JOURNAL *of the*

BOSTON SOCIETY OF CIVIL ENGINEERS

OCTOBER - 1941

VOLUME XXVIII NUMBER 4

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JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

Volume XXVIII OCTOBER, 1941 Number 4

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Entered as second-class matter, January 15, 1914, at the Post Office at Boston, Mass., under Act of August 24, 1912

Published four times a year, January, April, July and October, by the Society
715 Tremont Temple, Boston, Massachusetts

Subscription Price $4.00 a Year (4 Copies)
$1.00 a Copy

Acceptance for mailing at special rate of postage provided for in Section 1103, Act of October 3, 1917, authorized on July 16, 1918.

The Society is not responsible for any statement made or opinion expressed in its publications.

THE HEFFERNAN PRESS
WORCESTER, MASS.

JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

Volume XXVIII OCTOBER, 1941 Number 4

RECENT IMPROVEMENTS IN THE DESIGN OF SUSPENSION BRIDGE CABLES

BY BLAIR BIRDSALL*

(Presented at a meeting of the Boston Society of Civil Engineers, held on May 21, 1941.)

FOREWORD

IN RECENT years many improvements have been made in the design of suspension bridge cables. These improvements have been reported to the profession from time to time in articles discussing the particular bridges on which these improvements were first made. It has occurred to the writer that it would be desirable to review all these improvements in a single paper for the convenience of those interested in the problem.

INTRODUCTION

There are today three types of suspension bridge cables; namely, Parallel Wire Cables, Parallel Strand Cables and Parallel Rope Cables. For those familiar with the terminology of the wire rope trade, these names are clearly descriptive. For those who are not familiar with these terms, it is important to note that a strand is a group of wires twisted together and a rope is a group of strands twisted together. Both of these are shop operations. Depending on which type of cable is to be built, individual wires, strands or ropes are laid parallel in the field.

*Assistant Chief Engineer, Bridge Department, John A. Roebling's Sons Company, Trenton, New Jersey.

Parallel Rope Cables are similar to Parallel Strand Cables and will not be separately discussed in this paper. Cables consisting of one strand or one rope are simply special applications of the Parallel Strand and Parallel Rope Cables. Cables of this type can be used only for very small structures and will not be discussed.

Parallel Strand Cables

This type of cable did not come into wide use until after the development in 1928-1929 of the process known as prestressing. Before that time, the unpredictable stretch inherent in twisted wire strands made it difficult to use them for suspension bridge cables. This is due to the fact that, for easy and proper bridge erection, an accurately predicted cable stretch is necessary. In 1928 during the studies on the George Washington Bridge erection footbridges, it became evident that a way had to be found to remove the unpredictable stretch in the ropes which were to support the footbridges. Due to the long span of 3,500 ft., a small uncertainty in the modulus of elasticity would have placed the footbridges in the wrong position with respect to the free hanging curve of the main cables. This problem resulted in the development of prestressing.

The process consists of stretching a piece of new rope or strand along a specially constructed track. A tension equal to approximately one-half its ultimate strength is applied to the member. This tension is well above any that will exist in service and it is held long enough to insure that all of the wires have been drawn into close contact with one another. After this treatment, the member is so stabilized that it has a definite modulus of elasticity and its stretch under tension can be predicted with assurance. The prestressing set-up is also useful as a means of establishing the governing points along a member, such as cable band and tower saddle positions. After prestressing the tension is reduced to the average working tension of the member. It is then measured and marked as required. This greatly reduces the amount of field work required.

This opened a wide field to the use of pre-fabricated strands in tension structures. Without prestressing, a structure such as the Skyride at the Chicago World's Fair would have been impossible. Parallel Strand Cables have been installed on the Grand Mere, Bucksport, St. Johns, Maysville, Deer Island, Thousand Islands, Davenport,

Lion's Gate, and many other suspension bridges. In the cables of all these bridges, the strands were arranged in what is known as the "closed" construction. A cross section of one of these cables shows a group of mutually tangent circles arranged in the form of a hexagon. Usually wood or aluminum fillers are used to build this hexagon out to its circumscribing circle, and the whole is wrapped with galvanized wire.

In 1933, with the completion of the San Rafael Bridge in the Dominican Republic, a new development—the "open" construction—appeared in Parallel Strand Cables. This improved the economy of suspension bridges in the short span field. A section through one of these cables shows a group of independent circles arranged in layers to form a rectangle in outline.

The following are a few of the desirable features of this construction: Wrapping is not necessary, since all strands are accessible for painting. Cable bands may be made up of several parts, some of which are placed between the layers or tiers of strands. Thus, several friction surfaces are put to use by the clamping bolts instead of the usual two surfaces. For this reason a smaller number of bolts is required to prevent slippage.

The radius and therefore the length of the tower saddle is a function of the bearing pressure which is exerted on the bottom layer of strands or wires in any cable. In the case of the closed Parallel Strand Cable, the pressure exerted by all the strands of the cable must be carried down to the saddle through the bottom layer of strands. In the case of the open construction, the saddle can be so designed that the pressure of all layers may be carried down to the saddle between the tiers of strands. Thus, the bottom layer must withstand only its own pressure. In this way the length and bulk of the saddle may be reduced to pleasing and economical proportions. Finally, the open construction has considerable artistic value on a small bridge. To the casual observer, a small cable of the open type has the appearance of a much larger cable. This improves the architectural lines of the bridge and provides the layman with a greater feeling of security.

Parallel Wire Cables

This is the familiar type of cable which has been used on all the great monumental structures from and before the Brooklyn Bridge

to the Bronx-Whitestone Bridge. The most recent improvements in the design of this type of cable started with the George Washington Bridge. In the construction of a cable of this type, thousands of individual lengths of wire must be spliced end to end. It had always been common practice to make a splice by cutting or rolling threads on the wire ends and joining them by means of a right and left hand threaded nipple. This necessarily reduced the strength of the wire at that point.

In an effort to improve the efficiency of this connection, the following procedure was developed: Serrations are pressed on the wire ends in such a manner that a cross section at any point contains the full area of the wire. The wire ends are inserted in a sleeve. The sleeve is subjected to such a high radial pressure that its metal flows into contact with the wires and engages the serrations. The result justifies the expensive procedure: Every splice of this type tests as good as or better than other types of splice and a large majority display 100% efficiency.

Experience with the cables of the George Washington Bridge indicated the necessity for shattering another tradition of Parallel Wire Cable design. In the erection of such a cable it is convenient to place the individual wires in groups called strands. (Note: This type of strand which contains parallel wires should not be confused with the twisted strands of the Parallel Strand Cables.) In order to achieve the final desired result of a round cable, these strands are arranged in the form of a hexagon. When all strands have been completed and set in place, the whole cable is squeezed by means of a powerful compacting machine. During this process, the strands lose their indentity and the cable becomes a mass of wires arranged in the form of a circle.

Tradition established the practice of forming the above mentioned hexagon with a flat top and bottom and a vertex at each side. This arrangement was used for the George Washington Bridge Cables. During the compacting operation, a great deal of difficulty was encountered in the attempt to obtain a round cable. This focused attention on the problem and it was at once apparent that, with this disposition of the hexagon, it was necessary to force the top wires up against gravity and the bottom wires down against their normal tension in order to achieve a round cable. Up until that time this had not been

recognized as a serious difficulty. This is probably due to the fact that these cables were nearly 50% larger than the largest previous cables, which were on the Philadelphia-Camden Bridge.

The lessons learned during the building of the George Washington Bridge Cables were put to good use on the cables of the Golden Gate Bridge. Here the hexagon outlining the strands was turned through 30°, placing a vertex at top and bottom. A study was made in an effort to place all wires in such a position that during compacting they would all move horizontally with little or no vertical motion. As a result of this study, it was decided to distribute different numbers of wires to the various strands. Previous to this, it had been customary to place the same number of wires in each strand. Small strands were placed at the vertexes of the hexagon and other changes in the number of wires and placement of strands were made throughout the body of the cable.

This was a highly desirable change but was not in itself sufficient to insure a good completed cable. The finest theoretical strand and wire arrangement would be useless if the strands were not adjusted initially to equal tensions. To clarify this statement, it should be noted that a newly completed strand is adjusted into the partially completed cable simply by setting the strand in place at the support points, such as the tower saddles, and making sure that throughout its length it is parallel to the main body of the partially completed cable. It is obvious that, if the new strand and the body of the cable are not at the same temperature at this time, there will be a difference of length and a resulting uneven distribution of tension, when the temperature becomes uniform throughout the cable including the new strands. A similar difficulty arises if any slumping of the strands is allowed to take place at the tower saddles after adjustment.

The new arrangement of the hexagon made it possible to use a device known as a cable former, which overcame to a large degree the first difficulty mentioned above. One of these was installed every few hundred feet along the cable. It consisted of a frame which enclosed the cable on three sides, leaving the top open, and vertical fingers attached to the bottom of this frame extended up between the vertical tiers of strands. With the new arrangement of the hexagon, it was possible to do this with the smallest possible number of these fingers and with a minimum lateral displacement of the strands. Thus,

during construction, the main body of the partly completed cable consisted of several vertical tiers of strands, each separated by an air space of approximately 1″.

To insure uniform temperature conditions, all strand adjustment was done at night. During the day a large amount of heat was stored up in the main body of the cable. After sundown the comparatively small newly completed strands cooled off quickly and, due to the fact that the main body of the cable was ventilated as described above, its rate of cooling was not much slower than that of the small strand.

To overcome the second difficulty: namely, that of strand slumping in the main tower saddles, triangular zinc fillers were used to fill the triangular voids between the round strands in the saddles.

As a result of these improvements, the completed Golden Gate Bridge Cables were perfectly round and there was more assurance than ever before that the stress was uniformly distributed over the cross section.

There is not much question that all of these improvements are desirable. However, it is rather difficult to obtain a definite quantitative measure of their value. Nevertheless, this measure can be approached in the following manner.

The perfect cable is one in which all wires have the same tension. This condition will be most nearly realized when all wires of the cable are absolutely parallel. When all wires are parallel there will be a minimum percentage of voids in the interstices between the wires throughout the cable.

All the improvements just described have been developed in an attempt to approach as nearly as possible this perfect condition and the percentage of voids existing in the completed cable is a good measure of their effectiveness. Very often a short section of cable is made up in the laboratory in connection with the building of a large cable. All wires of each strand are carefully laid parallel. The strands are then arranged in the form of a hexagon and squeezed into a circular section by means of the compactor which is to be used on the prototype. It is obvious that the percentage of voids between the wires in the completed main cables cannot be less than that existing in this small sample. Thus, a comparison between the two on various bridges will give us a fair indication of the benefits resulting from the improvements which have been described.

