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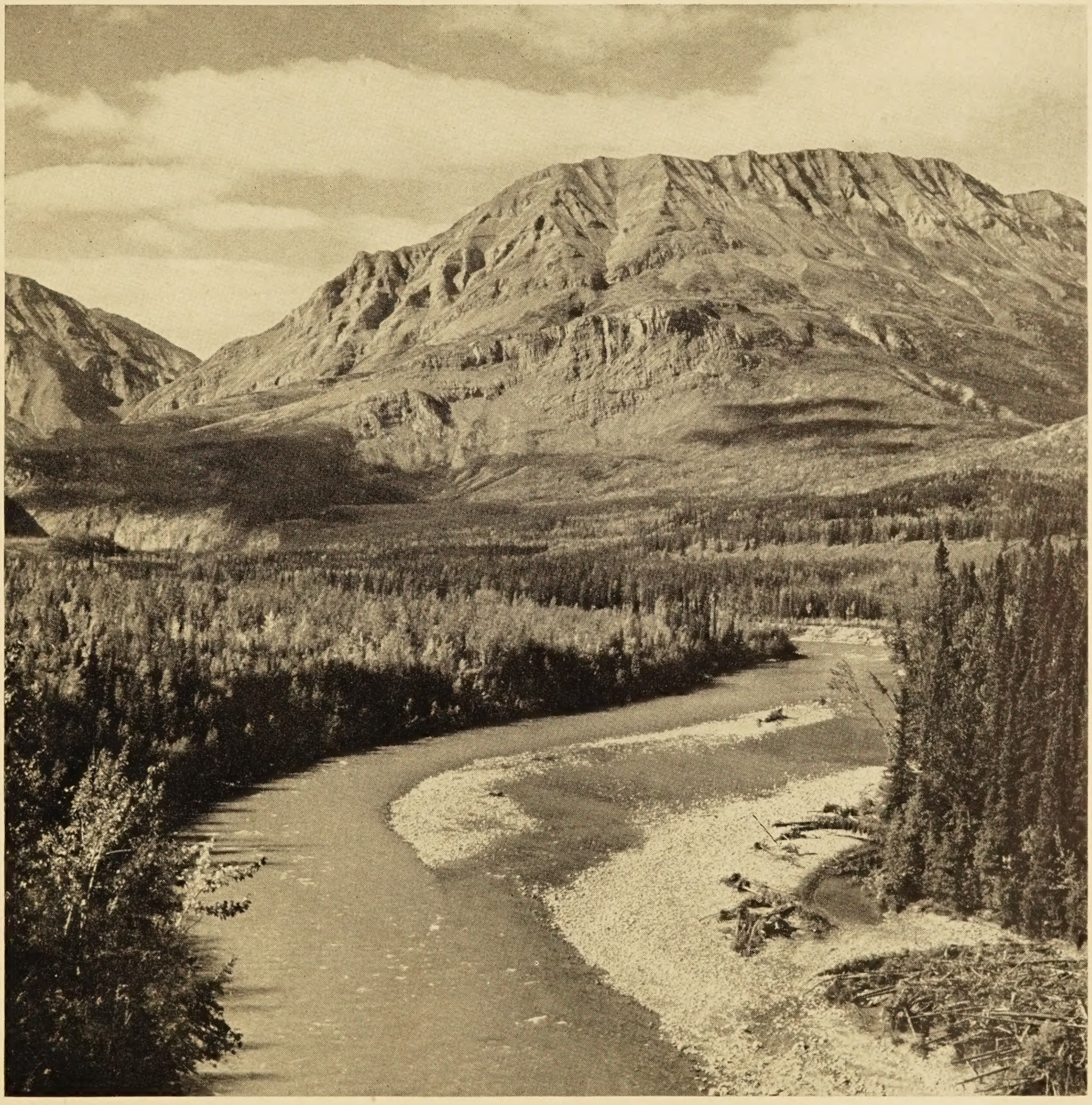

MOUNTAINS AND FORESTS, IMPENETRABLE ON FOOT, ARE READILY SURVEYED FROM THE AIR


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E. A. STROMBERG, *Editor*

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## *In This Issue*



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# AERIAL SURVEYING ON THE ALASKA HIGHWAY, 1942

Reported by WILLIAM T. PRYOR, Highway Engineer,[1] Department of Design, Public Roads Administration

THE ALASKA HIGHWAY crosses what was in 1942 perhaps the largest unmapped wilderness left on the North American Continent. The highway begins in Canada at the end of the 495-mile railroad reaching north and west from Edmonton, Alberta, to Dawson Creek in British Columbia. From Dawson Creek the highway extends in a northwesterly direction through British Columbia and Yukon Territory into Alaska where, at Big Delta, it joins the Richardson Highway, an existing road leading northwest about 100 miles to Fairbanks and southerly to Valdez. (See fig. 1.)

Work on the highway began in 1942 at a time when the Japanese were gaining a foothold in the Aleutian Islands and the American Continent was thought to be in danger of invasion by way of Alaska.

The agreement [2] between Canada and the United States called for the construction of a highway along a route joining the airports at Fort St. John, Fort Nelson, Watson Lake, Whitehorse, Boundary, and Big Delta; the respective termini connecting with existing roads in Canada and Alaska. The airports had been located at intervals along the best all-year flying route across the vast subarctic region between the principal centers in Canada and the United States and Fairbanks.

Completion of the highway in the shortest possible time was imperative, in the face of innumerable obstacles. Difficulties arose at the very beginning, for both the Public Roads Administration and the Army Corps of Engineers discovered that there did not exist any detailed maps showing the topography of the vast area through which a route had to be found—an expanse 1,500 miles long and 100 to 250 miles wide, comprising about one-quarter million square miles in the subarctic region.

The only maps available, covering the headwaters of the Mackenzie and Yukon Rivers, lacked planimetric detail and accuracy. They were at scales of 1:100,000 to 1:2,000,000 and even smaller, and little could be learned from them about the topography and general character of the area.

Trappers, Indians, and "bush" pilots, and others who were likely to be familiar with any part of the area along the projected route, were interviewed. The descriptions they gave only confirmed what was already known—that there would be found areas of muskeg (sphagnum swamp), high mountain ranges, permanently frozen soil, dense forests which everywhere obstruct the view and hinder the traveler, countless lakes, and numerous rivers which would be difficult to cross.

The airport locations, known by latitude and longitude, and the small-scale maps showing general locations of rivers and mountain ranges, were all the engineers had in the beginning for guidance. The 1,423-mile project was split into two parts: The southern division, with headquarters at Fort St. John, extended 634 miles from Dawson Creek to a point near Watson Lake; the northern division continued from that point for 789 miles to Big Delta, and had its headquarters at Whitehorse (fig. 1).

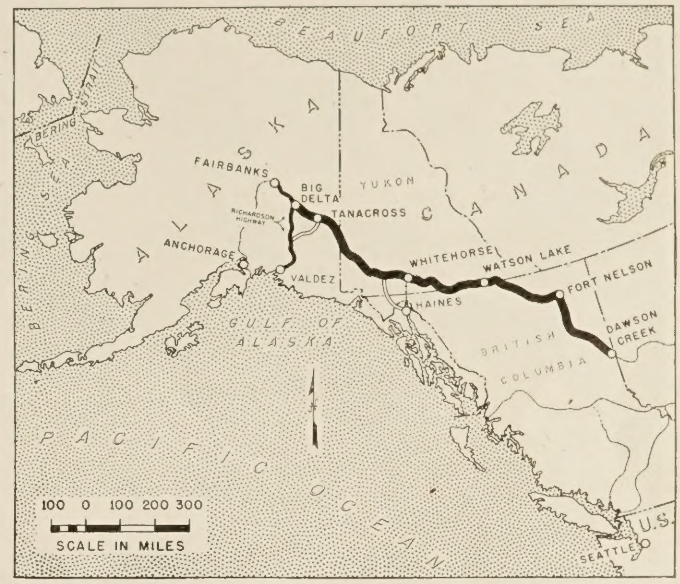


FIGURE 1.—MAP OF THE ALASKA HIGHWAY.

#### TRANSPORTATION DIFFICULTIES ENCOUNTERED

The problem of locating a highway in unmapped country is rarely encountered in the United States. As implied in the word "unmapped," there was almost complete absence of roads or even usable trails in the wilderness region through which the Alaska Highway was built. Surveying and mapping parties had little to guide them.

Transportation of men and supplies and equipment was always difficult. At the southern end of the highway a provincial road led northerly 60 miles from Dawson Creek to Fort St. John, and an old winter road which could be traveled only when the ground was frozen continued to Fort Nelson. This winter road is shown in figure 2, winding across the fairly level terrain beyond the east bank of the Prophet River.

North of Fort Nelson, overland travel was accomplished by primitive methods. Dog teams were used while snow remained on the ground and rivers and lakes were frozen over, but in summer the men often pushed forward on foot, carrying their supplies on their backs. Pack and saddle horses were sometimes used, but they were not easy to obtain and it was difficult to get them into the back country and to feed them there.

Motorboats and small wood-burning steamers were used during the summer wherever the route lay along the larger rivers and chains of lakes. A number of ferries were also placed in operation at river crossings.

[1] The author was on the staff of the construction engineer for the southern division of the Alaska Highway and was in charge of design during the period of operations covered by this article.
[2] Executive Agreement Series 246 of the Department of State.


FIGURE 2.—AN OLD WINTER ROAD CONNECTING FORT ST. JOHN AND FORT NELSON. ICE ON THE RIVERS AND LAKES CAN BE SEEN IN THIS AERIAL VIEW.

Only land-based airplanes flying from the existing airfields, or float planes flying from lakes or rivers (ski planes were used in winter) were free to come and go at will. Even they were handicapped at times by limited fuel supplies and bad weather. Aircraft were used to transport the field parties and their equipment to advance bases whenever possible.

HARSH CLIMATE FACED

The average annual temperature along the highway ranges from 35° Fahrenheit at Fort St. John to 26° at Fairbanks, as compared to 50° or more in the United States. Temperatures range from a high of 105° to 76° below zero. The winters are cold, long, and dry;

spring and fall are very short; and summer, from early June to September, is a cool wet season.

Daylight in midwinter lasts about 6 hours. Subzero temperatures prevail for weeks at a time, to be broken by chinook winds which raise temperatures to above 40° for a few days.

In summer there are as many as 20 daylight hours, and nights are short twilight periods. During early summer the midges, biting flies, and mosquitoes are extremely annoying pests.

Housing of survey crews was always a difficult problem, and sometimes men had to camp in the open. The tents (fig. 3) which the field parties often used for quarters, kitchen, mess hall, and offices were not comfortable nor did they afford complete protection under even usual climatic conditions of the region.

FIGURE 3.—A TENT CAMP WAS HOME AND OFFICE FOR SURVEY CREWS.

### TOPOGRAPHY A MAJOR OBSTACLE

Some idea of the nature of the country through which the Alaska Highway was located and built can be obtained from the illustrated examples that follow. For descriptive purposes the various kinds of topography are grouped in three classes, according to the difficulty they presented to the highway engineer.

*Impassable topography.*—In impassable topography a road suitable for all-year use could not be built in a reasonable length of time. Figure 4, showing a section of the Canadian Rockies about 100 miles southwest of Fort Nelson, illustrates this type of topography.

*Difficult topography.*—In this kind of topography special problems are encountered in locating and building a road. In the area north of the Canadian Rockies, shown in figure 5, A, the ground is fairly level as indicated by the numerous small lakes and the ox bows in the rivers; but there are many areas of muskeg, composed of water-saturated muck, fine silty soil, and decaying vegetation with a cover of moss, grasses, willows, and low-growing bushes. Stunted evergreens often grow scatteringly on these areas. Muskeg is an extremely


FIGURE 4.—IMPASSABLE TOPOGRAPHY—HIGH MOUNTAINS AND GLACIERS.

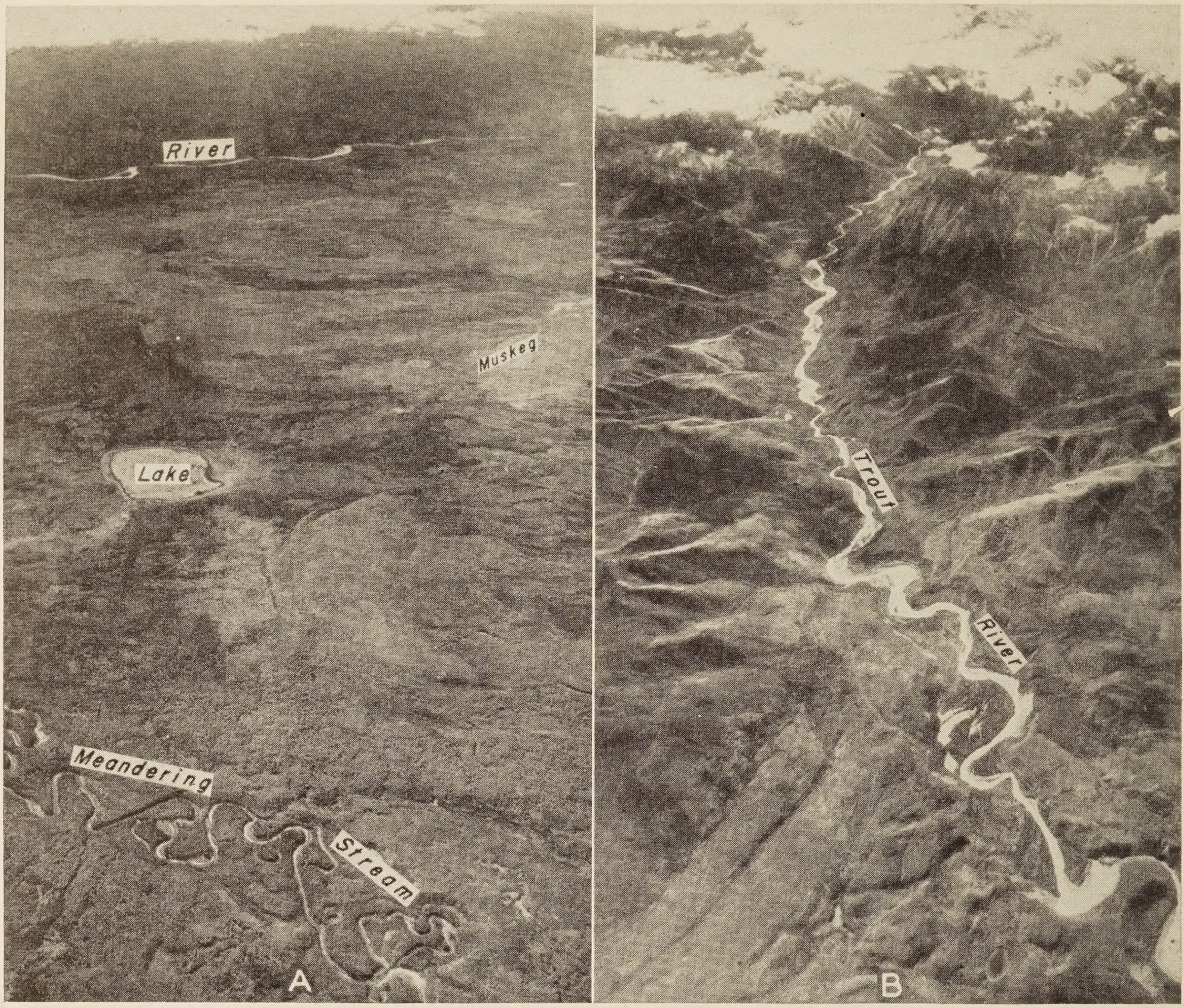


FIGURE 5.—DIFFICULT TOPOGRAPHY—(A) MUSKEG ON LEVEL GROUND, AND (B) A NARROW, TORTUOUS RIVER VALLEY.

unstable base for road building. It occurs not only in valleys and on level ground, but also on hillsides wherever water is retained. The subsoil underlying muskeg is always impermeable and frequently is permanently frozen (permafrost).

