input and output stages of a typical computer run used to obtain points on the operating-characteristic curve for this plan.

From the fifth column of the computer output shown in Figure 4, the average load ratios obtained by simulation are seen to be in very close agreement with the population load ratios given in the third column. The average pay factors in the last column are used to plot the operating-characteristic curve for this plan in Figure 5. It can be observed from Figure 5 that, between load ratios of R_L = 0.60 and R_L = 1.60, the operating-characteristic curve matches the desired pay function quite closely. Although it begins to fall below the desired curve above R_L = 1.60, this is not considered to be a serious drawback because a pavement would seldom exceed this level of quality in actual practice. Below R_L = 0.60, the operating-characteristic curve drops rapidly to the minimum pay factor of F = 0.60. As explained in more detail in the companion paper in this Record, this is believed justifiable for such seriously defective pavement.

Although the operating-characteristic curve for this plan fits the desired pay function reasonably well, this is by no means the only pay schedule that could have been developed. Depending on the sample size, the critical load ratio below which the minimum pay factor is assigned, and the region within which a close fit is desired, a pay equation with a somewhat different intercept and slope might be appropriate. If an extremely close fit were required, a second-degree pay equation could be used although it is doubtful that the slightly better fit would justify the added complexity.

Example 2

The previous example contains no requirement on individual test results to guard against isolated sections of poor quality within a lot. For this next example, the same sample size of N = 5 will be used, the lot load ratio will be computed as before by first averaging the thickness and compressive strength results before entering them into the AASHTO equation, but an additional requirement will be imposed on each sublot. If the load ratio computed from the single values of thickness and compressive strength from a sublot is less than R_L =

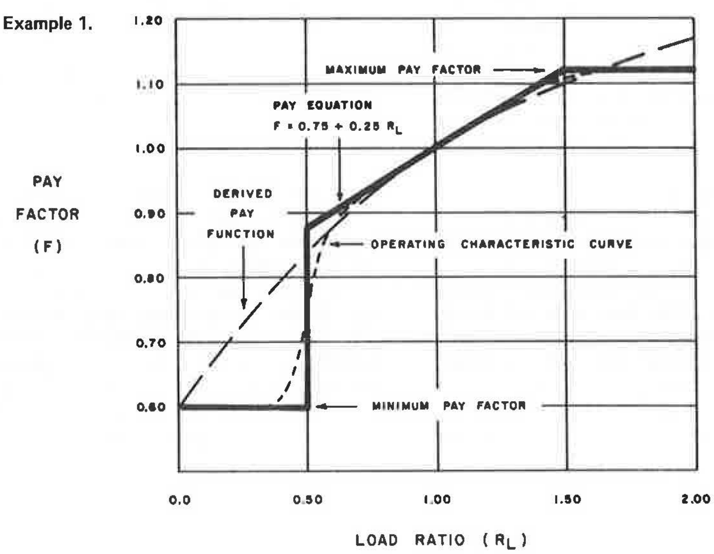
Figure 3. Input for typical computer simulation run.

```
run aashto13
EXECUTION BEGINS...

ENTER DESIGN VALUES FOR THICKNESS (INCHES) AND COMPRESSIVE STRENGTH (PSI)

9 4000

ENTER A AND B OF F = A + B(LOAD RATIO) AND MAXIMUM PAY FACTOR

0.75 0.25 1.12

ENTER LOWER LIMITING LOAD RATIO AND MINIMUM PAY FACTOR

0.50 0.60

ENTER RETEST LOAD RATIO AND NUMBER OF ADDITIONAL SAMPLES

0 0

ENTER STANDARD DEVIATIONS FOR THICKNESS AND COMPRESSIVE STRENGTH

0.25 500

ENTER MINIMUM, MAXIMUM, AND STEP SIZE FOR THICKNESS

8.50 9.50 0.25

ENTER MINIMUM, MAXIMUM, AND STEP SIZE FOR COMPRESSIVE STRENGTH

2500 5000 500

ENTER NUMBER OF LOTS PER RUN, SAMPLE SIZE, AND SEED NUMBER

500 5 1234567

ADVANCE PAPER TO NEW PAGE, DEPRESS SPACE BAR, AND RETURN CARRIAGE
```

Figure 4. Output for typical computer simulation run.

Population Means: Thickness	Population Means: Compressive Strength	Load Ratio Computed from Population Means	Simulation Results: Retest Frequency	Simulation Results: Average Load Ratio	Simulation Results: Average Pay Factor
8.50	2500	0.317	0.0	0.318	0.600
8.50	3000	0.433	0.0	0.435	0.645
8.50	3500	0.564	0.0	0.571	0.850
8.50	4000	0.709	0.0	0.718	0.929
8.50	4500	0.867	0.0	0.873	0.968
8.50	5000	1.038	0.0	1.040	1.010
8.75	2500	0.377	0.0	0.379	0.607
8.75	3000	0.516	0.0	0.522	0.778
8.75	3500	0.671	0.0	0.672	0.910
8.75	4000	0.843	0.0	0.839	0.960
8.75	4500	1.031	0.0	1.037	1.009
8.75	5000	1.235	0.0	1.251	1.062
9.00	2500	0.448	0.0	0.454	0.682
9.00	3000	0.611	0.0	0.613	0.878
9.00	3500	0.796	0.0	0.793	0.948
9.00	4000	1.000	0.0	1.007	1.002
9.00	4500	1.223	0.0	1.232	1.057
9.00	5000	1.465	0.0	1.457	1.102
9.25	2500	0.529	0.0	0.529	0.781
9.25	3000	0.723	0.0	0.723	0.928
9.25	3500	0.941	0.0	0.942	0.985
9.25	4000	1.182	0.0	1.192	1.047
9.25	4500	1.446	0.0	1.459	1.101
9.25	5000	1.732	0.0	1.743	1.119
9.50	2500	0.624	0.0	0.622	0.872
9.50	3000	0.852	0.0	0.858	0.964
9.50	3500	1.109	0.0	1.117	1.029
9.50	4000	1.394	0.0	1.405	1.091
9.50	4500	1.705	0.0	1.719	1.118
9.50	5000	2.041	0.0	2.041	1.120

Figure 5. Simulation results for Example 1.



0.75, two additional cores will be taken from the sublot. The test results from these cores will be averaged with the original values to determine the thickness and compressive strength associated with that particular sublot. The pay factor for the lot will then be computed in the usual manner except that if the load ratio for any sublot is less than the minimum required value of $R_L = 0.50$, the sublot will be treated as a separate lot and assigned the minimum pay factor of $F = 0.60$. In this event, the pay factor for the remainder of the lot will be computed by using the remaining test results.

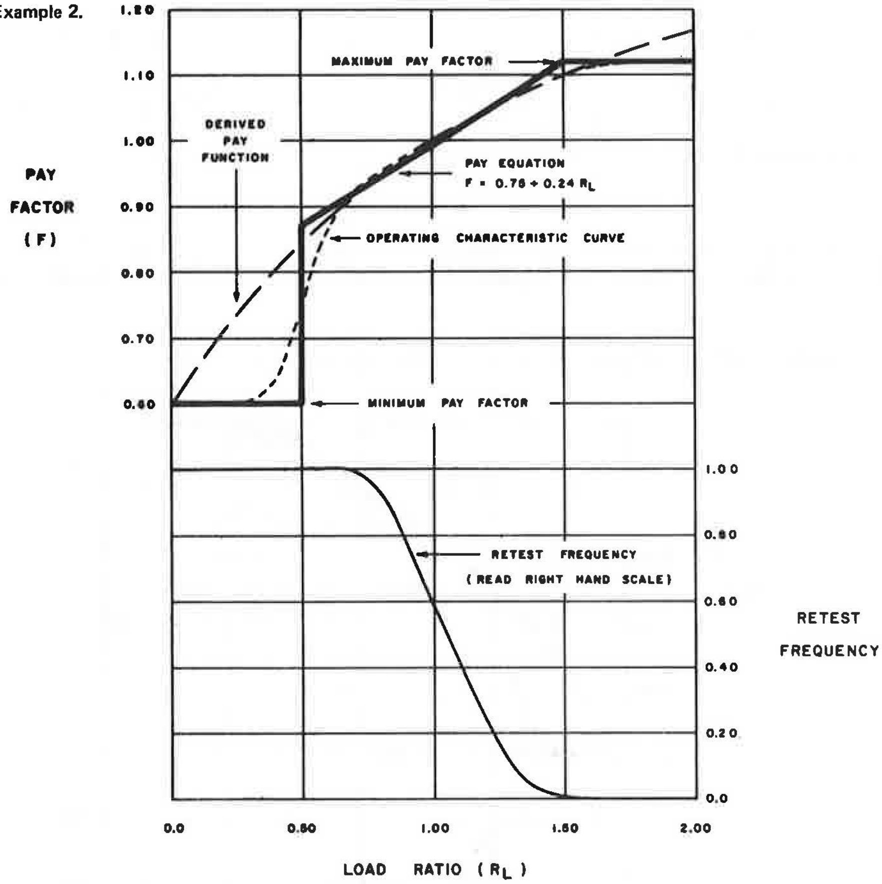
Because this acceptance procedure is different from that used in the first example, a slightly different pay equation is required. By trial and error, Equation 6 was found to be suitable for this application. The maximum pay factor of $F = 1.12$ continues to be satisfactory and the minimum pay factor remains unchanged at $F = 0.60$ for load ratios less than $R_L = 0.50$:

