INVESTIGATIONS INTO THE EFFECT OF FREEZING
ON A TYPICAL ROAD STRUCTURE

by

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INVESTIGATIONS INTO THE EFFECT OF FREEZING ON A TYPICAL ROAD STRUCTURE

ABSTRACT

Provision has been made for the construction of typical road structures in a concrete-lined pit installed in the floor of a large refrigeration room at the Road Research Laboratory. Pavements of standard thickness employing normal surfacing, base and sub-base materials can be laid on any imported soil, the level of the water-table within which can be varied as required. Temperature conditions in the room can be adjusted to give rates and depths of frost penetration similar to those prevailing during the prolonged cold spells which are a feature of unusually severe winters in Britain. Heave, and loss of strength on thawing, associated with the subgrade and unbound base and sub-base materials, can be related to the properties of the materials, the depth of frost penetration and the position of the water-table.

The Report describes a series of tests carried out on a pavement consisting of a 4-inch asphalt surfacing and an 8-inch wet-mix limestone base laid directly on a silty clay subgrade of liquid limit 30 per cent. Levels of water-table 4 ft, 2 ft and 1 ft below road surface level were used.

Although the soil used was classified by the Laboratory freezing test as frost susceptible, serious frost heave and loss of strength on thawing occurred only when the water table was closer than 3 feet to the road surface. This conclusion should apply to any cohesive soil of liquid limit exceeding 30 per cent, but other tests indicate that it would not apply to other more permeable frost susceptible materials such as chalk.

The wet-mix base was not classified by the Laboratory freezing test as frost susceptible and very little heave occurred in it. There was, however, a small intake of water into the base from the subgrade and this caused a significant although temporary loss of strength in the base on thawing. This loss of strength was related more to the initial moisture content and density of the base than to the level of the water-table.

Further tests using more frost susceptible base and sub-base materials are proposed.
1. INTRODUCTION

In countries where the winter conditions are very severe, some damage to roads by the action of frost is usual and is catered for by an annual maintenance programme. In Great Britain very cold winters sufficiently severe to cause widespread damage to road foundations are experienced on average at intervals of about ten years. Even during such winters the depth of frost penetration seldom exceeds 18 in and it should therefore be generally economic to adopt a design for new roads which removes altogether the danger of frost damage, provided there is sufficient knowledge of the frost susceptibility of road materials under the climatic conditions prevailing in Britain.

The potential frost susceptibility of soils and other road materials can be compared in a standardised laboratory test developed at the Road Research Laboratory. Compact specimens of the material, 6 in long and 4 in in diameter, are frozen from one end whilst the other end is kept in contact with free water maintained at a temperature slightly above freezing point. The amount of heave which occurs is used as a comparative measure of the frost susceptibility.

Experience gained over the past 20 years during investigations of road failures due to frost suggests that materials which heave less than 0.5 in after being subjected to this test for 250 hours are unlikely to lead to frost damage in roads, but materials which heave more than 1 in are very liable to give trouble under some circumstances. However, thickness of pavement, drainage conditions and intensity of traffic are other factors which determine the likelihood and extent of frost damage. During the comparatively short natural periods of frost it is impossible to gain anything but a general impression of the importance of such factors; systematic field measurements are not feasible. To overcome this difficulty, provision has been made at the Road Research Laboratory for the construction and observation of typical road structures built into the floor of a large refrigeration room. This Report describes the first series of tests carried out with this facility.

2. THE MECHANISM OF FROST HEAVE IN ROAD STRUCTURES

When ice and water are present together in soil or other porous materials, the equilibrium suction in the water becomes independent of the moisture content, and is uniquely related to the temperature alone. This relationship is represented by the equation.

\[ pF = 4.1 + \log_{10} t \]

where the suction is expressed on the pF scale and \( t \) is the number of degrees centigrade below freezing point.\(^4\)

It follows from this equation that high suctions may arise in the freezing zone, and that once freezing has commenced in soil, or any other porous material containing water, suction gradients are created tending to draw water into the freezing zone, to give rise to frost heave.
Frost heave in the case of roads depends on the presence of three factors:

1. frost susceptible materials
2. an adequate moisture supply, and
3. sufficiently low temperatures to cause some freezing of water in the susceptible materials. If any one of these factors is absent, significant frost heave will not occur.

When thawing takes place in a road foundation affected by frost heave the ice present in the freezing zone melts to cause a considerable increase in moisture content in that part of the road structure. This may reduce the bearing capacity to near zero, until such time as the excess moisture can drain back to the water-table under the action of gravity. Damage may therefore arise either from the disruptive effect arising from the development of the ice lenses, or from the weakening of the structure attendant on the thawing process.

3. SCOPE OF THE INVESTIGATION

During the field observations made during the cold winters of 1940/41, 1946/47 and 1962/63 it has been clear that the amount of heave at the road surface and the danger of subsequent failure depends on:

1. the types of materials forming the subgrade, sub-base and base of the road,
2. the depth of the water-table,
3. the total thickness of the pavement in relation to the traffic carried, and
4. the duration of the cold spell and its severity.

The intention is to explore these aspects under controlled conditions. So far, tests have been completed on one road structure only, using three levels of water-table. The structure consisted of a 4-in asphalt surfacing on a wet-mix limestone base 8 in thick laid directly on a silty clay sub-grade. The levels of water-table were 1, 2, and 4 feet below the road surface. The pavements were laid in a concrete-lined pit set in the floor of a refrigerated room in which air temperature as low as \(-20^\circ\text{C}\) could be achieved.

4. EXPERIMENTAL METHODS

4.1 Construction of the pit

In an excavation made in the floor of the cold room, a concrete-lined pit 9 ft 6 in long, 5 ft wide and 5 ft deep was constructed. The walls and floor of the pit were coated with waterproof paint to prevent any movement of water into and out of the pit. Fig. 1 shows a diagram of the pit with principal dimensions.

