CONSTRUCTION OF EMBANKMENTS ACROSS PEATY SOILS

By L. Casagrande,* *Member*

Introduction

In response to an invitation by the U.S. Waterways Experiment Station, the author submitted in 1964 a study on mass displacement of soft organic soils by a combination of surcharging and blasting. It included the following topics:

1. Summary of American Practice.
2. Summary of German Developments including brief description of major applications.
3. Discussion of possible further developments of this method.
4. Literature as needed in connection with the above points.

The author considered himself qualified to undertake such a study because he made vital contributions to the procedures of displacement of peat by blasting which were developed in Germany before the second world war.

Although displacement of soft organic deposits (peat, marsh, swamp, muck, muskeg) for the purpose of building highway embankments on firm ground originated in the United States in the early Thirties [6, 26, 27],[1] it has become a neglected art on this continent. On the other hand, further developments of this method have continued in Germany since the author first introduced it into German highway construction practice. Therefore, as part of this study, the author visited Germany in 1964, specifically for the purpose of personally acquainting himself with the most recent developments in this field.

The following is a revised version of the above mentioned study.

Properties of Peaty Soils

The literature contains a number of detailed descriptions and classifications for peaty deposits (bog, marsh, muck, muckeg). An excellent

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[1] The numbers in brackets refer to the bibliography at the end of this article.

summary on this topic, including an up-to-date bibliography, has been prepared by Pihlainen [65]. Since the pertinent engineering properties of these materials are quite similar, for the sake of simplicity these highly organic materials will be referred to in the following as peaty soils.

Description of Peaty Soils

With some exceptions peaty deposits are brown to dark brown in color and very soft and spongy in the undisturbed state. Except for the surface crust, the natural water content of such deposits is very high. Although the fibers or leaves may be clearly visible in the undisturbed state, they frequently disintegrate, without much effort, into an amorphous mass during remolding. In spite of peaty soils being usually very soft in the undisturbed condition, they possess distinct brittleness. Upon remolding, they may lose all of their strength and turn into a more or less homogeneous, soupy state.

In the undisturbed condition peat possesses a very high void ratio and a relatively high permeability. Upon load application, or when disturbed, both the void ratio and the coefficient of permeability decrease to very much lower values.

At locations where the grandwater table is somewhat below the surface of a peaty deposit, a drying crust (meadow mat) may exist which has appreciably greater strength, much less moisture, and may be much less sensitive to disturbance.

Water Content

Depending on the origin and on the thickness of a peaty deposit, the water content ranges between wide limits. Whereas the water content of the crust may be as low as 100% (by dry weight), the peat beneath the surface crust may possess water contents ranging between several hundred and several thousand per cent. Since peat, as a rule, has only a negligible submerged weight, the decrease in water content with depth is barely perceptible.

In Table I are recorded the ranges of water contents for typical peat deposits on this continent and in several European countries. In spite of the wide range of water contents, there are great similarities between a typical Canadian peat [2] and an average English peat [40], or between a peat deposit in Northern Ireland [34] and one in Florida [23].

TABLE I

Reference	Location	Water Content %
Adams [1]	Canada	375 - 430
Anderson and Hempstock [2]	Canada (Alberta)	600 - 1350
Brawner [8]	Canada	200* - 300*
		800 - 1400
Casagrande [14]	USA (Mass.)	250 - 800
Colley [23]	USA (Florida)	480 - 900
Dücker [29]	Germany (Schleswig Holstein)	400 - 1250
Duncan, et al [30]	Northern Ireland	400 - 800
Erlenbach [33]	Germany	600 - 900
Feustel and Byers [34]	USA	3200
Flaate and Rygg [35]	Norway	>2000
Garras [38]	Germany	600 - 800
Hanrahan [41]	Ireland	700 - 1400
Hardy and Thomson [42]	Canada (Northwest)	470 - 760
Helenelund [43]	Finland	1000 ->2000
Johnson [46]	USA	200* ->2000
Lake [51]	Scotland	1850
Legget [52]	Canada	up to 1400
MacFarlane [55]	Northern Ireland	700 - 1200
Margason and Fraser [56]	Northern Ireland	300 - 1500
Micklebrough [58]	Canada	480
Moos, von and Schneller [60]	Switzerland	100* - 2100
Morton [62]	USA (New Hampshire)	300 - 650
Ripley and Leonoff [74]	Canada	220* - 1460
Root [77]	USA (California)	550 - 1000
Tresidder and Fraser [86]	Scotland (Shetland)	400 - 1600
Usinger and Garras [88]	Germany	400 - 1200
Ward [90]	England (Wales)	660 - 1300

* Near surface or along edge of deposit.

Plasticity

The plasticity of peat ranges from low plasticity for thoroughly weathered deposits to non-plastic for highly fibrous deposits. As shown on the plasticity chart in Fig. 1, all points are located below the A-line. The two heavy lines and the encircled area represent an average of numerous test results for thoroughly weathered peat in Northern Germany [29] and slightly fibrous peat deposits in the United States [12, 14], with liquid limits ranging between approximately 300 and 1000. With a smaller degree of weathering, i.e., with increase in fibrous matter, the points on the plasticity chart move down and to the left, as suggested by the two dashed lines in Fig. 1.

Although only few investigations have been concerned with the plasticity of peat, it is believed that such tests would provide the engineer with a simple method for estimating the character and the engineering properties of peat deposits.

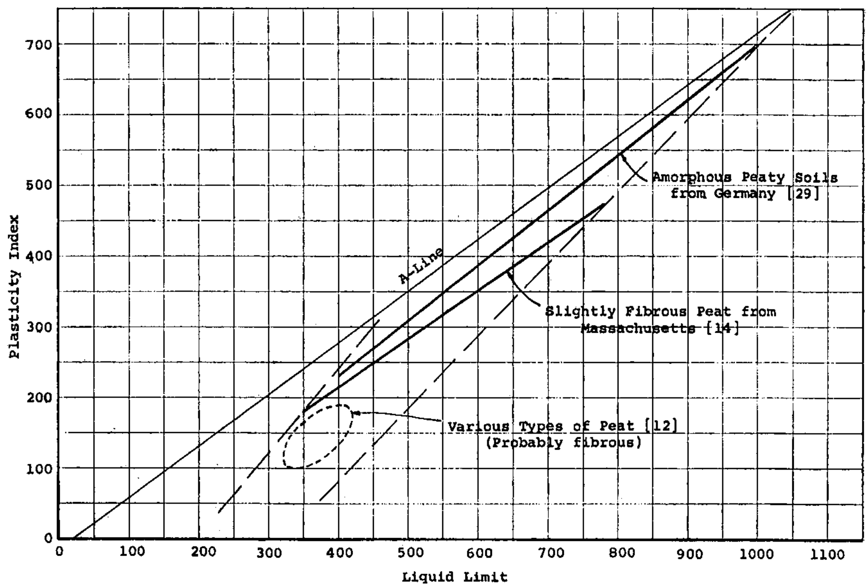


FIG. 1.—PLASTICITY CHART

Shear Strength

The results of investigations of shear strength of peaty soils are summarized in Table II and in Figs. 2 and 3. The majority of these investigations were made by means of in situ vane tests. Where unconfined compression tests were also made, the two sets of strength data agreed reasonably well.

The relationship of shear strength to depth for investigations from widely scattered sites are plotted in Fig. 2. It can be seen that the results for peat deposits in the Boston area [14] and in the vicinity of Vancouver, B.C. [49], are in reasonable agreement. They show a slight effect of surface drying which extends to a depth of about 10 or

TABLE II

Reference	Location of Peat	Shear Strength psf	Natural Water Content %
Anderson and Hempstock [2]	Canada (Alberta)	100 - 250	700-1400
Casagrande [14]	USA (Littleton, Mass.)	100 - 370	230- 750
	USA (Dedham, Mass.)	70 580	400- 800
	USA (Neponset, Mass.)	270	400- 550
	USA (Neponset, Mass.)	70 - 190	250- 380
	USA (Neponset, Mass.)	1000*	110
Dücker [29]	Germany (Schleswig Holstein)	20 -1000*	400- 800
Fraser [36]	Northern Ireland	280 - 560	680-1450
Hardy and Thomson [42]	Canada (Northwest)	100 - 600	470- 760
Lea and Brawner [49]	Canada (Alberta)	110 - 300	
Margason and Fraser [56]	Northern Ireland	340	790
Moos, von and Schneller [60]	Switzerland	100 - 300	220-1460
Ripley and Leonoff [74]	Canada	200 - 430	100-2100
Smith [82]	England	70 - 360	
Tresidder and Fraser [86]	Scotland (Shetland)	70 -1870	400-1600
Ward [90]	England (Wales)	125	800-1000

* Shear strength of drying crust.

15 ft, and below that one is dealing presumably with normally consolidated material which has a shear strength ranging between 400 and 500 psf at a depth of 20 ft. In contrast to these two investigations are the results obtained in Scotland [86] which are distributed over a very wide range as indicated in Fig. 2. Most of the results of these

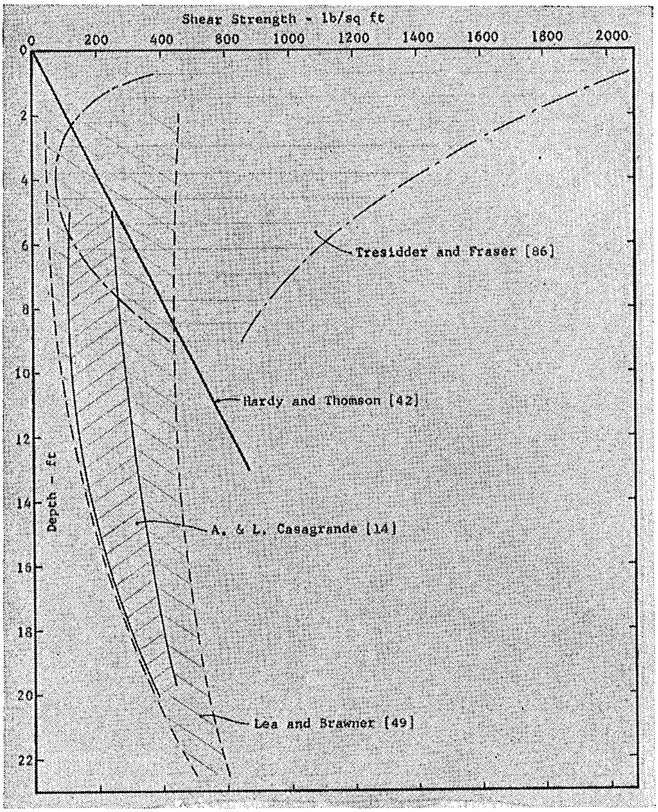


FIG. 2.—SHEAR STRENGTH FROM IN SITU VANE TESTS VS. DEPTH

vane tests, however, are concentrated in the middle of this area, and they indicate substantial preconsolidation by drying. Even at a depth of 9ft, the maximum depth at which tests were made in this investigation, the material is probably overconsolidated. Finally, the test results on muskeg reported by Hardy and Thomson [42] fall into a very narrow range which can be well averaged by the straight line shown in

Fig. 2 that starts with zero strength at the ground surface. Usually such a line would indicate a normally consolidated soil. However, it is very unlikely that a normally consolidated peaty soil would have a shear strength of 600 psf at a depth of only 12 ft. Therefore, some other factor must be responsible for the rapid increase of shear strength with depth at that site.