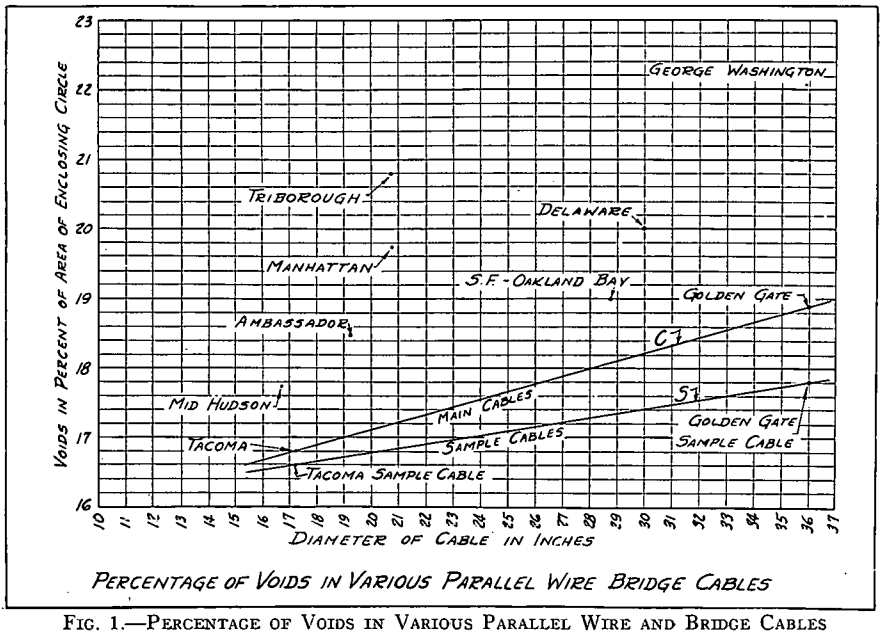

FIG. 1.—PERCENTAGE OF VOIDS IN VARIOUS PARALLEL WIRE AND BRIDGE CABLES

In Figure 1 the percentage of voids is plotted against the diameter of cable for several bridges. The percentage of voids which can be expected for sample cables of various diameters is indicated by line "S" joining the points for the Golden Gate and Tacoma-Narrows Bridge sample cables. These are the only two bridges on which the improvements described in this paper have been employed. Thus, line "C" joining the points for the main cables of these two bridges indicates the percentage of voids which can be expected for various sizes of cables when these improvements are employed. For all other bridges shown, the reduction of voids which could have been attained by the use of these improvements is indicated by the vertical distance between each point and line "C".

CONCLUSION

In discussing the various types of cables, the writer has not intended to convey the impression that any one type of cable is inherently superior to the others. Each has its own field of usefulness. The dividing lines between these fields are determined by economy. Gen-

erally speaking, Parallel Strand Cables, Open Construction, are economical up to spans of 500 or 600 feet. Parallel Strand Cables, Closed Construction, are economical for spans up to around 1,500 feet, and Parallel Wire Cables for all greater spans. These figures are only approximate and will be modified greatly by differences in loadings.

The improvements described in the paper have resulted in better cables of all types. Furthermore, they have made the suspension bridge more economical in the short span field and have made it possible for engineers to study bridges of longer and longer spans with the assurance that the cables can be made to conform with the design assumption of uniform tension throughout the cross-section.

GEOLOGICAL INVESTIGATION OF DAM SITES ON THE ST. MAURICE RIVER, QUEBEC

Illustrating the Objectives and Methods of Dam Site Investigations*

IRVING B. CROSBY, Member†

OBJECTIVES AND METHODS OF GEOLOGICAL INVESTIGATIONS OF DAM SITES

INTRODUCTION

THE investigation of a dam site is a problem in geology whether it is attempted by a geologist or by one untrained in geology. The first essential of such investigations is to interpret the characteristics, arrangement and structural relations of the geologic formations, whether rocks or soil, from scanty, fragmentary pieces of evidence. Even with every possible aid from engineering and scientific methods and related techniques, the evidence is never complete. Therefore recourse must be had to geologic principles by means of which the possibilities of variation in unseen strata or the effects of faulting and other dynamic action may be forecast. This requires most thorough training in all branches of geology, especially structural geology and physiography and also extensive and intensive experience with all types of geologic formations and geologic structures in all types of regions, for the problems of a dam site in a cold climate are radically different from those in a warm climate and those in arid regions have different problems from those in humid regions.

To obtain all possible evidence of value in solving the many complex problems, the engineering geologists must be familiar with engineering methods and with the many related techniques which may serve as tools in obtaining information. He must know when to use them and when their use might be misleading and must be able to interpret the results. He must also understand the new conditions which will exist when the dam is in use, must evaluate the effects of

*This paper is an elaboration of the paper entitled "Objectives and Methods of the Geological Investigation of Dam Sites" presented at the November, 1940, meeting of the Society.

†Consulting Engineering Geologist, 6 Beacon Street, Boston, Mass.

the new stresses on the rocks and forecast the effects of changed ground water conditions.

Due to the variable nature of geological formations and geological structure, the direct application of the precise and mathematical methods familiar to the engineer is seldom possible in the solution of geologic problems which require different methods of work and reasoning than those to which the engineer is accustomed. This essential difference between engineering and geological methods of work and reasoning is a serious obstacle to the complete understanding and cooperation between geologists and engineer which is so necessary to the successful investigation of dam sites.

Methods

Many sciences and techniques are involved in the examination of dam sites but as regards foundation conditions, the first and most important is field geology. A careful, thorough field examination of the site and of a considerable area about it is essential. The type of examination and the branches of geology used will vary with different conditions and with different objectives. The examination must be sufficient to permit not only answering direct questions asked by the engineers but to present to them all the data required in their studies. To obtain satisfactory results the geologist must have a free hand in planning his investigation, with adequate time to carry it out.

For a complete investigation upon which to base final designs, far more information will be necessary than for a reconnaissance. Many related techniques and methods may be necessary to obtain the essential geologic and foundation information. The number and choice of these techniques will depend upon the type of geologic problem. The related techniques which may be useful in obtaining information about geologic and foundation conditions of the site will not be discussed in detail here since they have been outlined in the writer's article in the Symposium on Masonry Dams recently published by the American Society of Civil Engineers.[1] They include, however, engineering methods of which borings are most commonly used, geophysical methods, procedures of soil mechanics, and physical and chemical tests.

[1]Irving B. Crosby. "Geological Problems of Dams," Proceedings, American Society of Civil Engineers, volume 66, pp. 869-890, May, 1940.

Where problems of soil mechanics occur, very close cooperation is necessary between geologist, soil mechanics expert and engineers. It should be the duty of the engineering geologist to determine the conditions in the ground, to locate the different materials, determine their extent, arrangement and relations, and describe their behavior in nature.

Objectives

The objectives of investigations of dam sites vary greatly from a reconnaissance to determine whether the site deserves further study, to a complete investigation to obtain all the information necessary for design and construction. In all types of investigations, however, the essential objective is to interpret conditions in the ground, to show their relations to the proposed engineering structure, to forecast the changes of conditions which completion of the engineering structure will cause and to point out the conditions which must be met by engineering design and procedure to insure a safe and satisfactory dam. Where the details of subsurface conditions cannot be forecast with reasonable accuracy, the limiting possibilities should be outlined and their probability explained. The range of conditions which must be met to insure a safe and satisfactory structure should be pointed out. In this way failure can be insured against and excessively conservative design which may fail to meet the actual conditions can be avoided.

The objectives of the different types of dam site investigations may be conveniently divided into Reconnaissance, Determination of Feasibility, and Complete Investigation. In practice, however, the objectives are not always so clearly defined. An investigation may start as a reconnaissance and pass through the different stages as the project develops. Thirteen years ago the writer began a reconnaissance of dam sites on the St. Maurice River in Quebec studying 32 sites which resulted in discarding many of them. Later detailed investigations were made at the more favorable sites and finally, last year, the third dam was completed.

Reconnaissance

The objectives of a reconnaissance investigation are to determine whether the proposed dam site deserves further investigation and to choose the most favorable of several sites if more than one are avail-

able. A reconnaissance investigation should be made very early in the study of the site, before detailed surveys are carried out. The writer knows of a case where an expensive and time-consuming survey of a site was completed without any geological examination. Later a geologist was called in and found that it would be practically impossible to build a safe dam at the site, but that a short distance upstream was a good dam site. If the geological investigation had been made earlier the cost of the survey would have been saved and much delay would have been avoided.

A reconnaissance investigation, as the term is used here, is almost entirely geological. Test pits to determine the character of overburden or to uncover concealed bed rock are the commonest aids. Whenever the depth to bed rock is an important question, the use of geophysical methods is very desirable and they are especially well suited to this type of investigation. The geologic problems most frequently encountered generally have to do with structural geology and physiography, and in the glaciated regions with glacial geology.

Feasibility Investigation

When it is known that a site has some possibilities and merits further investigation, it is often desirable to determine the feasibility of constructing a specific dam. This type of investigation should provide enough information for determining the most desirable size and type of dam and for making a preliminary estimate of costs. This will involve a thorough understanding of the geology of the site in order that the problems which will affect the choice of type of dam and the estimate of costs can be outlined. An outline of the investigation necessary before final plans are made is also desirable at this time.

The investigation of the feasibility of a site requires much more time than the reconnaissance investigation but is still predominantly geologic in character. Borings are usually needed and they may be supplemented by geophysical explorations. If it is found not feasible to carry the foundation of the dam to bed rock or if very deep excavations are to be made, problems of soil mechanics may arise at this stage of the investigation.

Complete Investigation

After it has been decided that it is feasible to build a specific

dam at the site and that it is desired to do so, complete investigation should be made of all the geologic and foundation conditions related to the design, construction and safety of the dam.

This stage of the investigation involves geology and many other methods and techniques. In fact, the complete investigation often becomes so complex that its essential geologic character is lost sight of. This is unfortunate since unless the geologic character of the problems is constantly in view, there is danger of omission or misinterpretation. The choice of methods, techniques and tests necessary for complete investigation varies so greatly with the character of the site, the type of geologic problem, and the character and size of dam that they will not be outlined.

Investigation of the foundation conditions of a dam site is an attempt to forecast underground geologic conditions. From the very nature of the problem the forecasts cannot be 100 per cent accurate. If properly carried out, however, any variation between actual conditions and forecasted conditions should be minor. It is very important, therefore, that as the foundations are exposed, they should be frequently examined by the geologist and variations from the forecasted conditions should be immediately discussed with the engineers. In this way expensive changes of plans and delays which have occurred in the construction of some dams can be avoided.

ST. MAURICE RIVER DAM SITE INVESTIGATIONS

The St. Maurice River and its Development

The investigation of dam sites on the St. Maurice River in Quebec, begun by the writer in 1928, illustrates every stage and objective of dam site investigation. A total of 32 dam sites on the St. Maurice and its tributaries have been investigated, dams have been completed at three sites and are planned at others.