The pit was divided into two parts, a large chamber to accommodate the road structure, and a smaller interconnected chamber for introducing and controlling the
water-table. The small chamber, which was provided with a wooden cover was separated from the main chamber by a concrete wall 4½ in thick. To connect the two chambers four equally spaced holes were positioned in a horizontal line close to the bottom of the wall. These holes permitted perforated brass tubes of 2 in diameter to pass through the wall and to extend for nearly the full length of the main chamber. The tubes were supported at a level of about 2 in above the floor of the pit and were embedded in a 6 in thick layer of medium gravel over which was placed a 3 in layer of fine gravel. Plate 1 is a general photograph of the large chamber showing the gantry used to support the in situ C.B.R. apparatus which was used for testing the subgrade and base of the road structure. Plate 2 shows the four perforated brass tubes at the bottom of the pit. The pit thus constructed enabled any road structure to be built on the gravel drainage layers and permitted the level of the water-table to be controlled from the small chamber.

4.2 Materials used in road structure

The soil used for the subgrade in the present series of tests was a silty clay of liquid limit 30 per cent, plastic limit 19 per cent and particle specific gravity 2.65 (Casagrande classification CL). The particle size distribution is shown in Fig. 2.

The wet-mix base was of a hard limestone graded to the requirements of Clause 807 of the Ministry of Transport Specification for Road and Bridgeworks, (1957). The particle size distribution is shown in Fig. 3, and the particle specific gravity was 2.69.

The surfacing was two-course rolled asphalt to the following specification:

**Basecourse** Finished thickness 2½ in. To B.S. 594:1958, Table 7, Schedule 4, stone content 65 per cent (½ in max size) crushed rock aggregate. Asphaltic cement to Table 1, Column 1, (40-60 pen. petroleum bitumen).

**Wearing course** Finished thickness 1½ in. To B.S. 594:1958, Table 6, Schedule 1, stone content 35 per cent (3/8 in maximum size). Asphaltic cement to Table 1, Column 3 (40-60 pen. 50/50 blend T.L.A. and petroleum bitumen). Added filler: limestone dust. No precoated chips.

To facilitate removal and re-laying of the road surface the material was precast in blocks 12 in square and laid in the manner described later in this note.

4.3 Construction of the road structure

The soil forming the subgrade of the road structure was obtained from a local borrow pit and was laid at its natural moisture content. It was placed in the pit in six equal layers of compacted thickness approximately 6 in. Each layer was compacted separately by a hand rammer supplemented by a Kango electric rammer. After the first four layers had been compacted, 24 boreholes of 4 in diameter, evenly spaced over the area of the subgrade, were augered through the soil down to the fine gravel below. A perforated hollow zinc drainage tube of 4 in in diameter and two feet in length was placed in each of these boreholes, the top of each tube being positioned level with the surface of the soil. The tubes were then filled with the same fine gravel as that used below the soil. The purpose of the gravel-filled drainage tubes
was to encourage equilibrium moisture conditions to be attained quickly when the level of the water-table was altered. Plate 3 shows the gravel-filled drainage tubes in position. After the drainage tubes had been installed, the remaining two layers of soil were spread and compacted. (A sample of the fine gravel was packed into a glass tube and this was set up vertically in the laboratory with the lower end in contact with a free water surface. The capillary rise of water up into the fine gravel was observed to be 1 inch. This meant that the level of the water-table in the drainage tubes in the road subgrade would not differ significantly from the level of the water in the small chamber.)

During laying, five moisture content determinations were made at random points in each layer, and an average moisture content for the layer was obtained to give a preliminary moisture content/depth profile for the subgrade, Fig. 4. The variation of moisture content observed in each layer was less than 1 per cent. After placing the soil, its surface was sealed with polythene sheeting for two days after which a borehole was sunk near the centre of the pit and moisture content measured at 3-in intervals of depth. These results are also shown on Fig. 4. The fluctuations of moisture content found within the layers reflect the variation of suction associated with density gradients produced within the layers during compaction.

Measurements of dry density, using the sand replacement method were made at two points in each layer of the soil after compaction. The range of measurements on all layers was 97.3 to 103.3 lb/cu ft., with an average of 99.7 lb/cu ft., for the soil as a whole. This corresponds to an air content of 8 per cent.

Before the base was laid the polythene sheeting was removed and 20 in situ measurements of C.B.R. were made on the surface of the subgrade, using a normal C.B.R. rig working off the gantry shown in Plate 1. The values ranged between 2 per cent and 3 per cent with an average value of 2.7 per cent. The individual results together with the associated moisture contents are given in Table 1, Columns 1 and 2. After these measurements were completed, the borehole was backfilled to the original soil density and the surface of the subgrade re-shaped to level.

The wet-mix limestone used for the base was laid in two layers, each of compacted thickness 4 in. A lightweight vibrating plate compactor was used. The finished surface was level to within ± 1/8 in. The moisture content of six representative samples of the wet-mix material taken during the laying of the base indicated a range from 2.7 to 3.6 per cent with an average value of 3.1 per cent. Two measurements of dry density were made on the compacted base. The values were 136.9 lb/cu ft, and 131.3 lb/cu ft giving an average of 134.1 lb/cu ft.

Although not strictly applicable to material containing an appreciable percentage of aggregate coarser than 3/4 in, the C.B.R. test was used to assess the bearing capacity of the wet-mix, because it was the only test practicable within the confined space of the refrigerated room. The values ranged from 20 per cent to 78 per cent with an average of 47 per cent. The individual values are given in Table 1, Column 3. The average moisture content at the surface, corresponding to these tests was 2.4 per cent (see Table 1, Column 4), i.e. 0.7 per cent less than average moisture content measured in the wet-mix material, as laid.

The two-course asphalt used for the surface was made up as pre-cast blocks using moulds 12 in square and 4 in deep. The blocks were placed with the wearing
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<td>44</td>
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**Mean:** 2.7 | 18.0 | 47 | 2.4 | 141 | 20.3 | 136 | 2.8 | 34 | 2.5 | 2.5 | 17.7 | 21 | 2.4 | 2.8 | 17.6 | 36 | 1.9

*After only 1 day's drainage*
course material uppermost on the compacted wet-mix base. Plate 4 shows some of
the blocks in position. The spaces between the blocks were filled with a plastic com-
pound and the joints were covered with adhesive tape, Plate 5. This method of making
the surface was chosen because it was impossible to operate surfacing plant in the
confined space of the cold room, and also to facilitate the removal of the surfacing to
enable the testing programme on the base and subgrade to be carried out.

Full details of the structure, showing the location of the drainage tubes are given
in Fig. 5.