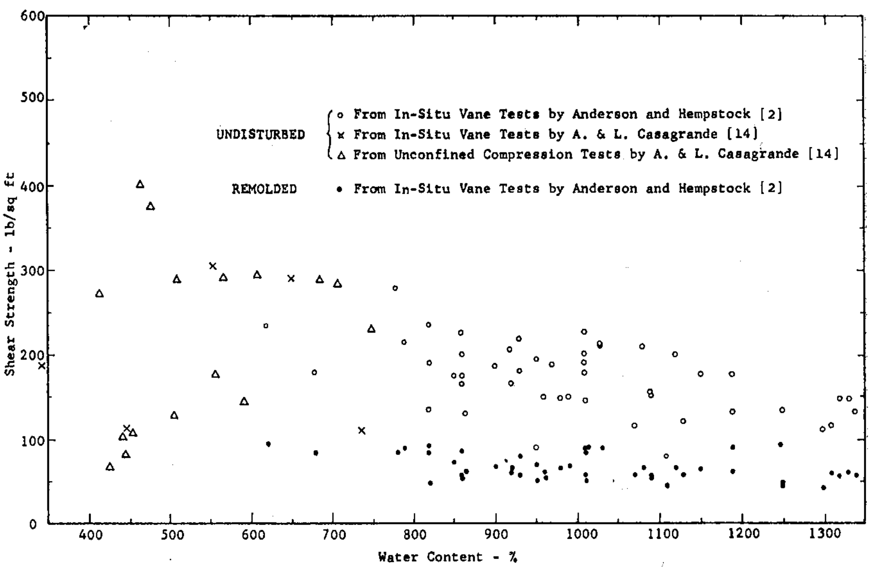


FIG. 3.—SHEAR STRENGTH OF PEATY SOILS

The relationship of shear strength and water content is reflected by the data in Table II and by Fig. 3. In Fig. 3 the results of two sets of investigations are plotted for which individual test results are available [2, 14]. In spite of the wide scattering of the test results, this plot shows the expected increase in shear strength with decreasing water content. It is of interest that the range for the strength of the remolded material is apparently independent of the water content, and that the loss in strength upon remolding is very much greater for the lower range of water contents, for which the strength drops to about one-third of the undisturbed strength. For the highest water content, remolding causes about a 50% loss in strength.

Void Ratio and Permeability

Both the void ratio and the coefficient of permeability of peaty soils range between wide limits. Whereas surface layers and shallow peat deposits which are subjected to air drying may possess void ratios between 2 and 5, for deep deposits they increase to between 5 and 28, with the majority lying between 5 and 15.

From the available information on permeability tests on undisturbed specimens of peat made prior to the start of consolidation

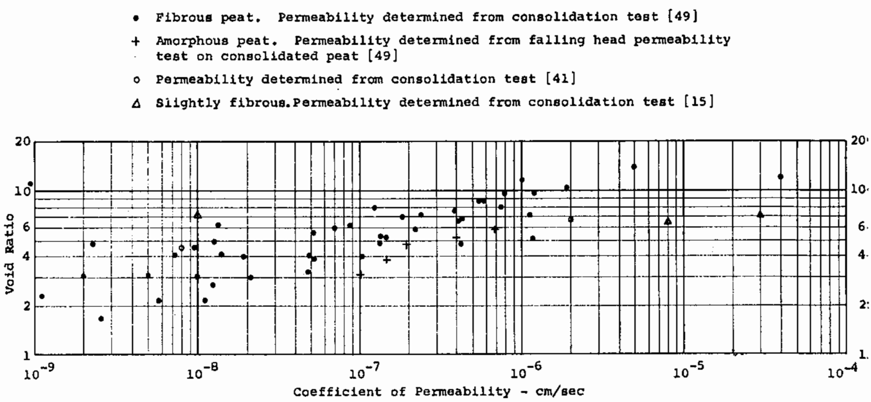


FIG. 4.—VOID RATIO VS. COEFFICIENT OF PERMEABILITY

tests one can conclude that the coefficient of permeability ranges between 10^{-2} and 10^{-4} cm/sec. Upon application of even small loads the permeability decreases rapidly. If the load is increased to 0.6 kg/sq cm the permeability gradually decreases to the order of 10^{-6} cm/sec after two days, and to the order of 10^{-8} cm/sec after a period of seven months [41].

Extensive investigations by Lea and Brawner [49] resulted on a log-log plot, Fig. 4, in a straight line relationship between void ratio and permeability. For comparison, in this figure are also shown the test results obtained by other investigators [15, 41], which fall into the same range.

A summary of the determinations of void ratios and coefficients of permeability for various peat deposits is contained in Table III.

TABLE III

Reference	Location of Peat	Effective Stress kg/sq cm	Void Ratio	Coefficient of Permeability cm/sec
Adams [1]	Canada	No load	3.4	
Casagrande [15]	USA (Mass.)	No load	7.1	3×10^{-5}
		0.02 for 12 hrs	6.6	8×10^{-6}
		0.16 for 12 hrs	3.1	1×10^{-8}
Colley [23]	USA (Florida)	No load	4.6-10.3	
Cook [24]	Canada (B.C.)	No load	2.8-13.1	
Hanrahan [41]	Ireland	No load	12 (12-25)	4×10^{-5}
		0.6 for 2 days	6.8	2×10^{-6}
		0.6 for 7 mths	4.5	8×10^{-9}
Lea and Brawner [49]	Canada (B.C.)	No load	8 -28	$10^{-2} - 10^{-4}$
Micklebrough [58]	Canada	No load	3.2- 9.7	
Root [77]	USA (Calif.)	No load	6 -17	
Thompson and Palmer [83]	USA (E. Coast)	No load	5.1- 7.1	
Ward [90]	England (Wales)		>16	

Primary Consolidation

Because of the relatively high permeability in the undisturbed state, primary consolidation develops rapidly [13, 41, 83, 90]. This is confirmed by numerous settlement observations on highway construction projects on this continent and in European countries which showed that primary consolidation of peat deposits develops practically simultaneously with load application [8, 46, 49, 61, 73].

Secondary Consolidation

After completion of the primary consolidation, the secondary compression of peaty soils proceeds usually along a straight line when plotting the settlements vs. log of time. However, some peats develop secondary compression which increases at a greater rate than such a straight-line relationship, so that predictions of settlements based on short-term laboratory tests may result in values which are too small.

Over a period of many years the secondary compression of a peat deposit often exceeds the primary consolidation. Furthermore, because of the high permeability of many peaty soils, one finds that most of the primary consolidation develops already during construction of the embankment. For these reasons the secondary compression of peaty soils is much more important than the time-lag due to primary consolidation [13, 77, 83].

Typical examples of the magnitude of the settlement due to secondary consolidation from observations are tabulated below [77, 83]:

Thickness of Peaty Soil ft	Height of Fill ft	Rate of Settlement per cycle on logarithmic time-scale ft
13	4.5	1.0
17	5.5	1.5
42	15	4.0
20	?	4.0

Difficulties and costly repairs result from such large settlements of highways due to secondary compression of peaty deposits [10, 28, 49, 61].

The literature contains numerous examples which show that secondary consolidation of peaty soils extend over hundreds of years. A number of medieval buildings in Europe which were built on such soils are still settling. Settlement observations on a 22 ft thick peat deposit in England resulting from draining this peat (i.e., by changing the submerged weight of peat into full weight condition) are reported by Chatwin [22] as follows:

Time Elapsed Years	Settlements ft
12	4.8
22	7.7
27	8.2
44	10.2
84	10.7

In an effort to control settlements of fills on peaty deposits, sand drains have been used extensively. In the judgment of the author, sand drains are of no value for the purpose of accelerating or controlling the large secondary compression of highly organic deposits. In fact, sand drains installed by the conventional method may cause more harm than good because their installation seriously disturbs the structure of the deposit and may result in much larger settlements.

Displacement of Peaty Soils by Blasting

In American highway construction the following methods for displacing peaty soils with explosives have been used:

(1) Toe shooting.
(2) Underfill blasting.
(3) Ditching.
(4) Relief method.

To these four basic procedures of peat blasting should be added a method which was developed in the State of New Hampshire. This procedure is a combination of the above methods and will, in this report, be referred to as:

(5) New Hampshire method.

Finally, in Germany a modified method of underfill blasting was developed which proved highly successful. Hence in this report reference will be made to:

(6) German method.

The basic principles involved in methods (1) to (4) are well described in the literature [5, 18, 21, 31, 44, 55, 81, 95]. Therefore, in this paper are discussed chiefly lesser known details of the various displacement methods. In addition, new concepts are presented that may contribute to greater efficiency and economy in the construction of highways over peat deposits.

FIG. 5.—JETTING OF SMALL CHARGES TO BREAK UP SURFACE MAT OR CRUST

Treatment of Surface Mat or Crust

Most highway engineers consider it good practice to break up the vegetative surface mat or drying crust before placement of any fill material. The purpose is threefold:

(1) To lessen the danger of cracks developing in the vegetative mat which may lead to sudden sliding during filling operations.
(2) To facilitate a uniform settlement of the fill.
(3) To prevent the crust from being trapped beneath the fill.

The dense surface mat is usually blasted with light charges which are either pushed or jetted to a depth of several feet at 1.5 to 3 ft spacing, as shown in Fig. 5. As a rule this operation extends over the full width and length of the proposed filling operation.