$$F = 0.75 + 0.24R_L \tag{6}$$

This acceptance procedure was also tested by computer simulation and the results are plotted in Figure 6. Although the pay equation produces $F = 0.99$ at $R_L = 1.00$, the retest provision bows the operating-characteristic curve upward sufficiently to pass through the point at which $R_1 = 1.00$ and $F = 1.00$, as it should. This effect is countered at the lower and upper ends by the minimum and maximum pay factors, which results in a good fit throughout the region between load ratios of $R_L = 0.60$ and $R_L = 1.60$. As in Example 1, the downward bias at the lower end is considered appropriate for such poor-quality pavement.

Although the operating-characteristic curve is quite satisfactory, this acceptance plan does have one serious drawback. As seen in Figure 6, the retest frequency is sufficiently high for normal construction that the plan would probably be con-

Figure 6. Simulation results for Example 2.



sidered impractical. For pavement that had a load ratio close to the desired value of $R_L = 1.00$, nearly 60 percent of the lots would require retesting. To correct this condition, the critical level of load ratio below which a retest is required must be set at a value substantially lower than $R_L = 0.75$. This modification will be made in the next example.

Example 3

This example is identical to the previous one except that the retest requirement is changed. First, a retest will not be required unless the load ratio for a single sublot is less than $R_L = 0.50$. Second, any sublot requiring a retest will be treated as a separate lot and N = 5 additional cores will be taken. Finally, the evaluation of the sublot will be based only on the new tests for strength and thickness; the original values will be discarded.

As in the previous example, the different acceptance procedure requires a different pay equation. In this case, Equation 7 was found to be appropriate. A maximum pay factor of F = 1.10 and the usual minimum pay factor of F = 0.60 for load ratios below $R_L = 0.50$ will be used with this equation.

$$F = 0.70 + 0.30R_L \tag{7}$$

The results of the simulation of this plan are shown in Figure 7. The operating-characteristic curve is very close to the desired pay function between load ratios of $R_L = 0.55$ and $R_L = 1.50$. However, unlike the previous example, the retest frequency is at an acceptably low level throughout the range within which most pavement would normally fall. At the design load ratio of exactly $R_L = 1.00$, for example, the retest frequency is about 5.0 percent.

Example 4

The remaining examples will all make use of individual sublot load ratios that will be averaged to obtain the load ratio for the lot. Each example will be constructed to be comparable with one of the previous examples to investigate the effect of the alternative method of computing the average load ratio.

This example is designed to be similar to Example 1. The maximum pay factor is F = 1.12, the minimum pay factor for load ratios below $R_L = 0.50$ is F = 0.60, and there is no retest provision. An appropriate pay schedule is given by Equation 8:

$$F = 0.745 + 0.25R_L \tag{8}$$

The simulation results are plotted in Figure 8. Although the pay equation produces F = 0.995 at $R_L = 1.00$, the operating-characteristic curve is bowed upward sufficiently to provide a good fit throughout the region between load ratios of $R_L = 0.60$ and $R_L = 1.60$.

Example 5

This example is designed to be comparable with Example 2 except that the critical value of load ratio below which a retest is required is set at a more practical level of $R_L = 0.50$. When a retest is required, two additional cores will be taken, which results in a total of three individual R_L values to be averaged together to obtain the load ratio for the sublot. If the average load ratio for any sublot is less than the minimum required level of R_L = 0.50, the sublot will be treated as a separate lot as described in Example 2. By using a maximum pay factor of F = 1.12 and a minimum pay factor of F = 0.60 for load ratios below R_L = 0.50, a suitable pay schedule is given by Equation 9:

$$F = 0.75 + 0.245R_L \tag{9}$$

The operating-characteristic curve and retest frequencies for this plan are shown in Figure 9. Although this plan is basically different from the one illustrated in Example 2, the resulting operating-characteristic curve is very nearly the same. However, unlike Example 2, the retest frequency is at a very tolerable level of about 5.0 percent for pavement with a load ratio close to the desired value of R_L = 1.00.

Example 6

This example is meant to be compared with Example 3. Whenever an individual load ratio for a sublot

Figure 7. Simulation results for Example 3.

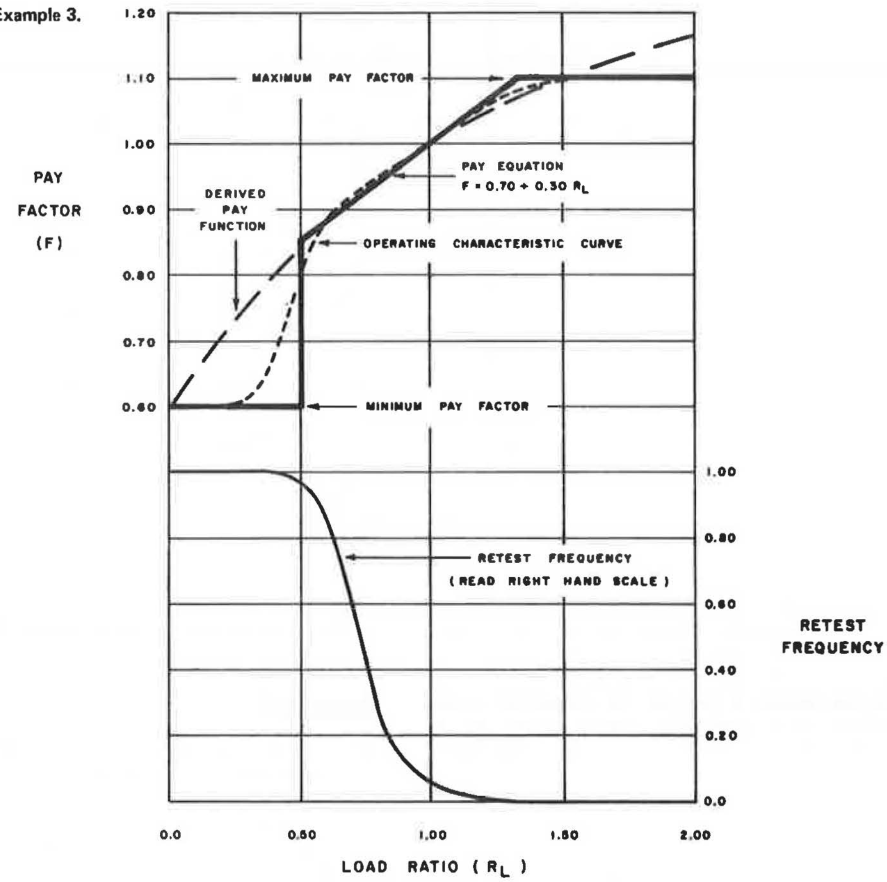


Figure 8. Simulation results for Example 4.