The St. Maurice River joins the St. Lawrence from the north midway between Montreal and Quebec and drains a large area of the Laurentian Plateau. (See map, Figure 1.) The river has its headwaters in a region of many lakes at an elevation of approximately 1300 feet above sea-level. It descends from the plateau level by a series of rapids separated by quiet stretches. It is thus an ideal power river, having natural storage in the lakes about its headwaters and con-

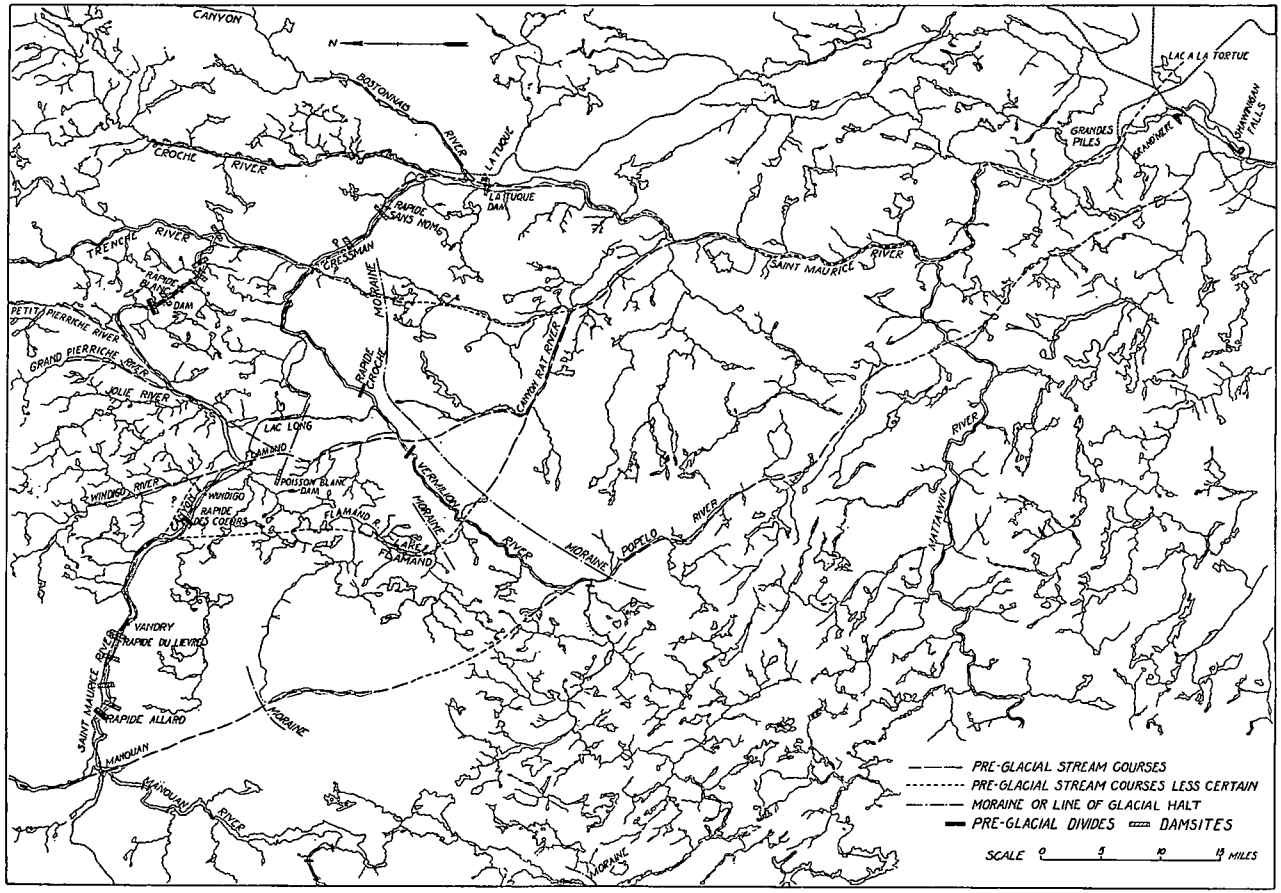

FIG. 1.—MAP OF THE ST. MAURICE VALLEY SHOWING THE PRE-GLACIAL DRAINAGE AND DAM SITES

centrated fall at a number of points. This fall is principally in the Laurentian Highland region, but part of it is in the St. Lawrence Valley where the St. Maurice River descends from the Laurentian granitic rocks to the St. Lawrence River.

Power development on the St. Maurice River began in 1900 at Shawinigan Falls in the St. Lawrence Valley where a head of 140 feet was developed by the Shawinigan Water and Power Company. Two other developments have since been made in the St. Lawrence Valley, at Grand-Mere, upstream from Shawinigan Falls, and at La Gabelle, downstream from the Falls. The Grand-Mere Dam is near the edge of the Highlands and backs the water up into the Highland section of the valley. A large storage dam was built near the headwaters of the River and smaller storage dams were constructed on tributaries, but the power possibilities of the Highland section of the river were undeveloped until 1930 except for a small, partial development at La Tuque by the Brown Corporation. The undeveloped head from the mouth of the Manouan River to the backwater of the Grand-Mere Dam was approximately 750 feet.

The dam sites and investigations described in this paper are all in the Laurentian Highland section of the St. Maurice Valley and are concerned with the development of the 750 feet of head between the mouth of the Manouan River and the foot of La Tuque Falls.

Geology of the St. Maurice Valley

The rocks of the St. Maurice Valley are old metaphoric and igneous rocks, schists, gneisses and granites. They are intersected by faults and the schists and gneisses have been folded. The Laurentian region was worn down by long-continued erosion to an undulating plain or peneplain with occasional hills rising above the general level. This plain was later uplifted and dissected by the streams which eroded valleys 500 to 1,000 feet deep in the surface of the peneplain. Flat-topped ridges and hills were, however, left between the valleys producing a nearly flat skyline. (See Figure 2.)

The dissection of this peneplain was cut short by the approach of the Continental Ice Sheet which covered the entire Laurentian region and modified the topography. Valleys were widened and straightened by the bevelling off of spurs but the ice did not produce new valleys. At the close of the glacial period, the ice melted away,

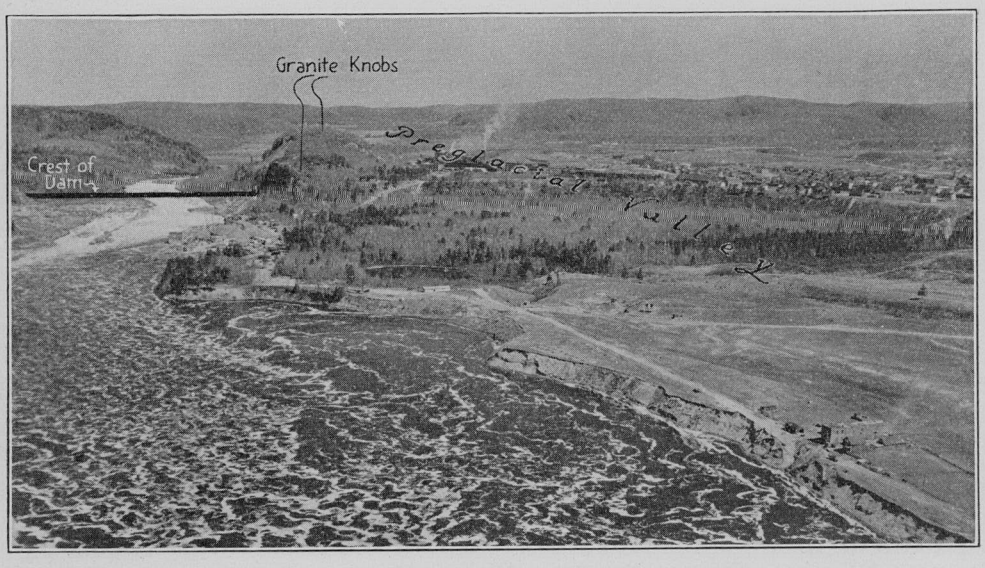

FIG. 2.—AIR VIEW OF LA TUQUE, SHOWING FALLS, DAM SITE, AND PRE-GLACIAL VALLEY. THE SKY-LINE IN THE BACKGROUND MARKS THE LEVEL OF THE PENEPLAIN.

leaving enormous quantities of debris—sand, gravel, clay and boulders —which had been picked up by the ice as it moved across the country. Deposition of this glacial debris was greatest in the valleys and least on the hills with the result that valleys were partially filled, pre-glacial stream courses were blocked and the reborn streams were frequently forced to take new courses, often far from their pre-glacial channels. Many of the present streams pass from one pre-glacial valley to another, crossing buried divides.[2]

The modern St. Maurice River consists of parts of several pre-glacial rivers. (See Figure 1.) The River flows east and southeasterly from the junction of the Manouan River to Flamand, crossing two pre-glacial divides. At Flamand it turns a sharp angle and flows northeasterly for 16 miles through a broad valley. A short distance below the mouth of the Petit Pierriche River, the St. Maurice makes a right angle turn to the southeast, the character of the valley changes suddenly, the river bed changes from sand to rock and the

[2]Irving B. Crosby. "Drainage Changes and their Causes in the St. Maurice Valley in Quebec," Journal of Geology, volume 19, pp. 140-153, February-March, 1932.

river plunges down a long series of rapids, the Rapide Blanc, in a narrow, rocky valley. At the mouth of the Trenche River, the St. Maurice makes another angle and flows southwesterly to Cressman and then turns again and flows southeasterly to La Tuque. The angular course of the river is due to control by the geologic structure. The straight stretches are along zones of weakness, possibly old fault zones.

The great difference in character of the different parts of the St. Maurice Valley is due to the fact that in places the river is flowing in the deeply filled valleys of a large pre-glacial stream, while in other places it is crossing a rock ridge or plunging down the narrow rock gorge of a small pre-glacial stream. These different characteristics of the valley give physiographic clews which make possible the outlining of the pre-glacial drainage system and the forecasting of subsurface conditions.

The geologic features of greatest importance to dam location, design and construction, in this area are: (1) faults, fault zones and zones of close jointing; (2) deeply buried pre-glacial valleys; (3) buried rock ridges and divides under existing streams which may offer feasible dam sites. The location and interpretation of these features without prohibitive subsurface exploration required working out the outlines of the physiography, glacial geology and structural geology of the region.

Geological Investigations

In 1928 investigation was begun by the Shawinigan Engineering Company of the possibility of developing this entire stretch of river by six to eight dams.[3] The writer was requested in October, 1928, to begin a geological investigation of the possible dam sites above La Tuque. Despite the lateness of the season, it was desirable to select the most favorable dam sites from Rapide des Coeurs to La Tuque and a reconnaissance was begun in November. (See map, Figure 1). Ten dam sites were studied and four of these were selected for further investigation.

In the summer of 1929 this reconnaissance investigation was extended to cover that part of the river between Rapide des Coeurs and

[3]C. R. Lindsey. "Considerations Governing the Location of Hydro-Electric Developments on the Upper St. Maurice River," The Engineering Journal (Engineering Institute of Canada), volume 16, pp. 293-300, July, 1933.

the mouth of the Manouan River, and more detailed geological examinations were made at some of the sites investigated the previous November.

Following these investigations, the Rapide Blanc dam site was selected for immediate development. This project required the construction of an earth dam at Poisson Blanc where there is a low divide between the St. Maurice and the Vermilion valleys. In the summer of 1930, further investigations were made at the Rapide Blanc and Poisson Blanc dam sites and construction of these dams was begun.

In 1938 the Shawinigan Engineering Company began construction of a dam at La Tuque for the St. Maurice Power Corporation. The writer had made a preliminary investigation of this site in 1928 for the Brown Corporation. In 1938 he made detailed investigation of the site and made periodic inspections of foundation conditions during the construction period.

The Reconnaissance Investigation

At the beginning of the reconnaissance investigation in November, 1928, the first trip along the river, by canoe and on foot from Rapide des Coeurs to Cressman, showed that in general the valleys were deeply filled with glacial deposits, that the most important problem was the selection of sites where rock was near the surface all the way across the valley and that it would be necessary to make use of physiographic studies to outline the pre-glacial drainage system and select sites where the present river crosses pre-glacial divides or rock ridges and rock is reasonably near the surface. The lateness of the season, the shortage of time, and the inaccessibility of some of the sites precluded the use of test drilling. Application of electrical prospecting to the investigation of dam sites had recently been successfully made at Fifteen Mile Falls, New Hampshire,[4] and it was used at several of these dam sites to great advantage but in many places it could not be used. The electrical prospecting apparatus was hauled in by hand on a toboggan to one of the sites.[5]

Despite the use of geophysical methods where possible, principal

[4]See "Electrical Prospecting Applied to Foundation Problems" by Irving B. Crosby and E. G. Leonardon, American Institute of Mining and Metallurgical Engineers, Technical Publication No. 131, 1928.