The breaking up of the surface mat to the full width of the embankment has the following important disadvantages:

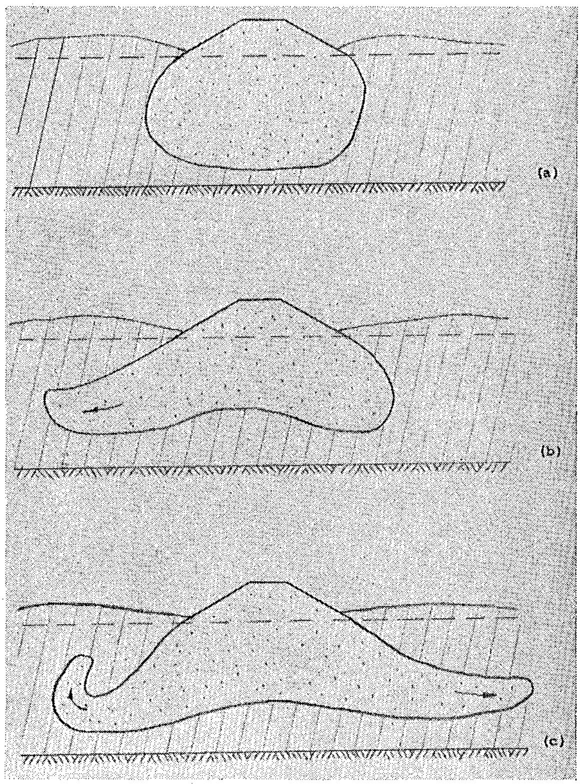


FIG. 6.—TYPICAL FILL SETTLEMENTS ON PEAT

(1) Placement of fill on the surface of peat or other highly organic soils may result in a portion of the fill being lost by excessive "spreading" [10, 28, 43]. Instead of the fill settling in a bulky manner, as shown by the cross-section in Fig. 6a, it may settle in a highly irregular manner and with excessive horizontal spreading, as shown in the cross-section in Fig. 6b. In extreme cases cross-sections of the type shown in Fig. 6c [43] are known to develop.

(2) Blasting of the surface mat disturbs the soft peat for some distance beyond the blasting operations and also to a certain depth beneath the mat. On a number of projects it was observed that the water content of the soft peaty soils had decreased noticeably in the vicinity of the applied fill. For

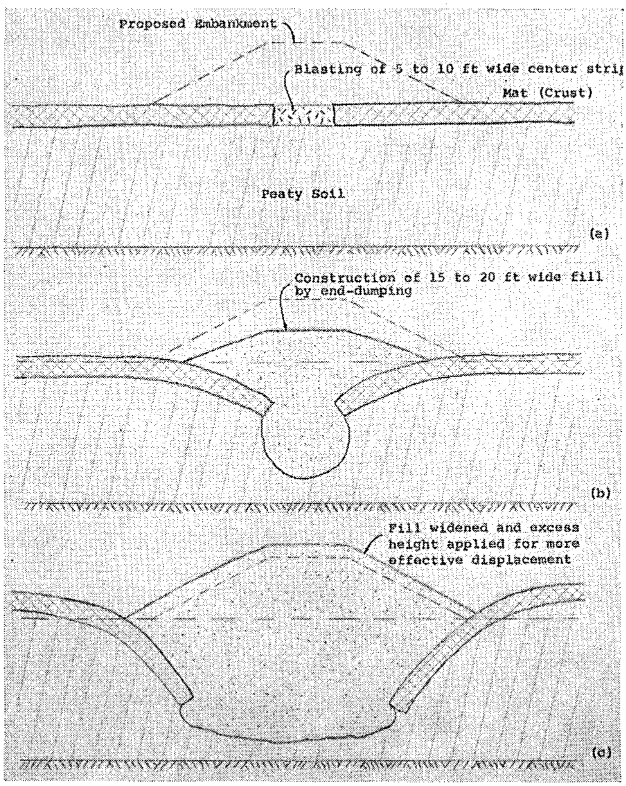


FIG. 7.—PROPOSED PROCEDURE FOR FILL CONSTRUCTION

this reason Morton [62] believes that blasting of the crust must not extend too far in front of the fill operation since the exposed unstable material "tends to stiffen." In his opinion this is due to rapid evaporation. However, this phenomenon is also known to occur at locations where the peat is completely submerged. Toe blasting operations for an embankment in Northern Ireland [36] resulted in a marked decrease in the water content to a distance of up to 200 ft. This was accompanied by a noticeable increase in shear strength within the affected areas. Similar observations were made on a second project in Northern Ireland [56].

It is the author's opinion that a procedure of breaking up the crust over a width of between 5 and 10 ft along the centerline, as shown in Fig. 7a, rather than over the full width of the embankment, and end-dumping a narrow fill at a fast rate and well ahead of placing the fill over the full embankment width, ensures that a narrow core of fill penetrates deep into the peaty soil. This is illustrated in Fig. 7b. Spreading of this core is inhibited by the restraining effect of the undamaged portions of the mat. Once this narrow core of fill is completed over the full length, it can gradually be widened without danger of sudden slides or excessive spreading. Placement of additional fill will result in the mat being deflected downward. Such rotational deflection gradually leads to an increase in width of the submerged portion of the fill, as illustrated in Fig. 7c. Completion of the desired width of displacement can then be achieved with explosives, as discussed under a subsequent heading.

The method of constructing first a narrow fill along the centerline over the full length of the peat deposit has been applied by the Michigan State Highway Department for peat in excess of 20 ft in depth [10]. It has also been used successfully on several German projects [16, 88].

Toe Shooting

The standard procedure of toe shooting consists of disturbing the peat by blasting ahead of the fill for a distance of between 25 and 50 ft per operation. The author considers it preferable to shoot only one row of charges at a time. These charges are arranged close to the toe of the fill, as illustrated in Fig. 8. Individual charges should be small enough so as not to throw out peat, including adjacent fill. The desirable quantity of explosives per hole should be determined by experimentation and will for average conditions not exceed $\frac{1}{2}$ in lbs, where h is the effective depth of the charge as shown in Fig. 8.

The standard procedure of toe shooting is slow and will not always guarantee full displacement of the peat. Experience has shown that large pockets of peat are frequently trapped beneath the fill and cannot be displaced at a later stage without excessive efforts and costs.

Cognizant of these difficulties, German engineers have in recent years developed a new method of toe shooting. It makes use of the desirability of increasing the confining load above individual charges, thus enabling the use of larger charges which in turn permit great efficiency and therefore also greater economy. This procedure consists of the following steps:

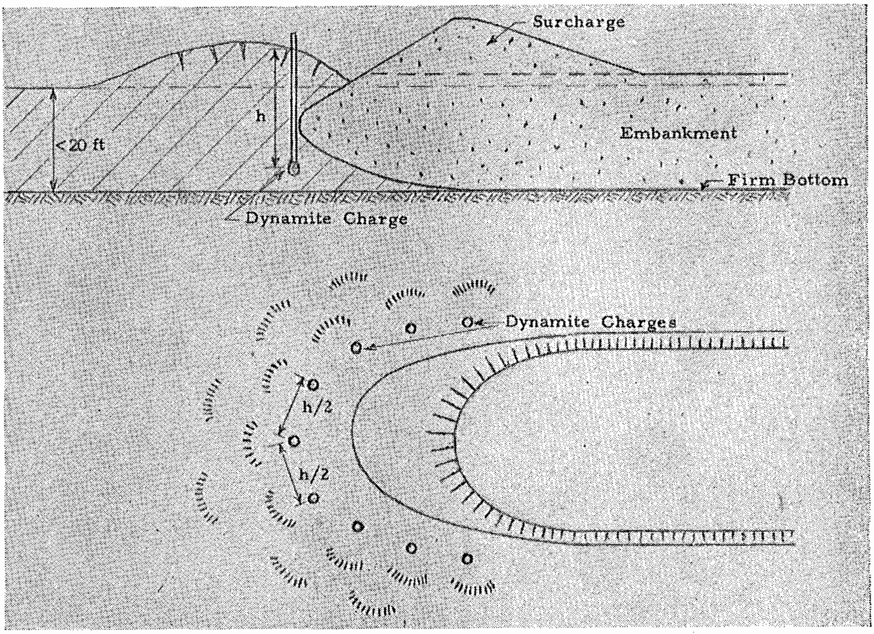


FIG. 8.—DISPLACEMENT OF PEAT BY SURCHARGING AND TOE SHOOTING

(1) As illustrated in Fig. 9a, ahead of the completed embankment already resting on firm bottom, enough sand is spread to permit the crew to operate on the surface of the peat. Depending on the consistency of the peat, the thickness of this sand platform ranges between 10 in. and 2 ft.

(2) From the surface of this sand platform, 10 in. diameter holes are jetted to firm bottom, using a jetting device described below. Depending on the depth of the peat stratum and on the available degree of confinement, the jetted holes are spaced between 6 and 15 ft, as illustrated in Fig. 9b.

(3) Immediately upon completion of one hole, the charges are lowered to the bottom of the hole. It was found that if the surface of

the slurry which develops during the jetting operation is kept above groundwater level, such jet holes will remain open long enough for the charges to be lowered without collapse of the hole. Depending on the depth of the hole, between one and five 20 lb units of explosives are used per hole. These units are delivered by the explosives manufacturer already assembled in cylindrical waterproof cardboard containers.

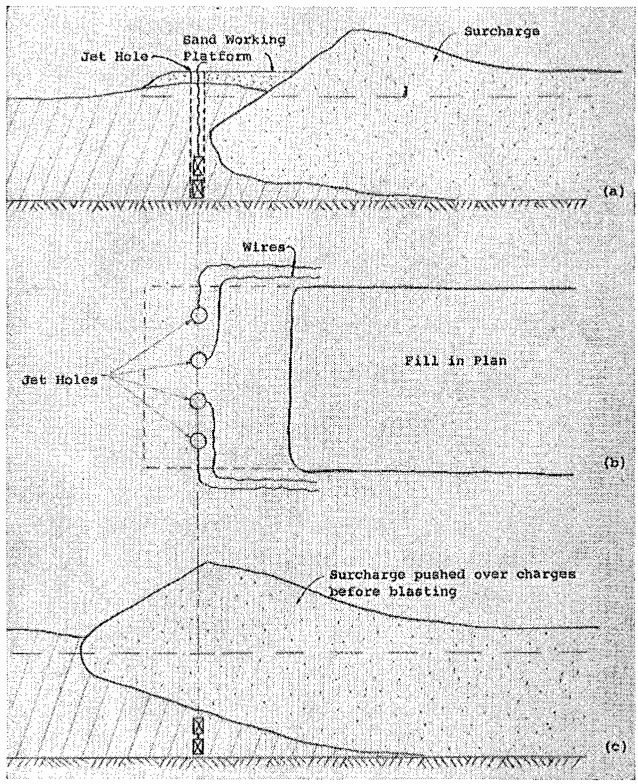


FIG. 9.—GERMAN PROCEDURE OF TOE SHOOTING

(4) As illustrated in Fig. 9b, on the ground surface wires are carried to both sides of the proposed fill so that they will not be damaged during subsequent filling operations. For depths of peat greater than 30 ft it was found expedient to protect the electrical circuits by stringing the wires through plastic tubing.