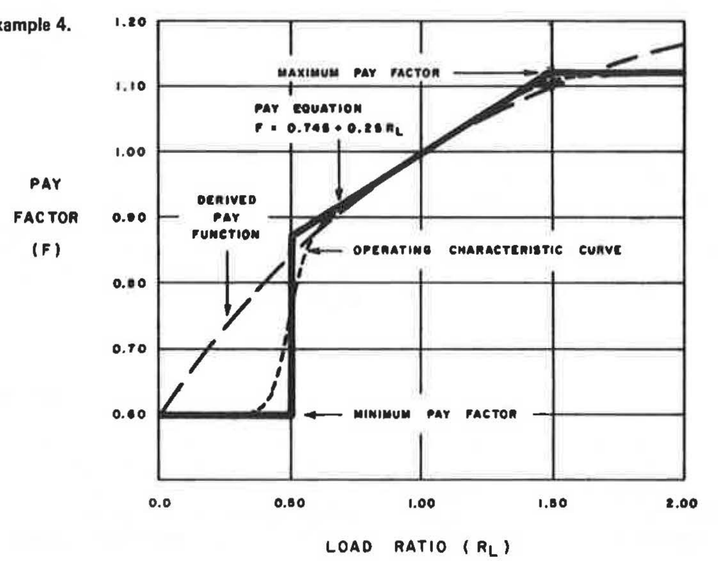


Figure 9. Simulation results for Example 5.

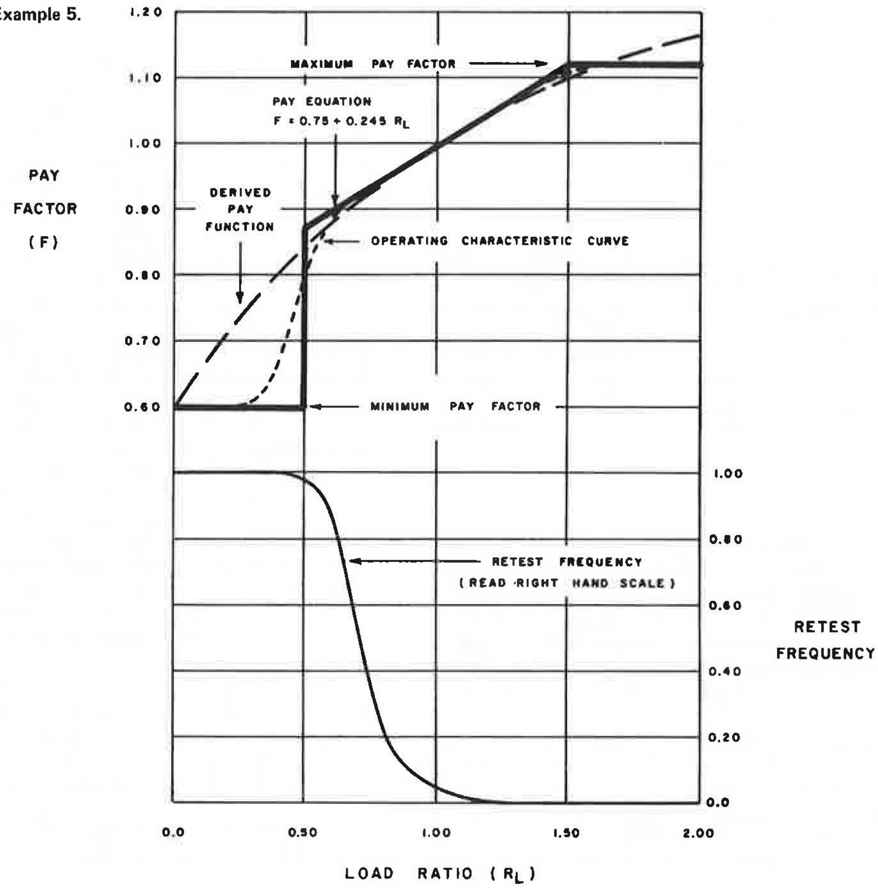
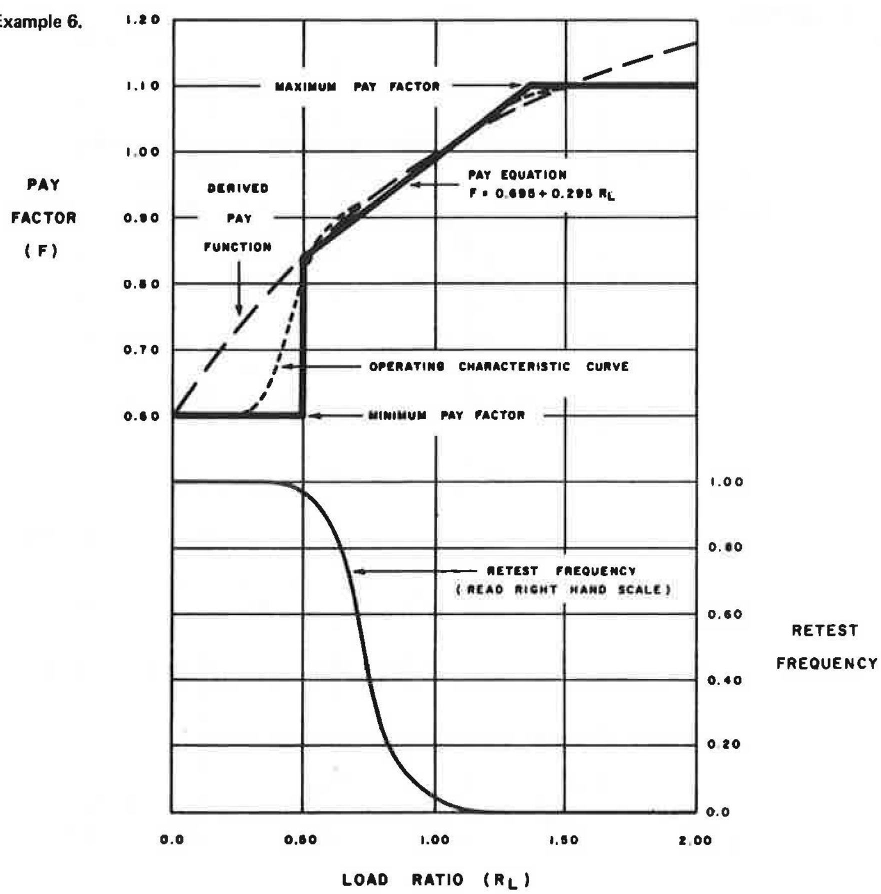


Figure 10. Simulation results for Example 6.



is below R_L = 0.50, the sublot will be treated as a separate lot, N = 5 additional cores will be taken, and the original test value for the sublot will be discarded. With a maximum pay factor of F = 1.10 and a minimum pay factor of F = 0.60 for load ratios below R_L = 0.50, Equation 10 was found to provide an appropriate pay schedule:

$$F = 0.695 + 0.295R_L \quad (10)$$

The simulation results for this example are plotted in Figure 10. The operating-characteristic curve matches the desired pay function just slightly better than the curve obtained in Example 3, although the results are so nearly the same that there may be no practical difference. Since the retest provisions are identical, so are the retest curves that indicate a normal retest frequency of about 5.0 percent.