[5]The seismic method of geophysical prospecting had not at that time been developed for dam site investigation. It has since been highly developed and has proved successful in dam site investigations similar to those described here.

dependence was, of necessity, upon physiographic study which was continued during December when deep snow prevented ordinary geologic work. Extensive use was made of aerial photographs, wherever available, and finally a long airplane flight was made over the region in order to obtain additional information and to correlate the observations made in different parts of the area. The use of aerial photographs and aerial observations was of very great value. Their value was increased by the fact that there was no topographic map of the region and that the best map was very sketchy.

The study of the pre-glacial drainage showed that the present river is composed of parts of four or five pre-glacial rivers, that there are at least three places where the river is now crossing a rock divide between pre-glacial valleys and several other places where the river has cut across a bend in its old valley and has become entrenched upon a rock spur. These places were selected for dam sites with reasonable assurance that bed rock would not be too deep even though borings had not yet been made at any of them and electrical prospecting had been used at only two of the sites. The rock divides are at Rapide Allard, Rapide des Coeurs and Rapide Blanc. (Fig. 1.)

The clearest case of glacial disarrangement of the drainage is between Rapide des Coeurs and Rapide Blanc. The St. Maurice River flows southeasterly from Rapide des Coeurs to Flamand where it turns a right angle and flows sluggishly to the northeast through a broad, sand-floored valley. The tributary streams come from the northeast and join the river at acute angles, indicating that this part of the valley was originally made by a stream flowing to the southwest. The course of the pre-glacial stream was blocked by a glacial moraine south of Flamand. When the ice disappeared, the reborn river found its old valley dammed by this moraine, backed up, forming a long lake which overflowed a rock divide at the head of Rapide Blanc. This lake became filled with silt and sand, producing a flat-floored valley. A boring at Flamand did not reach rock at an elevation of 765 feet, or 85 feet lower than the rock divide at the head of Rapide Blanc, proving that the pre-glacial valley sloped in a direction opposite to that of the present stream. When the post-glacial St. Maurice poured over the divide at the head of Rapide Blanc, it plunged down a narrow rocky gorge which had been made by a small stream, a tributary of the pre-glacial Trenche River. This physiographic study

proved conclusively that the river is flowing on bed rock in Rapide Blanc, that there is no buried gorge and that feasible dam sites are available.

At the foot of Rapide Blanc, a few miles from the mouth of the Trenche River, the character of the St. Maurice River changes, the banks are sand instead of rock, there is an extensive sand plain between the two rivers and indications are that rock is deeply buried. Immediately below the mouth of the Trenche River, the St. Maurice turns a right angle and flows through a narrow gorge with rock walls. It was not possible to use the electrical prospecting method in the gorge at the actual dam site on account of the ruggedness of the topography, but the method was used successfully on the flat, sand plain between the St. Maurice and Trenche rivers and bed rock was found to be 130 feet beneath the level of the river. This deep buried valley was made by the pre-glacial Trenche which was joined by the small stream coming from Rapide Blanc. It seemed very improbable that the Trenche turned the extremely sharp angle necessary to go through the gorge at the dam site and evidence was found that the pre-glacial valley was buried under a moraine to the west of the dam site. It was concluded therefore that rock was probably near the surface in the gorge and that it was a feasible dam site. This was later confirmed by drilling in the river and bed rock was found to be only 20 feet beneath the surface of the water.

The downstream dam of the series was planned at Rapide Sans Nom between Cressman and La Tuque. The river is here flowing in a gorge between rock banks and there is a rock terrace 80 feet above the water on the left-hand side. It looked at first as if there could be no question but that the river was flowing on rock at this point, but physiographic studies indicated that the pre-glacial stream was at a much lower level at this place and that the bed rock gorge, though narrow, was probably deeply filled with glacial debris. Investigations by the electrical method along the river banks showed that rock was more than 100 feet beneath the surface. It was impossible to locate a dam anywhere along this stretch of the river without crossing a buried gorge and it was decided to use this site. The other sites which were selected in the reconnaissance investigation were Rapide Allard on a bed rock divide, Rapide du Lievre where the buried valley is shallow, Rapide des Coeurs where the river crosses a rock divide,

near the head of Rapide Blanc where the river crosses a rock divide, and the mouth of the Trenche where the river has cut across a rock spur which projected into the pre-glacial valley. It was therefore found feasible to avoid buried valleys at four out of the six dates.

At La Tuque Falls also, the river has cut down upon a rock ridge at one side of its deeply buried pre-glacial valley. The conditions at the Rapide Blanc Dam, Poisson Blanc Dam and the La Tuque Dam will now be described in greater detail.

Rapide Blanc Dam Site

The first dam site examined at Rapide Blanc was two miles below the head of the rapids. Rock is exposed on both banks and there are ledges on the left bluff but few on the right bluff and those visible were much fractured. The rock is granite gneiss intersected by three systems of joint cracks, two of which are practically vertical. The third set of joint cracks dips 20° to the right, or practically parallel to the slope of the right bluff. The combination of frost action and gravity in this bluff has tended to loosen blocks, open joint cracks and cause blocks to slide into the river. On the left side of the river, these inclined joints dip into the bluff,—gravity and frost action counteract each other and there is no tendency towards opening of cracks and moving of blocks. Several hundred feet upstream from the proposed dam axis on the right abutment is a very peculiar canyon which apparently was made by a special type of fault action known as keystone faulting.[6] There was evidence that fault movement had occurred in this canyon since the glacial period, probably during the last 10,000 years.

The presence of this fault combined with the broken condition of the rock in the right bluff made this site undesirable. Search was therefore made for a better site and one was found half a mile downstream. Here the rock is similar but the inclined joints are not well developed, their relation to the river and to the bluff is entirely different, and there is no tendency for blocks to slide down the joint planes towards the face of the bluff. No faults were found in the vicinity of the site and there was no evidence that the solidity of the rock had been affected by movement on the fault upstream. Exten-

[6]Irving B. Crosby. "Further Evidence of Keystone Faulting," Journal of Geology, volume 38, pp. 184-186, February-March, 1930.

sive ledges were exposed, geologic investigation was relatively simple and extensive test drilling was not required. This proved to be a very satisfactory dam site. Bed rock was at or near the surface and was in good condition except that it was intersected by joint cracks. Considerable grouting was done to seal these cracks, but on the whole relatively little foundation treatment was required.

Construction of the dam was begun in 1930 and it was finished in 1934. The elevation of the base of the dam is 770 feet, the elevation of high water is 905 feet and of the roadway on top of the dam, 918 feet, and the operating head is 112 feet.[7] The dam and power plant were designed and built by the Shawinigan Engineering Company, Mr. Sven Sveningson, Chief Engineer. The writer was Consulting Geologist.

The Rapide Blanc Dam formed a reservoir 29 miles long and required a small dam near Lac Poisson Blanc to close a low gap in the reservoir rim. Both the Rapide Blanc and Poisson Blanc dams have been in use since 1934.

Poisson Blanc Dam Site

The Rapide Blanc Dam raises the water-level to elevation 905 and backs it up 29 miles to Rapide des Coeurs. South of Flamand at the head of Poisson Blanc Creek, the elevation of the height of land is only 895 feet. From this divide Augur Creek flows south to the Vermilion River. A dike was therefore required to close this low place in the rim of the Rapide Blanc reservoir. The most feasible place for such a dike was a little north of the divide where Poisson Blanc Creek flowed through a narrow gap in the hills. The elevation of the creek at this point was 884 feet. This gap was divided by a rock hill into two parts, a western part 500 feet wide and a narrower eastern part. Both parts were floored with sand. During the reconnaissance investigation, physiographic evidence was found indicating that depth to rock here was considerable and that the overburden was sand and gravel. Therefore subsurface conditions were investigated by four borings and numerous test pits. It was found that the elevation of rock in the deepest place was 814 feet, or 70 feet below the surface. The borings showed that the overburden was sand and

[7] L. A. Duchastel. "The Rapide Blanc Development." The Engineering Journal (Engineering Institute of Canada), volume 16, pp. 433-437, October, 1933.

gravel. Permeability tests on samples from the test pits indicated that the material was very pervious and seepage estimates indicated that the seepage under this dam would be undesirably large.

In order to reduce the seepage, a cut-off trench was excavated to a depth of 40 feet and two rows of sheet piling were driven to rock. Excavation of the cut-off trench proved that the sand and gravel were very pervious. The dam is built of sand with a concrete cut-off wall resting upon hard-packed sand. The eastern part of the pass was closed by a similar, smaller dike. The length of the main dike is 550 feet, its crest elevation is at 915 feet, giving a height of 31 feet from the lowest part of the valley. Although the Poisson Blanc Dam is small, considerable foundation exploration and treatment were necessary.

La Tuque Dam

Two factors were of prime importance at the La Tuque dam site: the pre-glacial disarrangement of the drainage, and the structure of the rock. The St. Maurice River is here flowing southward in a broad, sand-floored valley. The valley walls, composed of gneiss, rise some 500 feet to the general level of the dissected peneplain. On the west side of the valley, separated from the west valley wall by a saddle, are two granite knobs. (Figure 2.) At the close of the glacial period this valley was filled with sand to a little above the level of this saddle. When the post-glacial river took its course down the valley, it happened to flow between the granite knobs and the valley wall. It cut down rapidly in the sand but soon became entrenched upon bed rock in the saddle, with the result that erosion upstream was practically stopped while erosion of the sand continued downstream, producing falls and rapids where the river poured over the rock ridge in the saddle. Downstream from the falls, the river swung from side to side, eroding a broad valley and leaving a sand plain half a mile wide, upon which stand the town and the Brown Corporation Mill, between the river above the falls and the lower valley below the falls. (Figure 2.) This unusual topographic situation has provited an excellent opportunity for power development. Although the St. Maurice Valley is nearly a mile wide here, the main valley is blocked by a natural dam of sand half a mile wide, and the river is

entrenched in a narrow gorge which could be closed by a relatively short dam.

It was suspected on physiographic evidence that bed rock was very deeply buried in the center of the main valley. Investigation as to the depth of rock was made by the electrical prospecting method on the floor of the lower valley below the falls and rock was found to be 410 feet beneath the surface, or practically at sea level.

A concrete gravity dam has been bulit across the gorge near the lower end of the falls where the elevation of the river bed was 400 feet. The dam is 1337 feet long, and consists of west bulkhead, sluice section, intake section and power house, and east bulkhead.[8] The height of the main dam is 100 feet, but from the power house foundations to the top is 160 feet. Before construction of the dam, the elevation of the river above the falls was 480 feet and the drop was 100 feet. By means of tailrace excavation a total head of 114 feet has been obtained.

The rock under the river at the dam site and in the mountain west of the river is gneiss but the two knobs east of the river consist largely of granite. The principal problem of this site has to do with faults. In the preliminary geological investigation in 1928 geologic evidence was found of a fault on the east side of the river under the talus along the west bases of the granite knobs. When this fault was later investigated by drilling and excavation, it proved to be a fault zone 200 feet wide, dipping down under the granite knobs at an angle of 47° from the horizontal. Since this zone extends up and down the river for an unknown distance, probably a matter of miles, there was no possibility of locating the dam so that it would avoid this fault. The fault zone is at the extreme eastern or left end of the dam, between the river and the granite knob. Three nearly parallel faults, probably branches from the main fault zone, were found under the river. One of these showed on the surface of the river bed as a small gorge ten feet deep. The power house excavation showed that 20 feet deeper this fault and the other faults were merely cracks half an inch wide, tightly filled with clay gouge.