A different type of difficult topography is shown in figure 5, B, a photograph of the Trout River Valley near its confluence with the Liard. The river flows north out of the Canadian Rockies through rugged ground. The highway was built along this river valley, and tortuous alinement and steep grades were necessary. Water seeping and flowing down the slopes under a cover of vegetal debris caused slides in spring and summer and formation of ice in winter.

*Comparatively easy topography.*—Figure 6 shows comparatively easy topography in an area north of the Canadian Rockies where the ground is level to rolling. The pioneer road followed an easy route on the near side of the Liard River. Eskers and drumlins, left by the receding glaciers, are a definite indication of the presence of granular soils. Eskers are long narrow ridges of gravel or sand and silt deposited by subglacial streams. Drumlins are elongate or oval hills of glacial drift.

#### PURPOSE OF THE PIONEER ROAD

Plans for construction of the Alaska Highway required that a route connecting the airports designated as control points be selected by the Public Roads Administration. A pioneer road was to be built by Army Engineer troops following the line selected for the final road as closely as might be practicable. This road was to be used for movements necessary in constructing the final highway.

Actually, location of the route and construction of the pioneer road began almost simultaneously and progressed concurrently throughout the construction season of 1942. Much of the pioneer road was located before the exact line of the final road had been fixed.

Construction crews were soon close at the heels of the survey parties. At the very beginning it became evident that usual methods of ground reconnaissance and survey would not suffice. The only hope of keeping up with the work schedule established to meet the war emergency lay in the use of aerial methods which permitted seeing and studying the topography far in advance of construction. Aerial methods enabled the engineers to examine the unexplored territory much more quickly and fully than was possible by ground reconnaissance, and without the difficulties and delays which hampered ground operations.

#### EVOLUTION OF AERIAL SURVEYING

The use of aircraft and aerial photography in highway location has passed through progressive stages of evolution. At first aircraft were used for observation flights over wide areas, much as reconnaissance parties might scout the topography on the ground. A logical advance was the taking of aerial oblique photographs: Bird's-eye views showing topography in perspective, much as the aerial observer sees it.

Then came the practice of taking vertical photographs in series along a predetermined line of flight. These photographs can be matched and assembled in a mosaic to form a composite picture of the area photographed. Next followed the application of the stereoscopic principle, using successive pairs of vertical photographs to see the topography in three dimensions.

The most recent development in aerial surveying, as applied to highway work, is the making of accurate large-scale contour maps from vertical photographs that have been adjusted to controls established by ground surveys.

All of these methods may be combined in their proper sequence for use in highway location. On the Alaska Highway all methods except the making of contour maps were used. Because of the haste which the exigencies of war required and the inexperience in aerial survey methods of most of the location engineers, full advantage could not always be taken of the aerial survey methods. Sometimes the survey parties, hard pressed by the pioneer road construction crews, could not wait for the taking of needed additional aerial photographs. Where the photographs were available in time, and as the engineers gained experience, the aerial survey procedure was always found to be a time-saving means of locating the most practicable route for the pioneer road.

#### RECONNAISSANCE BY AERIAL OBSERVATION

Observation flights for highway reconnaissance are generally planned from available topographic maps, but for the Alaska Highway no suitable maps existed. The flights were therefore made along compass courses between the airfields, whose positions had been determined. Lakes, rivers, mountain peaks and passes, and identifiable muskeg were observed on these flights; and the more prominent landmarks were selected as guides for aerial photographic flights and for the ground survey crews.

While akin to field reconnaissance, the aerial observation flights made it possible to see in a few hours an expanse of area which, in wilderness such as that along the Alaska Highway, could be examined on the ground only by weeks or months of arduous travel. Densely timbered areas, which were impossible to study on the ground except for occasional glimpses from a hill or treetop, could be scanned easily from the air.

#### OBSERVATION RECORDED BY OBLIQUE PHOTOGRAPHS

The Army Air Corps during the fall of 1941 took many aerial oblique photographs over a wide area between the airfields in Canada. These photographs were taken from an average elevation of 20,000 feet with a camera having a 6-inch focal length lens. Generally the photographs were high-obliques, so-called because the horizon appears in the picture.

Figure 7 is a high-oblique photograph of the Peace River near its emergence from the Canadian Rockies west of Hudson's Hope. Study of such photographs along the river showed that the valley might have been a feasible route for the highway, but the final location was chosen to follow the line of airfields.

A valuable development evolved from the taking of high-obliques is trimetrogon photography, producing left and right high-obliques combined with a vertical between them. A set of photographs, taken simultaneously by three 6-inch lens cameras from an altitude of 20,000 feet, provides horizon-to-horizon coverage. The effectively useful portions of the photographs cover an area 25 to 50 miles wide. In figure 8 a set of trimetrogon photographs, matched together, shows a mountain divide between two broad river valleys. Study of trimetrogon photographs taken along this divide would aid in choosing a pass between the river valleys for more detailed aerial examination. The Air Corps took many of these trimetrogon photographs as well as single high-obliques.

The oblique photographs recorded for further use what the observers had seen during their reconnaissance

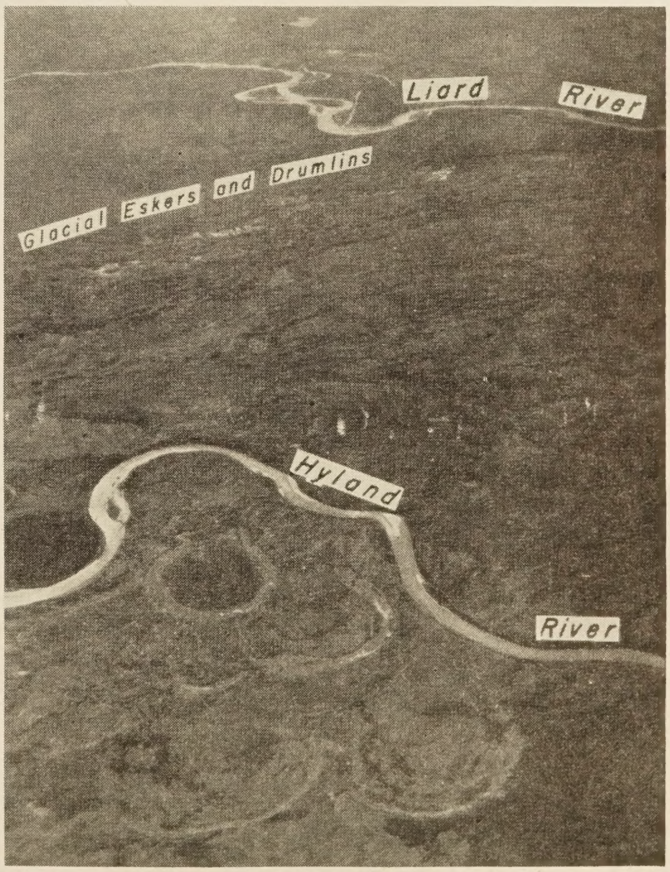


Figure 6.—Comparatively Easy Topography—Rolling Ground with Glacial Deposits.


Figure 7.—The High-Oblique Photograph Gives a Perspective Picture.

Figure 8.—Trimetrogon Photographs Have Horizon-to-Horizon Coverage.


FIGURE 9.—A LARGE-SCALE VERTICAL PHOTOGRAPH; SCALE 4 INCHES PER MILE.

flights. Possible routes for the highway could be selected from them and impassable obstacles avoided. Flight lines for taking strips of vertical photographs were planned along tentatively chosen routes and the pilots were guided on such flights by identifiable landmarks on the obliques.

#### DETAILED STUDIES MADE WITH VERTICAL PHOTOGRAPHS

During the spring and summer of 1942 the Army Air Corps took aerial vertical photographs on flight lines along many of the possible routes observed in aerial reconnaissance or on the high-oblique photographs.

The verticals were taken with a 6-inch lens camera, generally from an altitude of 10,000 feet. Taking into account the average elevation of the topography, such vertical photographs had a scale of about 4 inches per mile. Figure 9, showing an area along the Muskwa River, is a vertical at this scale. Slide areas and drainage channels can be seen clearly; even individual trees can be recognized.

Vertical photographs of considerably smaller scale were also taken, but were of much less value in locating the pioneer road. Figure 10 is a vertical of the area at the confluence of the Liard and Turnagain Rivers, at a scale of about 2 inches per mile. The general character of the topography can be seen, but much of the detail needed for location study is not visible.

Figure 11 shows the locations of some of the vertical photographs taken on flights along possible routes chosen for detailed study in the area south and west of Fort Nelson—only every other photograph of each flight line is represented, and some flight lines are omitted altogether, in order to avoid crowding. The overlaps and gaps in the flight lines reflect the haste with which the photography was necessarily done, and the inability to plan the work completely in advance because of the lack of suitable maps such as would be available for similar work in the United States.

Uncontrolled mosaics were made simply by matching the conjugate images of adjacent vertical photographs, as shown in figure 12. The pioneer road had already been pushed through this area, and appears as a white line on the edges of some of the photographs. An uncontrolled mosaic made from verticals taken on several

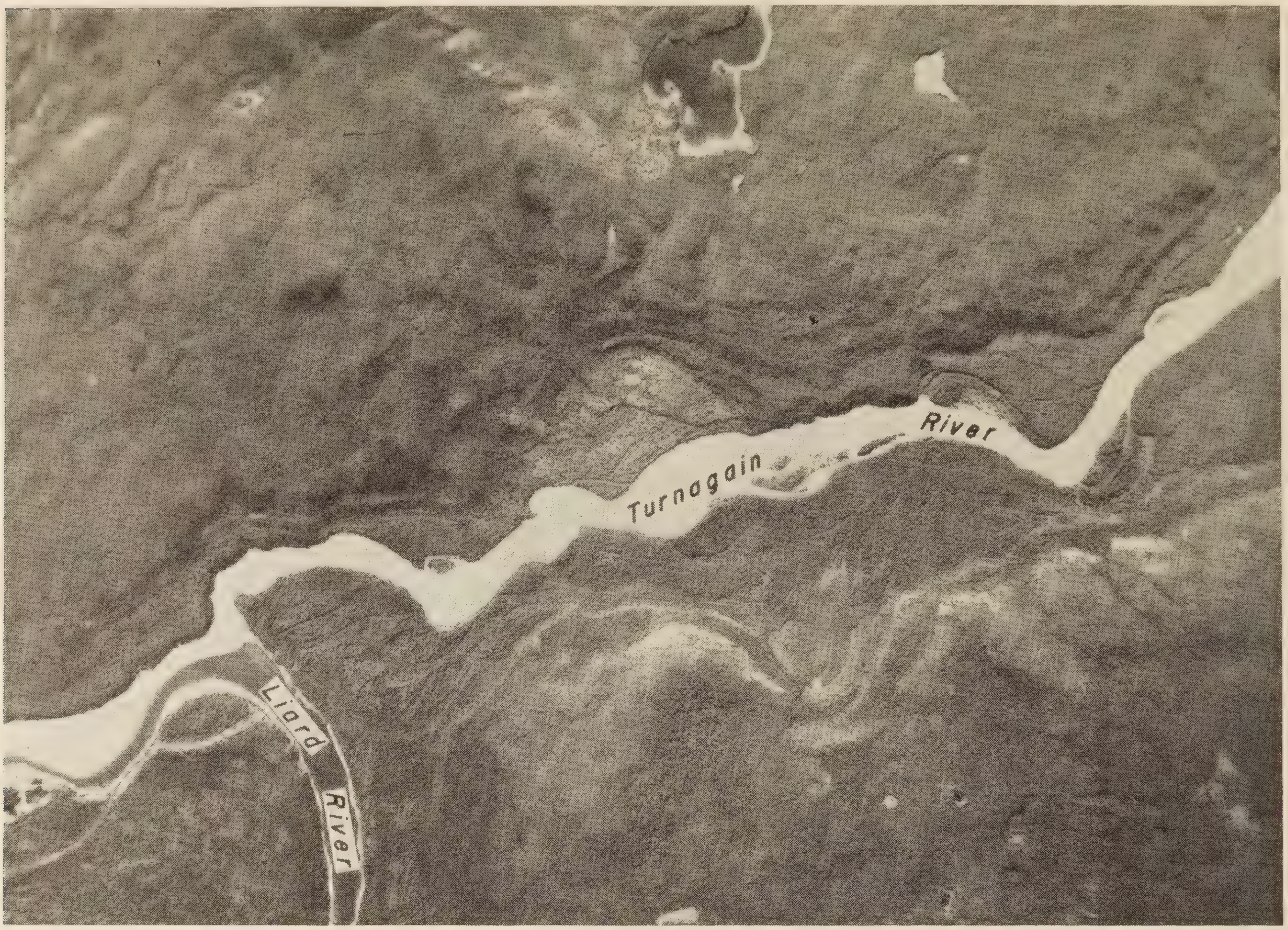


Figure 10.—A Small-Scale Vertical Photograph; Scale 2 Inches per Mile.

overlapping flights is shown in figure 13. These pictures show the area at the confluence of the Tetsa and Muskwa Rivers.

Controlled mosaics to accurate scale could not be made on the Alaska Highway. The shortness of time and the great difficulty of making ground surveys in the wild and inaccessible area made it impossible to establish the precisely located ground controls necessary for the assembly of controlled mosaics.

On the Alaska Highway the uncontrolled mosaics were usually assembled by the location engineers in their field camps. With a mosaic in hand, the engineer could locate his ground position and orient himself by compass bearings on prominent landmarks identifiable in the photographs. Distances could be scaled only approximately but planimetric accuracy at this stage of the location work was less important than finding suitable crossings of rivers and mountain passes, choosing between ridge and valley routes, locating passages across or around muskeg, and identifying soil and drainage conditions.

The aerial photographs for use in location of the Alaska Highway were taken with a 6-inch focal length lens camera, but experience gained by the War Department during World War II indicates that photographs taken with a 24-inch lens have a number of advantages for this type of work. With the longer focus lens photographic flights at an altitude of 14,400 feet will provide photographs at a scale of 600 feet per inch, permitting direct measurement of distances with an engineer's "60" scale. Scaling is not precise but is more nearly accurate than on the photographs taken with a 6-inch lens because the displacement of images due both to tilt of the plane and to variation in ground relief is reduced by the higher altitude of flight and the longer focal length of lens. The photographs cover an area about a mile wide on the standard 9- × 9-inch negative and all essential ground details are clearly distinguishable. The 14,400-foot altitude is advantageous for flying since it is above the roughest air zones yet is not so high as to require the use of oxygen masks.

### STEREOSCOPE PROVIDES PERCEPTION OF DEPTH

A photograph or mosaic, being a plane surface, has only two dimensions. Any simulation of a third dimension, which may occur to varying degrees, results from the perspective features of the picture and the subconscious interpretation of recognizable indicators of depth such as shadows.

