A Compendium on Class I, Type C Mixes

Ellis G. Williams
Kentucky Highway Materials Research Laboratory

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TO: D. V. Terrell
   Director of Research

You will recall our attending a meeting of the Specifications Committee almost two years ago, at which time there was a rather lengthy discussion of the density in bituminous concrete paving mixtures and the possibility for altering our Class I specification to provide a surface mix with increased density. This was followed up in the spring of 1951 with some tests and other discussions that culminated in adoption of a specification for a surface mix designated as Type C.

The specification was first applied to a resurfacing project on several Louisville streets in October, 1951. Some difficulties in laying the mix developed at that time, and slight modifications in the grading were made to facilitate placement just for that job. No change was made in the specification. Under hot weather and heavy traffic this past summer, the surface was displaced at several locations on these streets, and naturally there was immediate concern about the dependability of mixes that could be made under this specification.

Earlier the Research Lab and the Testing Lab had worked jointly on laboratory tests with various Type C and Type B mixes. When difficulties arose on the Louisville streets, this work was enlarged somewhat and cores were taken from the streets in an attempt to find the source of trouble. More recently, in conjunction with another Type C surfacing project on U.S. 31W at Muldraugh Hill, the two Labs worked together on designing the mix as well as in sampling at the time of construction. Still further tests were made cooperatively with the Asphalt Institute Laboratory in New York.

These various investigations, supplemented by fundamental analyses, have been recorded in memorandum reports prepared by E. G. Williams at different times during the past year. At my suggestion he has assembled these reports and the information from the Asphalt Institute into the
attached Compendium dealing with Type C mixes. Actually there is a great deal of data pertaining to Type B mixes also, and in addition various combinations of aggregates - crushed and uncrushed - are represented in the mixes.

Essentially, Mr. Williams and the Asphalt Institute show that there is nothing wrong with the Type C from a specification standpoint. It is evident that the Type C is much more sensitive to effects of improper asphalt contents, over tacking between courses, and errors in grading than is the Type B mix. In other words, proper design and control are more critical (but fundamentally of no greater importance) with Type C than with Type B. However, if it is handled carefully Type C offers the properties which were sought when a new mix was first discussed.

Early in December Mike Logan of the Bureau of Public Roads suggested that we take cores from the Muldraugh Hill Project to check densities, particularly since the finished surface on that project had an open-textured appearance. This was done, and the results tabulated on page 31 of the Compendium show a variation from 3.5 to 5.3 percent voids in the compacted mix. The surface texture as illustrated in Fig. III-2, belies such a degree of density. That is accountable in the fact that crushed limestone was used for both fine and coarse aggregate, and the workability of the mix was reduced to the point where a smooth-textured finish was not possible. Undoubtedly, this will be characteristic of Type C containing all-crushed aggregates, and for that reason the tendency will be away from crushed fines in mixes designed for city streets even though they offer the best stability and flow properties to resist heavy traffic.

I am certain that those who have a primary interest in our bituminous concrete mixes will find material of considerable value in this Compendium, and I recommend it as a source of information on Type C mixes.

Respectfully submitted,

L. E. Gregg
Assistant Director of Research

LEG:ddc
Attached
Copies to: Research Committee Members
Mack Galbreath (3)
Commonwealth of Kentucky
Department of Highways

A COMPENDIUM ON CLASS I, TYPE C MIXES

by

Ellis G. Williams
Research Engineer

Highway Materials Research Laboratory
Lexington, Kentucky

December, 1952
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PART I

COMPARATIVE PROPERTIES AND PHYSICAL RELATIONSHIPS
MEMO TO:  L. E. Gregg
Assistant Director of Research

SUBJECT: Comparative Properties and Physical Relationships of Class I, Type C Surface Mixes.

Recently the Class I, Type C Surface Mix has been the subject of numerous discussions resulting from present paving operations on Muldraugh Hill (U.S. 31W and U.S. 60) south of Louisville, and from performance of pavements placed heretofore on some Louisville Streets. Apparently, past difficulties have created some doubt about the suitability of Type C surfaces in heavy traffic areas for which it was intended. Type C does not represent the ultimate in stable mixtures, but it should be entirely adequate for the purpose intended.

Some of the questions which have arisen suggest that restatement of fundamentals involved may clarify most, if not all, of the difficulties encountered. While these fundamentals pertain to paving mixtures in general, the principal consideration here is, of course, Class I, Type C surface. Since Type C is really a modification of the well known Type B surface, the latter is included for comparison.
Fig. I-1. Gradation Specifications for Class I, Type B and Type C Surface Mixtures
mixture lowest. Percentagewise this decrease - based on Mix Z (Class I Type B) - is 2.3 percent for Mix Y (Louisville streets) and 4.0 percent for Mix X (Muldraugh Hill). In this respect, the difference between the Louisville and Muldraugh Hill mixtures results from the ability of crushed fines to produce greater densities due to the angularity of the particles.

<table>
<thead>
<tr>
<th>Mix X</th>
<th>Mix Y</th>
<th>Mix Z</th>
</tr>
</thead>
<tbody>
<tr>
<td>Voids - 3.5%</td>
<td>Voids - 4.4%</td>
<td>Voids - 4.0%</td>
</tr>
<tr>
<td>Asphalt - 13.2%</td>
<td>Asphalt - 12.6%</td>
<td>Asphalt - 13.4%</td>
</tr>
<tr>
<td>1/2&quot; to 3/8&quot; - 6.3%</td>
<td>1/2&quot; to 3/8&quot; - 6.7%</td>
<td>1/2&quot; to 3/8&quot; - 4.1%</td>
</tr>
<tr>
<td>3/8&quot; to No. 4 - 27.1%</td>
<td>3/8&quot; to No. 4 - 27.0%</td>
<td>3/8&quot; to No. 4 - 31.0%</td>
</tr>
<tr>
<td>4 to 8 - 14.6%</td>
<td>4 to 8 - 14.5%</td>
<td>4 to 8 - 12.3%</td>
</tr>
<tr>
<td>8 to 16 - 10.4%</td>
<td>8 to 16 - 10.2%</td>
<td>8 to 16 - 10.3%</td>
</tr>
<tr>
<td>16 to 50 - 13.2%</td>
<td>16 to 50 - 13.4%</td>
<td>16 to 50 - 15.7%</td>
</tr>
<tr>
<td>50 to 100 - 4.6%</td>
<td>50 to 100 - 4.5%</td>
<td>50 to 100 - 5.0%</td>
</tr>
<tr>
<td>100 to 200 - 2.9%</td>
<td>100 to 200 - 3.0%</td>
<td>100 to 200 - 2.1%</td>
</tr>
<tr>
<td>-200 - 4.2%</td>
<td>-200 - 4.2%</td>
<td>-200 - 2.1%</td>
</tr>
</tbody>
</table>

Fig. I-2. Block diagrams illustrating the volume relationships of voids and aggregate fractions in the three Class I mixes.
Sketch II-1 illustrates the load bearing structure of the mix from which most of the stability of any mixture is derived. In this case, as the graph shows, the aggregate voids constitute 16.7 percent of the total volume. This is the aggregate condition at optimum asphalt content in the Muldraugh Hill mix. The aggregate size fractions are uniformly distributed, with each succeeding size fraction effectively filling voids in the coarser fraction. In this way a dense structure with good load-supporting characteristics is attained. Such a structure may be consolidated further under traffic, but consolidation in this way is minimized.
L. E. Gregg

December 8, 1952

Sketch II-5 illustrates quite clearly the action of the trapped volatiles. Near the bottom of the surface course the intrusions are very similar to those that prevailed in Condition IV (See Sketch II-4). The jagged fingers which penetrate into the mix and in some cases to the surface, are caused by upward flow of the volatiles in the form of vapor.

The condition along the bottom of the surface course has resulted in complete filling of the voids and consequent plasticity already discussed under the heading Excessive Tacking. However, the condition here is potentially worse because of the reduction in viscosity of asphalt cement in the mix brought about by combination with the diluant. In extreme cases this may produce viscous liquid asphalts within the mix, as has been observed in cores cut from Seventh Street in Louisville.

The upward migration of vapor apparently proceeds along interconnected voids. If this migration is blocked, the concentrated vapor softens the surrounding asphalt. Conceivably, continued of this entrapment will ultimately lead to opening of new channels through solution if nothing else. In time the vapor will escape, but observations indicate that this is a long time.