Example 7

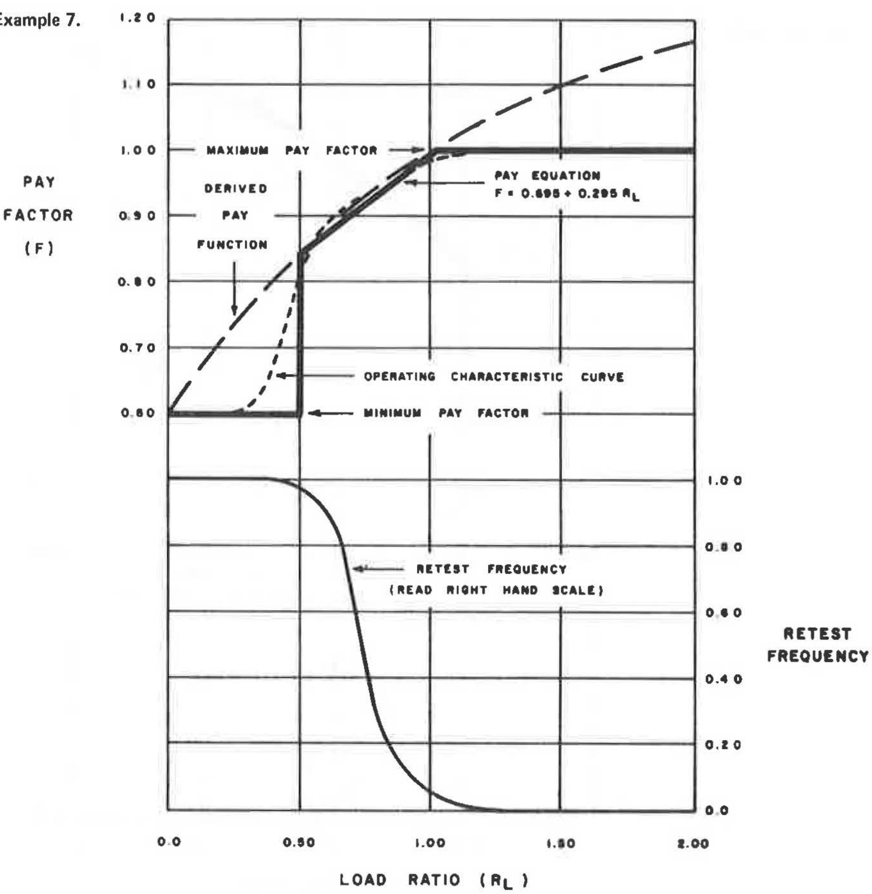
It was stated in an earlier section that if a highway agency did not elect to apply a bonus provision, the maximum pay factor could be limited at F = 1.00 (100 percent). To see what effect this would have on the operating-characteristic curve, this example duplicates the conditions of Example 6 except that the maximum pay factor is reduced from F = 1.10 to F = 1.00.

The results are plotted in Figure 11. As expected, the operating-characteristic curve matches the desired pay function very well except at the upper end, where it becomes increasingly biased downward. For a load ratio exactly at the design value of R_L = 1.00, the expected pay factor is approximately F = 0.985. The retest frequency curve, of course, remains unchanged.

Although the downward bias of 1.5 percent at the desired quality level seems relatively small, it was demonstrated in a recent paper (8) that with specifications based on the concept of percent defective, this can force contractors and producers to supply a level of quality substantially above that which is desired or economically justifiable from the highway agency's standpoint. For acceptance procedures based on the concept of load ratio, the effect is similar but somewhat less severe. In Figure 11, it is seen that a load ratio of about R_L = 1.20 or more is required to achieve an average pay factor of F = 1.00. By substituting typical values into the AASHTO equation, it is found that this would require either an increase in pavement thickness of about 0.3 in or an increase in compressive strength of approximately 500 psi.

An acceptance procedure such as this is misleading at best. Unless the contractor knows the degree of overdesign required, even good-quality work may receive a sufficient number of pay reductions to substantially reduce the expected profit margin on the job. Fortunately, there are two ways in which this undesirable feature can be corrected; both require that the average pay factor be 100 percent when the work is exactly at the desired quality level. The first method is simply to permit bonus pay factors as was done in the first six examples. It can be observed in each of Figures 5 through 10 that the operating-characteristic curve passes through the point at which R_L = 1.00 and F = 1.00, as desired. The second method permits pay factors in excess of 100 percent to be averaged with lower pay factors but is not a true bonus provision be-

Figure 11. Simulation results for Example 7.



cause the overall pay factor for specific intervals of time is still limited to a maximum of 100 percent. This method has been described in detail in an earlier paper (4).

ALTERNATIVE METHODS OF COMBINING MULTIPLE PAY FACTORS

The computer programs developed for this work provided the capability to conduct an interesting secondary study. By using a typical pay equation similar to those developed in the seven examples, it was possible to compute appropriate pay factors for various combinations of thickness and compressive strength and then compare these with the pay factors that would result from the several other methods that have been used to combine multiple pay factors. The results obtained by using five alternative methods are listed in Table 1. The product, average, minimum, and maximum methods are self-explanatory. For the cumulative method, the individual pay adjustments are summed to determine the total adjustment.

Assuming that the method based on the AASHTO equation is the fundamentally correct approach, the

Table 1. Comparison of alternative methods of combining multiple pay factors.

THICKNESS			COMPRESSIVE STRENGTH			METHOD BASED ON AASHTO EQUATION		PAY FACTOR DERIVED FROM INDIVIDUAL PAY FACTORS				
VALUE	LOAD RATIO	PAY FACTOR	VALUE	LOAD RATIO	PAY FACTOR	LOAD RATIO	PAY FACTOR	PRODUCT	AVERAGE	MINIMUM	MAXIMUM	CUMULATIVE
8.50	0.71	0.93	3000	0.61	0.90	0.43	0.86	0.84	0.92	0.90	0.93	0.83
8.50	0.71	0.93	3500	0.80	0.95	0.56	0.89	0.88	0.94	0.93	0.95	0.88
8.50	0.71	0.93	4000	1.00	1.00	0.71	0.93	0.93	0.96	0.93	1.00	0.93
8.50	0.71	0.93	4500	1.22	1.06	0.87	0.97	0.98	0.99	0.93	1.06	0.98
8.50	0.71	0.93	5000	1.46	1.12	1.04	1.01	1.03	1.02	0.93	1.12	1.04
8.75	0.84	0.96	3000	0.61	0.90	0.52	0.88	0.87	0.93	0.90	0.96	0.86
8.75	0.84	0.96	3500	0.80	0.95	0.67	0.92	0.91	0.95	0.95	0.96	0.91
8.75	0.84	0.96	4000	1.00	1.00	0.84	0.96	0.96	0.98	0.96	1.00	0.96
8.75	0.84	0.96	4500	1.22	1.06	1.03	1.01	1.01	1.01	0.96	1.06	1.02
8.75	0.84	0.96	5000	1.46	1.12	1.23	1.06	1.07	1.04	0.96	1.12	1.08
9.00	1.00	1.00	3000	0.61	0.90	0.61	0.90	0.90	0.95	0.90	1.00	0.90
9.00	1.00	1.00	3500	0.80	0.95	0.80	0.95	0.95	0.97	0.95	1.00	0.95
9.00	1.00	1.00	4000	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
9.00	1.00	1.00	4500	1.22	1.06	1.22	1.06	1.06	1.03	1.00	1.06	1.06
9.00	1.00	1.00	5000	1.46	1.12	1.46	1.12	1.12	1.06	1.00	1.12	1.12
9.25	1.18	1.05	3000	0.61	0.90	0.72	0.93	0.94	0.97	0.90	1.05	0.95
9.25	1.18	1.05	3500	0.80	0.95	0.94	0.99	0.99	1.00	0.95	1.05	0.99
9.25	1.18	1.05	4000	1.00	1.00	1.18	1.05	1.05	1.02	1.00	1.05	1.05
9.25	1.18	1.05	4500	1.22	1.06	1.45	1.11	1.10	1.05	1.05	1.06	1.10
9.25	1.18	1.05	5000	1.46	1.12	1.73	1.18	1.17	1.08	1.05	1.12	1.16
9.50	1.39	1.10	3000	0.61	0.90	0.85	0.96	0.99	1.00	0.90	1.10	1.00
9.50	1.39	1.10	3500	0.80	0.95	1.11	1.03	1.04	1.02	0.95	1.10	1.05
9.50	1.39	1.10	4000	1.00	1.00	1.39	1.10	1.10	1.05	1.00	1.10	1.10
9.50	1.39	1.10	4500	1.22	1.06	1.70	1.18	1.16	1.08	1.06	1.10	1.15
9.50	1.39	1.10	5000	1.46	1.12	2.04	1.26	1.23	1.11	1.10	1.12	1.21

Table 2. Evaluation of alternative methods of combining multiple pay factors.