An actual object or area is seen by the two eyes from slightly different angles, and this converging dual sight results in stereoscopic vision—the full perception of depth. If two photographs of an object taken from different camera positions are examined

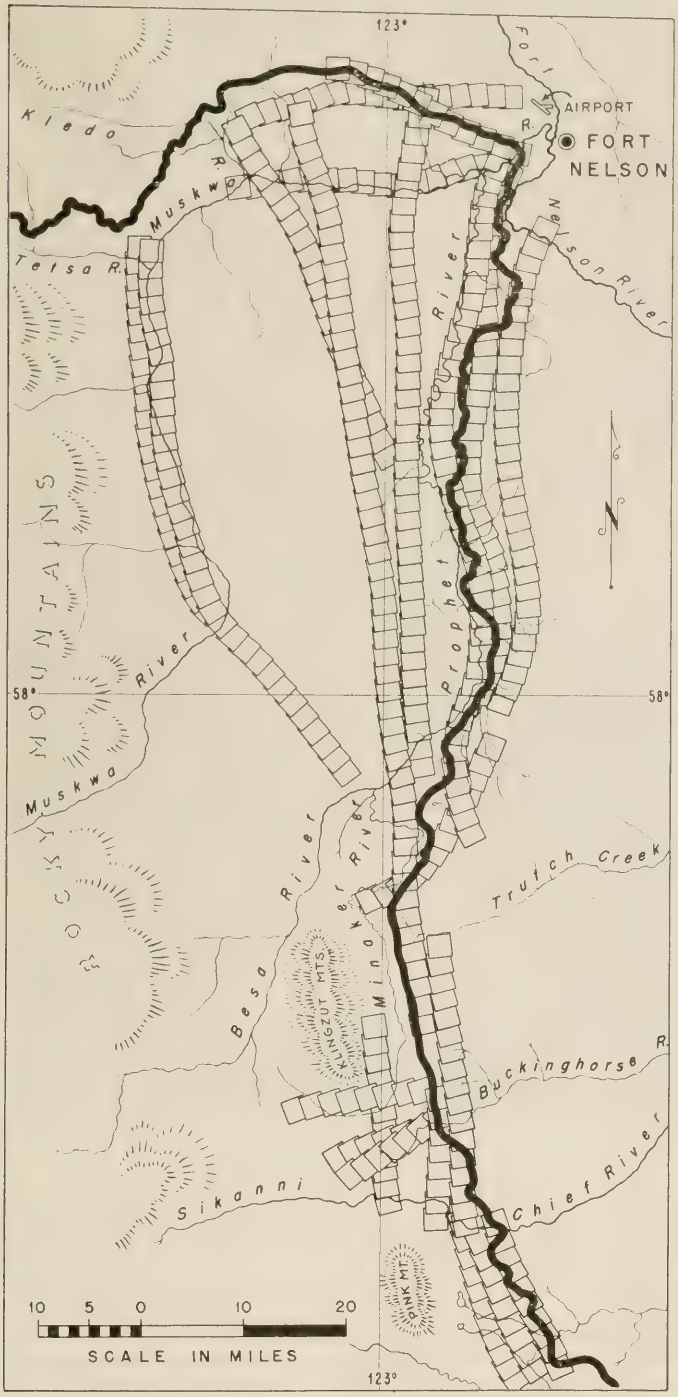


FIGURE 11.—LOCATION OF SOME OF THE FLIGHT LINES AND VERTICAL PHOTOGRAPHS TAKEN ALONG THEM ON ONE SECTION OF THE ALASKA HIGHWAY.

through a stereoscope, the effect of two-eye vision of the object is produced. Under the stereoscope the photographs seem to fuse together and take on the appearance of an actual three-dimensional model.

The topographic relief of an area may be studied in detail by the stereoscopic examination of aerial vertical photographs, and in this way the fullest use is made of the photographs for highway location work. Complete appreciation of this can be gained only by actually using a stereoscope.

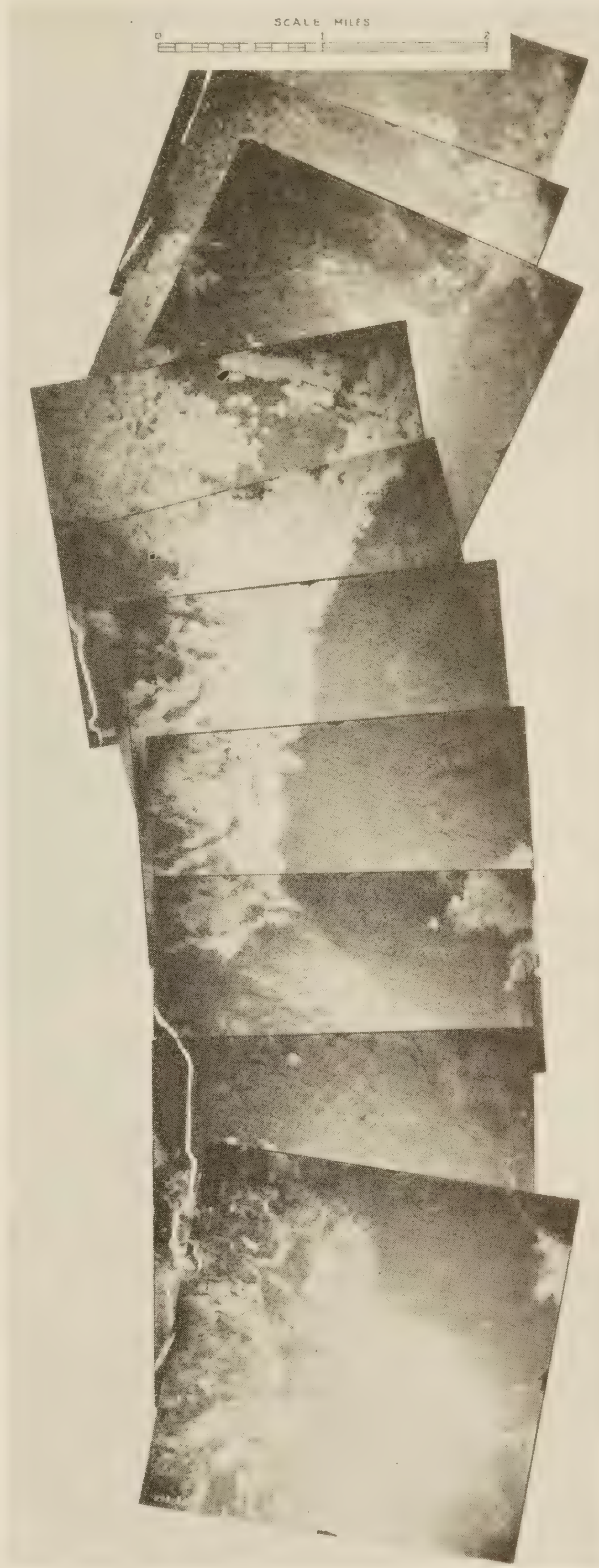


FIGURE 12.—AN UNCONTROLLED MOSAIC MADE BY MATCHING A SERIES OF VERTICAL PHOTOGRAPHS.

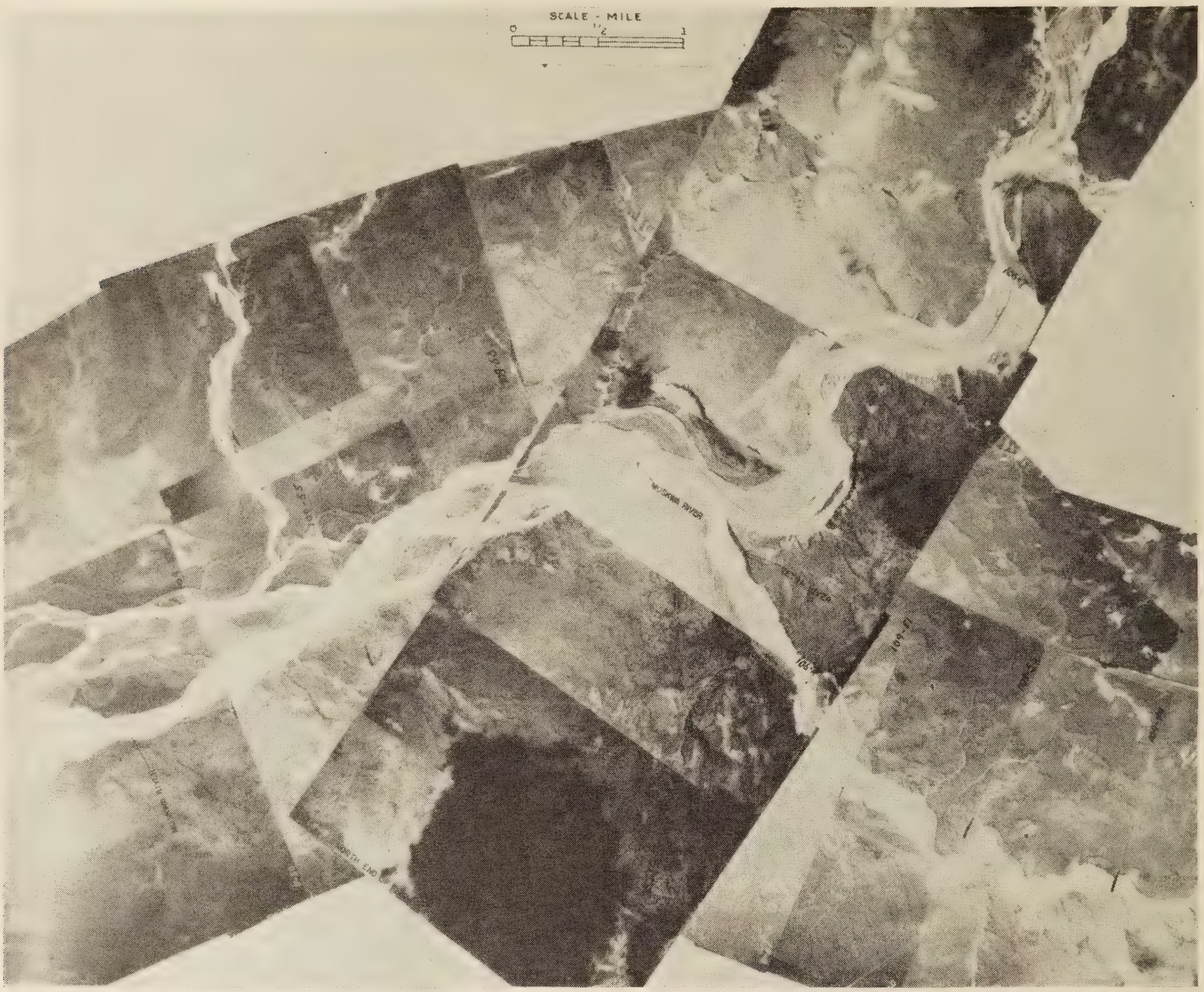


FIGURE 13.—AN UNCONTROLLED MOSAIC MADE OF PHOTOGRAPHS FROM SEVERAL OVERLAPPING FLIGHTS.

The parlor stereoscope, familiar to everyone a generation ago, is an application of stereoscopic observation developed largely for instructive entertainment. Several types of instruments, such as the simple lens stereoscope shown in figure 14 set up over a pair of photographs, are now available for engineers. When properly oriented, spaced, and mounted the pair of photographs, covering the same area but taken from different camera positions, is called a stereogram.

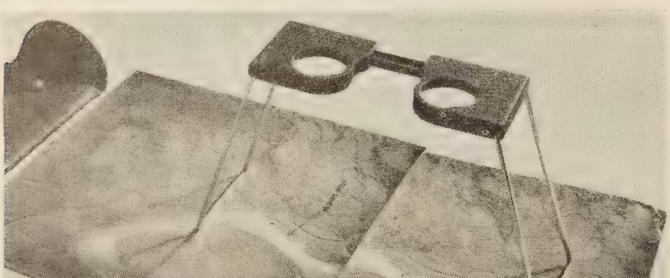

FIGURE 14.—AN ENGINEER'S STEREOSCOPE—AN AID IN SEEING PICTURED TOPOGRAPHY IN RELIEF.

A typical stereogram is shown in figure 15. The two larger circles represent the centers of the photographs—it was from above these points that the pictures were taken. The smaller circle drawn on the left photograph marks the transferred center of the right photograph. A mountainous area, much of it above the timber line, is shown in this stereogram. Though an idea of the direction of slopes can be guessed from the shadows, no real judgment of relative height or depth can be made until the three-dimensional image is seen under the stereoscope.

Some of the field engineers working on the Alaska Highway developed an ability to examine pairs of vertical photographs stereoscopically without the aid of an optical instrument. They could thus do their work more expeditiously and did not have to carry the lens stereoscope.

This ability to see stereoscopically without optical aids can be acquired by practice. At first, a cardboard


FIGURE 15.—A TYPICAL STEREOGRAM—A PAIR OF PHOTOGRAPHS ORIENTED FOR STEREOSCOPIC STUDY.

10 or 12 inches high should be placed on edge along the dividing line between the two photographs of the stereogram. A well-defined feature should be selected as a focal point: For example, the small lake in figure 15 which appears below the circled center in the right photograph and the circled transferred center in the left photograph. The forehead is rested on the top of the cardboard, with each eye looking at the selected point on the photograph beneath it. Both eyes must be focused as though looking beyond the plane of the stereogram until the chosen points appear to fuse together. The entire area within view will then be seen in three dimensions.

Some adjustment in the spacing and orientation of the two photographs and in the height of the eyes above them may be necessary. After some practice the cardboard may be dispensed with. Photographs are best studied stereoscopically under uniform lighting. They should be oriented so that the shadows in them incline toward the observer.

#### SOIL AND DRAINAGE CONDITIONS INTERPRETED

Photographs appear in realistic detail when seen stereoscopically. The valleys become depressions, peaks and ridges rise in full height, trees reach skyward, and all topographic features fit into their proper elevational position. The location engineers can see where routes might feasibly be placed; where obstacles would bar the way.

An additional and equally important purpose served by stereoscopic study of aerial photographs is the recognition of soils and drainage conditions from their relation to vegetation and topography, as shown in table 1. The engineers on the Alaska Highway had little knowledge of these relations at the beginning of the work and, in fact, none had been established for the subarctic region. These relations could be determined only by examination of the aerial photographs at identifiable places on the ground where the drainage conditions could be directly observed and the soils sampled and tested.

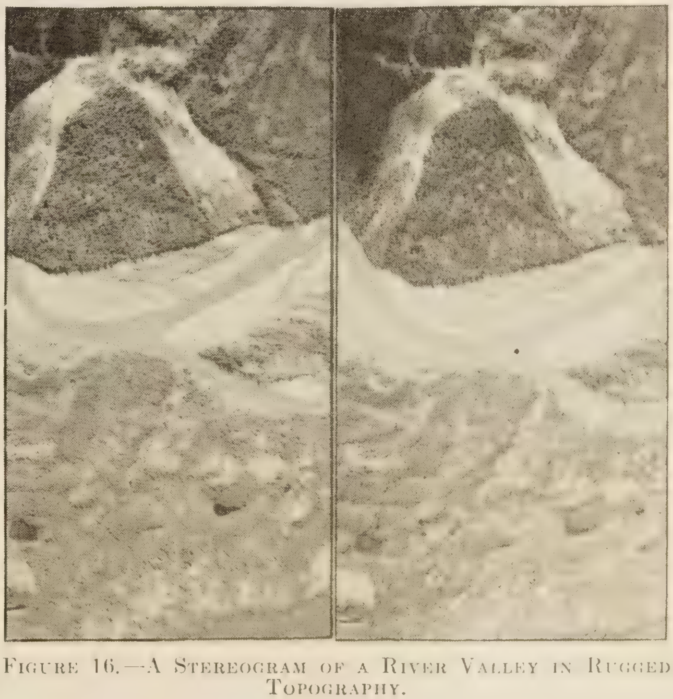

FIGURE 16.—A STEREOGRAM OF A RIVER VALLEY IN RUGGED TOPOGRAPHY.