(5) Fill material is then pushed over the area of the jet holes

until the full height of fill including the desired surcharge is reached. This step is illustrated in Fig. 9c.

(6) Then blasting is carried out. The process is repeated in the same order.

This method does not require the fill to be extended in a "V" shape but makes use of a blunt head of fill, as illustrated in Fig. 9b.

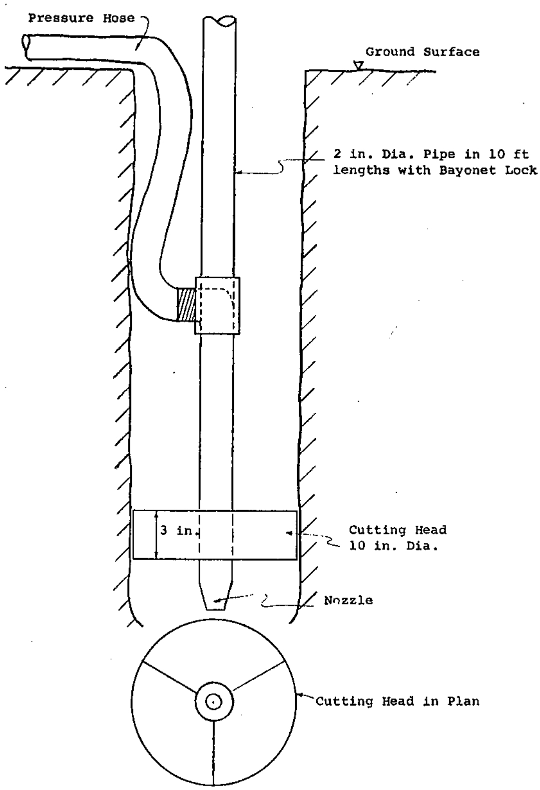


FIG. 10.—SKETCH OF GERMAN JETTING DEVICE FOR TOE SHOOTING

Fig. 10 shows a sketch of the jetting device used for the German method of toe shooting. It consists of a cutting head, a pressure hose connected to the jetting pipe approximately 2.5 ft above the cutting head, and 10 ft lengths of pipe which can be extended quickly by means of a simple locking device. The 1 in. nozzle extends approximately 3 in. below the lower edge of the cutting head. This jetting device is handled by a two-man crew using a water pressure of up to several hundred psi. As soon as the pump is started the jetted hole develops by the up and down movement of the jetting device. Penetration to depths of over 70 ft is usually achieved within several minutes. The author watched jet holes of 30 ft depth being completed within approximately 1 minute.

Currently this method is being employed very successfully along the Autobahn near Emmerich, close to the Dutch border. In April, 1964 the author inspected this work in progress, and also other peat blasting projects which have been completed in recent years. Without exception this method seems to have worked very successfully, both regarding the effectiveness of displacing peat and costs.

Underfill Blasting

The American procedure of underfill blasting is carried out as follows. The vegetative mat is thoroughly broken up by blasting. Then the fill is placed and explosives are pushed through the fill well into the peat by means of 1.5 to 5 in. diameter pipes driven or drilled through the fill. For great depths of peat, explosives experts [5] recommend settling the peat by blasting in stages, handling 10 to 15 ft of peat at a time, and in sections of embankment between 100 and 200 ft in length.

The literature contains descriptions of numerous applications of this method for depths of peat ranging up to over 50 ft and using charges of up to 100 lb per hole [25, 45, 64]. One of the largest projects of this kind was an 1800 ft long peat crossing of over 50 ft in depth [45].

As will be described under a separate heading, a modified method of underfill blasting was developed in Germany in the Thirties. Large pre-assembled charges are jetted through the sand fill into position and blasting is performed in one operation over the full length of the peat crossing, settling the fill to full depth. In this manner the cost per unit of displaced peat decreases with increase in depth of peat.

Ditching

This method has been used in the past for peat deposits not exceeding 15 feet in thickness. A ditch is blasted along the centerline of the proposed embankment to as nearly the full depth of the peat as practical and in sections not exceeding 50 ft. Such ditches have been blasted to widths up to 50 ft in one operation. Each section of the ditch is filled by end-dumping immediately following the blast.

To the author's knowledge this method has not been used in recent years. The reasons for this are probably twofold:

(1) Soft peat of relatively shallow depth can usually be displaced by surcharging, toe shooting, or by a combination of both.

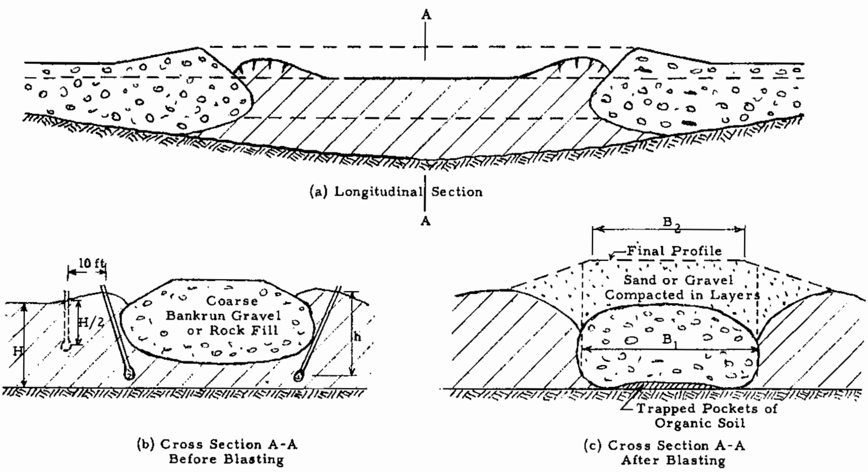


FIG. 11.—UNDERFILL BLASTING—NEW HAMPSHIRE PROCEDURE

(2) Development of large equipment has gradually made excavation of peat to firm bottom competitive.

Relief Method

This method involves the blasting of ditches on both sides of the fill so as to enable the peat to escape from underneath the fill. Normally this method is combined with underfill blasting [5, 31, 44].

New Hampshire Method

For depths of peat of between 10 and 40 ft the New Hampshire Highway Department has developed an economical method of settling fill to firm bottom. Where the peat possesses a surface crust it is first disrupted by blasting in short sections. Then the fill is placed by end-dumping. The two end sections of the embankments, where the thickness of soft material is less than 10 ft, are usually settled to hard bottom by surcharging only. The main body of the fill is then placed over the full length as indicated by the dashed lines in Fig. 11a. The soft material which remains beneath the fill is displaced by means of the following procedure of underfill shooting. Along both sides of the fill dynamite charges are placed at 10 ft centers felow the toe of the fill as indicated in cross-section A-A in Fig. 11b. This is achieved by means of 1.5 in. pipe that is pushed down with a special handle in such a manner that the bottom of the pipe is located beneath the toe of the fill. The quantity of dynamite in lbs per charge is about twice the depth H in feet.

If considerable quantities of soft materials have piled up on both sides of the fill (Fig. 11b), or if the meadow mat or crust has not been blasted effectively, thus causing too much interference with lateral displacement of the liquefied material, a second row of small charges is placed about 10 ft from the main row, as indicated in Fig. 11b, and fired a fraction of a second after the main rows. In some older procedures which employ two or more rows on each side of the centerline, the outer rows were exploded first. However, experience in New Hampshire has proved that a greater efficiency is achieved when the main row is exploded with resistance on both sides.

The dashed lines in the cross-section in Fig. 11c indicate the final profile of the fill which is built upon the settled fill. In the experience of the New Hampshire State Highway Department, if relatively thin lenses of the compressible material are caught beneath the fill, they consolidate quickly and do not cause objectionable settlements after completion of the pavement.

The New Hampshire Highway Department found it advantageous to use for the main body of the fill as coarse material as possible, either a very bony bankrun gravel or even rockfill. Coarse granular fill has the following advantages:

(1) The fill settles with much less spreading than experienced by a sand fill;
(2) Coarse fill has the ability to effectively arch across peat pockets that may be caught beneath the fill.

The additional fill which is placed upon the settled fill, shown by the dashed area in Fig. 11c, may consist of any suitable granular material, and is placed and compacted in layers.

German Method

In connection with the construction of the German Autobahnen, from 1934 to 1940, numerous crossings of soft organic deposits had to be made, some of which were over a thousand feet long and with a maximum depth of about 60 ft. After some experimentation with

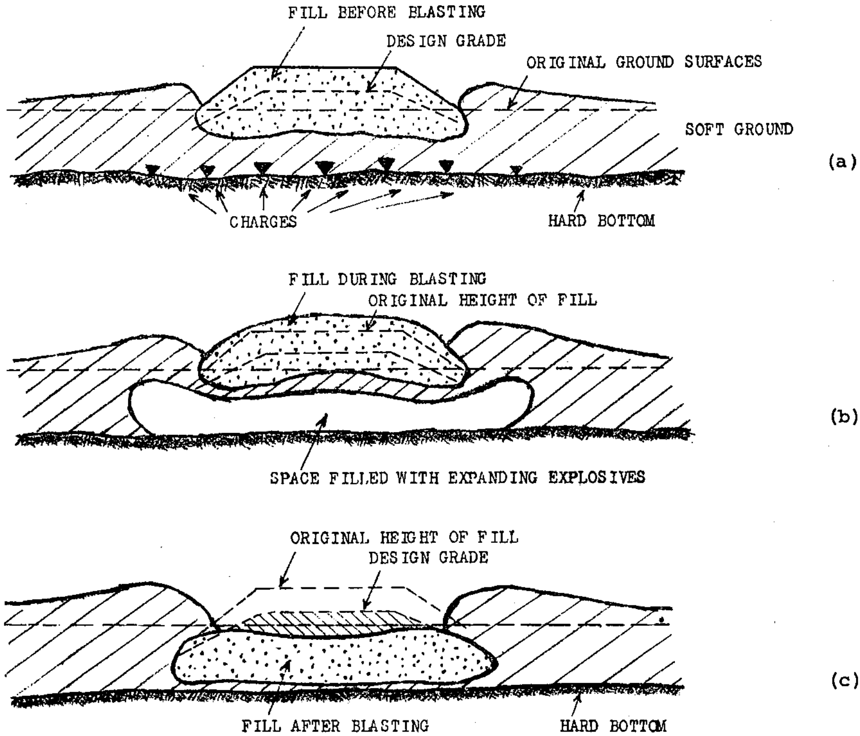


FIG. 12.—GERMAN PROCEDURE OF UNDERFILL BLASTING

the American procedure of underfill blasting [19], the following method evolved. Depending on the consistency of the surface crust, either a narrow strip is disrupted along the axis of the proposed embankment followed up without delay with construction of a narrow fill which is gradually widened, as illustrated in Fig. 7; or the entire fill is placed for the full length of the crossing, on top of the undisturbed mat, as shown in Fig. 12 [16, 17, 18, 20]. The height to which the fill is placed is so controlled that after blasting only minor additional filling is required. The thickness of peat beneath the fill can readily be determined by jetting a pipe through the fill, and observing the color of the returning wash water and the resistance of the jet pipe.