4.4 Equipment installed for measuring pore water pressure and temperature

During construction, equipment was placed at various points in the structure to
measure pore water pressure and temperature. To enable the leads and connections
to be accommodated, a narrow space across the full width of the pit was left open in
the surfacing at one end. When all the instruments had been installed this space was
filled with well-compacted fine cold asphalt.

The pore water pressure in the subgrade both before and after freezing was
measured by a remote reading tensiometer of a type developed at the Laboratory. 5
During the construction phase, the brass unit containing the sintered glass plate was
installed at a point 1 ft below the formation level, and the brass connecting tubes were
led out of the pit. A calibrated bourdon gauge was used to record the pore pressure.
The fixed levels of the gauge above the surface of the road and of the measuring
point below the surface were determined using a level and staff. This enabled the
difference in hydraulic head to be calculated, and applied to all readings of the gauge
to determine the true value of pore water pressure at the point where the measuring
unit had been installed.

Temperatures in the road structure during the progress of the experiments were
measured by thermistors designated as type F, 23 which have a resistance of about
2000 ohms at 20°C. To prevent damage to the glass thermistors during construction
of the road section they were inserted, with flexible leads attached, into brass tubes
sealed at one end and containing petroleum jelly. The open end of the brass case was
sealed with an epoxy resin adhesive which secured the flex and prevented the ingress
of moisture. Fig. 6 shows a cross-section of the assembled thermistor together with
the circuit diagram of the substitution bridge used for measuring the resistance. The
bridge and the distribution board for the 24 thermistors used were located in an ad-
jacent room and are shown in Plate 6. Prior to installation, individual calibration
curves were obtained for all the thermistors used over the temperature range —20°C
+ 20°C. Fig. 7 shows a typical calibration curve for a thermistor of the type used.
Thermistors were installed on the vertical axis of the pavement at various depths in
the soil and the wet-mix base. A limited number were also located near the two ends
of the structure. The exact location of the thermistors is given in Fig. 8. The brass
tubes containing the thermistor elements were placed horizontally so that the depth
of measurement could be accurately established.

4.5 Measurements made at each level of water-table investigated

After the pore water pressure of the subgrade had become stabilized at the level
of water-table required, the moisture contents and strengths of the base and the sub-
grade were measured before freezing was commenced. During freezing, tempera-
tures within the road structure were recorded and the depth of the zero isotherm
plotted against time. Any associated surface heave was recorded against time.

When the depth of the zero isotherm became constant under the particular tem-
perature conditions used for the cold room, the moisture content and strengths of
the base and subgrade were measured by the methods discussed below to give values
corresponding to the frozen condition. Thawing was then commenced by increasing
the cold room temperature and measurements were possible of the moisture contents
and strengths of the base and subgrade in the partially and fully thawed conditions.
These measurements were repeated after a period of from 7 to 21 days had been
allowed for drainage.

5. DETAILS OF TESTS

5.1 Test 1 (Water-table 4 ft below surface)

At the commencement of the first test, water was introduced into the small pit to
bring the water-table to a depth of 4 ft below the road surface. The tensiometer was
filled and brought into operation at the same time.

The soil used to provide the subgrade had been taken from a location where the
water-table was effectively 4 ft below the surface. It was anticipated therefore that
there would be little moisture exchange between soil and water-table in the first test.
In fact the pore water pressure was a little positive of the equilibrium value for the
position of the water-table, at the measuring point of the tensiometer, and slow drain-
age to the water-table occurred. This is indicated by the pore water pressure meas-
urements shown in Fig. 9 (c). Observations of pore water pressure were made for
about 5 weeks before freezing was commenced. During this period the pore water
pressure very nearly reached equilibrium with the water-table. Studies of the mois-
ture content/suction relations for the soil showed that the changes of pore pressure
which occurred after the introduction of the water-table would have resulted in less
than a 0.2 per cent overall change of moisture content of the soil, and in fact once
the level of the water-table had been established, no measurable change in level,
which could be associated with moisture movements in the soil, occurred. In view of
this it was decided to accept the measurements of strength already made in the soil
and in the base as applicable for the water-table at the 4 ft depth.

It was known from observations made during the very cold winters of 1946/7
and 1962/63 that the zero isotherm under pavements reached a depth of 20-24 in and
that it attained this depth by progressive downward movement over a period of 4-6
weeks. In the experiments it was proposed to simulate these conditions as far as
possible. The first test was in this respect somewhat exploratory. Whilst pore water
pressure conditions were reaching equilibrium the room temperature was reduced to
a value a little above freezing point to simulate the cold air temperatures which nor-
manly precede a prolonged frost. At the end of this period the pore pressure gauge
was disconnected to prevent damage by freezing and the datum level of the pavement
was recorded. This was done by measuring the distance between the centre of each
surfacing block and a long straight-edge placed across the iron girders secured along
each side of the pavement section, Plate 1. The air temperature was then reduced in stages by increments of 4° or 5°C, readings of temperature at all points in the road structure being taken at frequent intervals during the period. The increments of temperature chosen, the periods for which they were applied and the corresponding temperatures beneath the centre of the pavement are shown in Fig. 9 (a). From the temperature readings, the depth of the zero isotherm was calculated, Fig. 9 (b). During the freezing period the level of the centre of each surfacing block was recorded daily and the heave calculated. The mean heave for the surface as a whole is plotted against time in Fig. 9 (d). Whilst freezing was in progress no change in the level of the water-table was recorded and no water was added.

The lowest temperature used in the cold room was −17°C, which gave a depth of frost penetration of the order required. It will be seen from Fig. 9 (a) that after this temperature had been maintained for some days the room temperature was increased to −12°C. This was due to a failure of a cooling compressor and the higher temperature had to be maintained for about six weeks whilst repairs were effected. The room temperature was then again reduced to −17°C and maintained at that value until equilibrium temperature conditions had been established in the road structure. This circumstance caused a temporary delay in the downward movement of the zero isotherm and in the rate of surface heave.

When equilibrium conditions had been established, several of the asphalt blocks were removed and nine measurements of in situ C.B.R. and associated moisture content were made on the surface of the wet-mix base. A boring was also made through the base and moisture contents determined at 2 in intervals of depth. The surface of the subgrade was exposed in four areas and in situ C.B.R. values determined on the frozen soil together with the associated moisture contents. After this the structure was replaced as far as possible in its original condition and thawing was commenced a few days later.