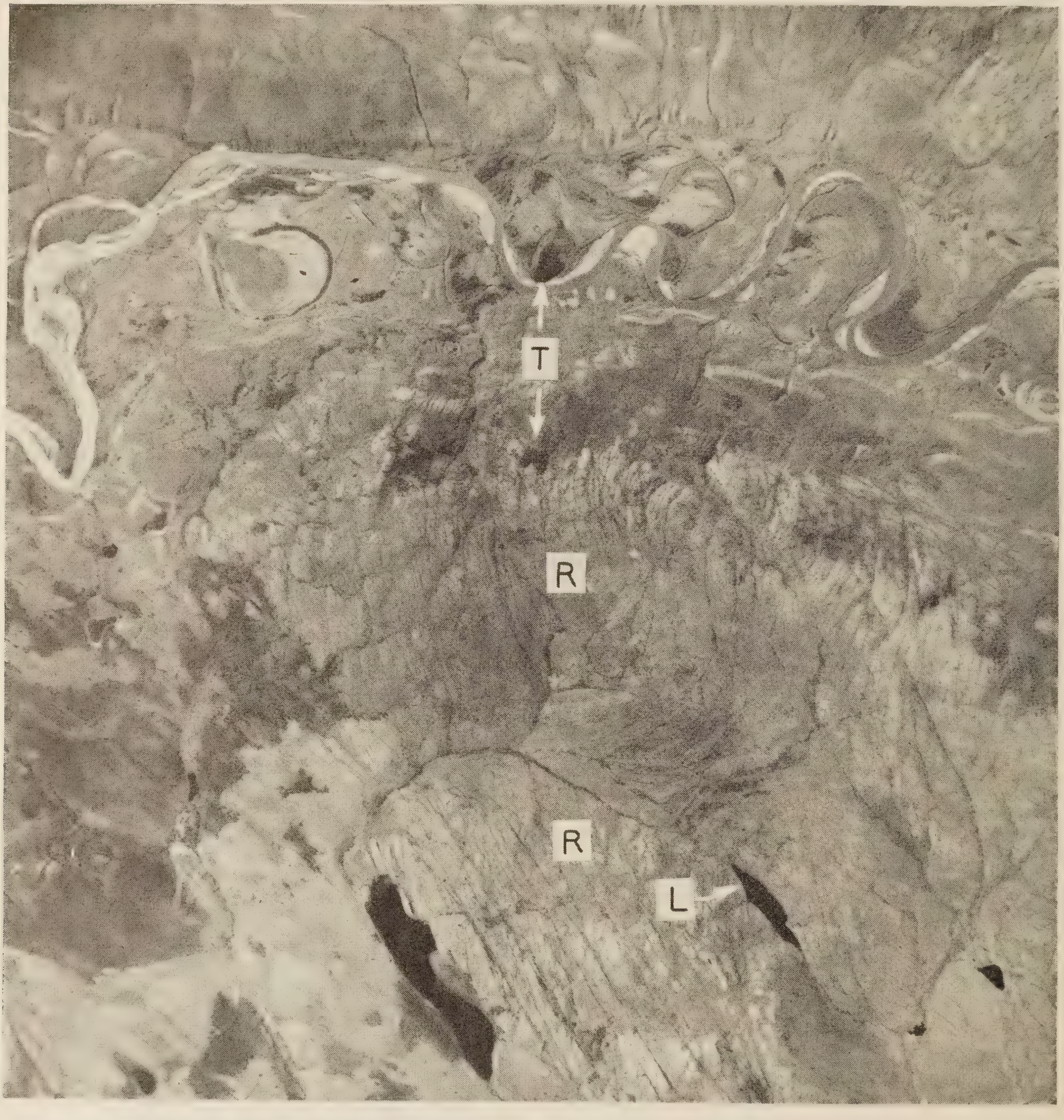


FIGURE 17.—GLACIATED, RUGGED TOPOGRAPHY. MUCH OF THE BEDROCK IS EXPOSED

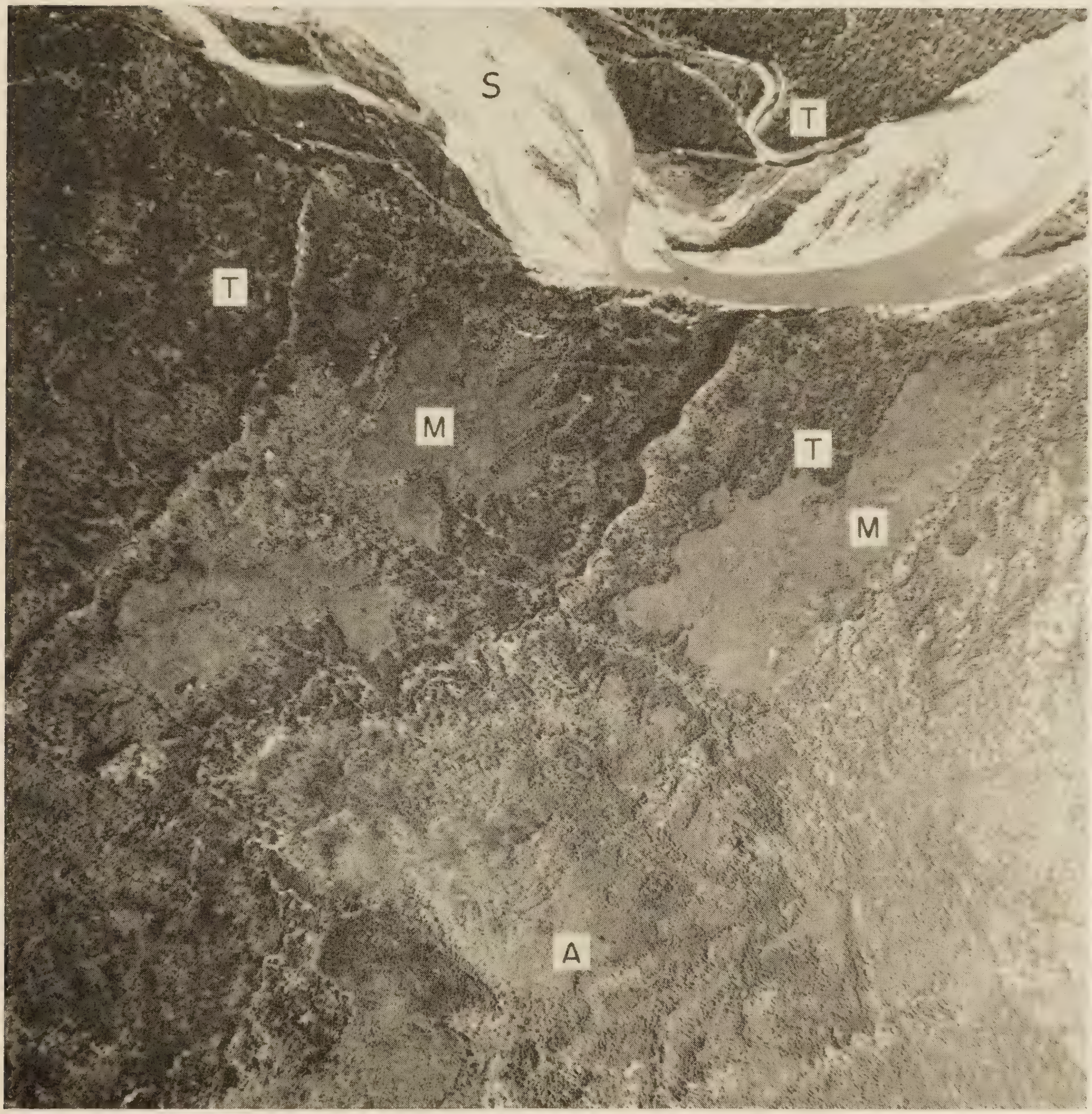


FIGURE 18.—A LEVEL PLATEAU CUT ACROSS BY A RIVER VALLEY.

TABLE 1.—*Vegetation as an indicator of soils and drainage* [1]

| Type of vegetation | Soil | Drainage |
|---|---|---|
| Scrub spruce, willows, dwarf birch, bushes, moss. | Muskeg (sphagnum swamp), acid soil, impermeable base materials.[2] | Poor drainage or standing water. |
| Willows, birch, alder | Sandy, silty, or sandy and gravelly. | Poorly drained or stream watered. |
| Balsam, cottonwood, white spruce. | River and stream deposits of silt, sand, and gravel. | High water table, wet sloughs between ridges of ground. |
| Evergreens (mostly white spruce) of large size for the locality. | Firm soils, mostly impermeable, on uneven but not rugged relief. | Drainage rather poor. If well watered, the soil may be granular and drainable. |
| Quaking aspen, in low relief areas. | Clay and silt, or lacustrine clays. | Muddy if worked. Water table is close to ground surface. |
| Quaking aspen, in rolling relief and on south slopes. | Permeable soils, gravelly sand and silts or clay. | Drainable but often damp or wet. |
| Jack pine | Sandy and gravelly, relief is usually uneven. | Dry or drainable. |
| Mixed evergreens of spruce, fir, and pine on glacial deposit areas. | Granular soils of gravel, sand and silt. | Permeable and drainable. |

[1] The indications given are based on a study of average topographic conditions, and are subject to local variations. Aspen and jack pine tend to come in on all burned over areas in stage reforestation. Jack pine tends toward the more dry and granular soils, and aspen to south exposures. On very wet areas both types are soon replaced by other kinds of growth.
[2] Impermeable base in British Columbia may be fine clay and silt; in Alaska and Yukon it is usually permanently frozen ground.

Gradually experience was accumulated and the location engineers learned to interpret the nature of soils and drainage from the types of vegetation and topography seen stereoscopically in the vertical photographs. Some of these interpretations are illustrated in the examples that follow.

The stereogram, figure 16, shows a river valley in rugged topography. On the far side of the river, tall evergreens are growing on a slide area. The slide indicates unstable ground. On the near side, the steep banks sloping down to the river are covered with aspen and small evergreens. The small lakes lying on the shelf halfway up the slope are a sign of stable soil.

The vertical photograph shown in figure 17 covers an area roughly gouged by the ice-age glacier. A large part of the area pictured is exposed solid rock, *R*. The clear-water lake, *L*, appears dark because of aquatic plant growth which absorbs light. Scattered evergreen groves, *T*, grow in deposited soil, well watered but not undrainable, and usually of sandy or gravelly nature. The tallest timber (mostly spruce) is at the river bends and ox bows and indicates granular, watered soil. The meandering character of the river signifies a nearly level flow line. Between the river and the lakes there is a high rock divide.

The vertical photograph, figure 18, shows a level plateau above a river valley. White areas in the river bed, *S*, are sand, silt, and gravel, flooded during high water but dry when the picture was taken. The tallest evergreens, *T*, grow along the river where there is well watered but drainable granular soil. Aspen groves, *A*, with some scattered evergreens signify good drainage though the soil may not be granular. A ground view of this type of vegetation on rolling topography is shown in figure 19, A. Here the soils are likely to be permeable and drainable and usable for road building. Aspen often appears as a stage of natural reforestation after a forest fire.

The light-colored areas, *M*, in figure 18 are muskeg. The few trees growing within these areas are small and scattered. Muskeg is a certain indication of poor drainage, a considerable covering of organic material, and an impervious underlying base. The sticky ooze exposed whenever the muskeg cover is cut can be seen in figure 19, B. The pioneer road construction crews avoided muskeg whenever they could.

The stereogram, figure 20, shows glaciated topography with scattered evergreen groves. The eskers and drumlins among the small lakes cause the appearance of "pockmarks" and "knob and kettles." Glacial deposits of this character are definite signs of materials suitable for road building. The pioneer road can be seen in this picture winding across the rough topography. The more direct alinement built subsequently can also be seen.

Topography of the type illustrated in figure 20 is shown as it appears from the ground in figure 21. The largest trees grow in the watered valleys such as that crossed by the high fill in the center of the picture.

#### PIONEER ROAD LINES SELECTED FROM PHOTOGRAPHS

When the general line of a portion of the highway was determined, perhaps along a river valley or following a ridge, the location engineer in the field used the vertical photographs of the area in which he was working to plan his advance from point to point. Assembled as an uncontrolled mosaic, the photographs could be used to select several possible routes between the points chosen as controls.

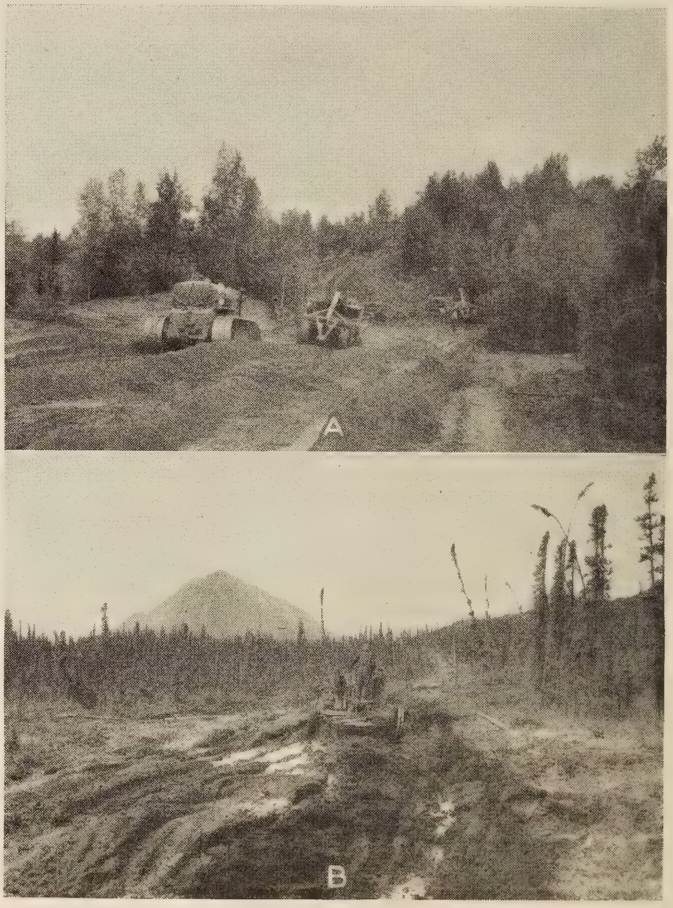


FIGURE 19.—(A) MIXED VEGETATION ON ROLLING TOPOGRAPHY INDICATES SOILS LIKELY TO BE GOOD ROAD MATERIALS. (B) MUSKEG MAKES ROAD CONSTRUCTION DIFFICULT.

Each trial line was then studied in detail and was adjusted to the topography and to soil and drainage conditions as seen and interpreted by stereoscopic examination of successive pairs of photographs. The final lines were then drawn with a wax crayon on each vertical as illustrated in figure 22, which shows a glaciated area along the Rancheria River. The line had already been located to point *A* by study of adjacent photographs. From this point there are several possibilities: Route *A—B—C* is direct but crosses irregular ground; route *A—D—E* is longer but lies on fairly smooth topography; an alternate *A—D—B—C* combines parts of the two routes. The choice of going through *E* or *C* would depend on study of adjoining photographs.