FIG. 13.—TORPEDO CHARGE READY TO BE JETTED THROUGH FILL

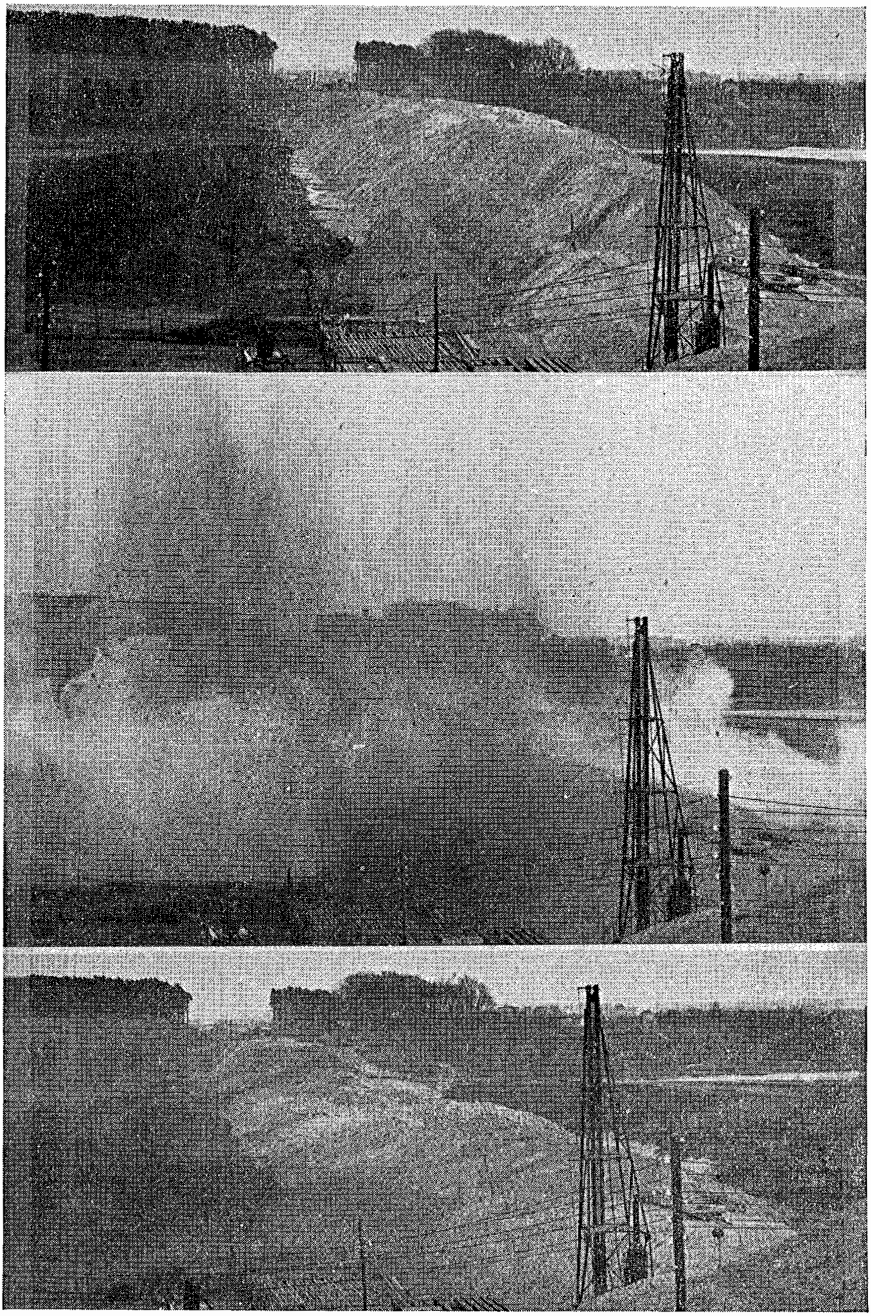
Then, charges in much greater units than is customary in the United States are jetted into position through the fill down to firm bottom, and are fired simultaneously over the full length and width of the fill, as illustrated by Figs. 12a and 12b. Such a simultaneous explosion of charges below the fill is highly effective in destroying the strength of the soft material. Furthermore, the explosion lifts the whole fill bodily for several feet, Fig. 12b, and then the mass drops down upon the underlying disrupted peat which is thereby effectively displaced as indicated in Fig. 12c.

Several methods were developed for the purpose of jetting preassembled charges through the fill and to the bottom of the underlying peat. In the method shown in Fig. 13, a torpedo shaped metal

FIG. 14.—ECCENTRIC JETTING DEVICE IN OPERATION

case containing the explosive is connected to a jetting pipe in such a manner that the pipe can be salvaged after the charge is in place. Another device that proved very satisfactory is shown in Fig. 14. It consists of four pet nozzles which form a sleeve to which the jet pipe is attached. This device is jetted to a depth at which the charge is to be placed. Then, with the jet remaining in operation to keep the hole open, the case containing the explosive is lowered by means of an electric cable and finally the jetting device is withdrawn. Both devices worked well only if the fill did not contain large amounts of coarse gravel; otherwise this coarse gravel would gradually collect at the bottom of the jet hole, finally blocking further progress.

In the German procedure three methods of fill construction were used.



(a) Before

(b) During

(c) After

FIG. 15.—UNDERFILL BLASTING AT GOLLINGWIESE

(1) Embankment constructed to the full width and length and blasted in one operation (Fig. 12).

(2) As illustrated in Fig. 7b, first a narrow fill is constructed along the centerline to the full length. After blasting of this fill it is then widened on both sides and settled to firm bottom by blasting. Depending on the dimensions of the embankment and the depth of peat, this widening and additional blasting may be done in one operation or in several stages.

TABLE IV

Method of Displacement	Peat Displaced cu yd	Amount of Explosives lb	lb/cu yd Displ. Peat
Filling	10,400		
1st Blasting	11,700	8,100	0.7
Surcharging	1,300		
2nd Blasting	6,500	4,200	0.6
Surcharging	1,300		
3rd Blasting	7,800	6,100	0.8
Surcharging	500		
4th Blasting	2,100	5,100	2.4
Surcharging	400		
5th Blasting	3,500	11,400	3.3

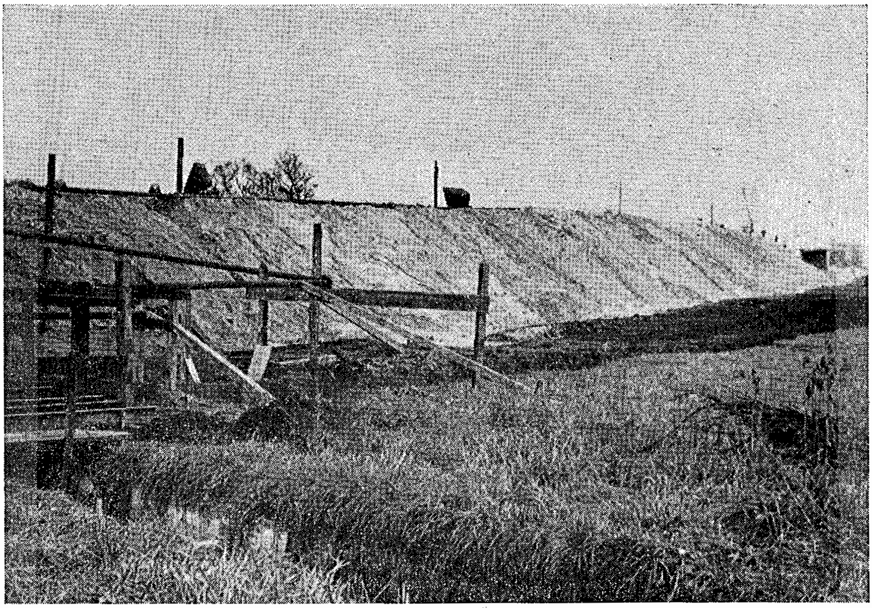
(3) First a narrow fill is constructed along one side of the embankment to the full length, and then settled to firm bottom by blasting. The fill is then gradually widened and blasted again in either one or several additional stages.