	DEVIATIONS FROM CORRECT PAY FACTOR FOR VARIOUS METHODS				
	PRODUCT	AVERAGE	MINIMUM	MAXIMUM	CUMULATIVE
	-0.02	0.06	0.04	0.07	-0.03
	-0.01	0.05	0.04	0.06	-0.01
	0.0	0.03	0.0	0.07	0.0
	0.01	0.02	-0.04	0.09	0.01
	0.02	0.01	-0.08	0.11	0.03
	-0.01	0.05	0.02	0.08	-0.02
	-0.01	0.03	0.03	0.04	-0.01
	0.0	0.02	0.0	0.04	0.0
	0.0	0.0	-0.05	0.05	0.01
	0.01	-0.02	-0.10	0.06	0.02
	0.0	0.05	0.0	0.10	0.0
	0.0	0.02	0.0	0.05	0.0
	0.0	0.0	0.0	0.0	0.0
	0.0	-0.03	-0.06	0.0	0.0
	0.0	-0.06	-0.12	0.0	0.0
	0.01	0.04	-0.03	0.12	0.02
	0.0	0.01	-0.04	0.06	0.0
	0.0	-0.03	-0.05	0.0	0.0
	-0.01	-0.06	-0.06	-0.05	-0.01
	-0.01	-0.10	-0.13	-0.06	-0.02
	0.03	0.04	-0.06	0.14	0.04
	0.01	-0.01	-0.08	0.07	0.02
	0.0	-0.05	-0.10	0.0	0.0
	-0.02	-0.10	-0.12	-0.08	-0.03
	-0.03	-0.15	-0.16	-0.14	-0.05
AVERAGE:	-0.001	-0.007	-0.046	0.035	-0.001
STANDARD DEVIATION:	0.013	0.054	0.056	0.066	0.020
RANK FOR ACCURACY:	1 - 2	3	5	4	1 - 2
RANK FOR PRECISION:	1	3	4	5	2
OVERALL RANK:	1	3	4 - 5	4 - 5	2

values in the last five columns of Table 1 can be used to judge which of the other methods is most appropriate in this particular case. To accomplish this, the deviations from the values obtained by the fundamental method have been tabulated for each of the other methods in Table 2. The summary statistics for each column are printed at the foot of the table. For a method to be judged both accurate and precise, the average deviation must be close to zero and the standard deviation should be small. On this basis, the five methods have been ranked for accuracy and precision and the overall rank has been determined by weighting these two separate ranks equally.

What emerges from this rather cursory investigation is evidence that the method of multiplying individual pay factors together is equal or superior to any of the other methods that were tested, at least for this particular application. This is encouraging, not only because this approach is widely used, but also because it suggests a method by which additional quality characteristics not included in the AASHTO equation might be incorporated into acceptance procedures for rigid pavement.

SUMMARY AND CONCLUSIONS

A method has been outlined by which the AASHTO design equation can be used to develop acceptance procedures for rigid pavement. By computing the expected load-bearing capacity from the as-built characteristics of the pavement and comparing this with the design loading, a ratio is obtained that forms the basis for a rational pay schedule. Sensitivity tests were performed to confirm the reliability of this approach, and several different acceptance procedures were developed and tested by computer simulation. In all cases, it was possible to make the operating-characteristic curve conform closely to the desired pay function.

It was demonstrated that the limitation of pay factors to a maximum of 100 percent biases the operating-characteristic curve downward, which makes it difficult for contractors to know how to bid or perform under such a specification. This situation can be alleviated by allowing a bonus provision or by permitting pay factors greater than 100 percent to be used to offset other pay factors less than 100 percent.

Finally, a secondary study was conducted to compare various methods currently in use for combining multiple pay factors. Under the assumption that the method based on the AASHTO equation is fundamentally correct, it was demonstrated that the method of multiplying the individual pay factors together is among the best of the other methods that were tested.

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Interest on Capital Invested in Construction as Delay Damages

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The potential for contractors to recover extended financing costs that result from a construction delay is investigated. Legal case histories arising from the federal courts and boards of contract appeals are reviewed, and recent developments related to federal construction contract procedures are presented. Legal case studies are cited that indicate that delay damages can be recovered under the suspension-of-work clause even though no written directive is issued. Delay damages under the change clause are generally not recoverable, although the general conditions of construction contracts of the General Services Administration and the Department of Defense do permit recovery of cost of delays related to change orders. Legal precedents are reviewed that suggest that interest on borrowed funds that was necessitated by a delay can also be recovered. Regulations that prohibit recovery of interest on borrowed funds governed by most federal construction contracts are reviewed. These have been challenged and upheld in the U.S. Court of Claims. Since 1976, boards of contract appeals have awarded imputed-interest damages. These damages result when a contractor is required by a delay to increase the capital investment in a construction project. This increased investment represents a loss of profit because these funds could otherwise be invested in short-term securities and treasury notes. Cost Accounting Standard 417, effective December 1980, provides for the recovery of imputed-interest damages resulting from a delay. The calculation procedure presented in CAS 417 is illustrated with a construction example. It is shown that on a project that costs $2 380 750 and experiences a three-month suspension-of-work delay, the contractor is entitled to $29 702 in imputed-interest damages in addition to any other damages that may have been incurred.

Acceptable cash flow for a construction contractor is largely dependent on ability to achieve satisfactory progress with regard to the project schedule. Unanticipated delays in the construction process, regardless of the cause or responsible party, will likely result in additional direct and indirect costs for the contractor. Additional direct costs caused by schedule delays have been cited in numerous damage claims. Whenever these disputes have resulted in litigation, courts have shown a general tendency to award extra compensation. A distinct exception to this trend is the awarding of extended finance charges and interest damages as a result of the delay.

OBJECTIVES AND SCOPE

The objectives of this paper are to define the damages to contractors resulting from financing costs incurred when the work is suspended or when work must be performed that is beyond the scope of the original construction contract. The potential for recovering these damages in litigation proceedings will be assessed by presenting recent legal trends and case studies. New developments in cost accounting standards that describe a procedure for computing financing costs will also be illustrated.

The objectives of this paper were achieved through a study of legal case histories obtained from the legal library of Pennsylvania State University and other unpublished documents and reports. Important aspects of key cases were clarified by interviewing government lawyers involved in construction litigation cases for the U.S. Department of Defense (DOD). The concepts in this paper are limited to interpretations presented in Board of Contract Appeals (BCA) hearings and courts of law. Decisions of contracting officers and out-of-court settlements are not included. Essentially all key decisions, especially recent ones, have been rendered in disputes against the federal government. In the private sector few significant cases involving questions about financing costs have been settled in court. Nevertheless, it would appear logical that federal contracting procedures could easily be extended to state and local governments and to the private sector of the construction industry.

TERMINOLOGY

To understand claims involving government contracts and legal case studies, the following definitions will be used:

1. Contracting officer: An official of the agency awarding a contract who has the power to decide disputes arising during the performance of a construction contract. The contracting officer's decision is conclusive unless appealed to the agency's BCA.
2. Board of Contract Appeals: Any one of 11 different contract appeal boards representing various federal agencies. Appeal of a decision by a contracting officer is heard by BCA. The hearing resembles a courtroom proceeding. The BCA referenced in this paper is the Armed Services Board of Contract Appeals (ASBCA), whose jurisdiction is DOD contracts. Decisions of BCA can be appealed to the U.S. Court of Claims.
3. Court of Claims: A federal court that decides cases involving claims against the United States Government. Decisions of the Court of Claims can be appealed to the United States Supreme Court.

FINANCE COSTS AS DIRECT DAMAGES

A review of federal appeals board cases and Court of Claims decisions involving claims for delay damages indicates that the potential exists for recovering both direct and consequential damages. Direct damages include home and field office overhead, equipment expenses, escalation of material and labor cost, and loss of efficiency. Consequential damages are not directly related to the delay and include loss of bonding capacity, restrictions on the contractor's ability to perform other jobs due to limited working capital, and loss of profit. Consequential damages can be somewhat vague and often lack "specific rationale," or traceability to the delay. Direct damages are more easily recovered because they can usually be linked to the delay. The fact that contractors must be able to accurately show a cause-effect relationship was demonstrated in the 1975 case of Roanoke Hospital versus Doyle and Russell, Inc. [214 S.E. 2d 155 (1975)]. This requirement is not always easily satisfied where intercorporate and intracorporate financing are involved.