The tentative locations selected on the photographs were examined on the ground by the survey engineers and the best route, modified if necessary to fit actual ground conditions, was flagged and blazed. The shortest route between control points such as river crossings, mountain passes, and passages around or through muskeg was followed wherever possible. High ground usually proved to be the best choice for rapid pioneer road building on the Fort St. John division of the highway.

Sometimes alteration or abandonment of the first trial lines selected from the photographs became necessary because of unsuitable soil conditions not evident in photographic interpretation. If possible, alternate lines were then selected by further study of the uncontrolled mosaic and stereoscopic examination of the vertical photographs. In this way the field engineers, in their tent camps, could choose tentative alternate lines without delay.

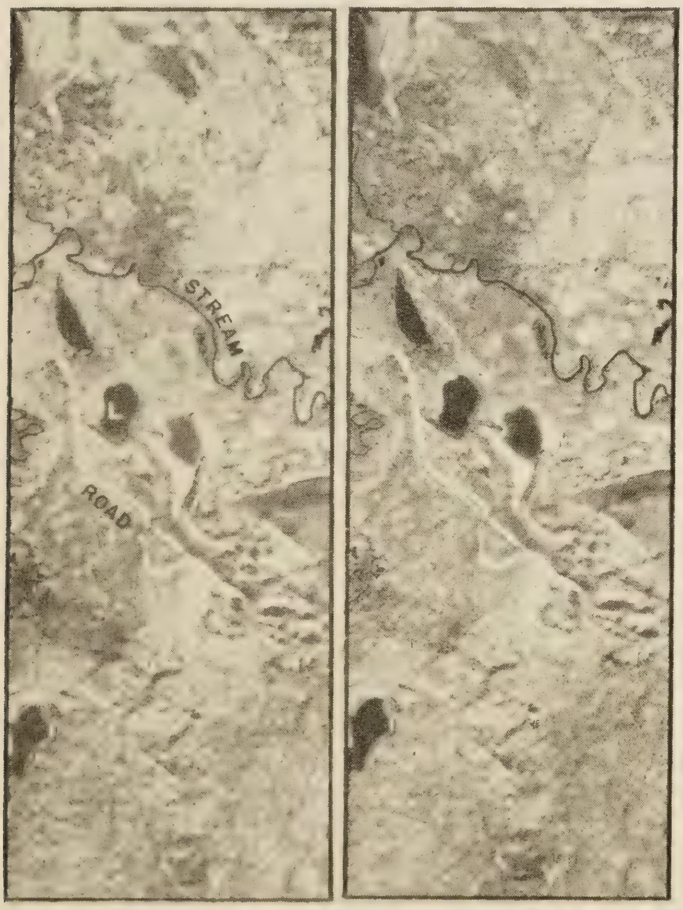


Figure 20.—Glacial Deposits Are Evident in This Stereogram.

Figure 21.—Glacial Deposits Provide Good Road Materials.

When a practicable route could not be found on the existing photographs, additional vertical photographs were requested along a new flight line. Often there was not time for this, however, as the construction crews frequently were close behind the survey parties. The location then had to be selected by ground reconnaissance which was usually difficult and occasionally resulted in overlooking a better route than the one actually selected.

###### FLAGGING AND BLAZING THE LINE

The line chosen by study of the photographs and tested on the ground was flagged and blazed for the bulldozer operators of the pioneer road construction crews to follow in the initial clearing operations. The location engineer used the mosaics and studied the photographs stereoscopically to keep himself oriented and the line aimed at a fixed objective. Where the topography was so rugged or irregular that acceptable grades could not be selected from the photographs with assurance, an Abney level was used.

If ground elevation control points had been established in advance, working grade lines could have been located on the paired photographs even in rugged topography with a simple elevation measuring instrument such as a parallax bar or a stereocomparagraph.

Occasionally the bulldozers would get off the blazed line. Sometimes at night they lost sight of the markers, and at other times they diverted from the line to avoid a small area of muskeg or unstable ground not noticeable until a tractor started to cross it.

If the machines were far off course and well ahead of the diversion point they had to be directed forward to the blazed line. In the haste to push the road through, backing up to start over again could not be permitted.

Direction of the bulldozers back to the blazed line was not unduly difficult if they were still within the area covered by the aerial photographs, as a course could be found by stereoscopic study of the paired verticals. When the machines had strayed beyond photographic coverage, however, the location engineer had to find a way back to the line as best he could by ground reconnaissance and compass directioning. This was always troublesome and time consuming.


Figure 22.—Tentative Locations Selected on a Photograph.

### ALASKA HIGHWAY LOCATION A PROVING GROUND FOR AERIAL SURVEYING

The engineers using aerial survey methods for location of the pioneer road of the Alaska Highway had in their hands a new technique. Their ability to apply it, and at times their confidence in it, were limited during the early days of the work. There was no alternative to adoption of the methods, however, for it would have been physically impossible, in the short time allotted, to locate the road entirely by ground survey through the wild, rough country that had to be traversed.

By trial and error the engineers acquired experience and eventually became adept in the use of the aerial photographs. Practical procedures were worked out, and principles of photographic interpretation were evolved for the subarctic region, though some of these principles were not established until the location work was well under way.

Many difficulties were encountered as a result of the lack of initial planning of the aerial survey work, for which inexperience, nonexistence of useful maps, and the driving need for haste were responsible. Some of the road had to be located by ground survey because aerial coverage could not be provided in time. When the photographs were available, however, their value was unquestionable, particularly after the field engineers had become proficient in their use. The Alaska Highway may well be considered a proving ground for highway location by aerial survey methods. The successful application of the methods there has helped to give impetus to their use.

# RUBBER-ASPHALT JOINT FILLERS

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by RICHARD H. LEWIS, Senior Chemist

**Rubber-asphalt blends have been used for some years as joint fillers in concrete pavements and in general have been found superior to the more common types of hot-poured materials. The failure of some rubber-asphalt compounds to serve satisfactorily has been traced to overheating these materials and to holding them at high temperatures for long periods at the time of pouring. Such treatment causes chemical changes that alter the physical properties of the materials and consequently impair those characteristics which make the materials efficient joint fillers.**

**Laboratory tests indicate that rubber-asphalt compounds should be heated to closely controlled optimum temperatures predetermined for each particular blend. Batches should not be heated for extended periods, and an expeditious mechanical method of pouring should be employed.**

**Experiments show that the prolonged flow test required in the present specification for rubber-asphalt materials can be supplanted by the easier and quicker softening point test. The cone penetrometer test, also required at present, can be replaced by the standard needle penetration test.**

FOR SOME YEARS joint-filling materials composed of blends of asphalt and rubber or rubbery synthetics have been used for sealing joints in concrete pavements. The service performances of joint fillers of this type, as indicated by inspection reports, show variable behavior. In some instances their performance has been superior to that of the more common types of hot-poured fillers, such as blown asphalt, refined asphalts, both petroleum and native, and mineral-filled asphalt. In other instances, their performance has been very unsatisfactory. Such variable behavior has been noted not only on the same projects but also on different projects with materials from the same source.

Producers have claimed, and present specifications indicate, that these materials may be heated satisfactorily for pouring without special precautions in the usual type of asphalt kettle, and that they can be poured satisfactorily at temperatures below 450° F. Laboratory studies and field experience do not substantiate these claims. Tests on materials from various sources have shown that some of these materials are difficult to liquefy and are of a ropey or nonuniform consistency unsuitable for pouring at the temperatures designated by the producer or provided for by existing specifications.

#### MATERIAL EXTENSIVELY USED FOR FILLING JOINTS IN CONCRETE RUNWAYS AND TAXIWAYS

Heating these materials in asphalt kettles in the field or over direct heat with gas burners or electric hot plates, which is common laboratory practice, often results in local overheating which causes an incipient decomposition of the rubber constituent of the blend. It has been found that even when the material is carefully heated in an oil-jacketed container precautions must be taken to control the softening process so that overheating does not occur.

In a report [1] by R. B. Jennings, which covers his inspection of installations of rubber-asphalt joint fillers in the eastern area of the United States, and more specifically the pouring of the material on the runways and taxiways at the Naval Air Test Center, Patuxent River, Md., the cause of the variable behavior of this material as a joint filler is attributed principally to damage of the material in heating. He claims that the use of the asphalt kettle, together with the retention of large quantities of the material at a high temperature for a long time, alters materially the physical properties of the filler causing the following types of failures:

(1) On crowned runways or taxiways or wherever grades were in excess of 1 percent the material exhibited a tendency to flow in joints during hot weather.

(2) The material lost resiliency and would not readily recover its original shape when deformed.

(3) The material had a surface stickiness; it would adhere to the tires of vehicles passing over it; and wind-blown sand and gravel striking the surface would adhere to it.

(4) Sand and small stone adhering to the surface of the joints settled into the jointing material by reason of their higher specific gravity and, in some cases, displaced the joint material leaving an incompressible material in the joint which would result in spalling of the concrete.

The major part of Jennings' report [1] deals with the development of equipment and methods for heating the joint filler to pouring consistency without subjecting the material to direct heat. Mechanical methods are also described which expedite the filling of the joint. The modifications in the preparation and handling of this material on more recent work have resulted in much improvement in the behavior of the installed joints.

#### FEDERAL SPECIFICATIONS PROVIDE FOR RUBBER-ASPHALT JOINT

This report deals primarily with the heating and pouring of rubber-asphalt joint-filling materials. Data obtained from a laboratory study of five commercial joint fillers from four sources are presented. Two of these materials, samples B and C, represent two different shipments from the same producer. All of these materials were submitted to meet Federal Specification SS-F-336, the test requirements and test methods for which are given below.

D. GENERAL REQUIREMENTS.

**D-1.** Material furnished under this specification shall be suitable for melting in the usual type of asphalt kettle. It shall be capable of melting readily and uniformly to a pouring consistency at a temperature of not more than 450° F. and shall comply with the detail requirements specified in section E. The material shall not be damaged when heated to the temperature required for satisfactory pouring.

[1] New Developments in Pouring Concrete Pavement Joint Filler, by Commander R. B. Jennings (C. E. C.) U. S. N. R., presented at the American Road Builders' Association annual convention, Chicago, Ill., January 1946. This report, somewhat modified, was printed as The Design and Maintenance of One and a Half Million Square Yards of Paving Slab to Prevent Pumping of Subgrade Through Paving Joints, in the American Road Builders' Association Technical Bulletin No. 103, 1946, pp. 14-25.

TABLE 1.—*Effect of pouring temperatures on test characteristics of rubber-asphalt joint fillers*

| | Characteristics of material A at pouring temperature of— | | | | Characteristics of material B at pouring temperature of— | | | | Characteristics of material C at pouring temperature of— | | | | Characteristics of material D at pouring temperature of— | | | | Characteristics of material E at pouring temperature of— | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 400° F.[1] | 450° F. | 500° F. | 550° F. | 400° F. | 450° F.[1] | 500° F. | 550° F. | 400° F. | 450° F.[1] | 500° F. | 550° F. | 400° F. | 450° F.[1] | 500° F. | 550° F. | 400° F. | 450° F. | 500° F.[1] | 550° F. |
| Time of heating to pouring temperature ... min. | 35 | 35 | 30 | 40 | 55 | 50 | 35 | 60 | 50 | 40 | 40 | 55 | 40 | 40 | 40 | 80 | 55 | 50 | 50 | 70 |
| Penetration, needle, 100 gm., 5 sec.: | | | | | | | | | | | | | | | | | | | | |
| At 59° F. | 52 | 60 | 56 | 59 | 34 | 39 | 48 | 32 | 48 | 60 | 57 | 32 | 36 | 34 | 36 | 39 | 38 | 48 | 57 | 58 |
| At 77° F. | 85 | 102 | 111 | 120 | 58 | 66 | 87 | 65 | 86 | 95 | 113 | 79 | 59 | 63 | 66 | 82 | 63 | 79 | 108 | 109 |
| At 95° F. | 145 | 166 | 196 | 231 | 97 | 113 | 157 | 298 | 132 | 154 | 205 | 293 | 100 | 116 | 116 | 160 | 99 | 135 | 171 | 190 |
| Slope of log-penetration-temperature curve | 0.0124 | 0.0123 | 0.0151 | 0.0165 | 0.0126 | 0.0128 | 0.0133 | 0.0269 | 0.0122 | 0.0114 | 0.0154 | 0.0267 | 0.0123 | 0.0148 | 0.0141 | 0.0170 | 0.0116 | 0.0125 | 0.0133 | 0.0143 |
| Penetration, cone, 150 gm., 5 sec.: | | | | | | | | | | | | | | | | | | | | |
| At 59° F. | 43 | 47 | 36 | 47 | 34 | 42 | 43 | 25 | 51 | 54 | 56 | 32 | 31 | 26 | 25 | 33 | 37 | 42 | 52 | 49 |
| At 77° F. | 76 | 79 | 81 | 104 | 53 | 64 | 69 | 47 | 79 | 84 | 100 | 74 | 49 | 47 | 49 | 64 | 50 | 69 | 90 | 84 |
| At 95° F. | 111 | 125 | 138 | 159 | 81 | 97 | 135 | 132 | 109 | 128 | 170 | 169 | 75 | 76 | 78 | 115 | 75 | 105 | 133 | 142 |
| Softening point (ring and ball) ... ° F. | 192 | 167.5 | 149.0 | 139.1 | 229.6 | 200.3 | 164.3 | 141.8 | 183.7 | 176.9 | 154.9 | 147.2 | 212.9 | 187.3 | 190.0 | 150.4 | 246.2 | 239.5 | 188.2 | 148.1 |
| Ductility test at 41° F.: | | | | | | | | | | | | | | | | | | | | |
| Ductility, 5 cm./min ... cm. | 11.5 | 12.5 | 6.5 | 8.5 | 13.5 | 14.5 | 15.0 | 12.5 | 19.0 | 22.0 | 21.5 | 12.0 | 5.5 | 5.0 | 5.0 | 5.5 | 23.5 | 42.5 | 61.0 | 44.0 |
| Length of broken specimen after 15 min ... cm. | 3.6 | 4.2 | 3.3 | 4.7 | 1.8 | 2.0 | 3.8 | 8.7 | 3.9 | 4.4 | 6.4 | 7.1 | 4.4 | 2.7 | 3.2 | 3.7 | 7.5 | 10.7 | 15.4 | 18.0 |
| Rebound after 15 min ... cm. | 7.9 | 8.3 | 3.2 | 3.8 | 11.7 | 12.5 | 11.2 | 3.8 | 15.1 | 17.6 | 15.1 | 4.9 | 1.1 | 2.3 | 1.8 | 1.8 | 16.0 | 31.8 | 45.6 | 26.0 |
| Percentage rebound ... percent. | 69 | 66 | 49 | 45 | 87 | 86 | 75 | 30 | 79 | 80 | 70 | 41 | 20 | 46 | 36 | 33 | 68 | 75 | 75 | 59 |
| Ductility test at 77° F.: | | | | | | | | | | | | | | | | | | | | |
| Ductility, 5 cm./min ... cm. | 16.0 | 15.0 | 11.5 | 10.0 | 16.0 | 17.0 | 15.0 | 11.5 | 19.5 | 25.0 | 20.5 | 12.5 | 9.0 | 10.5 | 5.0 | 8.0 | 27.0 | 40.0 | 114.0 | 228 |
| Length of broken specimen after 15 min ... cm. | 7.3 | 7.2 | 6.6 | 5.2 | 6.2 | 7.1 | 6.2 | 7.3 | 9.4 | 12.0 | 8.3 | 7.0 | 5.3 | 7.5 | 3.0 | 6.3 | 8.6 | 10.0 | 31.0 | 85 |
| Rebound after 15 min ... cm. | 8.7 | 7.8 | 4.9 | 4.8 | 9.8 | 9.9 | 8.8 | 4.2 | 10.1 | 13.0 | 12.2 | 5.5 | 3.7 | 3.0 | 2.0 | 1.7 | 18.4 | 30.0 | 83.0 | 143 |
| Percentage rebound ... percent. | 54 | 52 | 43 | 48 | 61 | 58 | 59 | 37 | 52 | 52 | 60 | 44 | 41 | 29 | 40 | 21 | 68 | 75 | 73 | 63 |
| Flow at 140° F. in 5 hours ... cm. | 0.3 | 1.2 | 18.3 | 15.4 | 0 | 0.1 | 1.4 | 20.6 | 0.3 | 0.6 | 2.4 | 15.4 | 0 | 0.2 | 0.4 | 14.3 | 0 | 0.1 | 0.4 | 10.1 |

[1] Minimum suitable pouring temperature for this material.