Typical examples of the German procedure are summarized below:

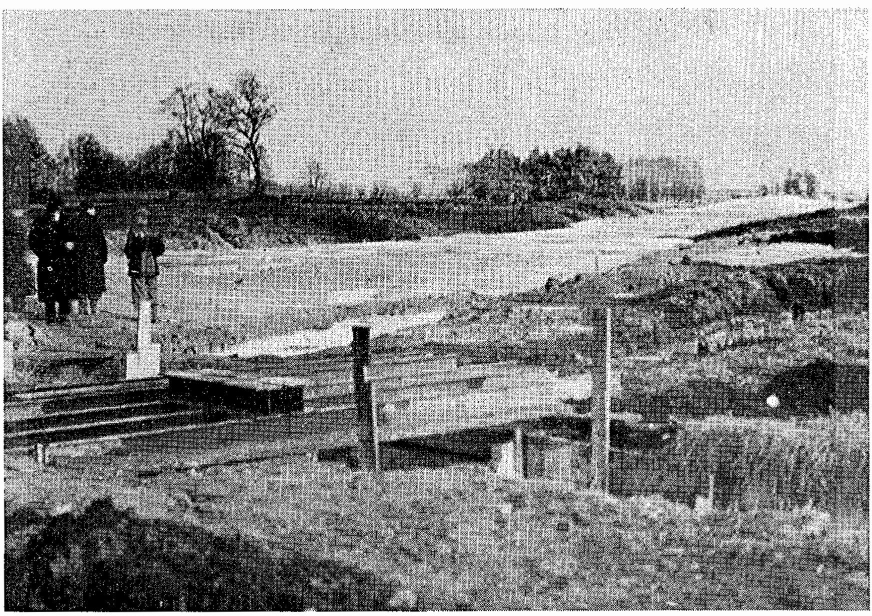
Gollingwiese [88]

The length of peat crossing was approximately 300 ft and the greatest depth about 40 ft, with the top 10 ft consisting of a rather tough vegetative mat. The crest width of the embankment was 75 ft.

The peat was excavated from both ends to a depth of 20 ft. Then the sand fill was constructed to the full width and length. In order to displace as much as possible of the peat without blasting, an excess surcharge of 15 ft was used. Since the firm bottom was sloping toward one side of the fill, several stages of blasting were required



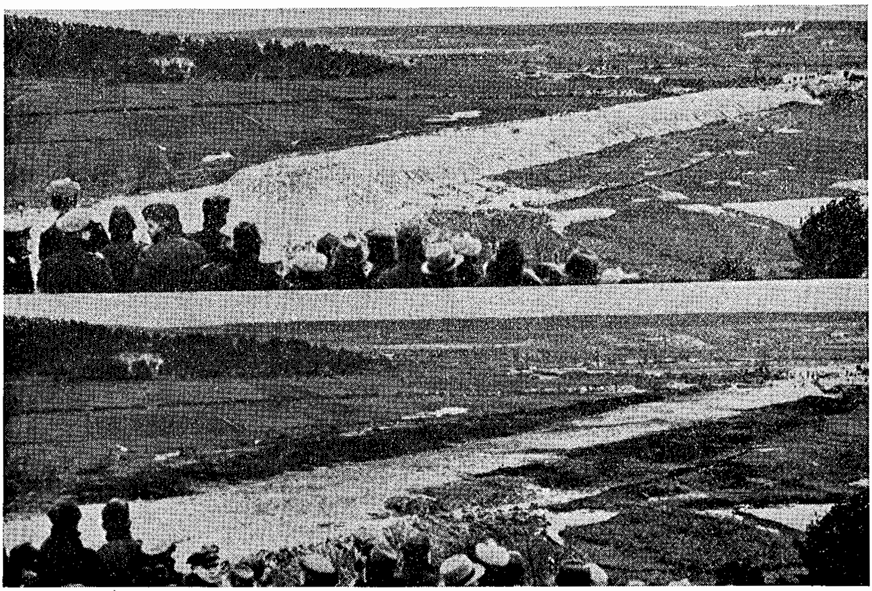
(a) Before Blasting



(b) Seconds Later

FIG. 16.—UNDERFILL BLASTING NEAR SAARMUND—FIRST BLAST

before the desired width of peat was displaced. The total amount of peat displaced was 45,000 cu yd, and a total of 35,000 lb of explosives were used. The condition before, during, and after blasting is illustrated by the photographs in Figs. 15a, b, and c. Fig. 15b shows large amounts of sand being thrown out which is indicative of excessive concentrations of explosives having been used.



(a) Before

(b) After

FIG. 17.—UNDERFILL BLASTING NEAR SAARMUND—SECOND BLAST

In Table IV are compared the quantities of peat displaced during filling operations and achieved by the various stages of blasting. As revealed by the last column in this tabulation, the amount of explosives used per cu yd of displaced peat increased rapidly with each subsequent blast.

Saarmund [*16*]

This peat crossing is about 700 ft long with an average depth of 25 to 30 ft and a maximum depth of 35 ft. The water content of the peaty soil ranged between approximately 400 and 1100%.

The initial sand fill had a crest width of 30 ft and a height of approximately 25 ft above the original peat surface. This fill was settled by blasting to its full length in one operation using four rows of charges which were jetted into position. After having been widened along both sides to the full width of 75 ft, this fill was settled by additional blasting. The largest individual charges were 250 lb and the smallest ones 65 lb. The total amount of peat displaced was approximately 200,000 cu yd using approximately 70,000 lb of explosives. The conditions prior and after the first blast are shown in the photographs in Figs. 16a and b. The effect of the second blast on this project is illustrated by Fig. 17. From personal observations during these blasting operations the author concludes that also in this instance excessive concentrations of explosives were used.

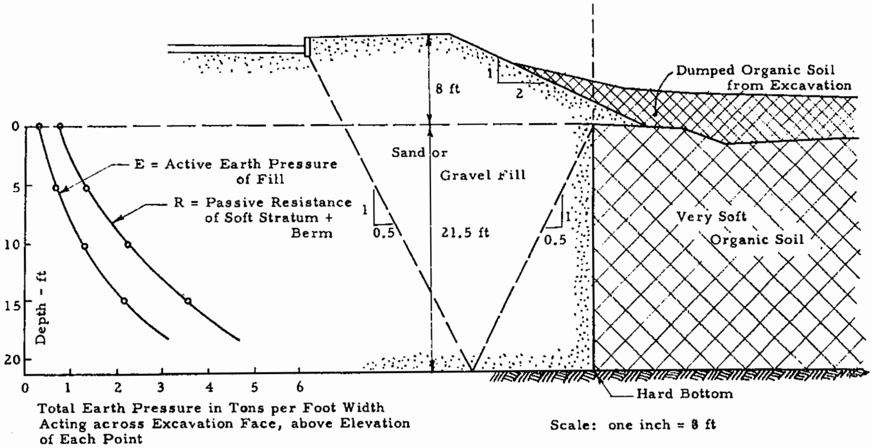
Konigsberg [33]

This several hundred foot long peat crossing with a depth of up to 60 ft was blasted in stages starting with a central dike of approximately 30 ft in width. After settling this fill by blasting, it was widened and blasted in two additional stages. The total peat displaced was approximately 130,000 cu yd and the total quantity of explosives was about 70,000 lb.

Required Width of Peat Displacement

For many years highway engineers were of the opinion that because of the small strength of peat the slope of the fill extending below the original peat surface should be approximately the same as above ground. Measured by this standard and depending on the method of peat blasting used, the bottom width of the fill after displacement of the peat may either turn out to be excessive or inadequate.

In recent years highway engineers on this continent and in Europe made the observation that in cases where according to previous requirements the base width was deficient, very few or no difficulties developed on the finished slopes of such embankments. This is not surprising considering the fact that during filling and blasting operations the peat will consolidate and increase in strength for some distance from the slopes, as has been discussed under a previous heading. There is no doubt that because of increase in resistance to lateral movement the body of fill need not be as large as required for stability above ground. German engineers are now satisfied with an average slope of 1 on 0.75 from the crest of the fill to the toe of the fill at firm bottom. This requirement is based purely on empirical knowledge. Obviously if an attempt were made to analyze such a problem one would have to take into consideration a number of variables including consolidation and strength characteristics of undisturbed and partially disturbed peaty soil, depth of the peaty deposit, dimensions of the proposed fill and the method of blasting. In contrast to such a difficult undertaking, analysis of the required base width of a fill, which is constructed by excavation of the peat, is a much simpler problem. If the results of such an analysis are applied to

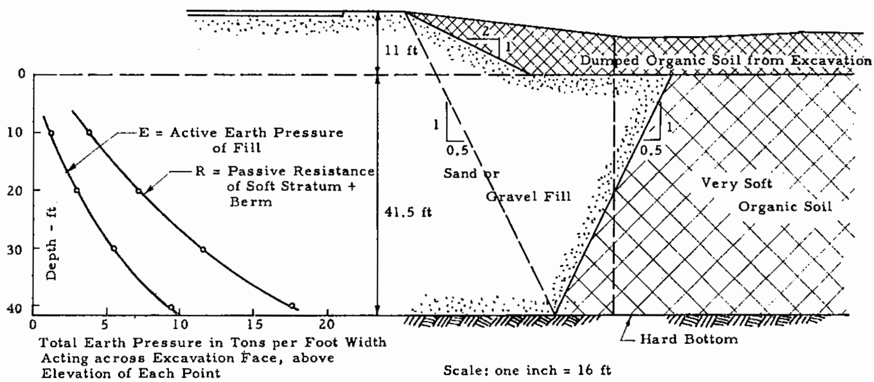


DEDHAM SITE—ROUTE 1
FIG. 18.—ANALYSIS OF ACTIVE EARTH PRESSURE OF GRANULAR FILL AND OF PASSIVE RESISTANCE OF SOFT STRATUM

the necessary width of fill, as developed by blasting, such a procedure would then be on the conservative side.

There are a number of rules-of-thumb in existence in the United States for the required width of excavation of peat in highway construction. The State of Massachusetts, for instance, has been using until recent years several methods for arriving at the desired width of excavation. For the example shown in Fig. 18, the design width was determined by a line sloping 1 on 0.5 from the edge of the pavement to hard bottom, and from that point another inclined line with the same slope in opposite direction to its intersection with the ground surface. Through this intersection a vertical line is drawn which establishes the width of excavation.

When using the above rule-of-thumb it was found that the shoulders and sometimes also the outer portion of the paved areas were subject to settlements and cracking. Although these effects were not very serious, the Department of Public Works found it expedient to support a research project [14] in order to provide assurance that highways built in this manner will not be subject to severe sliding. An analysis of the active and passive earth pressures for the example in Fig. 18 showed that sliding could not develop for these conditions. On the other hand, all portions of a sand and gravel fill, located outside the angle of repose drawn from the toe at hard bottom, are obviously



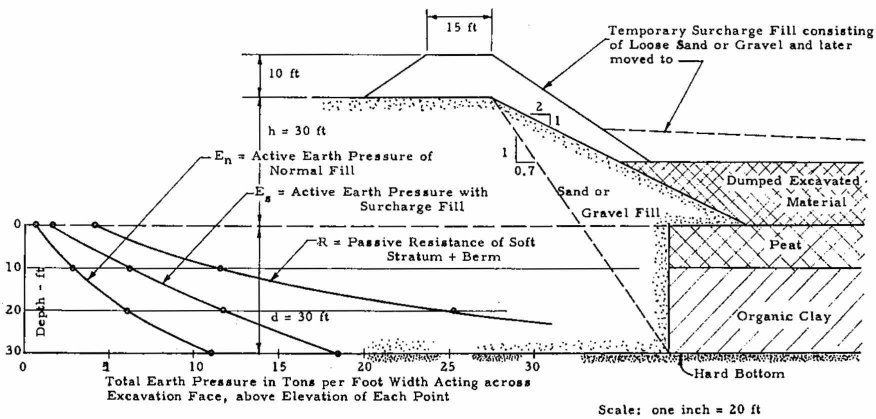
LITTLETON SITE—ROUTE 2
FIG. 19.—ANALYSIS OF ACTIVE EARTH PRESSURE OF GRANULAR FILL AND OF PASSIVE RESISTANCE OF SOFT STRATA

subject to movement due to gradual lateral consolidation of the adjacent peat.