In litigation proceedings, the potential for recovering extended financing costs is largely a function of predictability. Direct damages are predictable because they are a natural outgrowth of construction delays. In essence there is a cause-effect relationship. The case of Roanoke Hospital versus Doyle and Russell is especially significant because the court accepted the concept that extended interest costs constitute direct damages. A second case that establishes a precedent for recovering these costs is Hammermill Paper Company versus Rust Engineering Company [243 A. 2d 389 (1968)]. In this 1968 case, extended financing charges were allowed because they were documented with specific rationale. Therefore, establishing the link between delays and damage is a necessary condition for recovery. Furthermore, the damages must be clearly and accurately documented. When either of these two conditions fails to exist, damages wil not likely be awarded [Clark versus Ferro Corporation, 273 F. Supp. 230 (1964)]. Therefore, legal case history indicates that financing costs will not be awarded just because a delay has occurred. It is also important to note that in both the Roanoke Hospital and the Hammermill Paper Company cases, financing costs were treated as damages that were separate from any other damages that may have occurred.

RECOVERY OF DELAY DAMAGES

Contract Provisions Related to Damage Delays

As a general rule, a contractor is entitled to recover damages resulting from a delay caused by an owner, provided the following conditions can be established: (a) a delay initiated by the owner or the owner's agent to the work of the contractor in fact occurred, (b) the contractor was damaged by the delay (cause-effect relationship), (c) the delay was beyond the control of the contractor, and (d) damages are clearly and accurately documented (specific rationale). Should any of these conditions be absent, recovery of damages will likely be denied. However, there are situations where contract provisions may influence the decision of an appeals board or court in a damage claim. It is important, therefore, to understand the contract provisions related to delay damages. Three specific types of provisions will be reviewed. These are clauses related to suspension of work, changed conditions, and provisions prohibiting the recovery of damages.

Suspension-of-Work Clause

Three situations are of interest regarding suspension-of-work clauses. The first condition is when an owner invokes the suspension-of-work clause in writing. For the contractor to recover damages, the following provisions apply:

1. The delay must be for an unreasonable period of time,
2. The delay must not be a result of a concurrent delay caused by the contractor,
3. There will be no cost adjustment if the circumstances require or exclude an equitable adjustment under other provisions of the contract,
4. An allowance for profit is not included in any cost adjustment under this provision,
5. Claims for time extensions must be made in writing no more than 20 days after the commencement of the delay, and
6. Claims for cost adjustments must be submitted in writing as soon as possible after the suspension ends and no later than the date of final payment under the contract.

These conditions are included in most federal construction contracts [10 U.S.C. 2301-2314 (1979)] and in the general conditions of construction contracts of the American Institute of Architects (1), the Associated General Contractors of America (2), and the National Society of Professional Engineers (3).

Although the above contract provisions relate to written suspension-of-work directives, the owner is not necessarily protected from claims when written directives are not provided. This principle was established in the 1972 case of Carl M. Halverson, Inc., versus United States [461 F. 2d 1337 (1972)] where the suspension-of-work order was communicated verbally but not in writing. The trial commissioner ruled that a change order required because of an error in the contract drawings, related to the relocation of a creek, resulted in a suspension of the work of the contractor in a priority work area. Therefore, the contractor was entitled to recover the increased costs resulting from such suspension.

It is unusual for construction contracts to be void of suspension-of-work clauses. Nevertheless, a contractor may still be entitled to recovery of damages under a breach of contract. In fact, in the 1970 case of Chaney and James Construction Company, Inc., versus United States [421 F. 2d 728 (1970)], the court handed down the opinion that there is no difference between the remedies available under a suspension-of-work clause or a breach of contract. The opinion stated in part that "since the suspension of work clause is an administrative substitute for an action at law for a breach of contract...the contractor should be entitled to get the same relief under the suspension of work clause that he could get in the absence of the clause if he sued for breach-of-contract". This opinion is most significant, since it relates to change orders.

Change Orders

The general conditions of most contracts allow the owner to make changes in the scope of the work covered by the contract. Changes may cause delays in the construction process by disrupting the sequence of construction operations, altering previously completed work, or extending the schedule because of an increase in the amount of work to be performed. Although compensation for scope changes may be handled in a straightforward manner, claims for delay damages are not so simple.

In 1943, the U.S. Supreme Court set forth guidelines for recovering damages resulting from changes. The landmark case of United States versus Rice has become known as the Rice Doctrine [United States versus Rice, 317 U.S. 61, 63 S. Ct. 120 (1943)]. The Supreme Court ruled that cost adjustments are the result of altered specifications or changes and are not applicable to consequential damages. Delay damages are covered by time extensions. In essence, the court said that damages cannot be recovered under the contract provisions related to a change clause. Lower courts have consistently upheld this decision and have allowed damages to be recovered only if a breach of contract could be demonstrated (4). The most likely way to recover damages resulting from change orders is to demonstrate that there was a suspension of work, which is essentially a breach of contract.

No Damages for Delay Clauses

Provisions that preclude the recovery of delay damages may be included in some state, municipal, and private contracts (5). Delay damages are denied in lieu of a time extension for completing the work. Such provisions have been the subject of much litigation. However, courts have ruled that these clauses are not against public policy and therefore are enforceable. There are many exceptions, and enforceability varies from state to state (4).

Developments in Federal Contracting Procedures

In 1968, the General Services Administration and DOD amended the general conditions of their respective construction contract documents to allow for the recovery of costs of delays and disruptions (the ripple effect) caused by change orders (under the change clause) and changed conditions (under the differing-site-conditions clause). This would appear to be contrary to the Rice Doctrine, which precludes the recovery of delay damages under change clauses. However, when delays occur because the change orders are not timely, this constitutes an informal or involuntary suspension of work, and damages can be awarded. It is not necessary that a written suspension-of-work order be issued. Moreover, in 1972, a court decision in Tri-Cor, Inc., versus United States [458 F. 2d 112 (1972)] ruled that a contractor performing government work is entitled to an allowance for profit under these two clauses.

The ASBCA has issued rulings that emphasize the importance that contracting officers render timely decisions and issue change orders that affect work along the critical path of a construction schedule. For example, ASBCA ruled that a contracting officer unreasonably delayed a project by taking eight days to render a decision on certain disputed work. The contractor was subsequently awarded damages under the suspension-of-work clause even though no written order was issued. In another decision, ASBCA ruled that a contractor was entitled to reimbursement for the increased costs incurred when suspension of work was required because of a government delay in issuing required change orders (6).

Interest Damages and Bell Case

Previous court decisions appear to have signaled an opportunity for the recovery of extended financing costs (interest) as delay damages. A landmark Court of Claims case in this area was Joseph Bell versus United States [404 F. 2d 975 (1968)], which was decided in 1968. The court awarded compensation to a contractor for the additional interest he had to pay on a loan over the extended period of time that was caused by a slowdown initiated by the owner. In allowing recovery of the interest, the court held that the increased interest was undoubtedly an increased cost of performance attributable to the change. It is important to note that the court stated that the contractor was not seeking interest on the money owed to him by the owner.

This case was significant in that it established a precedent for recovering interest on borrowed funds as a type of damage. Subsequent BCA and Court of Claims decisions that have awarded interest damages have been based on this precedent. The Bell case set in motion the development in 1970 of regulations prohibiting recovery of interest damages on DOD contracts. However, more recent cases in both BCA and the Court of Claims have permitted recovery of imputed-interest damages. Imputed interest is interest on the capital invested in facilities under construction. These developments have spawned considerable uncertainty in the law.