During thawing the measurements of temperature within the structure were continued and the upward movement of the zero isotherm determined. To enable C.B.R. and the moisture content determinations to be made on the base and subgrade in the partially and fully thawed conditions the room temperature was controlled to delay thawing as necessary. The temperatures during thawing are shown on an enlarged time scale in Fig. 10, and the times at which C.B.R. and moisture content measurements were made are shown on that figure and on Fig. 9 (a). Different areas of the structure were used for each set of measurements to avoid the effects of disturbance. Ten days after completion of the thaw two borings were made through the wet-mix base and moisture content measurements at 2 in intervals of depth were made. Boreholes were also sunk through the full depth of the soil 6 days and 20 days after the completion of the thaw and moisture contents measured at 3-in intervals of depth. Table 1 gives the C.B.R. and associated moisture content values measured on the subgrade and base throughout the test. The results are summarised in Table 4. Fig. 11 (a) shows the moisture content/depth profile through the wet-mix base whilst frozen and ten days after thawing, compared with the average moisture content before freezing. Fig. 12 shows the moisture content/depth profiles through the subgrade 6 days and 20 days after thawing compared with the profile before freezing.

5.2 Test 2 (Water-table 2 feet below surface)

Six weeks before the commencement of the second test the water-table was
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<th>BEFORE FREEZING</th>
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<th>FULLY THAWED</th>
<th>PARTIALLY DRAINED</th>
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<td></td>
</tr>
<tr>
<td>4.1</td>
<td>16.9</td>
<td>25.0</td>
<td>2.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.9</td>
<td>17.8</td>
<td>33.0</td>
<td>3.6</td>
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<td></td>
</tr>
<tr>
<td>3.9</td>
<td>17.5</td>
<td>45.0</td>
<td>3.6</td>
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<td></td>
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<tr>
<td>3.2</td>
<td>17.6</td>
<td>35.0</td>
<td>4.1</td>
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<td></td>
</tr>
<tr>
<td>3.3</td>
<td>17.1</td>
<td>34.0</td>
<td>2.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.6</td>
<td>16.9</td>
<td>18.0</td>
<td>2.9</td>
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<td>2.9</td>
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<tr>
<td>3.6</td>
<td>17.9</td>
<td>25.0</td>
<td>4.1</td>
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<tr>
<td>3.6</td>
<td>17.6</td>
<td>48.0</td>
<td>3.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MEAN 3.5</td>
<td>17.8</td>
<td>33.0</td>
<td>3.3</td>
<td>10.0</td>
<td>24.8</td>
</tr>
</tbody>
</table>
raised to a level 2 ft below the road surface. It was thus at the same level as the pore water pressure gauge which registered zero pore water pressure within a few hours. At the same time it was clear that the soil above the water-table would need time to equilibrate and using the experience gained in the first test five weeks were allowed for this purpose.

At the end of that period the original wet-mix base was removed together with the thermistors contained in it. The subgrade was then reinstated to the original level and covered with plastic sheeting. A week later the sheeting was removed and a borehole was sunk and moisture content samples taken at 3 in intervals of depth to give the moisture content/depth profile, before freezing, shown in Fig. 13. Twenty measurements of in situ C.B.R. were made over the surface of the subgrade; these are given with the associated moisture contents in Table 2, Columns 1 and 2. A new wet-mix base 8 in thick was then laid as in the first test, the thermistors being re-placed in the original positions. The average moisture content of the material as laid was 3.3 per cent and the dry densities obtained from two measurements were 136.6 and 141.4 lb/cu ft., giving an average of 139 lb/cu ft. Twenty in situ C.B.R. tests were made on the surface of the compacted wet-mix base; the values and associated moisture contents are quoted in Table 2, Columns 3 and 4. After regulation of the surface of the wet-mix base the asphalt surfacing blocks were laid as in the first test and the pavement datum level was recorded. During this period the level of the water-table remained constant and no water was added. Before freezing the pore water pressure gauge was disconnected.

In this test the room temperature was reduced in one stage to −17°C and temperatures were recorded daily in the structure, together with the average heave of the pavement. The temperature conditions in the structure became stable after about 8 weeks. After this period there was some fluctuation of room temperature due to a compressor fault. Thawing was commenced when this had been corrected.

Fig. 14 (a) shows the temperature measurements made during freezing and thawing and it indicates the times at which measurements of in situ C.B.R. were made on the frozen, partially thawed, fully thawed and drained base and subgrade as in Test 1. The results of these measurements are given in detail in Table 2 and are summarised in Table 4. Fig. 14 (b) shows the depth of the zero isotherm plotted against time and Fig. 14 (c) the pore water pressure measurements before and after freezing. Finally Fig. 14 (d) gives the average heave of the structure and the amounts of water which were added at various times to maintain the level of the water-table.

Fig. 15 (a) gives the temperatures recorded by the thermistors located close to the two ends of the structure, for comparison with the measurements at the centre, Fig. 14 (a). Fig. 15 (b) compares the depths of frost penetration at the two ends and at the centre of the structure.

As in the first test, moisture content/depth profiles were determined in the wet-mix base whilst the material was frozen and after the thaw, Fig. 11 (b). Similarly, moisture content/depth profiles were determined in the subgrade after the thaw and 16 days later, Fig. 13.
### Table 3

Results of C.B.R. Tests—Test 3 (Water Table Depth 1 foot)