These faults in the river separate two areas of different geologic structure. On the west side the bedding of the gneiss dips diagonally

[8] J. A. McCrory. "Construction of the Hydro-Electric Development at La Tuque." The Engineering Journal (Engineering Institute of Canada), volume 24, pp. 54-63, February, 1941.

upstream at an angle of about 30° from the horizontal. On the east side of this fault the gneissic structure strikes up and down the river and dips to the east. The relation of the rock structure to the dam is ideal in the western part of the site. The bottom of the foundation excavation was naturally corrugated, due to the dip of the gneissic structure, thus providing excellent footing for the dam, making it proof against sliding.[9] The sluice section of the dam is on the western part of the site and the intake section and power house on the eastern part, the junction of these two sections being close to the westernmost of the three small faults. Under the intake section of the dam the gneissic structure and principal joints strike up and down the river, but the relation of the geologic structure to the dam is not so important here since, on account of the necessarily deep excavation for the draft tubes, the dam is set deeply into rock.

The fault zone at the east end of the dam is crossed by the east bulkhead section. The rock in this zone was cut by numerous fractures and secondary faults. Part of the rock in the upper part of this zone was decomposed. The foundation excavation was carried down along the footwall, which dipped 47° to elevation 380, or about a hundred feet beneath the original surface. The rock in the bottom of the excavation was fractured but not decomposed. It was consolidated by grouting to a depth of 20 feet in holes spaced on five-foot centers each way. A cut-off wall at the upstream face of the dam was carried down along the footwall of the fault zone to elevation 330. It was not considered necessary to extend this wall all the way across the fault zone which would have required excessively deep excavation, but a grout curtain, consisting of deep holes on five-foot centers, was extended to the hanging wall of the fault. The deepest of these holes went down to elevation 270. An inspection tunnel in the east bulkhead was designed to permit future grouting.[10]

Although the geological conditions at this site were far from ideal, they were overcome without serious difficulty and a satisfactory dam has been constructed. The dam and power plant were designed by the Shawinigan Engineering Company, Mr. J. A. McCrory, Chief

[9]For further discussion of this and other foundation features see Irving B. Crosby, "Discussion on Construction of the Hydro-Electric Development at La Tuque," The Engineering Journal, volume 29, pp. 297-298, June, 1941.

[10]J. A. McCrory. "Construction of the Hydro-Electric Development at La Tuque." The Engineering Journal (Engineering Institute of Canada), volume 24, p. 58, February, 1941.

Engineer. Mr. Hardy S. Ferguson was Consulting Engineer and the writer was Consulting Geologist.

The La Tuque development is a good example of the proper relation of geologist and engineers. The essential geological conditions were outlined by an investigation early in the study of the site. This showed it was a feasible site but that there was one serious problem which must be investigated and provided for. Later, detailed geologic investigations were completed. The geologist interpreted the subsurface conditions and pointed out their practical significance and the engineers designed the dam to meet those conditions. Frequent examinations were made during the construction period as different parts of the foundations were uncovered. The various geological conditions and problems were discussed with the engineers in the field and in the office. It was known in advance that certain unfavorable conditions existed at the La Tuque dam site, but by means of adequate investigation and competent engineering, these conditions have been overcome and a safe and satisfactory dam has been constructed.

SIMPLIFYING EARTHWORK COMPUTATION

By Thomas C. Coleman, Member*

In laying out a highway over new location, a major factor in the economy of the design of line and grade is the quantity of earthwork involved in the grading operation. Computing the earthwork is ordinarily a rather tedious procedure, that in addition to surveying and plotting cross sections of the original ground, also involves plotting cross sections of proposed construction, measuring the area of these with a planimeter or other device, and then computing the earthwork.

For a satisfactory estimate on which to base construction this work is necessary and justifiable, but there are cases when the amount of labor involved renders it rather cumbersome if not out of the question entirely. Such problems as the preliminary comparison of alternate routes, the effect of suggested changes of line or grade, or any case where speed of estimating is the essential requirement, would seem to justify the existence of a simpler method, even if some sacrifice is required in the matter of accuracy. The method herein proposed is offered as a step in that direction, the particular advantages being that the use of the planimeter is not required, and that it is not necessary to plot the cross sections of the proposed construction so long as a typical section is available.

The following discussion mentions only excavation, but applies equally well to the computation of embankment quantities, except of course when ledge excavation is mentioned.

For the present let it be assumed that at a certain section, say at station 1 + 00, the original ground, although not level, does give a cross section that can be satisfactorily approximated by a straight line, such as the line *AB* in Fig. 1. And let it be further assumed that it is required to excavate to lines *ACDB* for the proposed construction. Then the area of the quadrilateral *ABDC* will be a measure of the quantity of excavation necessary. In this figure it

*Junior Civil Engineer, Massachusetts Department of Public Works, 100 Nashua Street, Boston, Mass.

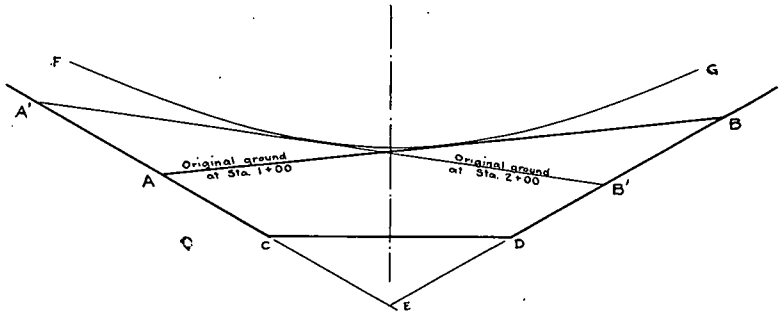

FIG. 1.

is to be noted that the line *CD,* being dependent only upon the width of the proposed highway, will generally be the same for every cross section on the project. In the case of the lines *AC* and *DB,* these represent the slopes of the cut and are determined by an assumed safe angle of repose for such slopes. They will therefore generally have the same slope on every section, although they will vary in length, since the length is determined by the depth of cut.

A common device in preliminary studies is to plot the proposed construction lines *AC, CD,* and *DB,* on a piece of tracing cloth or other transparent material, and superimpose this tracing cloth on the plotted cross sections of the original ground. In this way the effects created by various different conditions of grade and alignment can be observed. A somewhat similar method can be used to determine the quantity of excavation involved, by having a set of curves plotted on a sheet of transparent material in such a way that when superimposed on any cross section in the proper position, the area of the cross section, and therefore the quantity of excavation, will be indicated directly.

This can be illustrated by referring again to Fig. 1. The line AB, being the cross section of the original ground will have been plotted on the cross sections. The lines *AC, CD,* and *DB,* being proposed construction, will be laid out on a tracing and superimposed over the plot of the line *AB* in such a way as to show the proper relationship between the original ground and the proposed construction, thus in Fig. 1 the lines *AC, CD,* and *DB* will be drawn on the tracing, while the line *AB* will be visible through it. It is now re-

quired to lay out a set of curves on the tracing such that the area *ABDC* can be readily ascertained.

Now let it be assumed that when the tracing is superimposed over some other section, such as that at station 2 + 00, the original ground as seen through the tracing occupies another position such as *A'B'*, although otherwise the section is the same and involves the same amount of excavation as before. It follows that the area *ABDC* will be equal to the area *A'B'DC*. Now it is possible to draw a curve on the tracing, such as *FG*, that will fall tangent to *AB* in one case and *A'B'* in the other case, or in fact to any line that would give the same cross sectional area.

If a number of such curves are plotted on a sheet of transparent material, and then superimposed over plotted cross section of the original ground at any station whatever, the amount of earthwork involved can be determined immediately by observing which particular one of the curves is tangent to the line of the original ground, using a straight edge or piece of thread to smooth out irregularities if such a procedure seems desirable in order to obtain a better approximation.

At first glance it would appear that a set of these curves, after having been plotted, would be extremely limited in their use in that they would be useless for any project other than the one for which they were designed. This is not so, as will be seen from the following considerations. Again referring to Fig. 1, extend the lines *AC* and *BD* until they intersect each other at *E*, forming the triangles *ABE* and *CDE*. Now the triangle *CDE* depends only upon the line *CD* and the slopes of the lines *AC* and *BD*, all of which are the same for every section of the project, at least if the effect of banking of curves is neglected as being of minor importance. The triangle *ABE* is equal to the sum of the quadrilateral *ABDC* and the triangle *CDE*, from which it follows that the curve *FG*, being such that all of its tangents, such as *AB*, will cut off the same area of the quadrilateral *ABDC*, it will also be such that all of its tangents will cut off the same area of the triangle *ABE*. This is equivalent to saying that the shape of the curve *FG* is independent of the width of the proposed highway and depends only upon the rate of the proposed slopes. In regard to the rate of slope of *AC* and *BD*, these are governed by the safe angle of repose for the material composing the

slopes which is always more or less a matter of standard practice, and if varied at all is restricted to a few selected standards.

The next consideration is the matter of laying out the curve *FG*, or any other curve in the series. As a mathematical problem it can be stated as follows: Within the angle formed by two intersecting straight lines it is required to lay out a curve, such that any tangent to the curve will form a triangle with the two given straight lines which will have an area equal to the area of the triangle formed by any other tangent to the curve in the same manner. This is the problem that will now be analyzed.

For the purpose of discussion let it be assumed that the safe angle of repose for earth is two units in a horizontal direction to one unit vertically. In Fig. 2, let *EA* and *EB* represent the two

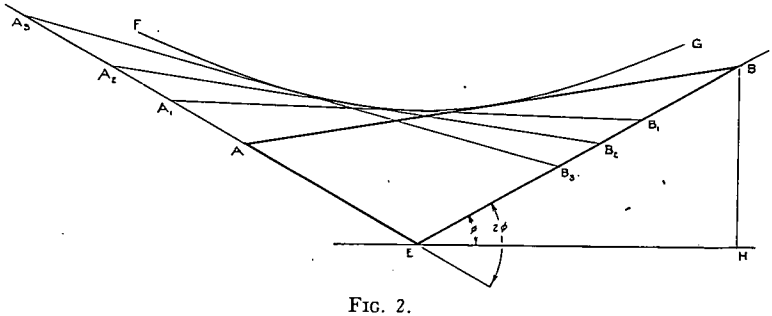

FIG. 2.

slope lines extending indefinitely. Let *EH* be a horizontal line and *BH* be a vertical line. Let *S* be the slope of the lines *EA* and *EB*, and let ϕ represent the angle that each of the slope lines makes with the horizontal. Then $S = \text{Tan } \phi$. In this particular case, $S = \frac{BH}{EH} = \frac{1}{2} = 0.5 = \tan \phi$. Then $\phi = \arctan S = \arctan 0.5 = 26° \ 34'$, and the angle between *EA* and *EB* $= 2\ \phi = 53° \ 08'$. The problem is to lay out the curve *FG* in such a way that when tangents be drawn to the curve, such as *AB*, A_1B_1, A_2B_2, etc., they will intersect the lines *EA* and *EB* such that the area of the triangle *EAB* = area of EA_1B_1 = area of EA_2B_2, etc.