**E. DETAIL REQUIREMENTS.**

**E–1.** Material furnished under this specification when tested in accordance with the methods described in section F shall conform to the following requirements:

(*a*) Pour point, not more than ... 450° F.
(*b*) Melting time, not more than ... 60 min.
(*c*) Penetration:
At 32° F., 200 g., 60 sec., not less than ... 0.28 cm.
At 77° F., 150 g., 5 sec ... 0.45 to 0.75 cm.
(*d*) Flash point, not less than ... 550° F.
(*e*) Flow, not more than ... 0.5 cm.
(*f*) Bond test.
There shall be no cracking of the material or failure in bond between the materials and the mortar test blocks at the end of five cycles.

**F. METHODS OF TESTING.**

**F–1.** The following methods of testing shall be used in determining compliance with the detail requirements specified in section E:

**F–1a.** *Preparation of sample.*

**F–1a (1).** A sample of the material weighing approximately 300 grams shall be selected in such a manner as to avoid inclusion of the surface layer. Of this quantity, 100 grams shall be melted, with continued stirring to a pouring consistency, in a clean container, using an oil bath or similar heating unit. The operation of melting shall be conducted as rapidly as possible without danger of local overheating. The remaining 200 grams shall then be added, in quantities of approximately 50 grams at a time, to the melted material until the entire quantity is of a sufficiently fluid consistency to be poured satisfactorily.

**F–1b.** *Pour point.*

**F–1b (1).** The minimum temperature at which the material will pour readily and uniformly when subjected to the procedure described in section F–1a (1).

**F–1c.** *Melting time.*

**F–1c (1).** The time in minutes required to melt the material to pouring consistency when subjected to the procedure described in section F–1a (1).

**F–1d.** *Penetration.*

**F–1d (1).** The penetration test shall be made in accordance with the procedure described in Federal specification SS–R–406, for the penetration test of bituminous materials, except that a penetration cone conforming to the following requirements shall be used in place of the standard penetration needle: The cone shall be constructed of stainless steel or brass with a detachable hardened-steel or stainless steel tip. It shall conform to the dimensions shown in figure 1 of A. S. T. M. Method of Test D 217–38T except that the interior construction may be modified as desired. The outside surface of the cone and tip shall be given a very smooth finish. The total moving weight of the cone and attachments shall be 150 g.

**F–1e.** *Flash point.*

**F–1e (1).** The test for flash point shall be made in accordance with the procedure described in Federal specification SS–R–406 for the flash and fire point test by means of open cup.

**F–1f.** *Flow test.*

**F–1f (1).** A portion of the sample prepared as described in Section F–1a (1) shall be poured into a suitable amalgamated mold 4 cm. wide by 6 cm. long, placed on a bright tin panel. The sample shall be poured to a uniform depth of 0.32 cm. After cooling at room temperature for 2 hours, the mold shall be removed and the panel containing the sample shall be placed in an oven maintained at 140° F. ± 2° F. for 5 hours. During the test the panel shall be mounted at an angle of 75 degrees with the horizontal. The total movement in centimeters of the specimen during the 5-hour test period shall be reported as the flow.

**F–1g.** *Bond test.*

**F–1g (1).** *Extension machine.*—The extension machine used in the bond test shall be so designed that the specimen can be expanded at a uniform rate of approximately 0.125 inch per hour for at least 4 hours. It shall consist essentially of one or more screws rotated by an electric motor through suitable gear reductions. Self-alining plates or grips, one fixed and the other carried by the rotating screw or screws, shall be provided for holding the test specimen in position during the test.

**F–1g (2).** *Mortar blocks.*—Two cement mortar blocks each 1 by 2 by 3 inches in size shall be prepared, using one part of portland cement to two parts by weight of clean, uniformly graded concrete sand, all of which passes a No. 4 sieve and not more than 5 percent of which passes a No. 100 sieve. Sufficient water shall be used to produce a flow of 100 ± 5 when tested in accordance with the procedure described in Federal Specification SS–C–158a, section F–4m (4). After curing 1 day in moist air and 6 days in water at 70° F., the 2- by 3-inch faces of each block shall be surfaced by grinding with a No. 30 HD carborundum stone until the aggregate is uniformly exposed. The blocks shall then be air dried at room temperature for 24 hours prior to use.

**F–1g (3).** *Test specimen.*—The mortar blocks, prepared as described in section F–1g (2), shall be placed on a plate of amalgamated metal and shall be spaced 1 inch apart by means of amalgamated metal strips placed at such distances from the ends that an opening 1 by 2 by 2 inches is formed. The test specimen is completed by filling this opening with the material prepared in accordance with the procedure described in section F–1a (1), followed by cooling in air at room temperature for 2 hours. The mortar test blocks may be held together by means of clamps, rubber bands, or other suitable means.

**F–1g (4).** *Extension at low temperature.*—The test specimen, prepared as described in section F–1g (3), shall be placed in an atmosphere maintained at 0° F. ± 2° F. for not less than 6 hours, after which the amalgamated plates and strip shall be removed and the specimen mounted immediately in the self-alining clamps of the extension machine described in paragraph F–1g

TABLE 2.—*Effect of time of heating at pouring temperature on test characteristics of rubber-asphalt joint fillers*

| | Effect of heating material A at 400° F.[1] for— | | | | Effect of heating material C at 450° F.[1] for— | | | | Effect of heating material D at 450° F.[1] for— | | | Effect of heating material E at 500° F.[1] for— | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 hours | 2 hours | 3 hours | 4 hours | 0 hours | 2 hours | 3 hours | 4 hours | 0 hours | 4 hours | 5 hours | 0 hours | 1 hour | 4 hours | 5 hours | 7 hours |
| Time of heating to pouring temperature....min.. | 35 | 40 | 65 | 45 | 40 | 30 | 40 | 35 | 40 | 45 | 40 | 50 | 40 | 50 | 35 | 40 |
| Penetration, needle, 100 gm., 5 sec.: | | | | | | | | | | | | | | | | |
| At 59° F. | 52 | 45 | 51 | 50 | 60 | 58 | 56 | 31 | 34 | 35 | 34 | 57 | 68 | 33 | 29 | 26 |
| At 77° F. | 85 | 74 | 97 | 91 | 95 | 82 | 101 | 62 | 63 | 66 | 65 | 108 | 95 | 64 | 59 | 48 |
| At 95° F. | 145 | 139 | 155 | 157 | 154 | 183 | 202 | 128 | 116 | 101 | 118 | 171 | 141 | 97 | 97 | 68 |
| Slope of log-penetration-temperature curve | 0.0124 | 0.0136 | 0.0134 | 0.0138 | 0.0114 | 0.0139 | 0.0155 | 0.0171 | 0.0148 | 0.0129 | 0.0150 | 0.0133 | 0.0088 | 0.0130 | 0.0146 | 0.0116 |
| Penetration, cone, 150 gm., 5 sec.: | | | | | | | | | | | | | | | | |
| At 59° F. | 43 | 39 | 47 | 41 | 54 | 52 | 46 | 28 | 26 | 26 | 27 | 52 | 58 | 30 | 26 | 23 |
| At 77° F. | 76 | 69 | 82 | 82 | 84 | 81 | 74 | 48 | 47 | 48 | 49 | 90 | 85 | 55 | 52 | 39 |
| At 95° F. | 111 | 116 | 132 | 132 | 128 | 143 | 162 | 112 | 76 | 78 | 87 | 133 | 126 | 86 | 85 | 59 |
| Softening point (ring and ball)....°F.. | 192.2 | 159.4 | 165.2 | 155.3 | 176.9 | 152.6 | 149.5 | 147.7 | 187.3 | 167.5 | 168.8 | 188.2 | 163.9 | 180.5 | 176.5 | 190.4 |
| Ductility test at 41° F.: | | | | | | | | | | | | | | | | |
| Ductility, 5 cm./min....cm.. | 11.5 | 10.5 | 13.5 | 14.0 | 22.0 | 18.5 | 11.5 | 10.0 | 5.0 | 4.5 | 4.5 | 61.0 | 35.0 | 8.5 | 7.0 | 7.0 |
| Length of broken specimen after 15 min..cm.. | 3.6 | 3.2 | 5.3 | 5.8 | 4.4 | 6.5 | 5.0 | 4.7 | 2.7 | 3.2 | 2.7 | 15.4 | 8.3 | 2.7 | 2.3 | 1.8 |
| Rebound after 15 min....cm.. | 7.9 | 7.3 | 8.2 | 8.2 | 17.6 | 12.0 | 6.5 | 5.3 | 2.3 | 1.3 | 1.8 | 45.6 | 26.7 | 5.8 | 4.7 | 5.2 |
| Percentage rebound....percent.. | 69 | 70 | 61 | 59 | 80 | 65 | 57 | 53 | 46 | 29 | 40 | 75 | 76 | 68 | 67 | 74 |
| Ductility test at 77° F.: | | | | | | | | | | | | | | | | |
| Ductility, 5 cm./min....cm.. | 16.0 | 14.5 | 19.0 | 21.5 | 25.0 | 19.5 | 13.0 | 15.0 | 10.5 | 6.0 | 6.0 | 114.0 | 83.5 | 11.5 | 10.5 | 9.0 |
| Length of broken specimen after 15 min..cm.. | 7.3 | 7.0 | 8.0 | 8.9 | 12.0 | 9.0 | 5.8 | 8.5 | 7.5 | 3.8 | 3.8 | 31.0 | 24.0 | 2.4 | 2.6 | 2.2 |
| Rebound after 15 min....cm.. | 8.7 | 7.5 | 11.0 | 12.6 | 13.0 | 10.5 | 7.2 | 6.5 | 3.0 | 2.2 | 2.2 | 83.0 | 59.5 | 9.1 | 7.9 | 6.8 |
| Percentage rebound....percent.. | 54 | 52 | 58 | 59 | 52 | 54 | 55 | 43 | 29 | 37 | 37 | 73 | 71 | 79 | 75 | 76 |
| Flow at 140° F. in 5 hours....cm.. | 0.3 | 2.7 | 1.2 | 5.3 | 0.6 | 5.6 | 8.5 | 15.7 | 0.2 | 1.4 | 2.7 | 0.4 | 3.0 | 0.5 | 1.0 | 0.3 |

[1] Minimum suitable pouring temperature for this material.

(1). The specimen shall then be extended at a uniform rate of 0.125 inch per hour for 4 hours. During this period the atmosphere surrounding the test specimen shall be maintained at a temperature of 0° F. ±2° F.

**F–1g (5).** *Recompression.*—After extension as described in section F–1g (4), the specimen shall be removed from the extension machine and allowed to remain in air at room temperature for 2 hours, after which it shall be compressed to its original thickness at the rate of approximately 0.1 inch per minute. The specimen shall be maintained in a horizontal position during compression.

**F–1g (6).** *Number of cycles.*—Five cycles of extension at low temperature followed by recompression shall constitute one complete test for bond as specified in section E–1 (f).

### TESTING PROCEDURE USED IN THIS STUDY

The effect of the pouring temperature on the test characteristics was determined in the first series of laboratory tests. In this series the materials were heated, according to method F—1a(1) of Specification SS—F—336, to the temperatures shown in table 1, and then poured immediately into the testing forms. In liquefying these materials, the oil bath was brought to a temperature 50° F. above the selected pouring temperature and controlled within ±5° F. thereafter until the filler material reached the designated pouring temperature. When 550° F. was used as a pouring temperature an air bath was substituted for the oil bath when material became fluid, to accelerate the heating.

The specification describes the pour point as the minimum temperature at which the material will pour readily and uniformly. The literature of one producer states that the material is considered at its pour point when it has a uniform consistency and is capable of being poured into a ½-inch joint. The temperatures at which the materials were considered as having a suitable consistency for pouring are given in table 1.

The minimum suitable pouring temperatures as determined in these tests were above the temperatures recommended by the producers and, with the exception of sample A, were equal to or above the specification requirement.

The effect of maintaining the fillers at the minimum suitable pouring temperature for various periods of time is shown in table 2. Because of insufficient material sample B was not included in this phase of the study. In preparing these samples for test, the oil bath was heated as previously described. As the materials reached the designated pouring temperature, the bath temperature was so adjusted as to keep the temperature of the filler material within ±5° F. of the pour point during the stipulated heating period.

The laboratory tests made on the filler materials and the test procedures used are those required by Federal Specification SS–F–336 with the following exceptions:

Flash point determinations were not made because of the tendency of these materials to foam and overflow the flash cup while heating and also because of the difficulty of maintaining a constant rate of increase in temperature.

All of these fillers passed the bond test when tested in the manner prescribed in the specifications. Because of insufficient material, however, the bond test was not made on materials subjected to the varying conditions of this study.

The following tests not required by the specifications were made also to show the behavior of these joint fillers under variable laboratory heating and pouring conditions.

Penetration tests using the standard penetration needle under a load of 100 grams for 5 seconds were made at three temperatures, 59°, 77°, and 95° F.

Cone penetration tests were not made at 32° F., 200 grams, 60 seconds. These were made, however, at 59°, 77°, and 95° F. with a load of 150 grams for 5 seconds.

Softening points were determined by the ring-and-ball method using glycerin instead of water for temperature control.

Ductility tests, using standard equipment, were run at 41° and 77° F. at a speed of 5 centimeters a minute. After breaking, the test specimens were left undisturbed in the ductility bath for 15 minutes, after which the total amount of contraction or rebound in the broken halves of the elongated specimens was determined. The amount of rebound has been suggested by one producer as a measure of the resiliency of the material.