In Fig. 19 is shown another example where the design width was determined by means of a rule-of-thumb which is slightly different from that shown in Fig. 18. In spite of the observed movements in the outer portion of this embankment, it was again found by comparison of active and passive earth pressure that there was no danger of major sliding. Finally, in Fig. 20 a third method is shown for determining the desired width of excavation. Also, in this instance the analysis showed that major instability could not occur. However, none of the embankments built with the use of the above rules-of-thumb are safe against cracking and subsidence within the outer portions of the fill on account of lateral deformation and consolidation of the peat stratum. The magnitude of this lateral yielding depends chiefly on the character of the peaty deposit. In many instances it may be sufficient to know the natural water content of the material in order to judge whether excessive yielding may be anticipated.

There are two ways one can reduce to a tolerable magnitude the lateral deformations of the fill:

(1) By increasing the width of excavation.
(2) By applying a temporary surcharge along the outer portions of the embankment, Fig. 20, which by increasing the active



NEPONSET SITE—SOUTHEAST EXPRESSWAY
FIG. 20.—ANALYSIS OF ACTIVE EARTH PRESSURE OF GRANULAR FILL AND OF PASSIVE RESISTANCE OF SOFT STRATA

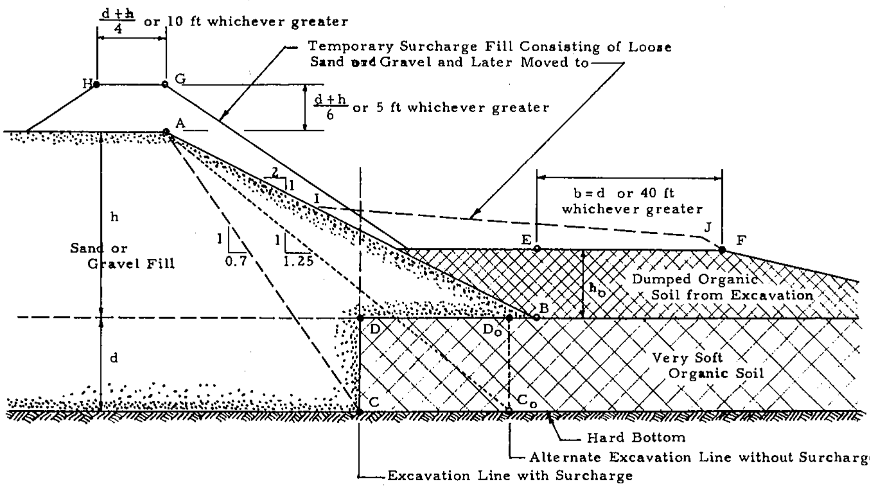
earth pressure will cause "prestressing" of the peaty soil. If such a surcharge is left in place for a period of several weeks and is then moved to the side of the embankment to form a permanent load on the outside of the vertical boundary between the natural fill and the peat, it would in addition result in an increase of the passive resistance of the peat.

This suggested procedure of surcharging should in most cases prove to be more economical than widening of the excavation. The surcharge fill need not extend over the full width of the embankment but only along the edges in the manner illustrated in Fig. 20.

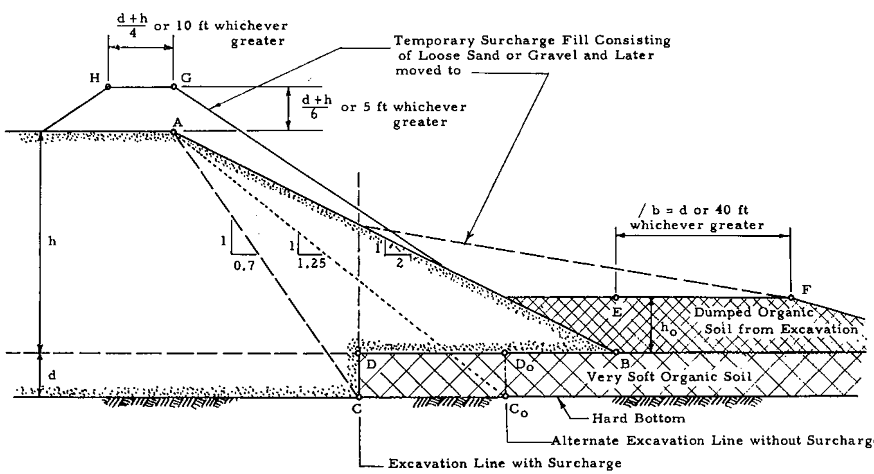
The analysis of the examples shown in Figs. 18 to 20 has led to the following two alternative rules for the required width of peat excavation [14]:

Rule I—Without Temporary Surcharge

From the edge of the crest of the embankment, point A in Fig. 21, draw a line sloping 1 on 1.25 to its intersection C_o with the hard bottom. The vertical line C_oD_o fixes the width of the excavation of the soft



PROCEDURE—H/D = 2
FIG. 21.—FOR H/D = 2 REQUIRED MINIMUM H_0 = 0.37H



PROCEDURE—H/D = 5
FIG. 22.—FOR H/D = 5 REQUIRED MINIMUM H_0 = 0.25H

stratum. The slope of the embankment should be approximately 1 on 2.

The above width of excavation assures full protection against objectionable settlements of the crest of the embankment.

In the example illustrated in Fig. 21, the toe of the embankment, point B, lies only slightly outside the fill. Therefore, there is no danger of substantial sloughing of the lower portion of the slope. However, when h/d is large, as, e.g., in Fig. 22, the portion of the toe of the embankment which is resting on the peat surface would not stand at the slope of 1 on 2. One may either allow the lower slope of the embankment to assume a considerably flatter equilibrium slope, or the toe can be protected against sloughing by building up a berm outside the embankment slope consisting of dumped excavation of the peat. The filling of this protective berm should not lag too far behind the elevation to which the embankment has been built up. The height of the protective berm may be determined by the method which is described under Rule II and illustrated in Fig. 25.

Rule II—With Temporary Surcharge

From the edge of the crest of the embankment, point A in Fig. 21, draw a line sloping 1 on 0.7 to its intersection with the hard bottom, point C. The vertical line CD establishes the width of the excavation of the peaty stratum, unless the theoretical toe of the embankment, point B in Fig. 21, is located closer to the center line of the fill, in which case the width of the excavation shall be fixed by the vertical through point B, as is the case in Figs. 23 and 24.

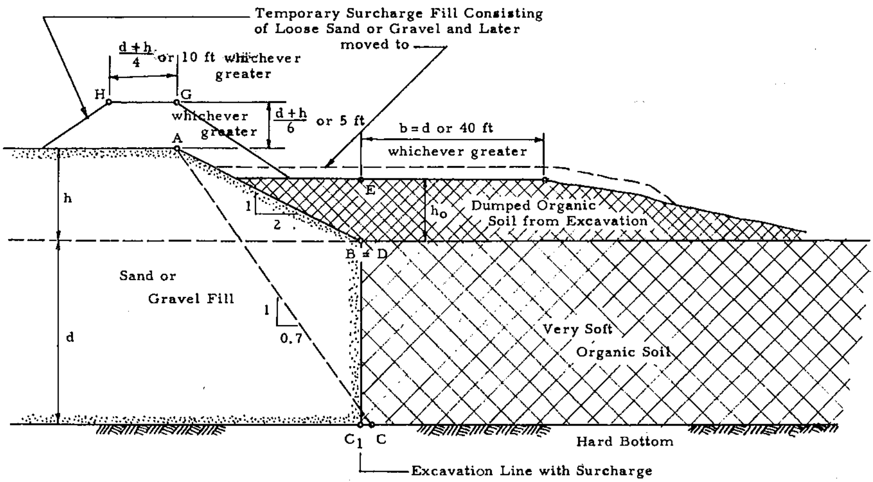
The design slope of the embankment should be 1 on 2. Simultaneously with the construction of the embankment above ground surface, an adjacent protective berm consisting of the excavated peat, should be constructed which should not lag too far behind the embankment in elevation. The minimum height, h_o, of this berm should be obtained from Fig. 25 as a function of the ratio h/d. The minimum width, b, of the berm should be equal to the depth, d, of the peaty stratum, or 40 ft, whichever is greater.

After completion of the design cross-section of the embankment the temporary surcharge is applied in the form of a dumped dike of a cross-section which is determined as follows:

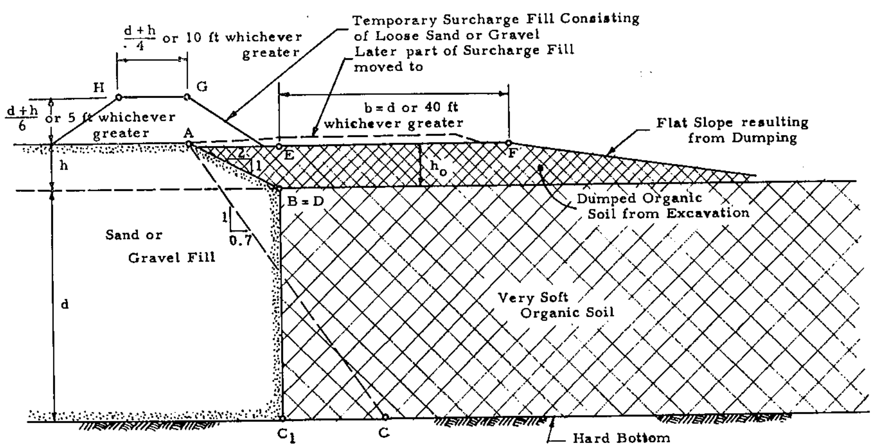
(1) The outer edge of the crest of the dike, point G in Fig. 21, is located directly above the outer edge of the crest of the embankment.