Charts on the left in Sketch II-5 show the original test properties of the mix; however, the change in properties brought about by filling of the voids are undoubtedly greater than implied by the graphs. The indicated changes are those to be expected only if the voids are filled
with an asphalt cement, and there is no added influence on the part of the dispersed volatiles. If, the volatiles become a factor – as they surely will – those voids are at least partly filled with a liquid asphalt, and the damage becomes greater. Unfortunately, we have no data or observations to establish the numerical value of this latter condition or to illustrate it in a relative way on charts or graphs.

Ellis G. Williams
Research Engineer
PART III

MULDRAUGH HILL PROJECT

Design and Cooperative Tests With The
Asphalt Institute
### Kentucky Department of Highways

#### Type C, Surface Course Composition

<table>
<thead>
<tr>
<th>Passing Sieve</th>
<th>Per Cent</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2 Inch.</td>
<td>100</td>
</tr>
<tr>
<td>3/8 Inch.</td>
<td>85-100</td>
</tr>
<tr>
<td>No. 4</td>
<td>50-70</td>
</tr>
<tr>
<td>No. 8</td>
<td>35-50</td>
</tr>
<tr>
<td>No. 16</td>
<td>20-40</td>
</tr>
<tr>
<td>No. 50</td>
<td>8-20</td>
</tr>
<tr>
<td>No. 100</td>
<td>5-12</td>
</tr>
<tr>
<td>No. 200</td>
<td>3-7</td>
</tr>
<tr>
<td>Bitumen</td>
<td>5.5 to 8.5</td>
</tr>
</tbody>
</table>

Adopted July 31, 1951
Mr. L. E. Gregg  
Assistant Director of Research  
Materials Research Laboratory  
Kentucky Department of Highways  
132 Graham Avenue  
Lexington 29, Kentucky

Dear Mr. Gregg:

We have completed a laboratory analysis and paving mix design for aggregates and asphalt recently sent to us. In your recent letter you note that these materials were obtained from your Muldraugh Hill project on U. S. 31 W.

Figure 1 shows the aggregate gradation limits for your Type "C" surface mix as given in your letter. It also shows the gradation of three trial mixes which we tested in our laboratory.

Our first trial used a 50-50 blend of the coarse and fine aggregates as received. The resultant gradation is shown in red on Figure 1. This gradation is about in the middle of your allowable limits on the No. 8 sieve, but runs out slightly on the high side with the No. 200 sieve. Single Marshall specimens were made with this gradation at 5, 6 and 7 percent of asphalt. Incidentally, asphalt content in this procedure is on the included or "wet weight" basis. Test results on these single specimens are shown in red on Figure 2. You will note that the flow value of these specimens is quite high and we considered that the mix might be improved by reducing the rather high amount of - 200 material.

The next gradation studied was the same 50-50 blend except that about half of the - 200 material was removed. Single specimens were again prepared at 5, 6 and 7 percent asphalt and tested. The aggregate gradation of this mix is shown in blue on Figure 1 and test properties are shown in blue on Figure 2. This mix indicated a slightly higher optimum asphalt requirement and the flow values were fairly near the upper limit of 20 as required by the Corps of Engineers.
A third gradation was then tried as shown in green on Figure 1. This gradation contained about 43% of coarse aggregate as received and 57% of the fine aggregate sample. Again, about half of the -200 material was removed. Test properties of single samples of this mix prepared at 5, 6 and 7 percent asphalt, are shown in green on Figure 2. This mix looked quite satisfactory on the basis of single samples so we then prepared duplicate specimens at four asphalt contents of the gradation shown in green on Figure 1. Asphalt contents for the specimens were 5.5, 6.0, 6.5 and 7.0. Results of tests on these specimens are shown on Figure 3. The optimum asphalt content for this mix is indicated to be about 6.5 percent by Corps of Engineers criteria. At this asphalt content the flow is about 16, stability is slightly over 2000 and void relationships are good.

We then checked this mix design by preparing Hveem Stabilometer specimens at 6.0 and 6.5 percent asphalt. Duplicate specimens at each asphalt content were prepared and gave the following test values:

<table>
<thead>
<tr>
<th></th>
<th>6.0% A. C. Spec. #1</th>
<th>Spec. #2</th>
<th>6.5% A. C. Spec. #1</th>
<th>Spec. #2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hveem Stability</td>
<td>48</td>
<td>44</td>
<td>27</td>
<td>46</td>
</tr>
<tr>
<td>Cohesiometer Value</td>
<td>468</td>
<td>370</td>
<td>569</td>
<td>572</td>
</tr>
<tr>
<td>Rt</td>
<td>94</td>
<td>94</td>
<td>88</td>
<td>97</td>
</tr>
</tbody>
</table>

Except for specimen No. 1 at 6.5% asphalt, all specimens had satisfactory test values.

I would therefore consider that the aggregate gradation shown in green on Figure No. 1 would make a satisfactory paving mix. Some of the -200 material, however, should be wasted. The mix is not particularly critical to asphalt content as is sometimes the case with crushed aggregate mixes. On roads of moderate traffic volume, I would recommend the full 6.5 percent indicated to be the optimum by the Marshall procedure. For heavy traffic conditions it might be advisable to cut the asphalt content back to about 6.2 percent. The Hveem specimens, prepared by kneading compaction, had a slightly flushed appearance at 6.5 percent and one of the two stabilometer specimens indicated that this percentage might be just a little too much. In deference to this one relatively low value and the slightly flushed appearance, I feel that about 6.2 percent might be advisable for heavy traffic conditions.

Concluding, I would recommend that you might give some consideration to the use of a sand for the -10 fraction if such is economically available. The use of two different materials in a mix will often provide better textured and non-skid qualities over the long run, perhaps due to differential rates of wearing. I am sure that an entirely satisfactory mix could be designed with sand or with a mixture of sand and stone fines if such a material is economically available. Also
you might eliminate the problem of wasting a part of the fines in the present aggregates.

I hope that these data will be of some benefit to you in your pavement construction.

With best personal regards, I am

Sincerely yours,

[Signature]
John M. Griffith
Engineer of Research
DESIGN AND CONTROL OF ASPHALT PAVING MIXTURES

FLOW - 1/100 INCHES

STABILITY - LBS

UNIT WEIGHT - LBS PER CU FT

TOTAL MIX

PERCENT TOTAL VOIDS FILLED WITH ASPHALT

PERCENT SOLIDS - TOTAL MIX

PERCENT VOIDS - TOTAL MIX
MEMO TO: L. E. Gregg  
Assistant Director of Research  

SUBJECT: Mixture Designs For Muldraugh Hill Surface Course by Research Laboratory and Asphalt Institute.  

The design test information pertaining to the Muldraugh Hill surface and prepared by the Asphalt Institute Laboratory has been received and reviewed thoroughly. Materials used in the design tests were furnished by this laboratory and came from sources supplying the project. Previously, the Research Laboratory had prepared a mix design for the project, and paving proceeded on the basis of this design.  

Research Laboratory Design Tests  

Aggregate used in the Research Laboratory design was crushed limestone for both the coarse (No. 9 stone) and fine (agricultural lime) fractions. This too came from the source which later supplied the project, but it was sampled several weeks earlier than the material sent to the Asphalt Institute Laboratory. The asphalt was PAC-5 for both sets of tests.  

Aggregate grading selected for the design approximates the center of the gradation specification for Class I, Type C surfaces and represents
a 50-50 blend of coarse and fine aggregates. Gradation of both the basic aggregates and the design mixes are contained in Table III-1.

The Marshall Stability Test was used for determining the asphalt content. Four test specimens prepared with the so-called "heavy" compaction, were made for each of four asphalt contents - 4.0, 5.0, 6.0 and 7.0 percent. Average values determined from the four specimens in each group are plotted as curves on the Marshall Stability charts in Fig. III-1. It should be noted that points which determined the position of these curves (design values) are not plotted on Fig. III-1, and the points that are plotted represent values established later when samples from the project were tested.

Optimum asphalt content was selected from these charts, and the test properties at optimum are contained in Table III-2.

Comparison of Data

Both laboratories used the same basic method - the Marshall Stability Test - but the Asphalt Institute Laboratory supplemented their work with Hveem Stabilometer Tests as a check on their Marshall design. The bitumen content recommended by us and used in the project was 5.5 percent, while that recommended by the Asphalt Institute was 6.2 percent.