<table>
<thead>
<tr>
<th>Before Freezing</th>
<th>When Frozen</th>
<th>Partially Thawed</th>
<th>Fully Thawed</th>
<th>Partially Drained</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Subgrade</td>
<td>Base</td>
<td>Subgrade</td>
<td>Base</td>
</tr>
<tr>
<td>C.B.R. %</td>
<td>M.C. %</td>
<td></td>
<td>C.B.R. %</td>
<td>M.C. %</td>
</tr>
<tr>
<td>2.5</td>
<td>21.8</td>
<td>15</td>
<td>1.6</td>
<td>115</td>
</tr>
<tr>
<td>2.8</td>
<td>19.9</td>
<td>14</td>
<td>1.6</td>
<td>150</td>
</tr>
<tr>
<td>1.5</td>
<td>19.1</td>
<td>10</td>
<td>1.9</td>
<td>145</td>
</tr>
<tr>
<td>1.2</td>
<td>19.2</td>
<td>22</td>
<td>1.8</td>
<td>125</td>
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<tr>
<td>1.3</td>
<td>19.6</td>
<td>10</td>
<td>1.8</td>
<td>135</td>
</tr>
<tr>
<td>2.3</td>
<td>17.4</td>
<td>19</td>
<td>1.8</td>
<td>115</td>
</tr>
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</tr>
<tr>
<td>2.5</td>
<td>17.4</td>
<td>10</td>
<td>1.5</td>
<td>100</td>
</tr>
<tr>
<td>2.5</td>
<td>20.9</td>
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<td>1.5</td>
<td>82</td>
</tr>
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<td>2.7</td>
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<td>3.6</td>
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<td></td>
</tr>
<tr>
<td>2.5</td>
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</tr>
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<td>1.1</td>
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<td></td>
</tr>
<tr>
<td>2.0</td>
<td>17.2</td>
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<td>2.0</td>
<td></td>
</tr>
<tr>
<td>1.7</td>
<td>19.0</td>
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<td>2.1</td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td>17.7</td>
<td>12</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>2.2</td>
<td>13.0</td>
<td>13</td>
<td>2.2</td>
<td></td>
</tr>
<tr>
<td>1.6</td>
<td>18.0</td>
<td>16</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>18.0</td>
<td>14</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>1.6</td>
<td>17.7</td>
<td>25</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>18.6</td>
<td>14</td>
<td>1.8</td>
<td>117</td>
</tr>
<tr>
<td>Layer Tested</td>
<td>Test No.</td>
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<td>2</td>
<td>3</td>
</tr>
<tr>
<td>-------------------</td>
<td>----------</td>
<td>---------</td>
<td>---------</td>
<td>---------</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SUBGRADE</td>
<td>BASE</td>
<td>SUBGRADE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C.B.R. %</td>
<td>M.C. %</td>
<td>C.B.R. %</td>
</tr>
<tr>
<td>Before Freezing</td>
<td>2.7</td>
<td>18.0</td>
<td>47</td>
<td>2.4</td>
</tr>
<tr>
<td>Frozen</td>
<td>141</td>
<td>20.3</td>
<td>136</td>
<td>2.8</td>
</tr>
<tr>
<td>Partially Thawed</td>
<td>34</td>
<td>2.5</td>
<td>21</td>
<td>2.8</td>
</tr>
<tr>
<td>Fully Thawed</td>
<td>2.5</td>
<td>17.7</td>
<td>21</td>
<td>2.4</td>
</tr>
<tr>
<td>After Drainage</td>
<td>2.8</td>
<td>17.6</td>
<td>36</td>
<td>1.9</td>
</tr>
</tbody>
</table>
5.3 Test 3 (Water-table 1 foot below surface)

Four weeks before the commencement of the third test, the wet-mix base used in the second test was removed. All boreholes in the subgrade were reinstated to the original density and the surface was regulated and covered with polythene sheeting. The water-table was then raised to a level 1 ft below the final road level (i.e. to formation level). The pore water pressure gauge was connected and almost immediately registered equilibrium with the position of the water-table. After a period of 7 days the sheeting was removed and twenty measurements of in situ C.B.R. and associated moisture content were made on the surface of the subgrade, Table 3, Columns 1 and 2, and moisture contents were determined at 3 in intervals of depth to give the moisture content/depth profile before freezing, Fig. 17.

Wet-mix material having an initial average moisture content of 2.5 per cent was laid as in the previous tests to an average dry density of 133 lb/cu ft., and sealed with polythene sheeting after installation of the thermistors. After four days the sheeting was removed and 20 determinations of in situ C.B.R. were made on the surface, the values are shown together with the associated moisture contents in Table 3, Columns 3 and 4. After surface regulation the asphalt blocks were placed and the initial levels taken.

As in Test 2, after the pore water pressure gauge had been disconnected, the room temperature was reduced to $-17^\circ$C in one stage and temperature recordings were commenced. Temperature conditions in the structure became stable after about 12 weeks and thawing was commenced after about 15 weeks. The test programme followed that adopted for the first two tests. Fig. 16 (a) shows the temperature records and gives the times at which strength and moisture content determinations were made. Fig. 16 (b) gives the depth of the zero isotherm, Fig. 16 (c) the pore water pressure measurements and Fig. 16 (d) the heave. The latter diagram also shows the quantities of water added to maintain the level of the water-table. Table 3 gives the results of all the measurements of strength and moisture content made on the subgrade and the base and these are summarised in Table 4. The moisture content profiles in the base and subgrade are given in Figs. 11 (c) and 17 respectively.

6. DISCUSSION

6.1 Depth and rate of frost penetration

The maximum depths of frost penetration in the three tests and the approximate times in which these penetrations were achieved are shown in Table 5. (In Test 1 the room temperature was lowered in stages and this influenced the rate of penetration. The rather greater penetrations in Tests 2 and 3 are probably associated with the higher moisture contents in the soil and seasonal changes in ground temperature conditions beneath and around the cold room.)
TABLE 5

Depths and rates of frost penetration

<table>
<thead>
<tr>
<th></th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum depth of zero isotherm (in)</td>
<td>25</td>
<td>28</td>
<td>27</td>
</tr>
<tr>
<td>Time for zero isotherm to reach lowest level (days)</td>
<td>130*</td>
<td>65</td>
<td>100</td>
</tr>
</tbody>
</table>

* temperature reduced in stages

Fig. 18 compares the three penetration/time curves, (Figs. 9 (b), 14 (b) and 16 (b)) with that obtained under a typical road pavement in the London area during the severe frost of 1962/63. In that period frost penetrations in the south of England were greater than in the previous cold winters of 1946/47 and 1940. In the laboratory tests the penetrations were several inches greater, but the rate of downward movement of the zero isotherm in the tests was similar to that occurring in practice. The conditions in the laboratory tests therefore were rather more severe than those likely to be encountered in the severest British winters experienced in the last few decades.

6.2 Moisture content and strength variations in the base

Fig. 11 shows that in all three tests the moisture content of the wet-mix base was higher whilst frozen than before freezing or after drainage. The average moisture contents before, during and after freezing are given in Table 6.