The cone penetrometer, which is required by Federal Specification SS–F–336 for the determination of the consistency of these asphalt-rubber blends, was devised initially for the control of greases and petrolatum. It

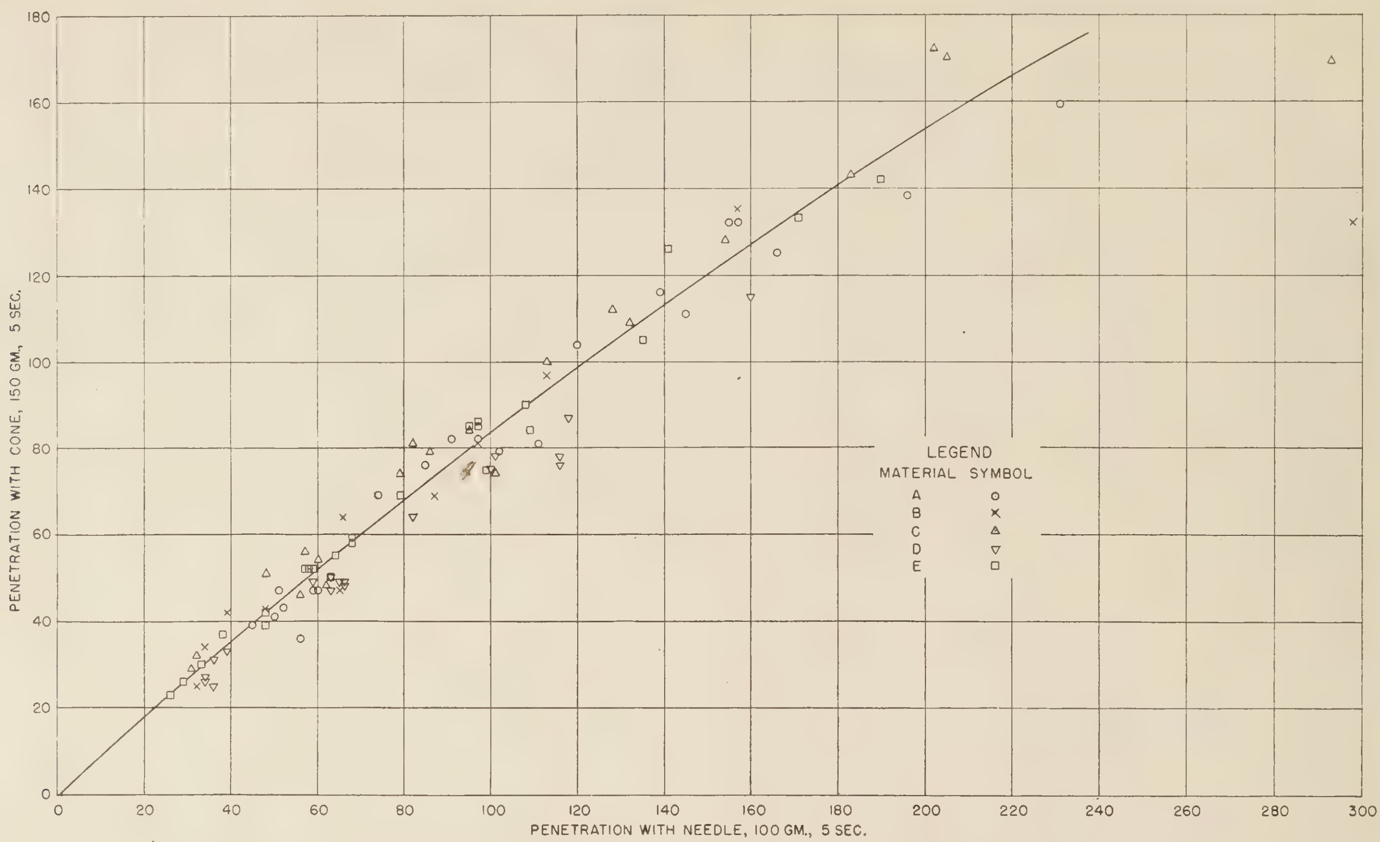


FIGURE 1.—RELATION BETWEEN PENETRATION WITH GREASE CONE AND STANDARD NEEDLE.

has been used, however, for determining the consistency of calking materials and other materials of nonuniform texture. The first rubber-asphalt joint-filling materials contained vermiculite or other flaky matter which to a certain extent prevented close checking of consistency with the standard asphalt penetration needle. It was found in this study that if the same number of tests are made with the standard penetration needle as are required by the A. S. T. M. method for the cone penetrometer, an average of the results is a satisfactory measure of the consistency of the filler material.

The results of the penetration tests, using the cone and needle penetrometers at three temperatures, are plotted in figure 1. Although there are a few points of variance the relation between the results of the two tests is very good. Since the standard penetration test is more familiar to highway engineers, the results obtained with the cone penetrometer will not be discussed further.

#### PENETRATIONS AT NORMAL TEMPERATURE ALTERED DURING HEATING

The data for the standard asphalt penetration tests given in tables 1 and 2 are plotted in figures 2 and 3. In figure 2 it will be noticed that as the pouring temperature increases, the penetration values at 95° F. increase, the increase being greater for samples B, C, and A than for samples D and E. The penetration values at 77° F. increase in like order but for samples B and C decrease for the material heated to 550° F. The effect of the pouring temperature on the penetrations at 59° F. is not as noticeable as for the other two temperatures. At this temperature no appreciable difference in the penetration values of the materials heated to various pour temperatures is obtained on samples A and D. In the case of samples B and C, although the penetrations at 95° F. of the materials heated to 550° F. greatly increase, the penetrations at 59° F. decrease. With sample E the penetration at 59° F. increases with the increase of the pouring temperature.

In figure 3 the effect of time of heating at the minimum suitable pouring temperature on the penetration values is shown to be more pronounced for samples C and E than for samples A and D. Prolonged heating of sample E tends to reduce the penetration values at the three test temperatures. In the case of sample C, the penetrations at 95° F. after 2 and 3 hours of heating increase but after a 4-hour heating period the penetrations at the three temperatures are considerably lower than values obtained for any of the other heating periods. The effect of time of heating on the penetration values of samples A and D show no consistent trend. A comparison of the data plotted in figures 2 and 3 shows that holding the materials at the selected pouring temperatures does not alter the penetration values as much as the subjection of the materials to higher temperatures before pouring.

In tables 1 and 2 the slopes of the penetration (plotted logarithmically)-temperature curves based on the penetration values obtained with the standard needle are given. In most cases, if the values shown are plotted to scale, there is approximately a straight-line relation between the temperature of test and the logarithm of the corresponding penetration.

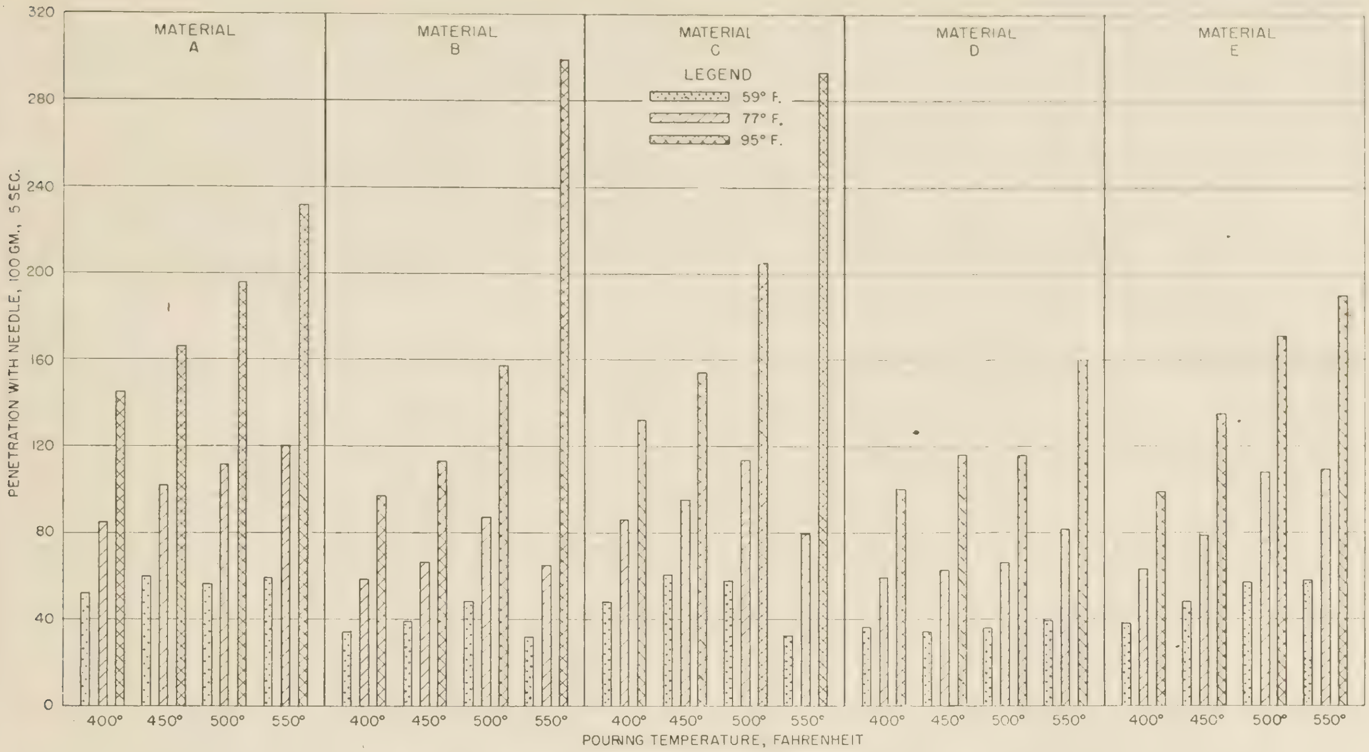


FIGURE 2.—EFFECT OF POURING TEMPERATURE ON PENETRATION.

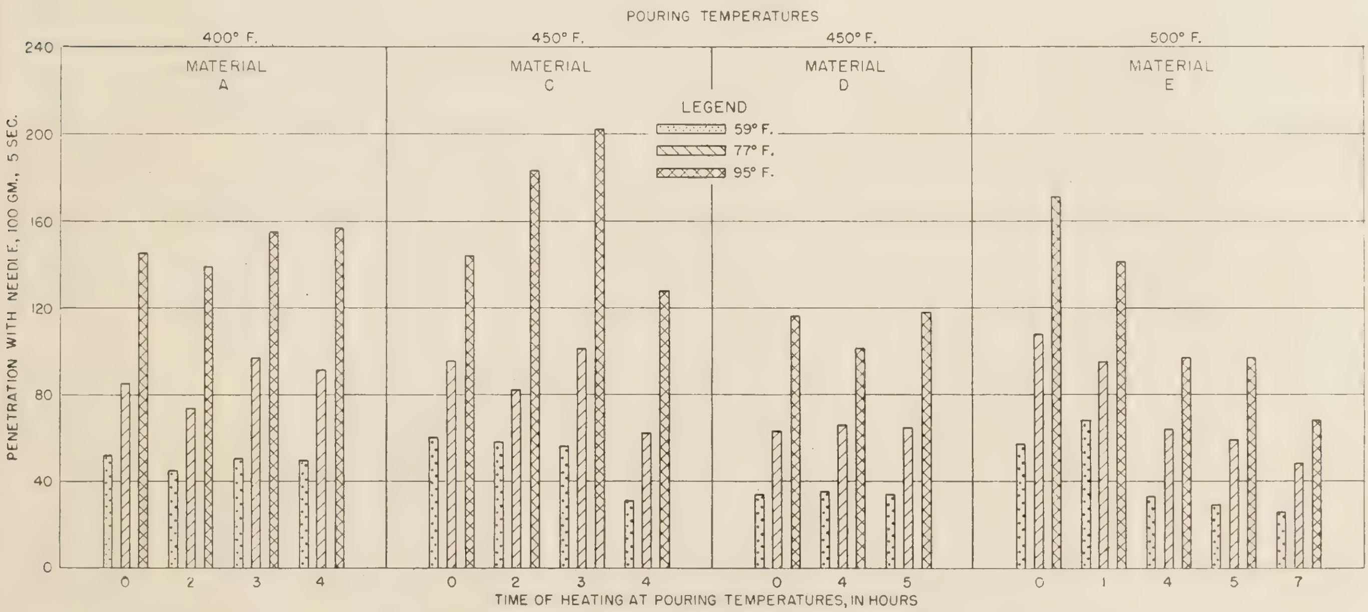


FIGURE 3.—EFFECT OF TIME OF HEATING AT MINIMUM SUITABLE POURING TEMPERATURE ON PENETRATION.

### TEMPERATURE SUSCEPTIBILITY OF MATERIALS ALTERED

It has been established [2] that the log-penetration-temperature curves of 50–60 and 85–100 asphalts assume the form of straight lines, and a true indication of the susceptibility to change in temperature of their consistencies is found in the slope of the lines. The slope, as reported in tables 1 and 2, can be calculated as follows:

$$\text{Slope}=M=\frac{\log p_2-\log p_1}{t_2-t_1}$$

where $p_2$ and $p_1$ are the penetrations at the temperatures $t_2=95°$ F. and $t_1=59°$ F., respectively.

[2] The Physical and Chemical Properties of Petroleum Asphalts of the 50–60 and 85–100 Penetration Grades, by R. H. Lewis and J. Y. Welborn, PUBLIC ROADS vol 21, No. 1, March 1940.

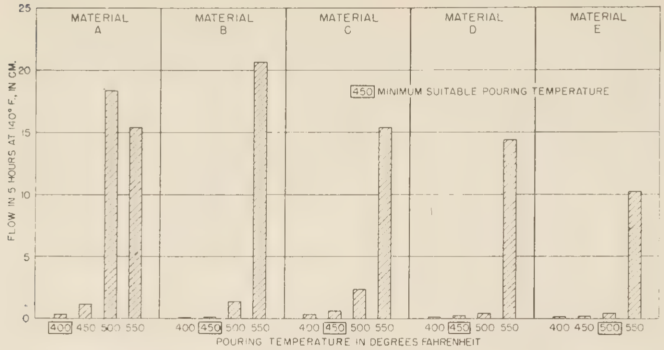


FIGURE 4.—EFFECT OF POURING TEMPERATURE ON THE FLOW TEST RESULTS.

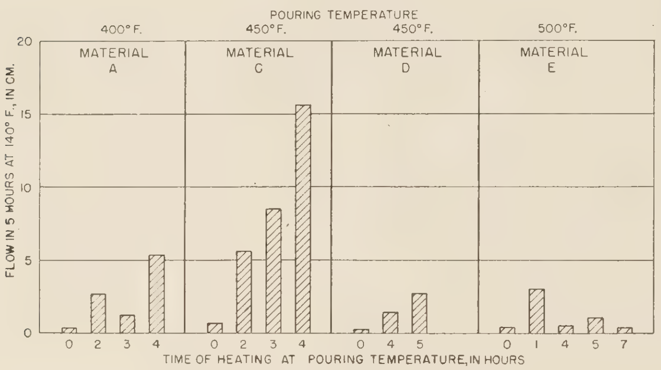


FIGURE 5.—EFFECT OF TIME OF HEATING AT MINIMUM SUITABLE POURING TEMPERATURE ON THE FLOW TEST RESULTS.

A study of these data as given in the two tables shows that as the pouring temperatures increase, the susceptibility of the poured rubber-asphalt blends to temperature change increases. The increase is highest for samples B and C and lowest for sample E. The effect of time of heating at pouring temperatures is generally progressive, is greatest for sample C, and is less pronounced and more variable for the other materials.