Without undertaking a proof at this time, it will be reasonably evident that the required curve will be symmetrical about the center line, it will not cross either of the slope lines *EA* or *EB*, but that as

it extends away from the center in either direction it will always approach one of these lines without ever reaching it. These conditions would seem to be satisfied by a hyperbola, and as a matter of fact the curve is a hyperbola, but the problem now is not so much to name the curve as to lay it out.

Perhaps the most convenient way to lay out a series of these curves is by means of coördinates, or by the use of a "grid," except that instead of making the lines of the grid run in the conventional horizontal and vertical directions, the grid used here will be what might be called an oblique grid, having one set of lines run parallel to the slope line *EA* and the other set parallel to *EB*, as shown in Fig. 3.

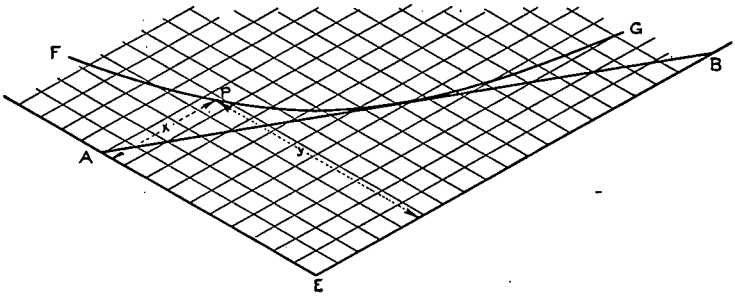

FIG. 3.

In this case the distance x will be measured on or parallel to the line *EB*, and y will be measured on or parallel to the line *EA*. Thus, in Fig. 3, the coördinates of the point *P* are x and y as indicated. It will now be shown that if x and y are measured in this manner the equation of the curve *FG* will be $xy = k$; where k is a constant, the value of which depends only upon the area of the triangle *EAB* and the slope angle ϕ.

In this oblique grid it will be noticed that the line *EA* becomes the y axis, in that $x=0$ for all points on this line, and similary for all points on the line *EB* (which becomes the x axis) $y=0$. Also on any line parallel to *EA* or *EB*, x or y, as the case may be, will have some other constant value.

In Fig. 4, let *M* be any point on the line *AB*, and let *ML* be parallel to *EA*, intersecting *EB* at *L*. Let $EL = x_a$ and let $LM = y_a$, thus making x_a and y_a the coördinates of the point *M*. Also let $EA =$

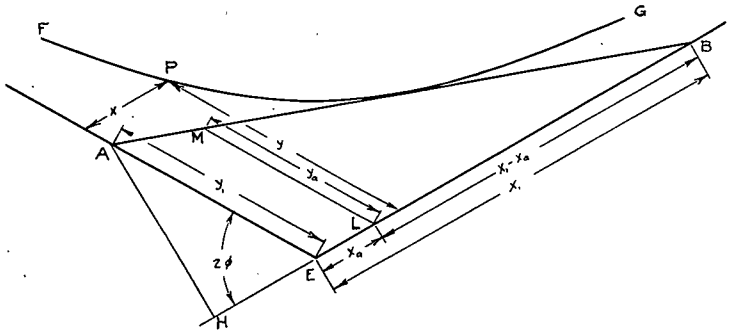

FIG. 4.

y_1 and $EB = x_1$. Since the triangle LMB is similar to the triangle EAB;

$$\frac{EB}{EA} = \frac{BL}{ML} \text{ therefore, } EB \times ML = EA \times BL$$

but $BL = x_1 - x_a$, which gives

$$x_1 y_a = y_1(x_1 - x_a) \tag{1}$$

This reduces to,

$$y_1 x_a + x_1 y_a = x_1 y_1 \tag{2}$$

Now if a perpendicular, AH, be dropped from the point A to the line EB (extended) the distance AH is equal to $y_1 \sin 2\phi$. Let the area of the triangle AEB be represented by Q, then,

$$Q = \tfrac{1}{2} EB \times AH = \tfrac{1}{2} x_1 y_1 \sin 2\phi \tag{3}$$

then,

$$x_1 y_1 = \frac{2Q}{\sin 2\phi} \tag{4}$$

For convenience let $Q' = \dfrac{2Q}{\sin 2\phi}$ and using this notation substitute into Equation 2 the value of $x_1 y_1$ that is obtained in Equation 4, which gives:

$$y_1 x_a + x_1 y_a = Q'. \tag{5}$$

In equation 5, x_1 and y_1 are constants defining the line AB, and x_a and y_a are coördinates of any point on the line.

For the present neglect the fact that AB is tangent to the curve FG, and solve for the intersections of the line AB with the curve FG. Let it be assumed that the equation of the curve FG is:

$$xy = k, \tag{6}$$

where k has a value that is as yet undetermined, and x and y are the coördinates of any point on the curve. At the intersection of the line AB with the curve FG, the values of x_a and y_a in Equation 5 become equal to x and y respectively, and the equation becomes;

$$y_1x + x_1y = Q'. \tag{7}$$

Solving Equations 6 and 7 simultaneously gives:

$$x = \frac{Q' \pm \sqrt{Q'^2 - 4x_1y_1k}}{2y_1}. \tag{8}$$

Equation 8 gives two possible values for x because no consideration was given to the fact that the line AB is tangent to the curve FG. Since the value of k is still to be determined, the line AB can now be made tangent to FG by assigning to k such a value that the two solutions for x coincide and form but a single solution. To accomplish this result the expression under the radical sign in Equation 8 must be made equal to zero, thus,

$$Q'^2 - 4x_1y_1k = 0 \tag{9}$$

but,

$$x_1y_1 = Q',$$

and substituting this in Equation 9;

$$Q'^2 - 4Q'k = 0, \tag{10}$$

and solving, gives:

$$k = \frac{Q'}{4}. \tag{11}$$

Returning to Equation 6, of the curve FG, and substituting in it the value of k from Equation 11,

$$xy = \frac{Q'}{4} = \frac{2Q}{4 \sin 2\phi} = \frac{Q}{2 \sin 2\phi}. \tag{12}$$

The significance of Equation 12 is that here is the equation for the curve FG, which has been made tangent to the line AB, but the equation of the curve does not depend upon the position of AB except in so far as the area of the triangle EAB, that is Q, depends upon AB. In other words the curve FG can be plotted tangent to the line AB when all that is known about AB is the area EAB that it encloses. The curve FG must therefore be tangent to all possible positions that AB may have in order to enclose that area. This is what was to be proven.

From Equation 12 any curve of the series can be plotted by

assigning the proper value to Q. It will also be an advantage to plot the points where the different curves intersect the center line, such as the point V in Fig. 5, which is the point at which the curve

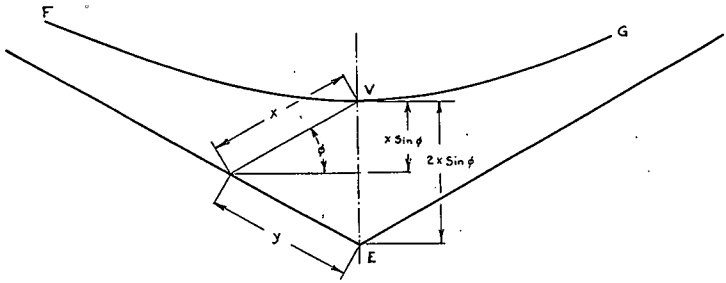

FIG. 5.

FG crosses the center line. To do this the distance EV should be computed. This can be done by making $x = y$ and the distance $EV = 2\ x \sin \phi$. Thus:

$$xy = \frac{Q}{2 \sin 2\ \phi}. \tag{12}$$

Making $x = y$, gives:

$$x^2 = \frac{Q}{2 \sin 2\ \phi} = \frac{Q}{4 \sin \phi \cos \phi} \tag{13}$$

Then:

$$x = \sqrt{\frac{Q}{4 \sin \phi \cos \phi}} \tag{14}$$

Making $EV = 2\ x \sin \phi$, gives:

$$EV = 2 \sin \phi \sqrt{\frac{Q}{4 \sin \phi \cos \phi}} \tag{15}$$

which reduces to:

$$EV = \sqrt{Q \tan \phi}. \tag{16}$$

After plotting the points on the center line, other points on the curves can be plotted from Equation 12 by laying out the oblique grid and assigning the various values to Q. It is of interest to note that if the values of Q are assigned, as they naturally would be, at uniform intervals, then the curves of the series will be spaced uniformly along any line of the grid. Thus, in Fig. 6, let the line OS

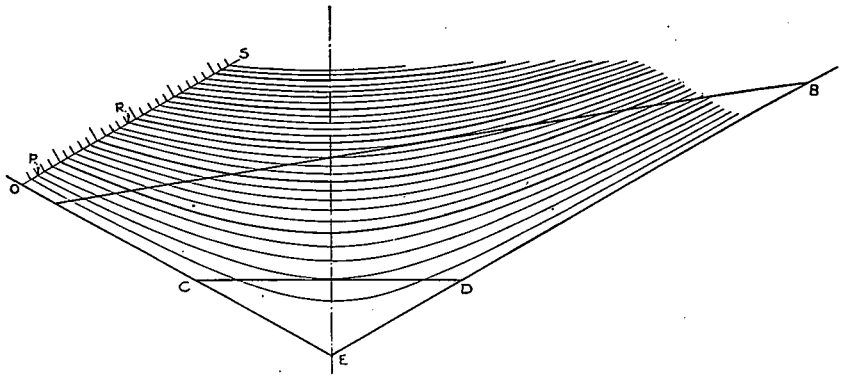

FIG. 6.

be parallel to the line EB, then the value of y will be constant for all the points on OS. According to Equation 12,

$$xy = \frac{Q}{2 \sin 2 \phi}, \tag{12}$$

and dividing through by y,

$$x = \frac{Q}{2y \sin 2 \phi}. \tag{17}$$

In other words, since y is a constant along OS, x becomes equal to Q divided (or multiplied) by a constant, or the value of x along OS will be directly proportional to the value of Q, the area of the triangle ABE. Furthermore, if x and y are measured in inches and Q in square inches, and if the grid is laid out so that the values of y are always equal to some even number of inches divided by $2 \sin 2 \phi$, this factor will cancel out in the expression for x (Equation 17) and the points can be plotted by simply spacing off an even number of points to the inch on each of the lines on the grid. That is, the unit used in laying out the grid in this case should be one inch divided by $2 \sin 2 \phi$. If this is done, then starting at the origin the first line of the grid will intersect the series of curves at points one inch apart; the second line will do the same at points one-half inch apart; the third line at points one-third of an inch apart, and so on.

This uniformity is of value in another respect. It will be recalled that in Fig. 1 the area $ABDC$, was to be obtained by subtracting the area CDE from the area ABE. Now in Fig. 6, if a

properly calibrated scale be laid out along the line *OS*, the distance *OP* will indicate the area *CDE*, and the distance *OR* will give the area *ABE*. Then the difference between *OP* and *OR*, which is the distance *PR*, will indicate the area *ABDC*. By properly selecting the line *OS* so that the distances will correspond to some standard scale, the distance *PR*, and therefore the area *ABDC* can be measured directly.

Various modifications might be considered. The curves might be plotted in ink of a different color than that used in plotting the cross sections to aid in differentiating the lines of one from those of the other. They might be plotted on the glossy side of the tracing cloth so that the line *CD* (Fig. 1) could be plotted on the dull side in pencil and erased afterwards, or for that matter it need not be erased. Readings might be in cubic yards for a given distance instead of square inches. Other variations may be worth while.