TABLE 6

Average moisture contents of wet-mix base

<table>
<thead>
<tr>
<th></th>
<th>Moisture content (per cent)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test 1</td>
</tr>
<tr>
<td>Before freezing</td>
<td>3.2</td>
</tr>
<tr>
<td>Frozen</td>
<td>5.1</td>
</tr>
<tr>
<td>After thawing</td>
<td>2.5</td>
</tr>
</tbody>
</table>

The increase in moisture content in the frozen base material indicates that water was drawn from the subgrade whilst the zero isotherm was passing through the base. The times taken for this to occur in Tests 1, 2 and 3 were 8, 4 and 7 days respectively. Moisture transfer from soil to base material would have been expected to increase with decreasing downward rate of movement of the zero isotherm in the base and with increasing height of the water-table. From this point of view the greatest movement of water into the base should have occurred in Test 3, which in fact shows the least movement. The likely explanation lies in the low initial moisture content of the wet-mix base in Test 3. In such material this could have re-
duced the unsaturated permeability of the wet-mix to a level where upward movement of moisture was restricted. The observations made show that in all three tests drainage of the wet-mix base after the thaw was comparatively rapid.

In each test the strength of the wet-mix base as indicated by the C.B.R. value increased by a factor of 3 to 6 on freezing, Table 4. Immediately after the thaw however the strength was \( \frac{1}{4} \) to \( \frac{1}{2} \) of the original value, but after drainage much of this loss of strength had been recovered. Although the moisture content measurements made in the surface of the base in connection with the C.B.R. tests do not indicate any large differences in the moisture condition of the base before and after freezing it is clear from the moisture content/depth profiles in the base that the loss of strength immediately after the thaw was due primarily to the increased moisture content of the base as a whole. The incomplete recovery of strength after drainage probably reflects a small relaxation of dry density in the base attendant on the freezing process.

6.3 Moisture content and strength variations in the subgrade

Table 7 shows for the three tests the average moisture contents of the soil above the lowest depth of the zero isotherm, before freezing, after thawing and after drainage.

<table>
<thead>
<tr>
<th></th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before freezing</td>
<td>20.5</td>
<td>18.9</td>
<td>20.2</td>
</tr>
<tr>
<td>After thawing</td>
<td>19.9</td>
<td>20.7</td>
<td>23.2</td>
</tr>
<tr>
<td>After drainage</td>
<td>19.8</td>
<td>19.7</td>
<td>20.1</td>
</tr>
</tbody>
</table>

In Test 1 the moisture contents indicate no increase in the amount of water present after thawing. The pore pressure gauge located slightly above the level of the maximum depth of frost penetration indicated a change in pore water pressure from \(-1.5\) ft of water before freezing to \(-0.5\) ft of water after thawing. The relation between suction and moisture content for the soil used in the tests was such that an increase of moisture content of less than 0.3 per cent would have resulted in the change of pore water pressure observed. The indication is therefore that in this test there was a tendency for water to move upward from the water table during freezing but the actual magnitude of the movement which occurred was not significant.

In Tests 2 and 3 the average moisture contents of the subgrade after thawing were significantly greater than before freezing; it appears however that the initial drainage was rapid and that the original moisture condition had been substantially re-established within 4 weeks of the thaw.
As with the wet-mix bases there was a large increase in strength while the soil was frozen in all three tests. In Test 1 however the strength after thawing was very similar to the original strength, which is consistent with no change of moisture content. In the remaining tests there was a considerable loss in strength immediately after the thaw (to about \( \frac{1}{2} \) the original value), but this was largely regained after drainage.

6.4 Heave of road structure

The maximum heaves in the base and in the subgrade can be deduced from Figs. 9 (d), 14 (d) and 16 (d). The values are shown for the three tests in Table 8.

<table>
<thead>
<tr>
<th>TABLE 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heave in the road structure</td>
</tr>
<tr>
<td>Heave (in)</td>
</tr>
<tr>
<td>Measured heave</td>
</tr>
<tr>
<td>Base</td>
</tr>
<tr>
<td>Subgrade</td>
</tr>
<tr>
<td>Total</td>
</tr>
<tr>
<td>Computed heave in base*</td>
</tr>
<tr>
<td>Due to expansion of water originally in base</td>
</tr>
<tr>
<td>Due to intake of water in base</td>
</tr>
<tr>
<td>Total heave in base</td>
</tr>
<tr>
<td>Computed heave in subgrade* †</td>
</tr>
<tr>
<td>Due to expansion of water originally in subgrade</td>
</tr>
<tr>
<td>Due to intake of water into subgrade</td>
</tr>
<tr>
<td>Total heave in subgrade</td>
</tr>
<tr>
<td>Total computed heave</td>
</tr>
</tbody>
</table>

* Assuming no water expanded or absorbed into the air voids
† Assuming all water above zero isotherm frozen
Frost heave in an unbound porous material can be regarded as the sum of two components arising from:

(i) the expansion on freezing of the water contained in the material and
(ii) the expansion due to water drawn into the freezing zone.

In a material containing air voids some of the expansion of the water could take place into the air voids and some of the water drawn into the material could also enter the air voids. The apparent change in volume would thus be less than that computed from the two factors discussed above.

For the base material used in these tests the average moisture contents before freezing and in the frozen state are both known. It is possible, therefore, to calculate the change of volume arising from the expansion of the water initially in the material and from the intake of water assuming that no expansion takes place into the air voids, i.e. that the air voids content remains constant. In Test 1 the initial moisture content of the wet-mix at a dry density of 135 lb/cu ft was 3.2 per cent. Hence the volume of water contained in each cubic foot of material was:

\[
\frac{135 \times 3.2}{100 \times 62.4} = 0.069 \text{ cu ft.}
\]

The change in height of a 1 foot thickness due to the expansion of this water (assuming no expansion into the voids) would be:

\[0.069 \times 0.1 \times 12 = 0.083 \text{ in}\]

and the change in height of an 8 in thickness would be 0.06 in. The average increase in moisture content due to the uptake of water was 1.9 per cent. Therefore, the volume of water taken up per cubic foot was:

\[
\frac{135 \times 1.9}{100 \times 62.4} = 0.041 \text{ cu ft}
\]

and the change in height of an 8 in layer would be:

\[0.041 \times 0.66 \times 12 \times 1.1 = 0.36 \text{ in}\]

The total heave in the base (again assuming no expansion into the voids) would therefore be:

\[0.06 + 0.36 = 0.42 \text{ in.}\]

These values together with the results of similar calculations for Tests 2 and 3 are included in Table 8. The computed values of heave in the base are about three times as great as the observed values, indicating (as might be expected in a material with an air voids content of about 14 per cent) that much of the volume change associated with the intake of water during freezing took place into the voids.