This discrepancy is wider than expected for identical mixes prepared in two different laboratories. Actually, the two mixes were not identical in grading, partly because they were prepared entirely independent of each other - the Asphalt Institute having no information on our earlier design at the time they did their work. Differences in gradation...
### Table III-1. Gradation Data

<table>
<thead>
<tr>
<th>Identity</th>
<th>% Asphalt</th>
<th>% Passing</th>
<th>1/2&quot;</th>
<th>3/8&quot;</th>
<th>No. 4</th>
<th>No. 8</th>
<th>No. 16</th>
<th>No. 20</th>
<th>No. 50</th>
<th>No. 100</th>
<th>No. 200</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse Agg. (1)</td>
<td>100.0</td>
<td>81.0</td>
<td>76.0</td>
<td>60.5</td>
<td>42.3</td>
<td>32.3</td>
<td>23.3</td>
<td>15.5</td>
<td>10.0</td>
<td>5.0</td>
<td>0.5</td>
</tr>
<tr>
<td>Coarse Agg. (2)</td>
<td>100.0</td>
<td>89.0</td>
<td>84.0</td>
<td>79.0</td>
<td>62.0</td>
<td>50.0</td>
<td>30.0</td>
<td>20.0</td>
<td>10.0</td>
<td>5.0</td>
<td>0.5</td>
</tr>
<tr>
<td>Fine Agg. (1)</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>86.3</td>
<td>63.8</td>
<td>40.6</td>
<td>20.4</td>
<td>7.5</td>
<td>3.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Fine Agg. (2)</td>
<td>100.0</td>
<td>92.0</td>
<td>82.0</td>
<td>66.0</td>
<td>50.0</td>
<td>38.0</td>
<td>18.0</td>
<td>4.5</td>
<td>2.5</td>
<td>1.5</td>
<td>0.5</td>
</tr>
<tr>
<td>A.I. Trial No. 1</td>
<td>100.0</td>
<td>92.0</td>
<td>72.0</td>
<td>52.0</td>
<td>23.2</td>
<td>13.5</td>
<td>10.0</td>
<td>5.0</td>
<td>2.5</td>
<td>1.5</td>
<td>0.5</td>
</tr>
<tr>
<td>A.I. Trial No. 2</td>
<td>100.0</td>
<td>92.0</td>
<td>72.0</td>
<td>52.0</td>
<td>23.2</td>
<td>13.5</td>
<td>10.0</td>
<td>5.0</td>
<td>2.5</td>
<td>1.5</td>
<td>0.5</td>
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<tr>
<td>A.I. Trial No. 3</td>
<td>100.0</td>
<td>92.0</td>
<td>72.0</td>
<td>52.0</td>
<td>23.2</td>
<td>13.5</td>
<td>10.0</td>
<td>5.0</td>
<td>2.5</td>
<td>1.5</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Notes: Aggregate (1) designates aggregate used by Research Laboratory while aggregate (2) is that used by the Asphalt Institute Laboratory. Trial No. 3 grading was used for design of asphalt content by the A.I. Laboratory.

### Table III-2. Marshall Stability Test Data

<table>
<thead>
<tr>
<th>Identity</th>
<th>% Asphalt</th>
<th>Stability Number</th>
<th>Flow 1/100&quot;</th>
<th>Unit Wt. lbs. cu.ft.</th>
<th>Compressed Mix</th>
<th>Filled W/Asphalt Only</th>
</tr>
</thead>
<tbody>
<tr>
<td>Research Lab. Design</td>
<td>5.5</td>
<td>1690</td>
<td>17.5</td>
<td>120.2</td>
<td>4.2</td>
<td>78.0</td>
</tr>
<tr>
<td>Asphalt Institute (a)</td>
<td>6.5</td>
<td>2110</td>
<td>16.5</td>
<td>147.5</td>
<td>4.6</td>
<td>76.0</td>
</tr>
<tr>
<td>Design (b)</td>
<td>6.2</td>
<td>2300</td>
<td>16.5</td>
<td>147.5</td>
<td>4.6</td>
<td>76.0</td>
</tr>
<tr>
<td>Field Sample A</td>
<td>5.5</td>
<td>1570</td>
<td>17.0</td>
<td>127.2</td>
<td>5.2</td>
<td>71.5</td>
</tr>
<tr>
<td>Field Sample 1</td>
<td>5.7</td>
<td>1080</td>
<td>17.0</td>
<td>127.2</td>
<td>5.2</td>
<td>71.5</td>
</tr>
<tr>
<td>Field Sample 2</td>
<td>5.5</td>
<td>1450</td>
<td>16.5</td>
<td>127.2</td>
<td>5.2</td>
<td>71.5</td>
</tr>
<tr>
<td>Field Sample 3</td>
<td>4.3</td>
<td>1374</td>
<td>12.5</td>
<td>127.2</td>
<td>5.2</td>
<td>71.5</td>
</tr>
<tr>
<td>Core No. 1</td>
<td>5.5a</td>
<td>1390</td>
<td>20.0</td>
<td>146.6</td>
<td>5.2</td>
<td>67.8</td>
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<tr>
<td>Core No. 2</td>
<td>5.3</td>
<td>1315</td>
<td>16.0</td>
<td>128.5</td>
<td>4.8</td>
<td>72.4</td>
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<td>Core No. 4</td>
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<td>126.0</td>
<td>5.3</td>
<td>72.9</td>
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<tr>
<td>Core No. 3</td>
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<td>1295</td>
<td>12.0</td>
<td>127.4</td>
<td>5.4</td>
<td>72.6</td>
</tr>
</tbody>
</table>

*Bitumen Content Assumed.

Notes: Asphalt Institute Design "a" is for moderate traffic and "b" for heavy traffic. Field samples were compacted on the job using the actual paving mix at the time of construction. Cores were cut when the pavement was approximately six weeks old. All specimens prepared with "heavy" compaction - 50 blows of 1 lb. hammer on both sides of the specimen.
Highway Materials Research Laboratory
Lexington, Kentucky

MARSHALL STABILITY TEST

Muldraugh Hill Surface Mix
- Samples prepared from field mix.
- Samples cored from the completed pavement.

TEST DATA

Optimum % Bitumen - 5.5
Unit Wt. - 150.2
Stability No. - 1690
Flow - 17.8
% Voids, Compacted Mix - 3.5
% Voids, Filled W/Bitumen

Fig.-III-1. Results of Marshall Stability Design Tests
For Type C Surface on Muldraugh Hill
Fig. III-2. Typical texture of Class I, Type C surface placed on Muldraugh Hill west of Louisville (U.S. 60 and U.S. 31W).
Coarse-textured surfaces, when accompanied by sufficient density and other desirable characteristics, possess good anti-skid qualities which are retained as the surface aggregate is worn down by traffic.

Class I, Type C surfaces made entirely with crushed aggregates can not have a fine surface texture at the correct asphalt content. Mixtures having natural sand or rounded material as the fine aggregate fraction have a somewhat finer surface texture than those containing only crushed aggregate. However, in either case the surface must be classified as relatively coarse.

Ellis G. Williams
Research Engineer
Mr. John M. Griffith  
Engineer of Research  
The Asphalt Institute  
801 Second Avenue  
New York 17, New York  

Dear Mr. Griffith:

I am writing with regard to your letter of November 21, and the data concerning designs of Type C surface mix which you sent with it. For the information of those who will receive copies of this letter, the materials used in your analyses came from our Muldraugh Hill project on U.S. 31W, and they were sent in accordance with arrangements made in my letter of October 30, to Mr. Walter F. Winters.

We have gone over your data very carefully, and are particularly pleased with the fact that you were able to run Hveem Stabilometer tests in addition to the tests by the Marshall method. All of our original information was obtained with Marshall test, and the job was carried out on the basis of that information.

Plots of our design data, along with a memorandum and additional results pertaining to a few samples compacted on the job are enclosed.  
Because of discrepancies in our design gradation (with 50-50 composition of coarse and fine aggregates) as opposed to your ultimate design with 43 percent coarse and 57 percent fine aggregate, there is no direct basis for comparison. Beyond this, the actual gradations on the job - as described in the attached memorandum - were of still greater difference particularly in the very fine fractions.