In side hill sections there is but one slope involved. In this case the line *CD* will intersect the line *AB*, as in Fig. 7. A special chart

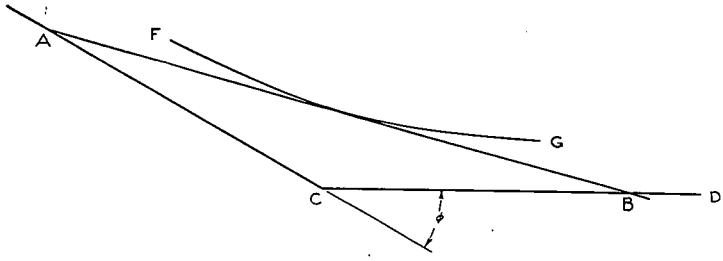

FIG. 7.

for this condition may be constructed by making the line *CD* level, with the line AC having a slope corresponding to the angle of repose, and then constructing a set of curves on the basis of having the lines *AC* and *CD* as axes of the oblique grid. In this case the angle between *AC* and *CD* will be ϕ instead of $2\ \phi$. There is however, an approximation introduced here in that side walks, berms, area taken up by surface and subgrade material, all constitute elements that discount the suitability of considering the grading surface as being a level straight line.

Another complication is encountered where ledge is involved, as

the side slopes in ledge are much steeper than in earth cuts. In Fig. 8, the area *ABDC* represents a cut section in ledge. The steep

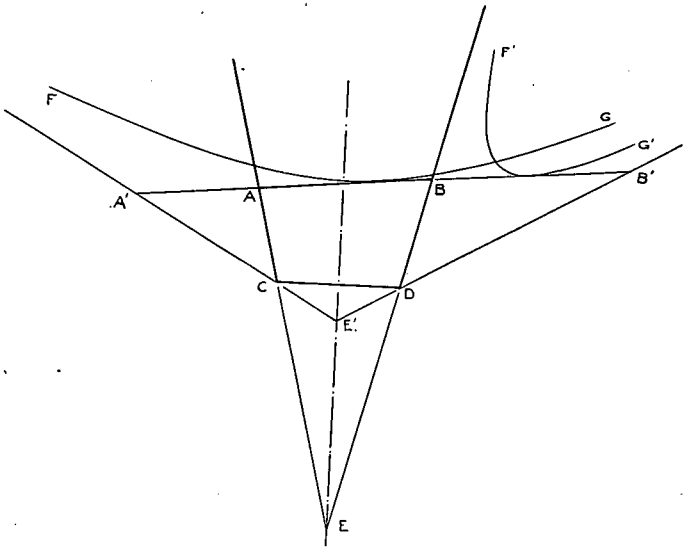

FIG. 8.

slopes involved in such a section, if extended to their intersection at *E*, would make a chart of prohibitive size, in which the major part of the area would be in the triangle *CDE*, which is of no particular interest. Furthermore it is quite conceivable that ledge would be encountered on one side of a cut and not on the other. Perhaps the most satisfactory method would be to apply the regular earthwork chart as at *A'E'B'*, finding the area by means of the curve *FG*, and then make a correction for ledge by having a small auxiliary chart with *BD* and *B'D* as oblique coördinate axes, from which the area *BDB'* can be found by means of the curve *F'G'*. The area *A'CA* would be determined in the same way as *BDB'*.

All of the discussion up to this point has been on the assumption that the line of the original ground can be approximated with a straight line. In some cases the original ground will fall along a line that is near enough to a straight line so that an accurate approximation can be made without difficulty. There may be times when the original ground is quite irregular. In such cases, if it is desired

to have a closer approximation than is to be readily estimated by eye, a simple geometrical construction can be used. In Fig. 9a let *AB* and

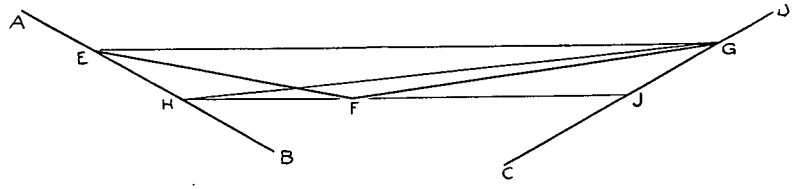

FIG. 9A.

CD be the proposed slope lines and the broken line *EFG* be the surface of the original ground, which in this case is approximated by a broken line having one angle point at *F*. It is desired to draw a straight line in such a manner that it will cut off the same area between the lines *AB* and *CD* as was cut off by the broken line *EFG*. Draw *EG* and through *F* draw *HJ* parallel to *EG*, which determines the points *H* and *J*. The line *HG* is the line desired and will indicate the correct area on the chart. A line connecting *E* and *J* would serve the same purpose. To prove that the line *HG* will cut off the same area as the line *EFG*, consider the area above the line *EFG*. This is equal to the area above the straight line *EG* plus the area of the triangle *EFG*. On the other hand if the area above the line *HG* be considered, it will be seen that it is equal to the area above the same straight line *EG*, plus the area of the triangle *EHG*. The area of the triangle *EFG* is determined by its base *EG* and its altitude, which is the perpendicular distance between the parallel lines *EG* and *HJ*. The triangle *EHG* has the same base *EG*, and the same altitude, and therefore the same area. This operation can be easily performed by means of a parallel ruler with a thread fastened to one blade. Setting one blade along the line *EG*, adjust the other blade to pass through *F*, and determining the points *H* and *J*. Drawing the thread from *H* to *G* will make it possible to read the area from the chart by means of the thread.

In Fig. 9b the broken line *EFGK* represents the original ground, and contains two angle points within the limits of the slope lines. The parallel ruler can again be used and the point *H* located as in Fig. 9a. Now by means of a pencil or needle point the blade of the ruler that has been occupying the position *HF* can be pivoted

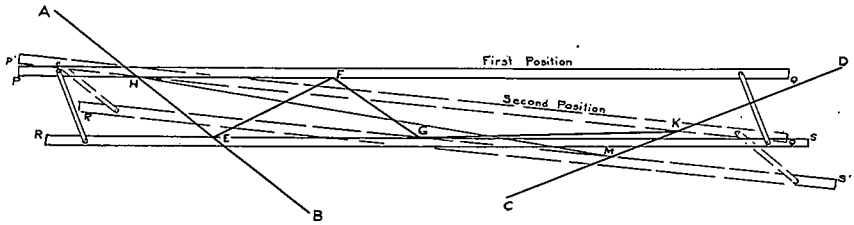

FIG. 9B.

at H and swung into the position HK. The other blade can now be adjusted so as to pass through G, and thereby caused to occupy the position GM, fixing the point M. The thread can now be drawn from H to M, indicating the desired area. The proof follows the same reasoning as before but carries it another step.

Fig. 9c illustrates a case in which the original ground has three

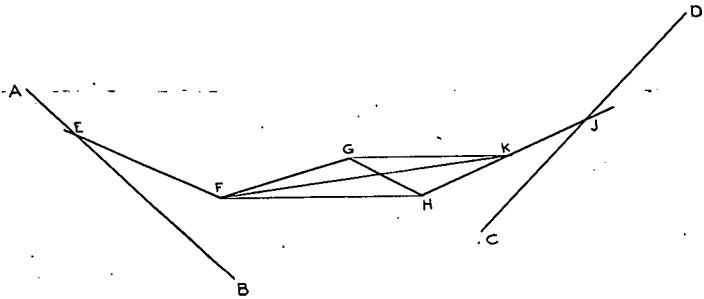

FIG. 9C.

angle points within the proposed slope lines. The slope lines are AB and CD as before, and the original ground is represented by the line $EFGHJ$. Draw FH, and through G draw a line parallel to FH, cutting HJ at K. The line $EFKJ$ will now replace the line $EFGHJ$ and has only two angle points between the slope lines, so it has already been dealt with.

This last construction differs from the case of one or two angle points in that it does not involve the slope lines; so it can be performed directly on the cross sections, and then the cross sections can be used as if the original ground had occupied the corrected position. The same reasoning applies if there are more than three angle points. They can be reduced to two angle points directly on

the cross sections by using the same procedure successively. It seems hardly to be recommended however in an approximate method such as this that elaborate procedures should be over extended. It is a case where a little judgment will replace a considerable amount of mechanical manipulation.

In appraising this method of computing earthwork it should be recognized that a certain amount of time will be required to lay out the necessary charts, but after they have been made they are available for future reference. One set of charts will suffice for all cases as long as the angle of repose of the earth slopes is assumed to be the same. This method is hardly suitable for shallow cuts or fills; but on the other hand when deep cuts or high fills are to be encountered, that is in those cases where the quantity of earthwork assumes a major importance, a rough preliminary estimate can be made rapidly if the proper charts are at hand. It is an advantage if the original ground is fairly smooth, but some unevenness is not a serious matter, and even considerable irregularity can be corrected. The question depends upon the degree of approximation that is to be permitted. Using this method a result should not be expected to be within about 5%, which is hardly sufficient for a precise estimate or as a basis of payment. On the other hand, in the matter of balancing quantities it is hardly probable that a result can come much closer in any event. It is only fair to remember that regardless of the method, computing earthwork is at best a matter of approximation, with many factors of uncertainty and personal equation affecting the result.

Supplement

The fact that the curve under discussion is actually a hyperbola can be demonstrated without too much difficulty. In Fig. 10, let XX' and YY' represent the usual rectangular axes with the origin at O, and let AO and OB represent the slope lines that form the oblique axes, and let these make an angle ϕ with the X axis. Let P be any point having coördinates with reference to the X and Y axes equal to x and y respectively. As x and y are now being used as rectangular coördinates, let s and t be taken as oblique coördinates. From P draw PD parallel to AO and PC parallel to YY'. From D draw DE parallel to XX' and DH parallel to YY'. Now if P is a point on the curve FG, its oblique coördinates must satisfy the condi-

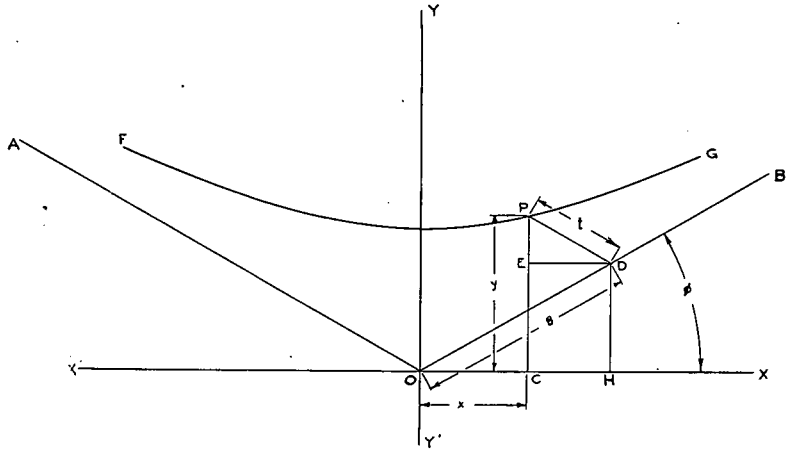

FIG. 10.

tion expressed in Equation 12, if x and y be replaced with s and t. This gives,

$$st = \frac{Q'}{4} \tag{18}$$

This can be shown to be the equation of the hyperbola by transforming it into rectangular coördinates and showing that it can be made to assume the form of the typical hyperbola equation, viz:

$$\frac{x^2}{a^2} - \frac{y^2}{b^2} = \pm 1 \tag{19}$$

where a and b can be any real quantities, positive or negative. Now examination of Fig. 10 will show that, $OD = s$, $PD = t$, $OC = x$, and $PC = y$. Also it follows directly that, $DE = CH = t \cos \phi$, $EP = t \sin \phi$, $OH = s \cos \phi$, and $DH = EC = s \sin \phi$.