Similar calculations of heave can be made of the subgrade. In this case however the average value of the moisture content of the frozen soil is not available; the
values available referring to the moisture condition after varying periods of drain-
age. Nevertheless a record is available of the total quantities of water added to
maintain the level of the water-table during freezing. These are 0, 19.5 and 65.5
gallons respectively for Tests 1, 2 and 3.

In Test 1 the average moisture content of the soil above the depth of maximum
penetration of the zero isotherm was 20.5 per cent, and the maximum depth of pene-
tration was 13 in into the soil. Thus the volume of water in each square foot of the
subgrade affected by freezing was:

$$\frac{13 \times 20.5}{12 \times 62.4} \text{ cu ft (assuming a dry density of 100 lb/cu ft for the soil.)}$$

and the upward movement on freezing would be:

$$\frac{13 \times 20.5}{62.4 \times 10} = 0.43 \text{ in.}$$

Quite apart from the assumption of constant air voids during freezing this estimate
would be expected to be rather excessive since all the water in the soil above the
zero isotherm would not be frozen.

No water was added in the first test to maintain the level of the water-table and
it can be assumed therefore that there was no significant uptake of water. The total
heave would thus not be expected to exceed 0.43 in in the subgrade. This is very
close to the measured heave in the subgrade of 0.45 in (Table 8), and suggests that
of the water originally contained in the soil little of the expansion on freezing took
place into the air voids.

For Test 2 the heave due to the expansion of the water in the soil calculated in
the same manner was 0.48 in, i.e. very similar to that in Test 1, but in this case
19.5 gallons of water were added and this would all have been absorbed into the
freezing zone. The volume of this water is $\frac{195}{62.4}$ cubic feet and spread over the
area of the road section, this corresponds to a depth of

$$\frac{195 \times 12}{62.4 \times 38.1} = 0.98 \text{ in,}$$

38.1 being the area of the road section in sq ft. The heave due to the intake and
freezing of this depth of water would be 1.09 inches, assuming that no water was
absorbed into the air voids. Thus the total computed heave in the subgrade would be
1.09 + 0.48 in or 1.57 in compared with the measured heave of 0.71 in (Table 8).
It can be concluded therefore, as might be expected, that much of the water taken up
by the soil due to the suction generated by the freezing process was fed into the air
voids and thus did not contribute to the heave. (The air voids content was 8 per cent
when the subgrade was placed.)

In Test 3 the heave computed from the expansion of the water initially contained
in the soil was 0.50 in, but in this test a total of 67½ gallons of water was added to
maintain the level of the water-table during freezing. Assuming no migration of the
water into the air voids this would have given rise to a heave of 3.6 in and a total
computed heave in the subgrade of 4.10 in. This compares with the measured heave of 3.02 in (Table 8). Again therefore it appears that about one third of the potential heave due to moisture migration was absorbed in the air voids.

7. CONCLUSIONS

1. Heave of the road structure, due to the uptake of water from the water-table, and loss of strength of the subgrade on thawing occurred only when the water-table was closer to the surface than 4 feet. From the results it appears that the heave and loss of strength would have been appreciable only when the water-table was closer to the surface than 3 feet.

The Road Research Laboratory frost heave test when applied to the subgrade material, Fig. 19 (a), indicated that it would be classified as frost-susceptible, the heave after 250 hours being between 0.60 and 0.75 in depending on the dry density. (The limiting criterion adopted is 0.5 in.) Heaves in this range are greater than would normally be measured on clay soils although on more silty materials values in excess of 1.5 in have been measured. Until further tests have been completed with other subgrades of more silty type it is not possible to say whether the conclusion stated above can also be applied to such soils. It should however be safe to apply it to any soil having a liquid limit in excess of 30 per cent (the liquid limit of the soil used in this test), and most British soils have liquid limits higher than this. 6

It must be stressed that the same conclusion could not be applied to chalks or to other formations made of materials such as shale. The unsaturated permeability of such materials is often very great and heave can occur when the water-table is at a considerable depth. (In tests recently completed it was shown that the magnitude and rate of heave of a soft chalk, in the Road Research Laboratory freezing test was largely independent of the depth of the water-table over the depth range 6 inches to 2 feet.)

The practical implication of the above conclusion is that if adequate steps are taken to keep surface water out of the road structure and to provide subsoil drains to maintain the water-table at a depth of 3 ft or more below road surface level, there is little likelihood of serious frost heave or loss of subgrade strength in cohesive soils of liquid limit in excess of 30 per cent.

2. No significant heave occurred in the wet-mix base material, which was defined by the Road Research Laboratory frost heave test as non-frost susceptible, Fig. 19(b). However during freezing there was some migration of moisture from the subgrade into the base. The magnitude of this movement was not apparently influenced by the level of the water-table, but appeared to be more associated with the initial moisture content of the wet-mix material—a factor which would materially affect the permeability of such a material.

3. Although the heave in the base and the uptake of water were both small, a loss of strength of between one-half and two-thirds of the original strength, occurred immediately after the material thawed. Although further tests using granular materials defined by the R.R.L. test as frost susceptible are necessary, it appears reasonable to suppose that with such materials the loss of strength on thawing would have been greater. It is important not to exaggerate the importance of this loss of
strength, which is only of a temporary nature. The fact however that it occurs in practice has been checked by studies, before and after freezing, of the transient deflection under a heavy wheel-load of normal roads having wet-mix bases. For several weeks after prolonged freezing the deflection may be 50 per cent greater than before freezing. This does not however appear to have been the immediate cause of any failures except in cases where a temporary surfacing of inadequate strength or thickness has been employed.

Freezing of granular sub-bases would result in a similar loss of strength and the imposition of increased stresses in the surfacing, base and subgrade. Until more is known about the practical importance of such increases in stress for the types of pavement now being employed it appears to be desirable to maintain the present requirements for non-frost susceptible sub-bases.