RECOVERY OF INTEREST ON BORROWED FUNDS

In the private sector of the construction industry where there are no regulations addressing the issue of interest damages, there is some precedent for recovering interest costs. The cases of Hammermill Paper Company versus Rust Engineering Company in 1968 and Roanoke Hospital versus Doyle and Russell in 1975 are examples. In another recent case (1979) interest costs were also awarded. The Atlas Concrete Pipe, Inc., was awarded interest damages because the defendant, Roger J. Au and Sons, Inc., failed to pay an outstanding open account of indebtedness [467 F. Supp. 830 (1979)].

Where regulations exist covering interest damages, such as at the federal and state government levels, the situation prior to 1980 was quite different. Following the Bell decision, which had awarded interest damages, federal regulations were developed to prohibit the payment of interest damages on contracts governed by the Defense Acquisition Regulations (DAR) and the Federal Procurement Regulations (FPR). Defense Procurement Circular (DPC) 79, dated May 15, 1970, mandates the application of DAR 15-106, Section 15 (Cost Principles, Pricing of Equitable Adjustments Under Firm Fixed-Price DOD Contracts) (32 C.F.R. 15.106). Two years later, on March 31, 1972, FPR also made DAR 15 mandatory for other government agency fixed-price contracts (41 C.F.R. 1-15.106). DAR Section 15 specifically states that interest on borrowed funds from external lending institutions is not an allowable adjustment to a contract price.

By using the Bell decision as a precedent, the legality of DAR Section 15 has been challenged in both the courts and BCA. The 1977 case of Framlau Corporation versus United States is of special interest [215 Ct. Cl. 185 (1977)]. The decision by the Court of Claims upheld the concept of administrative regulations that prohibit the awarding of interest on borrowed funds arising from a claim against the gpvernment, except where specifically allowed by a contract provision or authorized by statute. The significance of the decision is summarized as follows:

1. Administrative restrictions on recovering interest damages on borrowed funds are lawful, although these regulations can be overruled by statutes or by contract language to the contrary.
2. The decision did not specifically address the legality of interest damages in situations where there are no regulations or contract provisions. However, by stating that regulations can be overruled by contract provisions allowing interest recovery, the court appears to imply that such recoveries are legal.

With the Bell and Framlau cases serving as precedent, it would appear that a contractor can recover interest damages on borrowed funds provided there are no regulations or contract provisions to the contrary.

In the absence of limiting regulations or provisions, recovery of interest damages may be possible. However, a prerequisite is that the contractor must show that the delay or change in work created a clear necessity for borrowing additional funds and that these can be traced to the delay.

The burden of proof and traceability may not be too difficult for small and medium-sized contractors if they secure a loan from a financial lending institution. The situation is quite different for large construction firms, especially those with complex organizational structures. In many instances, such firms are subsidiaries or divisions of larger corporations and are often treated as satellite cost centers with the headquarters level being responsible for all capital investments and corporate financing. In most circumstances, past borrowing practices and the manner in which accounting records are arranged simply do not lend themselves to the degree of traceability that may be necessary. As a result, most large contractors have been unable to recover interest on actual borrowings as a cost of performance. Only in a few instances will there likely be satisfactory evidence to adequately support an interest claim involving a large corporation. Furthermore, the revision of intercorporate and intracorporate financing procedures appears unlikely, particularly those involved with federal contracts, because DAR 15 denies recovery of these damages.

RECOVERY OF IMPUTED-INTEREST DAMAGES

Establishment of Precedence

The Bell case and others dealt only with funds borrowed from an external lending institution. The issue of delay damages to a contractor whose own capital is invested in lieu of borrowing monies was not addressed.

It is important to note that the issue here is over the contingency funds of the contractor that are used as reserve capital when necessary but are otherwise invested in treasury notes and short-term securities. This situation is quite common among larger contractors. Interest on monies borrowed internally from this contingency fund is called imputed interest, and DAR 15 makes no mention of this form of borrowing. Imputed interest represents capital invested in the project by the contractor. Recent trends have been for the larger contractors to try to recover imputed-interest damages. The rationale is that imputed interest represents a loss of profit because the reserve funds could otherwise be invested.

The distinction between interest paid to an external lending institution and imputed interest is, in reality, a distinction between borrowing methods and hence is often related to the size of the contracting firm. An earlier case history that disallowed imputed interest but permitted interest on borrowing from external institutions in essence penalized the larger contractors in favor of the smaller ones.

Since the mid-1970s some contractors have been successful in recovering imputed-interest damages. The first shift in attitude occurred in the 1976 BCA case of New York Shipbuilding Company [ASBCA 16164, 76-2 BCA 11979 (1976)] where the plaintiff was able to recover these damages. Subsequent BCA cases have upheld this philosophy [Bailfield Industries, ASBCA 18057, 77-1 BCA 12348 (1977); Ingalls Shipbuilding Division, Litton Systems, Inc., ASBCA 17579, 78-1 BCA 13038 (1978); Fischbach and Moore International Corporation, ASBCA 18146, 77-1 BCA 12300 (1977)]. When the decisions are reviewed, there appears to be a trend toward not making a distinction between financing methods. This situation would suggest that the contractor is not required to show that actual borrowing was required but rather that the delay necessitated an increase in the capital invested in the project. Thus the concept of capital invested in a project has been introduced. Since, in these cases, the borrowing method does not seem to be at issue, it appears that the small contractor could also recover interest on borrowings.

Imputed Interest as Element of Profit

When court and BCA decisions are studied, two distinct viewpoints about imputed-interest compensations have emerged. The first viewpoint is that imputed interest is an element of profit, and, as such, the contractor is entitled to fair compensation for the increased investment due to changes in scope of the work. This is not necessarily a dollar-for-dollar recovery. Typically, BCA determines whether the profit associated with the additional work is in itself adequate to provide compensation for the contractor's investment. If so, no additional profit in the form of imputed interest will be allowed.

The 1976 decision in the New York Shipbuilding case marked the first time that imputed-interest damages had been awarded. The element-of-profit concept was established, and the damages were computed by using traditional interest formulas that take into account the principle, type and rate of interest, and the time. A year later, more definitive guidelines were set forth that are a function of progress payments. In the Fischbach and Moore International Corporation dispute, BCA awarded damages as an element of profit. The following considerations were applied: (a) normal progress payments and profit levels in the industry, (b) progress payments actually made on the particular contract, and (c) the amount of profit applied to the changed work.

The question of imputed interest should be placed in proper perspective, because in the 1977 Court of Claims case Framlau versus United States, an element of uncertainty was introduced. It should be recalled that the principal issues in this case were interest on funds borrowed from an external lending institution and the legality of DAR 15. Although recovery of damages was denied, the court stated that it is appropriate to apply different treatment to the recovery of interest on borrowings and imputed interest, because the contractor's cost of borrowing capital is clearly determinable, whereas the value to the contractor for the use of capital equity is not. This viewpoint would tend to make the recovery of imputed interest somewhat doubtful. However, in awarding imputed-interest damages to Fischbach and Moore International Corporation, BCA interpreted the Court of Claims decision as having considered only the question of interest and not profit for the use of capital. No subsequent cases regarding imputed interest have reached the Court of Claims, and whether or not the BCA interpretation would be adopted is quite uncertain.

Imputed Interest as a Cost of Performance

The second viewpoint about imputed interest is that it represents a cost of performance. Therefore, the contractor should be entitled to full recovery of imputed interest plus normal profit should the damage be the result of a change. Although no case history involving the construction industry is available, precedent seems to exist for treating imputed interest as a cost of performance. Cost Accounting Standard 417 (CAS 417) (Cost of Money as an Element of Capital Assets under Construction), dated December 15, 1980 (45 Fed. Reg. 48573), treats imputed interest as a cost. Also, in certain transactions, the Internal Revenue Code recognizes imputed interest as a cost that can be treated as an expense to a borrower and as an income to a lender. Thus, for tax purposes, imputed-interest charges are sometimes considered a cost of performance. This rule is applicable when loans or extensions of credit are made with no stated interest rate or when the interest is at less than the prevailing rate [IRC 482, Internal Revenue Regulation 1.482-2(a); Aristar, Inc., versus United States, 553 F. 2d 644 (1979); Kahler Corporation versus Commissioner of Internal Revenue, 486 F. 2d 1 (1973)]. Similarly, imputed interest is recognized by the IRS when property is sold or exchanged for deferred payments, and no interest, or an inadequate rate of interest, is quoted [Jeffers versus United States, 556 F. 2d 986 (1977)].