Then,

$$y = PC = EP + EC = t \sin \phi + s \sin \phi = (s + t) \sin \phi \tag{20}$$

and,

$$x = OC = OH - CH = s \cos \phi - t \cos \phi = (s - t) \cos \phi \tag{21}$$

From Equation, 20

$$s + t = \frac{y}{\sin \phi} \tag{22}$$

and from Equation 21,

$$s - t = \frac{x}{\cos \phi} \tag{23}$$

then,

$$s = \frac{x}{2 \cos \phi} + \frac{y}{2 \sin \phi} \tag{24}$$

and,

$$t = -\frac{x}{2 \cos \phi} + \frac{y}{2 \sin \phi} \tag{25}$$

$$st = -\frac{x^2}{4 \cos^2 \phi} + \frac{y^2}{4 \sin^2 \phi} \tag{26}$$

Rearranging the above equation, changing the signs, and substituting the value of st as given in Equation 18,

$$\frac{x^2}{4 \cos^2 \phi} - \frac{y^2}{4 \sin^2 \phi} = -\frac{Q'}{4} \tag{27}$$

Then dividing Equation 27 by $Q'/4$ gives,

$$\frac{x^2}{Q' \cos^2 \phi} - \frac{y^2}{Q' \sin^2 \phi} = -1 \tag{28}$$

In Equation 28, if $Q' \cos^2 \phi = a^2$, and $Q' \sin^2 \phi = b^2$, the equation will take the form of the typical equation of the hyperbola as shown in Equation 19, showing that FG is this type of curve.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

September 24, 1941.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the Boston City Club and was called to order by President, Albert Haertlein. This was a Joint Meeting with the Boston Post, Society of American Military Engineers. Seventy persons attended this meeting and supper.

President Haertlein in conducting the business meeting of the Boston Society of Civil Engineers announced the death of the following members:

Raymond C. Allen, who had been elected a member November 21, 1906, and who died June 20, 1941.

Louis F. Buff, who had been elected a member May 20, 1903, and who died August 29, 1941.

Henry C. Robbins, who had been elected a member May 19, 1905, and who died May 6, 1941.

Sidney Smith, who had been elected a member June 17, 1885, and who died June 26, 1941.

The Secretary reported on the Election of New Members.

Grade of Member: *Richard G. Bergstrom, Frank R. Berman, Fozi M. Cahaly, Warren M. Campbell, Arthur L. Dow, Chester R. Spielvogel.

Grade of Junior: †Norman B. Cleveland, †Paul M. Levenson, †Daniel W. Miles.

The President stated that a letter had been received from Prof. J. B. Babcock, Chairman of the Nominating Committee, in which this Committee presented the nomination of Lawrence G. Ropes for the office of Director, to fill the vacancy caused by the resignation of Waldo F. Pike. President Haertlein referred to the announcement in the notice of this meeting, that the election of a Director would be held at this meeting.

VOTED that the report of the Nominating Committee be accepted and the nominations be closed.

VOTED that the Secretary be directed to cast one ballot for the Nominee for the office of Director.

The Secretary cast one ballot and President Haertlein therefore declared that Lawrence G. Ropes had been elected a Director.

President Haertlein announced that the next meeting of the Society will be a Joint Meeting with the American Society of Civil Engineers and the usual Student Night, to be attended by the

*Transfer from Grade of Junior.

†Transfer from Grade of Student.

Student Chapters on October 15, 1941, and that the Speaker will be Commander R. V. Miller (CEC), USN—Officer in Charge of Construction of the Quonset Naval Air Station.

President Haertlein then turned the meeting over to Captain Jerome L. Spurr, President, Boston Post, Society of American Military Engineers, who introduced the speaker of the evening, Major E. A. Harwood, Corps of Engineers, United States Army, who gave a very interesting talk on "Engineering Economic and Military Components of Hemisphere Defense".

The meeting adjourned at 9 P. M.

EVERETT N. HUTCHINS, *Secretary*

APPLICATIONS FOR MEMBERSHIP

[October 20, 1941]

The By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission

ALBERT J. COLACEY, Revere, Mass. (b. June 27, 1914, Provinci Di Chieti, Italy). Central Evening High School, 1930-1936; Franklin Technical Institute, 1939-1941; Tufts College, evenings, 1941, one semester; 2nd semester beginning Sept. 22, 1941. Experience: began working for the Metropolitan District Water Supply Commission on Feb. 25, 1931, as a messenger clerk, tracing, drawing, designing, computations, layout work, checking, on the following subjects, Steel, Concrete, Dams, Reservoirs, Piping, Electrical, Topography, Roads, Charts, etc., resigned Feb. 15, 1941, as a Senior Engineering Aid. Worked about 6 months with Chas. T. Main and Samuel M. Ellsworth, tracing, drawing, checking, and some design on the following subjects, piping, electrical, sanitary, and architectural; four months with the U. S. Engineers, Park Square Building, Boston, as a Senior Engineering draftsman, structural and civil, drawing, checking and semi-in-charge of get out job. Resigned July 1, 1941, and began with Jackson & Moreland, Park Square Building, Boston, as a designing draftsman, structural (steel and concrete). Refers to *C. L. Coburn, W. F. Covil, D. O. Fisher, L. M. Gentleman, F. B. Wilkins.*

FREDERICK J. FOX, New York, N. Y. (b. August 20, 1894, Boston, Mass.). 1914-1917, draftsman and steel designer on apartment houses and small garages; 1917-1919, Construction Quarter Master Corps., of War Department as inspector of steel and building products at their source of manufacture; 1919-1923, Chief Engineer of Morgan & Glasses, a large Architectural firm, worked on the more intricate designs and especially Foundation problems; 1924, was consultant in property evaluation for a loan company, latter part of year and 1925, was Consulting Engineer on a large

Hospital and several 12 story fireproof apartment houses, also, one 23 story Hotel (Navarre); 1926-1927, Chief Engineer and Secretary of Kaufman Snyder Company, General Contractors on heavy foundation work; 1928-1929, worked for several Architect and Consulting Engineers on steel design and Foundation; 1929-1934, Secretary of the Arthur I. Kraft Company and President of the Lehigh Pile Foundation Corporation, executing several large public contracts and was the testing Engineer for the City of New York in determining the piles to be used later on the El. Express highway of New York, also constructed the difficult foundation of the Museum of Natural History; 1935 to present, Consulting Engineer building and designing commercial and industrial plants: just completed a dock on Bronx River. Licensed Engineer in New York State No. 9509, also hold a sewage disposal registry for a basic plan. Have designed and supervised some of the notable jobs in New York City and am at present a Consultant on all Construction designs and am employed by self.

Hugh P. Duffill, Duxbury, Mass. (b. June 20, 1898, Melrose, Mass.). Graduated from Massachusetts Institute of Technology in 1920. Experience, worked with the Illinois Highway Department as Structural Designer in their Bridge Office until October, 1921, when I returned to M. I. T. for graduate study and received a degree of Master of Science in Civil Engineering in 1922. Returned to Illinois Highway Department in same position, returning to Boston in 1923 as Structural Designer for the Boston and Albany Railroad. In 1927 went with Back Bay Cooperative Bank as Treasurer and in 1935 opened my office as Consulting Engineer, my work covering both Structural and general Civil Engineering Work. In 1940 employed with Stone and Webster Engineering Company of Boston, as Structural Designer. June 1941, employed with the Portland Cement Association as District Structural Engineer for the States of Massachusetts, Maine, New Hampshire and Vermont. Refers to *J. B. Babcock, C. B. Breed, J. D. Mitsch, C. M. Spofford, F. N. Weaver.*

Albert E. Sanderson, Jr., Wayland, Mass. (b. May 13, 1904, Waltham, Mass.). Graduate of the Waltham Public Schools; received B.S. degree at Northeastern University in 1940; now a candidate for M.S. at Harvard Graduate School of Engineering. Experience, during the course at Northeastern University, worked for the City of Waltham, engineering dept., as rodman, transitman and chief of party successively. Following graduation worked as draftsman for 6 months in Florida. From 1927 to 1930, employed at the Bethlehem Steel Company in Pennsylvania as a steel detailer and checker. From 1930 to 1935, was chief draftsman of the Boston Bridge Works, leaving to go into a non-engineering business. From 1935 to 1938, did some free lance engineering, mostly bridge and building design. In 1936 came to Northeastern University as night school instructor in drawing, accepting a full time appointment to the day division in 1938. At present instructor in Civil Engineering at Northeastern University, teaching the Surveying field work and design of structures. Refers to *C. O. Baird, E. A. Gramstorff, A. Haertlein, J. C. Moses.*

Transfer from Grade of Junior

Arvo A. Nelson, Marblehead, Mass. (b. February 18, 1909, Quincy, Mass.). Graduated from Northeastern University in 1929. Experience, June to September 1929, engineer and inspector for Turner Falls Power & Electric Company on construction at Cobble Mt. Power Station; September 1929 to February 1940, with Mass. Dept. of Public Works, Boston, Mass., estimating, de-

sign and for last 4 years there as specifications writer; February 1940 to July, 1941, Field Engineer for Portland Cement Association in northeastern Massachusetts; from July 1, 1941, to present time, office engineer for Portland Cement Association, Boston, Mass. Refers to *A. B. Appleton, R. W. Coburn, E. L. F. Davis, E. N. Hutchins.*

DAREL O. PACKARD, Arlington, New Jersey (b. March 27, 1911, Hull, Mass.). Graduated from Northeastern University in June, 1936, with a B.S. degree in Civil Engineering. Experience, September, 1936, to March, 1937, with The Semet-Solvay Engineering Corporation of New York City at their Owasso, Michigan, Shop, working as an inventory clerk and as an apprentice in the layout department of the Plate Shop. March, 1937, was transferred to the New York Office to take charge of the Reproduction Department; October, 1937, was transferred to the Engineering Department as a tracer and doing occasional drafting. June, 1939, entered the employ of E. I. DuPont De-Memoors & Co., Inc. (Plastics Division), at their Arlington, New Jersey, plant, employed in the Vinyl Resins Department on operations and later did some special work for the Industrial Engineering Division. January, 1940, was transferred from operations to Development and Control, supervising the operations and chemical control and development of the first process stage. July, 1940, was transferred to the Maintenance Engineering Department where my work consists of the drafting and design of repairs and alterations to present equipment and the layout and design of new and special equipment to be used in various parts of the Plant. Refers to *C. O. Baird, C. S. Ell, A. E. Everett, E. A. Gramstorff.*

ADDITIONS

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FRANK R. BERMAN, 1820 Commonwealth Avenue, Boston, Mass.

MAJOR ARTHUR L. DOW, Bangor Air Base, Bangor, Maine.

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PAUL M. LEVENSON, 19 Woolson Street, Mattapan, Mass.

DEATHS

SIDNEY SMITH ..... June 26, 1941

LOUIS F. BUFF ..... Sept. 29, 1941

JOURNAL

OF THE

BOSTON SOCIETY

OF

CIVIL ENGINEERS

VOLUME XXVIII

1941

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Published four times a year, January, April, July and October, by the Society
715 Tremont Temple, Boston, Massachusetts

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