8. ACKNOWLEDGEMENT

The work described in this report was carried out under the direction of Mr. D. Croney, head of the Pavement Design Section.

9. REFERENCES


2. CRONEY, D. Some cases of frost damage to roads. Department of Scientific and Industrial Research, Road Note No. 8, London, 1949 (H.M. Stationery Office).


PLATE 1

General view of large chamber showing gantry used to support in-situ C.B.R. apparatus
PLATE 2

Perforated brass tubes at the bottom of large chamber
PLATE 3
Gravel-filled drainage tubes in subgrade
PLATE 4
Some asphalt blocks in position on the surface of the compacted wet-mix base

PLATE 5
The surface of the completed road structure
PLATE 6
Apparatus used for determining resistance of thermistors
Fig. 1. CONCRETE-LINED PIT CONSTRUCTED BENEATH FLOOR OF COLD ROOM, SHOWING PRINCIPAL DIMENSIONS
Fig. 2. PARTICLE SIZE DISTRIBUTION OF THE SUBGRADE SOIL
Fig. 3. PARTICLE SIZE DISTRIBUTION OF THE WET-MIX LIMESTONE BASE
Fig. 4. MOISTURE DISTRIBUTION IN SUBGRADE AS LAID
Fig 5. DETAILS OF THE ROAD STRUCTURE AND THE POSITIONS AT WHICH DRAINAGE TUBES WERE INSTALLED IN THE SUBGRADE
Fig. 6. DETAILS OF CONSTRUCTION OF WATER-PROOF THERMISTOR CASE AND CIRCUIT DIAGRAM FOR MEASURING RESISTANCE OF THERMISTORS
Fig. 7. TYPICAL CALIBRATION CURVE FOR THERMISTOR
Fig. 8. VERTICAL LONGITUDINAL SECTION SHOWING THE POSITIONS AT WHICH THERMISTORS WERE INSTALLED IN THE ROAD STRUCTURE
FIG. 9. MEASUREMENTS OF TEMPERATURE, PORE WATER PRESSURE AND HEAVE-TEST: 1. WATER-TABLE 4 FEET BELOW SURFACE

(a) Temperatures in road structure during freezing and thawing periods

(b) Depth below road surface of zero isotherm during freezing and thawing periods

(c) Pore water pressure in soil at a point 2 feet below road surface before freezing and after thawing

(d) Average heave of road surface during test

Thermistors

<table>
<thead>
<tr>
<th>Depth (inches)</th>
<th>Condition of Wet-mix limestone base</th>
<th>Condition of Base</th>
<th>Moisture profile of subgrade</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6 in</td>
<td>1 ft-0in.</td>
<td>1 ft-0in.</td>
</tr>
<tr>
<td>2</td>
<td>1 ft-0in.</td>
<td>3 in</td>
<td>3 ft-6in.</td>
</tr>
<tr>
<td>3</td>
<td>3 in</td>
<td>6 in</td>
<td>6 ft-0in.</td>
</tr>
<tr>
<td>4</td>
<td>6 in</td>
<td>9 in</td>
<td>9 ft-0in.</td>
</tr>
<tr>
<td>5</td>
<td>9 in</td>
<td>12 in</td>
<td>12 ft-0in.</td>
</tr>
<tr>
<td>6</td>
<td>12 in</td>
<td>15 in</td>
<td>15 ft-0in.</td>
</tr>
<tr>
<td>7</td>
<td>15 in</td>
<td>18 in</td>
<td>18 ft-0in.</td>
</tr>
</tbody>
</table>

Temperatures during thaw are shown to larger scale on Fig. 10.

Cold room temperature temporarily lowered to retard thaw.

C.B.R. tests on fully thawed base 4 Nov.


Moisture profile of subgrade 17 Nov.

Moisture profile of subgrade 20 Nov.


C.B.R. tests on partly drained base 15 Nov.

C.B.R. tests on partly thawed base 17 Nov.

C.B.R. tests on partly thawed base 18 Nov.

C.B.R. tests on partly thawed base 15 Nov.

C.B.R. tests on fully thawed base 4 Nov.

C.B.R. tests on partly thawed base 20 Nov.

Moisture profile of subgrade 17 Nov.

Moisture profile of subgrade 20 Nov.

Temperatures during thaw are shown to larger scale on Fig. 10.
Cold room temperature varied near 0°C to retard thaw.

FIG. 10. TEMPERATURES IN THE ROAD STRUCTURE DURING THAW-TEST. 1. WATER-TABLE 4 FEET BELOW SURFACE
Moisture content (per cent)

0  2  4  6  8  10

Mean result

of 2
results
(10 days
after
thaw)

0
1
2
3
4
5
6
7
8
9
10

Test 1

Mean of 2
results
(15 days
after
thaw)

Mean of 3
results

Test 2

Initial moisture
content

Frozen

After thawing
and draining

Test 3

Mean of 2
results
(15 days
after
thaw)

Mean of 2
results

Fig. 11. MOISTURE DISTRIBUTION THROUGH WET-MIX LIMESTONE BASE
Fig. 12 MOISTURE DISTRIBUTION THROUGH SUBGRADE - TEST 1
Fig. 13. MOISTURE DISTRIBUTION THROUGH SUBGRADE - TEST 2
Freezing period

Air temperature fluctuating between -17°C and -11°C due to compressor fault

Thawing period

Thaw commenced

FIG. 14. MEASUREMENTS OF TEMPERATURE, PORE WATER PRESSURE AND HEAVE-TEST.
FIG. 15. EFFECT OF BOUNDARY CONDITIONS ON TEMPERATURE AND DEPTH OF ZERO ISOTHERM—TEST.2.
Fig. 17. MOISTURE DISTRIBUTION THROUGH SUBGRADE — TEST 3
Fig. 18. DEPTH OF ZERO ISOTHERM IN TESTS 1–3 COMPARED WITH MEASURED DEPTH UNDER CONCRETE ROAD DURING WINTER 1962/63
Fig. 19. LABORATORY FROST SUSCEPTIBILITY TEST ON SAMPLES OF SILTY CLAY AND WET MIX LIMESTONE USED IN ROAD STRUCTURE