CAS 417

The latest supplement to CAS 417, effective December 15, 1980, sets forth guidelines for computing imputed-interest damages. All contracts to which DAR 15, Cost Principles, is applicable provide for the recovery of imputed interest. This standard is intended to provide a consistent approach in determining imputed-interest damages resulting from construction delays. Damage computations are based on the capital assets that are committed to the project. CAS 417 is applicable to both borrowing from external lending institutions and internal or intracorporate borrowing situations. Thus, the regulations can be applied to any size contractor with equal fairness. Originally, the Cost Accounting Standards Board required that the contract be at least one year in length. This was based on the philosophy that the administrative costs of the contractor on short-term projects would typically be higher than the imputed-interest damage claim that could be recovered. This restriction has recently been removed since it is now apparent that imputed interest can be quite significant on projects lasting less than one year.

The CAS 417 details two alternative procedures for computing imputed-interest damages. The first method applies to those situations where the cumulative receipts or progress payments are approximated by an S-shaped curve. This situation is, of course, typical of the construction industry. The amount of an award is a function of the average of the cumulative monthly expenses of the contractor. The second method is used when the cumulative receipts or progress payments are approximated by a straight line. The damage amount is a function of the average of the cumulative expenses of the beginning and the end of the accounting period.

To illustrate the computational procedure of CAS 417, an example will be presented. The project to which the calculations are applied is the construction of the Navigation Systems Facility at the Naval Air Development Center, Warminister, Pennsylvania. The construction contract was awarded to G & C Enterprises, Inc., Bordentown, New Jersey, for $2 380 750.

A hypothetical schedule of receipts and expenditures was developed based on the actual construction schedule of the contractor. It is worthwhile to note that the critical-path method (CPM) of scheduling is very useful in documenting project receipts and expenditures. In this example, an early-start schedule was assumed. In developing the summary of planned project expenditures and receipts given in

Table 1. Planned project billings and receipts.

Date	Monthly Expenses ($)	Monthly Billings and Profit ($)	Monthly Receipts Less Retainage ($)
1979			
June	275 000	294 250	264 825
July	75 000	80 250	72 225
August	180 000	192 600	173 340
September	225 000	240 750	216 675
October	165 000	176 550	158 895
November	225 000	240 750	216 675
December	180 000	192 600	173 360
1980			
January	170 000	181 900	163 690
February	255 000	272 850	245 565
March	170 000	181 900	163 710
April	125 000	133 750	120 375
May	80 000	85 600	77 040
June	40 000	42 800	38 520
July	35 000	37 450	33 705
August	25 000	26 750	24 075
September	-	-	238 075
Total	2 225 000	2 330 750	2 380 750

Table 2. Revised billings and receipts after three-month delay.

Date	Monthly Expenses ($)	Monthly Billings and Profit ($)	Monthly Receipts Less Retainage ($)
1979			
June	275 000	294 250	264 825
July	75 000	80 250	72 225
August	180 000	192 600	173 340
September	225 000	240 750	216 675
October	165 000	176 550	158 895
November	225 000	240 750	216 675
December	180 000	192 600	173 360
1980			
January	170 000	181 900	163 690
February	15 455	-	-
March	15 455	-	-
April	15 455	-	-
May	255 000	272 850	245 565
June	170 000	181 900	163 710
July	125 000	133 750	120 375
August	80 000	85 600	77 040
September	40 000	42 800	38 520
October	35 000	37 450	33 705
November	25 000	26 750	24 075
December	-	-	238 075
Total	2 271 365	2 380 750	2 380 750

Table 1, the following assumptions were made:

1. The overhead and profit are 10 and 7 percent, respectively.
2. Billings are based on the consumption of resources rather than probable pay items.
3. Receipts are actual billings less 10 percent retainage, which is recovered at the time of final acceptance.
4. Billings are made as of the last day of each month; payments are not received until the 15th of the following month.

The following hypothetical situation is presented to demonstrate the effect of a delay on the interest damages due the contractor. Assume that a conflict arises over underground utility work that is in progress in the immediate vicinity of the project. The contracting officer issues a stop-work order effective February 1, 1980. The contractor is not allowed to resume work until May 1, 1980. In computing the revised schedule of expenses, the following assumptions are made:

1. Overhead costs continue during the delay.
2. Extra compensation for overhead is included in a separate damage claim.
3. After the delay, work resumes as previously planned.

The revised billings are summarized in Table 2.

In this example, it is assumed that the cash flow of the contractor is approximated by an S-shaped curve, and imputed interest is calculated for accounting periods that coincide with a calendar year. The cumulative expenses for the seven months in 1979 are as follows:

Month	Expenses ($000s)
June	275
July	350
August	530
September	755
October	920
November	1145
December	1325
Total	5300

Avg = $5 300 000 ÷ 7 = $757 143.

The imputed interest for 1979 is determined by multiplying the seven-month average by the interest rate established by the Secretary of the Treasury. The interest rate (assume 8.6 percent) must be prorated for the number of months (7/12) during the year in which billings were made. Thus,

1979 imputed interest = $757 143 x 7/12 x 0.086 = $37 983.

For the 1980 accounting period, assume that the interest rate quoted by the Secretary of the Treasury is 7.75 percent. The imputed interest for the second accounting period can be determined as shown below:

Month	Expenses ($000s)
January	1 495
February	1 750
March	1 920
April	2 045
May	2 125
June	2 165
July	2 200
August	2 225
Total	15 925

Avg = $15 925 000 ÷ 8 = $1 990 625.

Next, the imputed interest for 1979 is added to the end-of-month average balance for 1980. This is used to compute the 1980 imputed interest as follows:

1980 imputed interest = ($1 990 625 + $37 983) x 8/12 x 0.0775 = $104 811.

The total imputed interest for the project as originally scheduled is as follows:

Total original project imputed interest = $37 983 + $104 811 = $142 794.

Similar calculations are made for the disrupted schedule. Since the delay occurred entirely in the second accounting period, only the 1980 imputed interest will be affected. The revised imputed interest for 1980 is as follows:

Month	Expenses ($000s)
January	1 495
February (delay)	1 495 (no additional costs)

Month	Expenses ($000s)
March (delay)	1 495 (no additional costs)
April (delay)	1 495
May	1 750
June	1 920
July	2 045
August	2 125
September	2 165
October	2 200
November	2 225
Total	20 410

Avg = $20 410 000 ÷ 11 = $1 855 454.

1980 imputed interest (with delay) = ($1 855 454 + $37 983) x 11/12 x 0.0775 = $134 513.

Delayed project imputed interest = $37 983 + $134 513 = $172 496.

The imputed-interest damage is the difference between the imputed interest determined for the original and that for the disrupted billing schedules:

Imputed-interest damage = $172 496 - $142 794 = $29 702.

The additional costs associated with the overhead expenses that continued during the delay are not included in the $29 702. These damages must be claimed separately.

CONCLUSIONS

As a result of the research of BCA and court cases related to interest damages, several conclusions can be drawn. These are summarized as follows:

1. Recent case histories in both the federal and the private sectors of the construction industry reveal a distinct trend toward awarding interest damages for the additional capital invested in a project by a contractor.

2. Interest on funds borrowed from an external lending institution have been recovered in certain circumstances where the need for the additional borrowings could be traced to an owner-caused delay. However, in the federal construction contracts, regulations exist that prohibit the recovery of interest on borrowed funds. The legality of these regulations has been challenged and upheld.

3. Imputed interest on funds borrowed from a contractor's in-house contingency fund has been awarded on at least three occasions since 1976. As of December 15, 1980, federal regulations permit the recovery of interest damages on capital invested in construction regardless of the source of the additional required capital. These regulations would appear to establish precedence for recovering imputed interest in the private sector of the construction industry.

REFERENCES

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