I don't know how to account for the range of our stability numbers which are considerably lower than yours despite the greater unit weights and obviously greater density in our case. Certainly the difference is greater than any amount that could be attributed to the differences in gradation. The density, as reflected in percentage voids in the compacted mix, was given primary consideration in setting a 5.5 percent asphalt content on the job. This optimum value was well confirmed, also, by the other indicated properties.
Results from samples compacted on the job conformed rather well with the design test results, all things considered and the one cold sample (No. 4) disregarded. This, of course, could mean merely that our field compaction closely approximated laboratory conditions, and not shed any particular light on the differences in our data.

If, after you have gone over our information, you arrive at some conclusions about the differences I would welcome your comments. Our object, of course, is to work out fundamentals pertaining to the Type C mix so it will be usable to us. I might say that the comments you made concerning natural sand are quite pertinent because this mix is proposed mostly for use in the vicinity of Louisville at present, and that is where we have our greatest variety of natural sands. Actually for all hot mixes in total, more sand than crushed aggregate is used for fines in this state.

I could not tell from your letter whether copies had been sent to Messrs. Neiser, Bitterman, and Goshorn as requested in my letter to Mr. Winters. In view of the plotted data it may be that you were unable to do so. If that is the case, do you have any objection to making copies of your letter and graphs, and sending them to these three men? Also, if you have further comments on the data we are discussing, I would like to have copies sent directly to them so that they will be informed on all phases of the matter.

Very truly yours,

L. E. Gregg
Assistant Director of Research

LEG: mh

cc: J. O. Neiser, Assistant State Highway Engineer
J. A. Bitterman, Director, Division of Materials
John Goshorn, Asphalt Institute

Enc.
Mr. L. E. Gregg
Assistant Director of Research
Materials Research Laboratory
132 Graham Avenue
Lexington 24, Kentucky

Dear Mr. Gregg:

This letter is in further reply to yours of November 26 concerning the paving mix for the Muldraugh Hill project on U.S. 31W. Our meetings have now passed and we are just beginning to get caught up on some of our deferred correspondence.

As you point out, the variations in gradations between our design mix and yours does not allow us to make a direct comparison of the data. Perhaps the closest comparison that we could make would be between our second trial mix and your design gradation. In this case, however, we have only single specimens and at 1.0 per cent increments of asphalt and I would not wish to count too strongly on them for precise accuracy. The field specimens correspond more nearly with our first trial gradation with the high percentage of fines but this curve also is based on single samples at three asphalt contents. I would suggest that you give consideration to running a set of check data, duplicating our recommended gradation and the asphalt contents at which we tested. In this manner, we would have a direct basis for comparison which would furnish a check on accuracy of the procedure and reproducibility of test results.

I would not be too concerned about stability numbers, per se, insofar as the Marshall test is concerned. Personally, I view the Corps of Engineers procedure as one which attempts to provide a proper balance between several factors which should be considered in establishing proper mix proportions. Stability is only one of these factors and its principal value is to give an indication of optimum asphalt only from the stability standpoint. These procedures also judge optimum from the standpoint of maximum unit weight, voids in the mix and per cent of voids filled with asphalt. Averaging the optimum from the standpoint of these four factors gives a balanced optimum asphalt content which might have been different had any one of them been used alone. The actual Marshall stability number has very little significance to me and I believe that you will also find Charlie Foster in full agreement with this thought. On the test track at Vicksburg we had mixes which started out at about 150 lbs Marshall stability and when the asphalt content was right they
stood our heaviest traffic wheel loads. In other words, we were unable to find mixes which fail from lack of Marshall stability only; failing mixes invariably failed to meet the various other criteria established in the Corps procedures. These criteria were set on the bases of traffic test results with the limits so adjusted as to weed out unsatisfactory mixes. The minimum stability number of 500 was included more or less with a tongue in cheek attitude because we knew that a minimum stability limit would be expected. The principal value of the stability data, however, is to furnish one indication of optimum asphalt. I have long tried to de-emphasize the importance of this term insofar as Corps of Engineers procedures and the Marshall test are concerned. It's difficult though because the word "stability" seems to have a universal appeal to paving engineers. It is important to see to it that void relationships are within the proper range and the flow value is a good indicator of the plasticity characteristics of the mix. It is most important to establish the proper or optimum asphalt content. If all of the other criteria of the Corps are met, it would be difficult if not impossible to find a mix which would not meet stability requirements. Your design stability value is about 1700 lbs whereas ours with a slightly different gradation on the second trial mix is about 2100. I would not be particularly concerned about this variation. I am, however, a little concerned about the density difference since this affects three of the four factors by which optimum is determined.

This difference may be due to differences in gradation or in compaction techniques. It is for this reason that I have suggested that you run a set of tests duplicating our aggregate gradation and asphalt contents. If our results vary on truly comparable specimens then we should try to determine why the variation occurs. While at Vicksburg we took every opportunity to check reproducibility between different operators and found the procedures fairly reliable from that standpoint. I therefore feel that it would be quite worthwhile if you would run this check series and let's just see if we get different results on comparable conditions.

I plan on attending the Highway Research Board meeting and hope that you will also be there so that we can have a further discussion of this matter.

Very truly yours,

John M. Griffith
Engineer of Research
PART IV

INVESTIGATION OF FAILURES ON LOUISVILLE STREETS
MEMO TO: L. E. Gregg  
Assistant Director of Research  

SUBJECT: Paving Mixtures on Louisville City Streets - Class I, Type C and Laboratory Mixes Prepared Within the City-of-Louisville Specification for Asphaltic Concrete.

This memo includes the information pertaining to cores cut from Sixth and Seventh Streets in Louisville, reported in memo of July 21, and additional information pertaining both to Class I, Type C and the Louisville City Mix.

Three of the cores have been tested to determine gradation and asphalt content. The Type C course is not thick enough to permit determination of stability.

The three cores tested were No. 1 containing 7.8 percent asphalt, No. 5 containing 7.0 percent asphalt, and No. 9 containing 7.8 percent asphalt. It will be noted in the following discussion of core locations that No. 5 core is in an area showing no distress, while No. 1 and No. 9 are from areas where shoving is prevalent. Specimens tested were separated from the entire core at the joint between surface and underlying courses. It is entirely probable that a portion of the tacking material has penetrated the specimens, thus contributing to the very high asphalt contents.
Gradation of the aggregate in Cores No. 5 and No. 9 is within the Type C specification, and the aggregate in Core No. 1 exceeds the specification only a small amount. The average aggregate gradation for the project, as shown on the plant inspector's reports, varied slightly from the specification, but not to an extent considered detrimental. Fig. IV-18 shows these gradations and the specification limits.

The cores mentioned were cut July 18, 1952. Twelve cores were located on Seventh Street and two on Sixth Street. Specimens were obtained from areas on the verge of total failure, areas showing some distress but unfailed, and areas in excellent condition.

Inspection alone revealed a great deal of information which is highly pertinent to the condition of the pavement. Photos are included to show general location of each core on the street, and also to show the exact appearance of each core. Five of these photos, Fig. IV-8 thru IV-12, show top, bottom and three side views of each core. In addition, there are five close-up views, (Fig. IV-13 to IV-17) to illustrate particular conditions which are contributing to the stable or failing condition, as the case may be.

Core Locations and Pavement Conditions

Core No. 1 (Seventh Street, 200 feet north of Broadway) For this case, the location of this core is shown in Fig. IV-1 and the core in Fig. IV-8. The street is four lanes in width and the core was cut adjacent to the joint between the two eastern lanes. Shoving is prevalent.
only at and adjacent to the indicated joint, the remaining width of the pavement being satisfactory. Displacement is not serious at present, but is developing.

This section is underlain by approximately 1/2 inch of sheet asphalt which in turn is underlain by what is probably a binder course. This course, which is approximately 1 inch thick with a maximum aggregate size of 1 inch, is so loosely bound it could not be cored. The base is either paving stone or brick.

The new Type C surface course is 1 1/2 inch thick and is very rich in appearance. While some of this richness may be attributed to an original asphalt content that is excessively high, the principal cause of failure is without a doubt excessive quantities of RC-2 tack coat which could be identified by odor. Soft bitumen is present in the mix to within 1/8 inch of the top and to the extent that excess bitumen actually seeped from the core before it was transported to the Laboratory.