A series of blown asphalts commonly used as joint fillers in brick and concrete pavements have slopes of log-penetration-temperature curves ranging from 0.0090 to 0.0125, with an average of 0.0105. The log-penetration-temperature curves of 50–60 penetration asphalts from representative sources [2] have slopes ranging from 0.0172 to 0.0307 with an average of 0.0230, and for 85–100 penetration asphalts the slopes range from 0.0188 to 0.0324 with an average of 0.0242. It can be seen from the above values that the susceptibility of these rubber-asphalt blends initially is but slightly higher than that of blown asphalts; and that high temperatures tend to increase their susceptibility. The susceptibility of samples B and C when heated to 550° F. approximates that of the usual asphalt cement. Prolonged heating at the minimum suitable pouring temperature in most cases also increases the temperature susceptibility of these materials. It is apparent, therefore, that both the pouring temperature and time of heating must be carefully controlled to insure that these materials will have the proper consistency under all temperature conditions to perform satisfactorily as a joint filler.

[2] See footnote on p. 295.

### SOFTENING POINT INDICATIVE OF FLOW PROPERTIES OF JOINT FILLERS

The effect of high summer temperatures on the fluidity of the asphalt-rubber filler is presumed to be indicated by the behavior of the material in the flow test. The results of the flow test, as given in tables 1 and 2, are shown graphically in figures 4 and 5. Table 1 shows that, with the exception of sample C, the flow of the materials poured at the minimum suitable pouring temperatures is below the allowable maximum of 0.5 cm. permitted by specification. In the case of sample C, the flow is 0.1 cm. higher than is permitted. It is apparent that heating to temperatures 50° F. higher than the minimum suitable pouring temperature greatly increases the flow values of all these materials.

The flow values of the materials held at the minimum suitable pouring temperatures, as shown in figure 5, are somewhat erratic, only the results on samples C and D showing a steady increase in flow with increase in time of heating.

For control of the material during the filling of expansion joints one user of large quantities of this type of filler stipulates that material represented by samples taken from the heating kettles shall not be used if the flow is greater than 2 cm. It is apparent that under prolonged heating sample C is more likely to develop unsatisfactory flow characteristics than the other materials when held at an elevated temperature for a long time.

### RUBBER-ASPHALT FILLERS HAVE DUCTILITY CHARACTERISTICS DIFFERENT FROM THOSE OF USUAL JOINT MATERIALS

The softening point determination is most useful for the control of standard bituminous materials, both asphaltic and tar products. It is especially valuable in determining the resistance of these materials to the softening action of the sun or of artificial heat. It has been used, therefore, as a measure of the changes occur-

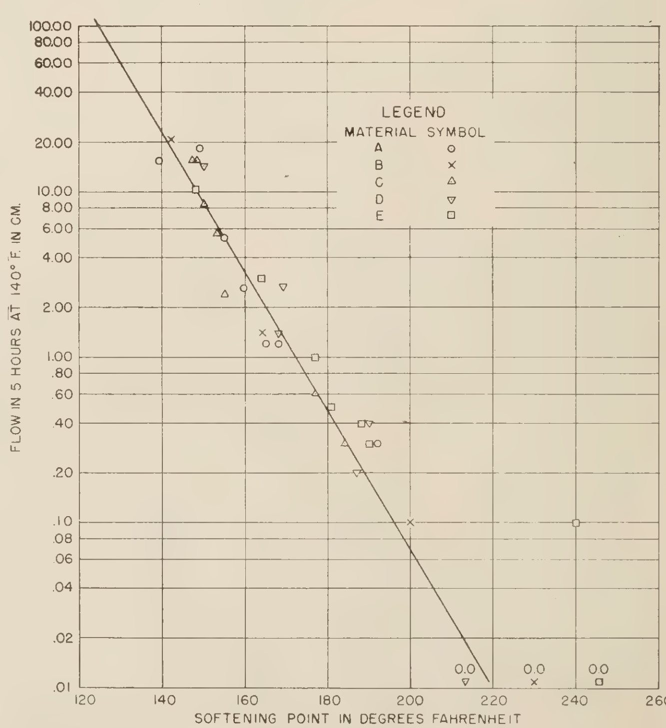


FIGURE 6.—RELATION BETWEEN SOFTENING POINT AND FLOW.

ring in these joint fillers. In making the determination of softening point of bituminous materials, a water bath is used for those softening under 176° F. and a glycerin bath for those softening over 176° F. In order to eliminate any variations in test results due to type of bath, glycerin only was used in this investigation.

The results of the softening point test plotted against the flow values obtained on the same material are shown in figure 6. A good relation between the two tests is shown. For these materials it is shown that when the softening point is over 180° F. the flow value in centimeters will be less than 0.5, and when the softening point is over 170° F. the flow will be less than 2 centimeters. Since the softening point determination can be made more easily and in less time than the flow test it would appear to be better suited as a field control test for these materials than the flow test.

The standard ductility test normally used for control of asphaltic material has been suggested by one producer as a control test for rubber-asphalt blends. It was claimed also that the contraction or rebound that occurs after the ductility specimen has broken is a measure of the resiliency of the joint filler. The ductility data given in tables 1 and 2 are shown in figures 7 and 8. In these figures the length of test specimen at time of break and the total length of the two halves of the test specimen after a 15-minute interval in the bath held at the control temperature are plotted. The percent of rebound shown in tables 1 and 2 is the difference between length of test specimen at time of break minus the total length of two halves of specimen after a 15-minute interval divided by length of test specimen at time of break multiplied by 100.

In figure 7 the effect of pouring temperature on the ductility of the various joint fillers is shown to be variable. Only in the case of sample E is there a definite trend. For this material there is a steady increase in ductility at 77° F. with increase in pouring temperature and a similar increase in ductility at 41° F. for pouring temperatures up to 500° F. The difference between ductility at 41° and 77° F. is not great for any of these materials except for the portions of sample E that were poured at 500° and 550° F. It is apparent that sample E becomes exceedingly ductile when heated to these higher temperatures. It should be noted that the producer of this material recommends that it be poured at a temperature between 380° and 400° F. which is considerably below the temperature of 500° F. which was indicated by this investigation to be suitable for its pouring. When poured at 400° F. this material is but slightly more ductile than the portions of sample C poured at 400°, 450°, and 500° F. The ductility values of the other materials do not show a consistent trend, but it is evident that the ductility values on portions poured at 500° and 550° F. are generally lower than the values of those poured at 400° and 450° F.

#### RUBBER-ASPHALT FILLER RESILIENT AT LOW TEMPERATURES

The effect of time of heating on the ductility of these materials is shown in figure 8. In the case of sample A the ductility tends to increase as time of heating increases, but with the other materials the ductility tends to decrease as the time of heating increases. The latter effect is especially true of sample E. The drop in ductility after 1 hour of heating is sufficient to indicate a definite change in structure and as the heating continues the ductility values are lower

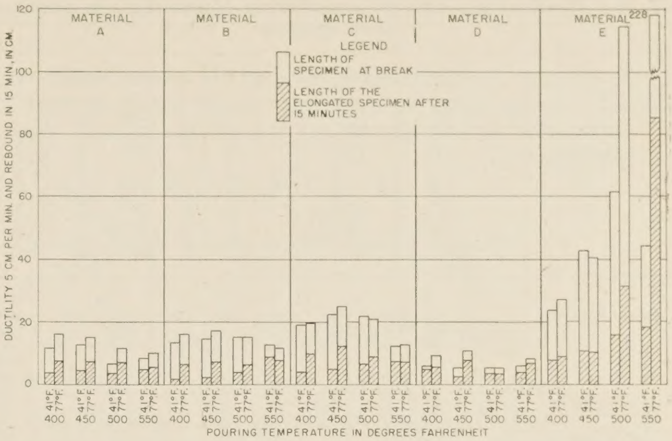


FIGURE 7.—EFFECT OF POURING TEMPERATURE ON DUCTILITY AND REBOUND AT 41° F. AND AT 77° F.

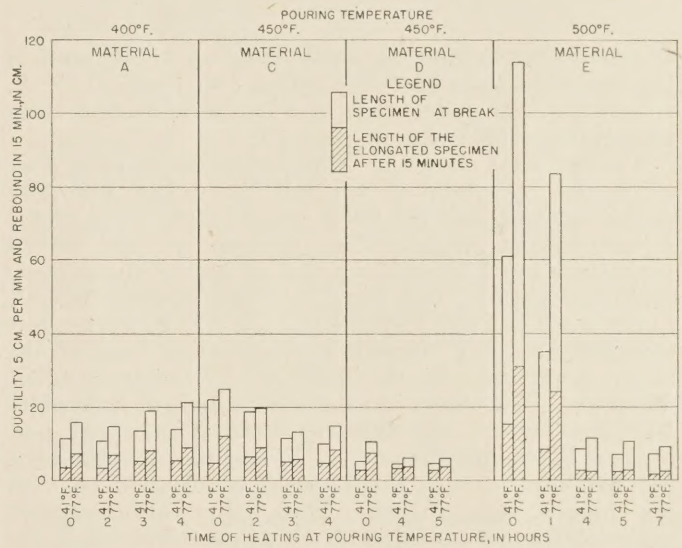


FIGURE 8.—EFFECT OF TIME OF HEATING AT MINIMUM SUITABLE POURING TEMPERATURE ON DUCTILITY AND REBOUND AT 41° F. AND AT 77° F.

than those obtained on any of the materials except sample D. Sample E, on heating to 500° F., develops ductility at 77° F. comparable to that of a steam-refined asphalt of comparable consistency. When tested at 41° F., the ductility is considerably higher than for an asphalt cement of comparable consistency. When this material is held at 500° F. for long periods, the ductility is reduced below the values obtained on the same material when heated to 400° and 450° F.

As previously indicated, the contraction in total length of the ductility specimen or rebound has been suggested as indicative of the resilient properties of these joint fillers. Table 1 shows that the percentage of rebound at 41° F. for samples A, B, and C definitely decreases as the pouring temperature increases. For samples D and E the percentage of rebound increases and then decreases as the pouring temperature increases. In 12 cases the percentage of rebound is greater for tests made at 41° F., in two cases equal to, and in six cases less than the percentage of rebound that occurs in tests made at 77° F.

Table 2 shows that with minor exceptions the percentage of rebound at 41° F. generally decreases as the time of heating increases. In 11 cases the percentage

TABLE 3.—*Ductility data on some typical blown asphalts and asphalt cements*

| Identification | Penetration 100 gm., 5 sec. 77° F. | Ductility test at 41° F. | | | | Ductility test at 77° F. | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Ductility 5 cm/min. | Length of broken specimen after 15 min. | Rebound after 15 min. | Percent rebound | Ductility, 5 cm./min. | Length of broken specimen after 15 min. | Rebound after 15 min. | Percent rebound |
| | | *Cm.* | *Cm.* | *Cm.* | *Percent* | *Cm.* | *Cm.* | *Cm.* | *Percent* |
| Blown Asphalts: | | | | | | | | | |
| 1 | 31 | 0.75 | 0.25 | 0.5 | 67 | 1.5 | 0.25 | 1.25 | 83 |
| 2 | 49 | .75 | .4 | .35 | 47 | 1.5 | .75 | .75 | 50 |
| 3 | 45 | 1.5 | 1.1 | .4 | 27 | 2.0 | 1.0 | 1.0 | 50 |
| 4 | 35 | 2.0 | 1.4 | .6 | 30 | 3.5 | 2.0 | 1.5 | 43 |
| 5 | 30 | 1.5 | 1.1 | .4 | 27 | 2.0 | 1.25 | .75 | 38 |
| 6 | 38 | 2.0 | 1.25 | .75 | 38 | 2.5 | 1.5 | 1.0 | 40 |
| 7 | 42 | 1.5 | 1.1 | .4 | 27 | 1.75 | 1.0 | .75 | 43 |
| 9 | 30 | 2.25 | 1.5 | .75 | 33 | 3.75 | 2.5 | 1.25 | 33 |
| Asphalt Cements: | | | | | | | | | |
| 21 | 41 | 3.25 | 2.3 | .95 | 29 | 79 | 56 | 23 | 29 |
| 8 | 45 | 3.25 | 2.6 | .65 | 20 | 103 | 82 | 21 | 20 |
| 13 | 58 | 5.0 | 4.0 | 1.0 | 20 | 179 | 106 | 73 | 41 |

of rebound is greater for tests made at 41° F. and in four cases less than that obtained in tests made at 77° F. In one case the percentage of rebound at each testing temperature is the same.

The test data shown in table 3 indicate that the ductility and resiliency of various blown and normally refined asphalt cements differ greatly from those of the asphalt-rubber fillers. The ductility of the blown asphalts is much lower than that of any of the rubber-asphalt materials, and those normally refined asphalt cements which have high ductility at 77° F. have a low ductility at 41° F. closely approximating that of the blown asphalts. In the case of the rubber-asphalt fillers, including those portions of sample E heated to 500° and 550° F., the ductility at 41° F. is a high percentage of the ductility at 77° F. and in some cases the low-temperature ductility equals or exceeds that at the higher temperature. Thus, the rubber-asphalt materials have a considerably greater ductility than the blown asphalts and, at low temperature, a considerably greater ductility than the normally refined asphalt cements. In contrast to the rubber-asphalt blends, table 3 shows that the percentage of rebound of the asphalts at 41° F. is always equal to or less than at 77° F. Generally the rubber-asphalt materials, as measured by the percentage of rebound, are more resilient at both 41° and 77° F. than the blown or normally refined asphalts. The better performance of these rubber-asphalt materials as joint fillers as compared to the performance of blown and normally refined asphalt cements may be attributed to the ductile and resilient characteristics which these materials possess at low temperature.

## CONCLUSIONS

A comparison of the test methods required by Federal Specification SS—F—336 and of the standard test methods used for the routine examination of asphalts justify the following conclusions:

(1) Provided the average of a sufficient number of individual determinations is taken, the standard penetration test using a needle provides as satisfactory a measure of the consistency of rubber-asphalt materials as the cone penetrometer test.

(2) The softening point test can be used satisfactorily to evaluate the flow properties of rubber-asphalt materials and because of the ease and speed of making this test it is definitely more suitable as a field control test than the flow test required by the specification.

These laboratory tests show that both the temperature to which these rubber-asphalt joint fillers are heated and the time for which they are held at elevated temperatures have the following effects on their physical properties:

(1) The consistency of these materials as measured by either the cone or standard penetration at various temperatures is altered.

(2) The susceptibility to temperature change of the consistency of these materials is altered by variations in pouring temperature and time of heating.

(3) The change in susceptibility, as indicated by tests for flow and softening point, may cause the poured joint to soften and flow under summer heat.

(4) The change in susceptibility of material may cause a failure of the joint filler at low temperatures because of brittleness or lack of cohesive strength.

(5) The resilient properties may be impaired.

For these reasons it is apparent that the users of this type of material should provide for closely controlled temperature during heating with no possibility for local overheating. The optimum temperature for the particular filler should be accurately determined and maintained during the heating period within narrow limits. The batch being heated should not be held at pouring temperature for extended periods. Some effective mechanical means of pouring to expedite the placement of the filler material should be developed. Since these materials if properly handled are superior to the joint-filling and sealing materials in general use, adequate means should be taken to insure that these materials are not unduly injured before they are placed in the joint.

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A revised edition of Public Control of Highway Access and Roadside Development, by David R. Levin, has recently been issued by the Public Roads Administration. The bulletin, originally issued in 1943, has been brought up to date by inclusion of pertinent State legislation through 1946. Much of the text has been rewritten and expanded. New material includes a model bill for a highway-development-rights law.

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