The method of application of the tack coat would encourage the use of excessive quantities of material and consequent showing failure regardless of the bitumen used. The tack was sprayed from a distributor bar in the manner usually referred to as "stringing". Application by this method generally encourages use of excess quantities of bitumen and the concentration in longitudinal trips of double applications. This latter is caused by overlapping in tacking adjacent lanes. Overlapping applications which possibly occurred in this case may well be the principal cause of failure.
Core No. 2 (Seventh Street, 210 feet north of Broadway) Core location No. 2 is shown in Fig. IV-1, and various views of the core are presented in Fig. IV-8. Also, the core is shown close up in Fig. IV-13 to illustrate a point in the types of failure observed. This is the companion to Core No. 1, and both samples represent pavement which has shoved along one joint between adjacent lanes. The condition was slightly less pronounced at Core No. 2 than at Core No. 1; likewise, intrusion of the tack into the Type L appeared less pronounced.

Approximately half the thickness of the old binder was cored with the remainder disintegrating during the coring operation.

Core No. 3 (Seventh Street, 150 feet south of Chestnut) The sample location and varied views of the core are shown in Fig. IV-2 and Fig. IV-8 respectively. Pavement at this point was placed in four lanes and the core was taken adjacent to the joint between the two eastern lanes. There is no vertical displacement in this area, but there is slight lateral displacement. The mat is laid on a stone or brick base and consists of $2\frac{1}{2}$ inches of binder course that looks good, $\frac{1}{2}$ inch of sheet asphalt which looks dry and shows cracking, and $1-5/8$ inch of Type C surface.

The binder and sheet asphalt courses are stable and probably brittle. The Type C appears to be slightly high in asphalt content, and here again the tack coat is very prominent. Indications are that the tack is providing a lubricated surface, and the fact that deformation is lateral merely indicates that the Type C mix has reasonably high stability in spite of the high asphalt content. No bleeding is evident on the surface; however, the escaping volatile material has softened the asphalt almost to the surface.
In this case, the principal difficulty lies in overtacking and volatile material in the bitumen used for tacking. Possibly the high asphalt content is a contributing factor, but this is doubtful since the stability of the mix has been sufficient to prevent shoving longitudinally.

Core No. 4 (Seventh Street, 135 feet south of Chestnut) This location is shown in Fig. IV-2 and the core in Fig. IV-9. A close-up of the core illustrating the excess of tacking material is contained in Fig. IV-14.

Inasmuch as this core is a companion to Core No. 3 and both represent the same pavement conditions, comments concerning Core No. 3 apply in this case.

Core No. 5 (Seventh Street, 300 feet north of Chestnut) The location and core are illustrated in Fig. IV-3 and Fig. IV-9 respectively. There is no distress of any sort in the pavement represented by this core and its companion Core No. 6. Surface texture is very similar to that observed in all other locations cored on Seventh Street. The mat consists of a stone or brick base, 2 inches of very dry-looking binder, ½ inch of sheet asphalt, and 1½ inch of Type C surface. The core was cracked transversely between the binder and sheet asphalt courses. Conditions in the core hole were not checked, so it is impossible to say whether this cracked condition existed in the pavement or was caused by coring. The former is more likely since there is no indication that the core "twisted off". Also, the brittle appearance of the mixtures indicate probability for such a separation of courses.
The tack coat is hardly discernable in this core. Also, as indicated by the data in Fig. IV-18, the measured asphalt content was considerably lower (7.0 percent) at two other locations where failure occurred. Finally, the curves in Fig. IV-18 show that gradation of aggregate in Core No. 5 was better than in the other two cores. Undoubtedly, these three things in combination account for the difference between failure elsewhere and satisfactory performance at this point.

Core No. 6 (Seventh Street, 310 feet north of Chestnut) The location of Core No. 6 is shown in Fig. IV-3, and the core is shown in Fig. IV-9. A close-up view illustrating material details is contained in Fig. IV-16. The sample is a companion to Core No. 5. No distress of any sort was noted at this location. The mat is made up of a stone or brick base, 1 inch of binder (which appears to be very high in fines, and contains a coarse aggregate with a maximum size of 3/4 inch), 1/8 inch sheet asphalt, 1 inch of binder similar to that below, 1/8 inch of sheet asphalt and 1-1/8 inch of Type C surface - which is the fifth course of bituminous mix.

The lower courses of this core appear to be dry and brittle. This is sustained by the presence of cracks in the two lower courses. The Type C course appears to be slightly rich, and there are slight indications of overtacking. Essentially the top course is well bound to underlying material.

Core No. 7 (Seventh Street, 200 feet north of Walnut) Surface conditions and varied views of the core from this location are shown in Fig. IV-4 and Fig. IV-10 respectively. A close-up of the core is contained
in Fig. IV-15. This core is located in an area having very pronounced though intermittent lateral displacement and some vertical displacement. The mat is made up of the usual base, 1\(\frac{1}{2}\) inch of Type A binder and 1 inch of Type C surface.

The Type C looks rich, and there is abundant evidence that it has been penetrated by the tack coat for more than half its thickness. The binder is a very loosely bound material having few fines and a low asphalt content. Obviously, it has low stability value.

The cause of distress here appears to be principally over tacking; however, the failure is probably aggravated by the composition of the underlying binder course material.

Core No. 8 (Seventh Street, 210 feet north of Walnut) The location of this core is shown in Fig. IV-4 and varied views of the core are presented in Fig. IV-10. This is a companion to Core No. 7, where surface conditions are the same. However, in this case the accumulated bituminous pavement is thicker. The mat here is made up of the usual base, approximately 1\(\frac{1}{2}\) inch of binder, 1 inch of sheet asphalt, 1\(\frac{1}{2}\) inch of very poor Type A binder (containing soft and disintegrated limestone particles and having a very irregular depth) and 1 inch of Type C surface.

The tack coat is very prominent, having penetrated almost the full thickness of binder and in places approximately half the surface course. Soft shale-like limestone in the binder course has tendencies to break down into a stiff clay when wet, and approximately 10 percent of the aggregate in that course appears to be of this material. The material is virtually
unbound, the bitumen having a brownish dead appearance and adhering poorly
to the aggregate. Undoubtedly, the binder course is the primary cause of
failure in this case.

Core No. 9 (Seventh Street, approximately midway between Cedar
and Liberty). The location from which Core No. 9 was taken is shown in
Fig. IV-5, and the core is shown in Fig. IV-10. This, by far, is the worst
section and represents virtually total failure. Shoving is very extensive
and amounts to several inches displacement both horizontally and vertically.
The mat consists of the usual base, approximately 4 inches of bituminous
material, and 1 inch of Type C surface.

The material between base and surface courses was impossible to
classify. The core twisted off immediately beneath the surface course.
Coring was continued, but the underlying material disintegrated. Samples
of this material were obtained. At least 75 percent (bases on visual es-
timate), of the underlying aggregate consists of a shale-like stone which
breaks down in the presence of water. This bituminous mix or combination
of mixes has virtually no stability and forms a lubricating course on
which the surface may readily move under load.

The only worthwhile corrective measure in this area is to remove
and replace all material down to a stable course which seems to be the base
course.

Core No. 10 (Seventh Street, approximately midway between Cedar
and Liberty). The core location and varied views of the core are contained
in Fig. IV-5 and Fig IV-11 respectively. This is a companion to Core No. 9,
and it was taken from the top of a ridge formed by severe shoving or rutting of the pavement. Vertical displacement is approximately 6 inches, and cracks occur on either side of this "hump". The mat consists of the usual base, 2\frac{1}{2} inches of very poor binder, and 1\frac{3}{4} inch of Type C surface.

The binder is approximately the same as that encountered in Core No. 9. In this case, however, the majority of the binder course remained intact and was extracted with the core. Fig. IV-17 is a close-up view of the core, illustrating conditions as follows:

The course is very weakly bound and the aggregate consists of approximately 50 percent shaley aggregate. This material, in the presence of water, has broken down and formed a lubricating course between base and surface courses. Disintegration appears to be most prominent along the base-binder contact surface. In any case, the entire mat is moving along the base since the binder lacks ability to resist any loading.

The tack coat here is excessively heavy and would, over a stable binder, provide a slippage plane and ultimate failure. However, in this case, it is at most a minor factor contributing to failure.

The only corrective measure with any chance of success is removal and replacement of the faulty binder.

Core No. 11 (Seventh Street, just south of Liberty). Location of the core is shown in Fig. IV-6, and varied views of the core are contained in Fig. IV-11. This core is located in a stable area in the east lanes. The two west lanes are unstable and in a condition very similar to that described for core location No. 9 and No. 10. The mat consists of the usual base, at least 1\frac{1}{2} inch of binder, and 1\frac{1}{4} inch of Type C surface.
The binder aggregate contains a great deal of soft and shaly particles. In this case the water has apparently not penetrated to the soft particles since they seem to be essentially intact. This is probably not the case in the adjacent west lanes since failure there is well developed. The tack coat is discernable but not exceptionally heavy, and it does not form a slippage plane at this location.

The probable reasons for failure in the west lane in this location are either slippage caused by over-tacking, or entry of moisture into the binder and consequent disintegration of the binder aggregate. This, of course, is conjectural, since we have no specimen taken from the west lane and have no assurance that the pavement in the west lane is similar to that on the opposite side of the street where cores were taken.

**Core No. 12** (Seventh Street, just south of Liberty). For views of this location and the core see Fig. IV-6 and Fig. IV-11 respectively. This is the companion to Core No. 11 and represents the same combination of courses but in different thickness as follows: Binder - 1 inch, Type C Surface - 1 1/2 inch. Remarks pertaining to surface condition under Core No. 11 apply here also.

**Core No. 13** (Sixth Street, 150 feet south of Liberty) Location of this core is shown in Fig. IV-7, and the core is illustrated in Fig. IV-12. In the east lane where the core was taken, the mat consists of a stone or brick base, 1 1/2 inch o. very poor binder, and 1 1/2 inch of Type C surface. This surface on Sixth Street appears to contain a higher percentage of course aggregate and has a coarser surface texture than the surface at all
locations studied on Seventh Street. Pitting of the surface resulting from disintegration of soft particles is the only type deterioration observed.

The tack coat in this case is not detrimentally heavy; however, there is evidence of some penetration of tack into both binder and surface course thus indicating a definite surplus of tacking material.

Core No. 14 (Sixth Street, 170 feet south of Liberty) This location is shown in Fig. IV-7 and the core is illustrated in Fig. IV-12. The entire discussion of Core No. 13 is applicable to this case also.

Summary The cause of failure in these pavements may be fixed with reasonable accuracy by visual inspection. The Type C surface within itself is not the cause of any failure observed. High asphalt contents have contributed materially to failure in some cases; yet, the principal causes of failure are excessive tacking or limited distribution of the tack, and unstable courses beneath the surface - usually faulty Type A binders.

Tacking was done with an RC-2 by the stringing method. This coupled with low air and pavement temperature during the period of construction, promoted the use of excess quantities of RC-2. Under those conditions volatiles were trapped within the mat. With the beginning of warm weather these volatile materials penetrated into both surface and binder courses. As a result, a very soft or even "fluid" condition has been created at some places in the pavements, and the tack coat itself has acted as a lubricating layer on which the pavement is being displaced by traffic loads.
L. E. Gregg

August 5, 1952

In certain areas some unsatisfactory aggregate has been worked into the binder course. This stone is soft and it breaks down in the presence of water forming an unstable lubricating course. Stripping of asphalt from the soft aggregate is prevalent also.

Laboratory Test Series

To facilitate comparison of Class I, Type C and Louisville City mixtures, a series of tests on mixes representing both specifications have been run. Basic aggregates for these tests were the same as those used in the street paving project, namely: crushed limestone (1\(\frac{1}{2}\)" to No. 8), river sand (No. 8 to No. 50), and bank sand (passing No. 50). As an added feature of the laboratory tests, an aggregate consisting entirely of crushed limestone was applied to the Type C mixes.

Three of the gradings (or 6 sets of samples) represented the fine limit, coarse limit, and center of the Type C specification range. The three Louisville mixtures tested represented the center gradation of the specification, a gradation on the fine side but not at the fine limit, and a coarse gradation but not the coarse limit. These gradations together with the Louisville specification limits (and the Class I, Type C specification limits for comparison) are shown in Fig. IV-20. Table IV-1 contains all data pertaining to gradation of the laboratory mixes and to the three cores tested in the pavement condition study. In addition, four typical gradings taken from the inspectors report at the time pavement was placed on the Louisville Streets are included.
With aggregates of the various gradings made up in the laboratory, a number of different specimens containing different percentages of asphalt were prepared using the light compaction in the Marshall Stability Method. Data from these design series run with these specimen in Table IV-2, and the results by groups are plotted graphically if Figs. IV-21 to IV-23 inclusive.

The trends in results show clearly that mixtures prepared within the two specifications react differently to changes in gradation of the aggregate. For the Class I, Type C stability increases and percentage asphalt, required to achieve desired void content, decreases as the mix is varied from the coarse side to the fine side of the specification. In contrast, as the coarse side of the Louisville specification is approached, stability generally increases and percentage of asphalt for desirable void content tends to decrease. The data indicate that properties of mixes representing the two specifications are approximately equal when the Class I is on the fine side and the Louisville mix is on the coarse side of the specification range. This must be regarded only as a broad generalization, in view of the gap that exists between the gradings for these two limits of the specifications.

Stability and flow values - but not unit weights and void relationships - for the 1a, 2a, and 3a Type C mixes (those containing natural sand fines) may be misleading to some extent. Expediency at the time the samples were prepared made it necessary that these be started through the test procedure immediately after they had been compacted and cooled to a
point lower than the 140°F. temperature of the water bath at the start of the test. On the other hand, all other specimens were allowed to cool in the molds, they were shoved from the molds several hours after compaction was completed, and they did not enter the test for a period of at least 24 hours. There is no definite evidence that this accelerated treatment would have an adverse effect; however, in all past tests on mixes approximating 3a the stability members were at least as high as 600. For that reason, stability and flow for 3a should be viewed skeptically, but all other properties including the optimum asphalt content are regarded as valid.

Table IV-2 includes a column pertaining to acceptable range of asphalt contents for the various mixes. This range is given somewhat as an indication of the sensitivity of the mixes to change in asphalt content, and not as absolute tolerances that should be intentionally followed. These limits are based wholly on values of flow (maximum 20) and percentage of voids in the compacted mix (2 to 6 percent) which have been generally regarded as suitable in the design criteria applicable to the Marshall Method and to the highest type of bituminous mixes. Of course, other factors in the overall design criteria, such as stability members, have been ignored in determining this so-called "suitable" range of asphalt contents listed in Table IV-2.

The series of samples containing all crushed aggregate is included to demonstrate the influence of angular fine material in mixes where high stability are important. Beyond just the fact that high stability values can be achieved, it is shown that these are achieved
at asphalt contents lower than those that produce maximum stabilities in mixes with rounded fine aggregate but having the same gradation. Actually, the variation in void content with change in percentage asphalt is the factor of primary importance, and the desirable void value is reached with lower asphalt contents in the mixes with crushed aggregate.

Some consideration should be given to the use of all crushed aggregate (either stone or gravel) in locations where high stabilities are required, even though it is recognized that workability is reduced and the finished surface texture is not of the type traditionally considered suitable for city streets. There is no reason to believe that the skid resistance would be decreased by use of crushed fine aggregate (if rigid control of the mix is exercised), and it seems logical that angularity of aggregate particles might increase skid resistance.

To briefly summarize the test findings, it may be said that quite satisfactory mixtures may be prepared with either specification. The Louisville specification is so broad that the possibilities for production of poor mixes are great unless design tests are run using the specific gradation planned for the job. Poor mixes can be avoided, of course, through experience in the selection of aggregate gradation and asphalt contents most desirable in the specification range. Probably that is the basis of the success which Louisville has had thus far in the use of this mix.
Fig. IV-1. Core locations No. 1 and No. 2, respectively 200 feet and 210 feet north of Broadway on Seventh Street. Pavement showing along joint cored.

Fig. IV-2. Core locations No. 3 and No. 4, respectively 150 feet and 140 feet south of Chestnut Street on Seventh Street. Lateral Displacement only.
Fig. IV-3. Core locations No. 5 and No. 6, respectively 300 feet and 310 feet north of Chestnut Street on Seventh Street. No distress of any sort.

Fig. IV-4. Core locations No. 7 and No. 8, respectively 200 feet and 210 feet north of Walnut Street on Seventh Street. Very prominent lateral displacement and slight vertical displacement.
Fig. IV-5. Core locations No. 9 (lower right) and No. 10, approximately midway between Cedar Street and Liberty Street on Seventh Street. Worst pavement condition observed; total failure from displacement. Core No. 10 from long high ridge with as much as 6 inches difference in pavement elevation.

Fig. IV-6. Core locations No. 11 and No. 12, just south of Liberty Street on Seventh Street. Failure in west lanes similar to that shown in Fig. 5, but pavement undamaged in east lanes where cores were taken.
Fig. IV-8. Varied views of cores No. 1, 2, and 3 taken from pavement on Seventh Street. All represent locations where there is surface displacement. There was no binder course placed at these locations in the 1951 paving, and in each case the core extended through the old sheet asphalt surface into the underlying binder.
Fig. IV-9. Varied views of cores No. 4, 5, and 6 taken from pavement on Seventh Street. Only core No. 4 represents failing pavement, there being no evidence of displacement at locations No. 5 and No. 6. Each core extends below the new Type C surface into old sheet asphalt and underlying binder material.
Fig. IV-10. Varied views of cores No. 7, 8, and 9 taken from pavement on Seventh Street. All represent prominent failures. The Type C surface at these locations was underlain by a Type A binder course.
Fig. IV-11. Varied views of cores No. 10, 11, and 12 taken from pavement on Seventh Street. In all cases the Type C surface was placed over a new Type A binder course. Failure is extreme at location No. 10.
Fig. IV-12. Varied views of cores No. 13 and 14 taken from pavement on Sixth Street near Liberty. Both Type C surface and Type A binder courses were placed at this location. There is no particular evidence of failure.
Fig. IV-13. Core No. 2 which represents a case of inherent stability in the new Type C surface and also in the old sheet asphalt and underlying binder. However, failure due to load and warm weather is being produced by slippage on the heavily tacked sheet asphalt. Evident in the core but not in the picture are dark rich areas along the joint between Type C and sheet asphalt, and similar appearances higher within the Type C surface suggest intrusion or migration of asphalt and volatiles from the tacked surface. Shoving is the principal form of failure at this location.

Fig. IV-14. Core No. 4 has the same essential features as core No. 2, but failure at this location is in the form of lateral displacement.
Fig. IV-7. Core locations No. 13 and No. 14, respectively 150 and 170 feet south of Liberty Street on Sixth Street. Pavement in good condition but surface pitted.
Fig. IV-17. The effect of disintegrating Type A binder is unmistakable in Core No. 10. Failure at this location is the worst observed, with some vertical displacements being as great as 6 inches.
### Table IV-1. Gradation Data

<table>
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<th>Identification</th>
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<th>1/4&quot;</th>
<th>3/16&quot;</th>
<th>5/32&quot;</th>
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<th>5/32&quot;</th>
<th>1/8&quot;</th>
<th>3/32&quot;</th>
<th>1/16&quot;</th>
<th>5/64&quot;</th>
<th>1/32&quot;</th>
<th>7/64&quot;</th>
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<td>Sixth St. Surface*</td>
<td>100</td>
<td>93.7</td>
<td>66.7</td>
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<td>36.8</td>
<td>11.2</td>
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<td>Seventh St. Surface*</td>
<td>100</td>
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<td>70.9</td>
<td>51.7</td>
<td>42.2</td>
<td>12.0</td>
<td>8.2</td>
<td>3.9</td>
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<td>Eighth St. Surface*</td>
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<td>67.8</td>
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<td>40.6</td>
<td>12.1</td>
<td>8.0</td>
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<td>Class I, Type A Binder**</td>
<td>700</td>
<td>97.5</td>
<td>69.5</td>
<td>44.4</td>
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<td>Core No. 1 Seventh St.</td>
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<td>94.1</td>
<td>70.5</td>
<td>51.1</td>
<td>41.2</td>
<td>15.4</td>
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<td>6.4</td>
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<td>Core No. 5 Seventh St.</td>
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<td>92.1</td>
<td>64.7</td>
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<td>38.3</td>
<td>15.7</td>
<td>10.7</td>
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<td>Core No. 9 Seventh St.</td>
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<tr>
<td>Class I, Type C Mix No. 1, 6a (Fine Limit)</td>
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<td>70.0</td>
<td>50.0</td>
<td>40.0</td>
<td>20.0</td>
<td>12.0</td>
<td>7.0</td>
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<td>Class I, Type C Mix No. 2, 6a (Center Gradation)</td>
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<td>92.5</td>
<td>60.0</td>
<td>42.5</td>
<td>30.0</td>
<td>14.0</td>
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<td>Class I, Type C Mix No. 3, 6a (Coarse Limit)</td>
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**Average of Type A Binder**

*From inspectors report at time of construction of Louisville streets.

### Table IV-2. Marshall Stability Test Data

<table>
<thead>
<tr>
<th>Mix</th>
<th>Grading</th>
<th>Optimum % Asphalt</th>
<th>Stability Number</th>
<th>Flow 1/10&quot;</th>
<th>Unit Weight</th>
<th>Total Filled</th>
<th>3 Yr. Voids</th>
<th>Acceptable Range</th>
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<td>Type G</td>
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<td>5.1</td>
<td>2250</td>
<td>16</td>
<td>149.5</td>
<td>3.9</td>
<td>77</td>
<td>4.7 to 5.6 (0.9)</td>
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<tr>
<td></td>
<td>6a</td>
<td>5.5</td>
<td>1020</td>
<td>18</td>
<td>166.6</td>
<td>4.0</td>
<td>78</td>
<td>5.3 to 6.3 (1.0)</td>
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<tr>
<td></td>
<td>2</td>
<td>5.8</td>
<td>1610</td>
<td>19</td>
<td>147.0</td>
<td>3.8</td>
<td>78</td>
<td>5.3 to 6.0 (0.7)</td>
</tr>
<tr>
<td></td>
<td>6a</td>
<td>6.5</td>
<td>780</td>
<td>18</td>
<td>144.5</td>
<td>4.4</td>
<td>77</td>
<td>5.8 to 7.3 (1.5)</td>
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<tr>
<td></td>
<td>3</td>
<td>6.5</td>
<td>1150</td>
<td>20</td>
<td>142.4</td>
<td>5.7</td>
<td>73</td>
<td>6.3 to 6.6 (0.3)</td>
</tr>
<tr>
<td></td>
<td>6a</td>
<td>7.0</td>
<td>2150</td>
<td>13</td>
<td>177.0</td>
<td>8.0</td>
<td>65</td>
<td>- - - - - - - -</td>
</tr>
</tbody>
</table>

**Specimens entered test immediately after compaction and cooling to temperature of 140°F.**

**Note:** Aggregate in mixtures 1, 2, and 3 is all crushed limestones. Mixtures 1a, 2a, and 3a were prepared with crushed limestone, river sand, and bank sand. All Louisville mixtures were prepared with crushed limestone, river sand, and bank sand. All specimens prepared with "light" compaction - 25 blows of 16 lb. hammer plus 7000 lb. static load.
Fig. IV-18. Gradation of Aggregate Recovered From Core Samples No. 1, 5, and 9
Fig. IV-19. Gradation of Three Type C Mixes Included In The Laboratory Test Series

Highway Materials Research Laboratory
Lexington, Kentucky

MECHANICAL ANALYSIS

Class I, Type C - Gradation of the three mixtures tested.
Fig. IV-20. Gradation of Three Louisville City Mixes Included In The Laboratory Test Series
Highway Materials Research Laboratory
Lexington, Kentucky

MARSHALL STABILITY TEST

TEST DATA
Optimum % Bitumen - 5.8, 6.5, 7.0
Unit Wt. - 146.8, 144.5, 137.0
Stability No. - 1020, 780, 350
Flow - 18, 17, 14
% Voids, Compacted Mix - 4.0, 4.4, 8.0
% Voids, Filled W/Bitumen - 78, 77, 65

Fig. IV-21. Results of Marshall Design Tests on Type C Mixes
Containing Limestone, River Sand, and Bank Sand.
Highway Material Research Laboratory
Lexington, Kentucky

MARSHALL STABILITY TEST

TEST DATA

- Class I, Type C - Mix Lou. 1
- Class I, Type C - Mix Lou. 2
- Class I, Type B - Mix Lou. 3

Optimum % Bitumen = 6.0, 7.0, 6.2
Unit Wt. = 1.52.6, 1.51.0, 1.53.5
Stability w/o = 800, 640, 1160
Flow = 16, 9, 16
% Voids, Compact Mix = 5.1, 4.7, 4.7
% Voids, Filled w/Bitumen = 72, 77, 77

Fig. IV-22. Results of Marshall Design Tests on Mixes Representing The Louisville City Specification on Containing Limestone, River Sand, and Bank Sand
PART V

COMPARATIVE PROPERTIES

OF

MIXTURES CONTAINING CRUSHED AND UNCRUSHED RIVER GRAVEL
MEMO TO: L. E. Gregg  
Assistant Director of Research

SUBJECT: Class I, Type C Surface Mixtures Using Crushed and Uncrushed River Gravel as Coarse Aggregate.

The use of crushed and uncrushed river gravel together with river sand and bank sand, in Class I, Type C surface mixes has been examined in anticipation of paving requirements. Uncrushed gravel is to be used during the present season in Western Kentucky and while no definite plans concerning crushed gravel are known at present, the comparative properties of the two materials should be considered. To facilitate these comparisons the familiar Type B surface is included.

Laboratory Investigation

Six gradings were prepared with both crushed and uncrushed aggregate. These gradings are contained in Table V-1. Gradings No. 1 through No. 3 are for Class I, Type C surface mixes, while gradings No. 4 through No. 6 represent Class I, Type B surfaces. Mixtures were prepared from all gradings using both crushed and uncrushed coarse aggregate. In all cases river sand formed that fraction of aggregate between the No. 8 and No. 50 screens, while the fraction finer than the
No. 50 screen consisted of bank sand.

The Marshall Stability Test was the basis for design of these mixtures. So-called light compaction - 25 blows of a 10-pound hammer plus a 7000-pound static load - was used in all tests. Design tests were run for each grading with both crushed and uncrushed coarse aggregate. This data is contained in Table V-2, and is recorded in a manner to facilitate comparison of mixtures having identical gradings but different particle shapes.

**Comparison of Type B and Type C Surface Mixes Using Uncrushed Gravel.**

Inspection of Table V-2 shows that although less asphalt was required in Type C mixtures, the stabilities were appreciably higher than for Type B mixtures. In the three Type C mixtures tested, stabilities increased from 600 to 870 as the grading was varied from coarse to fine. While all stabilities are satisfactory for medium traffic roads some advantage is gained in the denser mixes (finer gradings).

The Type B mixtures varied in stability from 200 to 690 as the grading was varied from coarse to fine. Stability of the samples having a coarse grading (200) reflects the serious lack of fines and indicates an undesirable mixture for surface courses. Neither type surface is desirable for use in heavy traffic areas, especially on principal urban streets.

Flow values for both Type B and Type C mixes were well below the maximum desirable value of 20. Type C mixtures had slightly higher and more desirable flow values than the Type B mixtures which had
tendencies toward brittleness. Where mat thickness is sufficient to pro-
vide essentially rigid support for the pavement, low flow values are not
detrimental. However, on medium traffic roads such mats are not usually
provided and flexibility of the pavement is an essential requirement.

Other test properties of both type surfaces and all gradings
were within a satisfactory range at optimum asphalt content. It should,
however, be noted that the percentage voids in the aggregate was generally
lower for the Type C than for the Type B mixtures, indicating the greater
aggregate density expected in this type mixture.

In general, it may be said that the advantages of Type C surface
mixes, using uncrushed gravel as coarse aggregate, as compared to Type B
surface mixes are: (1) lower asphalt requirements, (2) higher stabilities,
and (3) more desirable flow values.

Comparison of Type B and Type C Surface Mixes Using Crushed Gravel

Results in Table V-2 show that the differences in Type B and Type
C surfaces are not so pronounced when crushed gravel forms the coarse
fraction of the aggregate. Asphalt requirements for the gradings tested
were essentially equal as the mixtures were varied from coarse to fine
within their respective grading specifications.

For the entire range of the grading specifications, Type C has
a decided advantage since all stabilities were well within a safe range
for anything except the heaviest traffic, while Type B showed a serious
deficiency at the coarse grading limit. Stability numbers for the Type C
surface mixtures tested increased from 930 at the coarse limit to 1200
at center grading, and then decreased to 1020 at the fine grading.
This latter decrease was not expected, but the value was still well within the satisfactory range. Stabilities for the Type B surface mixtures tested increased from 380 at the coarse limit to 930 at the center grading, and continued upward to 1125 at the fine limit. This continued increase reflects increasing aggregate density as Type B mixes are varied from coarse to fine. Stability at the coarse limit is again very low, indicating a very questionable mixture for any use and definitely an unsuitable mix for areas of heavy traffic.

Flow values were satisfactory for all six gradings. Type C, however, had slightly higher values than comparable Type B mixes indicating greater flexibility for the Type C.

With the exception of those values for mixes made with grading No. 4 (Type B-coarse limit), all other test results were within satisfactory limits at optimum asphalt content. Aggregate with Grading No. 4 is too open-graded for use in surface courses, as indicated by the void content and reflected in the low stability of this set of samples. Generally, percentage of voids in the aggregate is slightly lower in Type C than in Type B mixes, indicating increased aggregate density in the Type C.

The principal advantages of Type C mixtures containing crushed gravel as coarse aggregate lie in increased uniformity throughout the grading range. The generally denser aggregate structure of the Type C mixes is beneficial, and in some locations slight advantage may be derived from the greater flexibility of pavements constructed with this mix.
Comparison of Crushed and Uncrushed Gravel as Coarse Aggregate in Type B and Type C Mixtures.

The test values which, at optimum asphalt content, reflect favorably the properties of mixes with uncrushed gravel as compared to those with crushed gravel are: (1) lower asphalt requirements, and (2) denser aggregate structurers. Those values which favor crushed aggregate are: (1) Major increases in stability, and (2) improved flow values.

This may be summed up by saying that uncrushed gravel is suitable for medium traffic roads provided the aggregate grading is sufficiently dense and so designed to accomplish desired flexibility; however, crushed gravel is better suited to heavy traffic conditions because of the advantages gained from particle shape. It should be emphasized that open-graded mixes with uncrushed gravel tend to be brittle and therefore pavements with these mixes require essentially rigid bases for good performance.

Summary

Type C mixtures require less asphalt than do Type B mixtures, when uncrushed gravel forms the coarse fraction of the aggregate. However, with crushed gravel there appears to be little difference between the Type B and Type C in asphalt requirements. Also, a given grading requires less asphalt when uncrushed aggregate is used in preference to crushed aggregate.

Stabilities produced by Type C gradings are generally higher than those produced by comparable Type B gradings. This is especially true when uncrushed gravel forms the coarse fraction of the aggregate.
All Type C mixes containing uncrushed aggregate were suitable for roads carrying medium traffic loads regardless of the grading that was used. On the other hand, no grading coarser than grading No. 5 should be used in the Type B mixes containing uncrushed gravel.

Flow values were generally satisfactory for all mixtures tested; however, Type B mixtures containing uncrushed gravel had low flow values indicating brittleness.

From the standpoint of density in the aggregate structure, Type C grading provided a greater density than Type B in all cases tested, and uncrushed gravel produced denser structures than crushed aggregate for a given grading.

Ellis G. Williams
Research Engineer
<table>
<thead>
<tr>
<th>Table V-1. Gradation Data</th>
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<tbody>
<tr>
<td><strong>% Passing</strong></td>
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<tr>
<td>Identity</td>
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<tr>
<td>Type C</td>
</tr>
<tr>
<td>No. 1</td>
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</tr>
<tr>
<td>No. 5</td>
</tr>
<tr>
<td>No. 6</td>
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Coarse Limits - No. 1 and No. 4  
Center Grading - No. 2 and No. 5  
Fine Limits - No. 3 and No. 6

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<tr>
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<tr>
<td>No. 3</td>
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</table>

Type B  
No. 4   | 7.0    | 380      | 16                 | 126.7| 8.4       | 65.0       | 23.7 *   |
|         | 6.5    | 200      | 13                 | 140.1| 5.0       | 73.0       | 18.9     |
| No. 5   | 6.1    | 930      | 16                 | 145.2| 4.3       | 77.0       | 20.2 *   |
|         | 5.8    | 535      | 15                 | 143.0| 3.8       | 77.0       | 17.7     |
| No. 6   | 6.0    | 1125     | 14                 | 146.2| 3.7       | 78.0       | 17.8 *   |
|         | 6.1    | 690      | 11                 | 143.4| 4.6       | 75.0       | 18.4     |

*Denotes crushed gravel as Coarse Aggregate.