(2) The height AG should be equal to $(d + h)/6$, or 5 ft, whichever is greater.

(3) The crest width of the surcharge dike should be equal to $(d + h)/4$, or 10 ft, whichever is greater.



PROCEDURE—H/D = 0.5
FIG. 23.—FOR H/D = 0.5 REQUIRED MINIMUM H_0 = 0.67H



PROCEDURE—H/D = 0.2
FIG. 24.—FOR H/D = 0.2 REQUIRED MINIMUM H_0 = 0.9H

In general, the surcharge fill should be left in place for a period of several weeks, unless settlement observations indicate that the settlements are continuing at an excessive rate. After sufficient consolidation has been achieved, the surcharge fill should be spread out over the protective berm, as indicated by the dashed line IJ in Fig. 21.

The application of the above rules to various other ratios of h/d

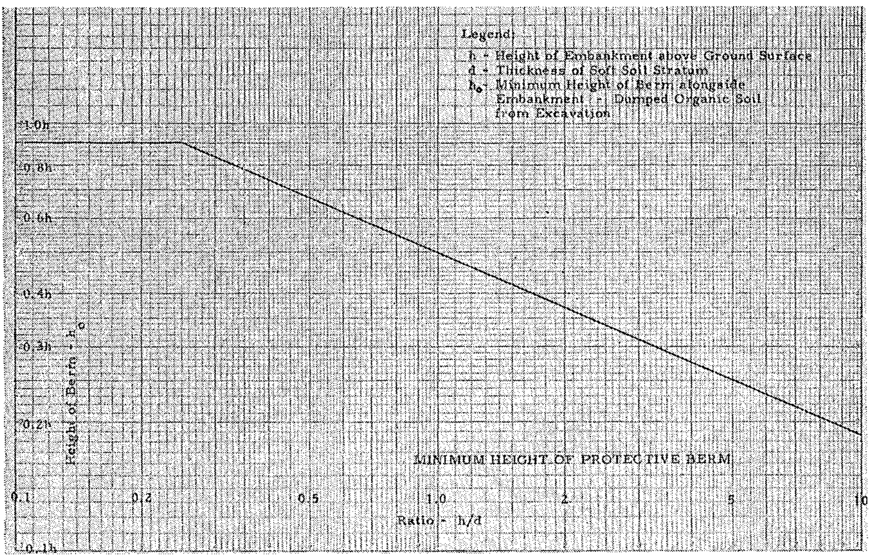


FIG. 25.—MINIMUM HEIGHT OF PROTECTIVE BERM

than those used in Figs. 20 and 21, are shown in Figs. 22, 23, and 24. It will be noted that in Figs. 23 and 24, i.e., for low embankments as compared to the depth of excavation, the width of excavation is governed by the vertical through the theoretical toe of the embankment, i.e., point B, and not point C.

When the thickness of the peaty stratum is small as compared to the height of the embankment, as, e.g., in Fig. 22, the percentage increase in the volume of fill by using Rule I as compared to Rule II, is so small that it will be more economical to use Rule I without the surcharge.

It is believed that the proposed Rules I and II are on the conservative side, and that on the basis of careful observations of embankments built in this manner, supplemented by the necessary tests of the soft materials, it may eventually be possible to modify these rules and permit smaller widths of excavation.

Explosives and Timing

For peat blasting operations the use of 40 to 60% gelatin dynamite has been standard practice. Small delays were commonly used both on this continent and in European countries for this type of work.

TABLE V

Thickness H of Peat Deposit	Distance D Between Rows of Charges	Spacing B Between Charges within a Row		
		Below Crest	Below Slopes	Outside Toes
<30 ft	0.75H	H	0.75H	0.5H
30 to 60 ft	0.75H	0.5H + 15 ft	0.5H + 7.5 ft	0.5H
>60 ft	45 ft	45 ft	38 ft	30 ft

TABLE VI

Thickness H of Peat Deposit—feet	Amount of Explosives W in. lb/charge Along Center Line of Fill	
	For Width of Crest <30 feet	For Width of Crest >30 feet
10	50	60
20	80	100
30	120	150
40	160	200
50	200	250
60	250	300

However, the author believes that the only benefit derived from delayed firing is a lessening of the shock set up by blasting large quantities of dynamite. Personal observations of blasting by the underfill method has convinced the author that the simultaneous firing of neighboring rows of charges without delay is more effective than the customary small delays between the blasting of outer and inner rows of charges.

On projects of peat blasting it is considered good practice to seek expert advice from explosives manufacturers. Such advice usually not only includes transport and storage, safety measures, installation of charges and the blasting operation itself, but also decisions regarding type of explosive, the spacing of individual charges, delays, etc. For large projects, test sections are strongly indicated. Excessive charges used in settling highway fills are not only wasteful but may result in decreased effectiveness of the blasting method.

Evaluation of the most important underfill blasting projects has led to tentative recommendations contained in Tables V and VI. Eventually improved versions of such recommendations will also have to take into account differences in the properties of the materials to be displaced.

New Types of Blasting Agents

Among the new types of explosives which have been developed in recent years, the most interesting one is a combination of ammonium nitrate and fuel oil. Although this blasting agent has so far not been used for peat blasting operations, it is believed that it would have important advantages for the underfill blasting procedure, for the following reasons:

(1) Slower reaction time, combined with development of appreciably greater volume of gas.
(2) Much greater safety in handling, in transport, and in storage.
(3) The cost is on an average 5 cents per pound as compared with 30 to 35 cents per pound for 40% gelatin dynamite.

Ammonium nitrate is very sensitive to moisture, but since preassembled charges can be sealed in water-tight containers, this should not be considered a serious problem. A second reservation could arise from the fact that for best results it requires adequate charge confinement. In applications of underfill blasting, this requirement may be sufficiently fulfilled by the combination of confining the blasting agent in containers, and also by the weight of an appreciable height of fill overburden.

Conclusions and Recommendations

The purpose of this section is to present the author's conclusions and recommendations for highway construction, for temporary road construction, for the construction of levees, and for future research.

1. *Construction of Highways Across Peaty Deposits*

The most efficient and most economical procedure for displacing soft, organic soil by sand, gravel or rockfill for the purpose of constructing a highway embankment, consists of the following steps:

(a) *Blast center strip*—A 5 to 10 ft wide center strip of the mat or crust of the peaty deposit is destroyed by blasting, as shown in Fig. 7a.

(b) *Dump initial fill*—A fill is built by end-dumping over the blasted strip with a crest width of 15 to 20 ft, and to such height that after the initial subsidence the crest will be about 3 ft above the adjacent ground surface, as shown in Fig. 7b.

(c) *Construct and settle by blasting the main fill*—The procedure will differ in detail depending on whether or not the explosive charges can be jetted through the fill material. The more desirable procedure is to dump the main fill first and then place the charges by jetting through the fill.

(1) *For sand or small gravel fill,* the initial fill (see Fig. 7c) should be widened and increased to the full design width and to a sufficient excess height so that the total volume will be about equal to the estimated requirement of fill volume after the blasting operation. For great thickness of the soft deposit or a narrow embankment it may not be possible to place the full estimated volume of fill within the desired width. Then the preassembled charges should be jetted through the fill down to the bottom of the soft stratum. Tables V and VI may serve as a basis for selecting the size and the layout of the charges. In general, the entire fill length should be blasted in one operation, as illustrated in Fig. 12. For large projects it is desirable to blast a test section at least 100 ft in length, principally for the purpose of checking the amount and distribution of explosives.

(2) *For coarse gravel and rockfill,* through which the charges cannot be jetted, it is necessary to install the charges before dumping of the main fill. The charges should be jetted on both sides and close to the initial fill, in a manner similar to that illustrated in Fig. 9a. The electric wires should be carried through plastic tubing well beyond the toe lines of the final fill. The main fill is then built and blasted as described in the preceding paragraph.

(d) *Explore shape of fill below ground surface and if necessary widen fill by supplementary blasting*—The exploration can be carried out quickly by means of jet probings either through the fill, or by slanting holes from both sides. For guidance in establishing a desired cross-section, Rules I and II as discussed in the section on REQUIRED WIDTH OF DISPLACEMENT, may be used. These rules are believed to be conservative. It is possible that a contact width of the fill along the firm bottom after blasting need not be greater than the crest width of the embankment.

2. *Construction of Temporary Roads Across Peaty Deposits*

The present empirical knowledge of construction of fills across peaty deposits is sufficient to suggest a procedure for the construction of a roadfill which would probably satisfy the following specifications: (1) Minimum volume of suitable fill material; (2) minimum time for construction; (3) safety against sliding; (4) settlements which can be readily repaired by constant maintenance operations. The author believes that these requirements can be fulfilled by using the following construction procedure:

(a) Blast a 5 to 10 ft wide center strip of the mat or crust of the peaty deposit, as shown in Fig. 7a.

(b) By end-dumping, build a fill with the desired crest width, but not more than 20 ft, and to a height such that after the initial subsidence the crest will be about 6 ft above the adjacent ground surface. If the desired crest width is more than 20 ft, the additional fill should be placed on both sides, and at the same time any deficiency in height which will develop during this operation, should be compensated by additional fill.

(c) Remove the top 3 ft of the fill by bulldozing it to both sides, creating shoulders with flat slopes which extend over the adjacent surface of the mat or crust of the organic deposit. The purpose of these flat shoulders is threefold:

(1) To increase the factor of safety of the main fill against sliding.

(2) To reduce the settlements of the main fill by creating counterweight berms.

(3) To form road shoulders which because of their flat slopes will provide a measure of safety against vehicles dropping off the narrow main fill, and which in addition can be used as emergency strips for disabled vehicles or light traffic. However, heavy vehicular traffic will have to be confined by appropriate markers to the crest of the main fill.

A road fill constructed in this manner will continue to settle and will require grading operations and occasional addition of fill.

A roadway so constructed would lend itself to further widening, or to the construction of a fill which is settled by blasting to firm bottom as described under a preceding heading.

For the major volume of the fill one can also use a variety of materials other than granular soils, such as stiff clay, clay shale, and badly weathered or disintegrated rocks. Such fill would finally have to be covered with a minimum thickness of a good quality granular fill, or with landing mats if granular soils are not available.

Certain procedures which have been used for settling fills through soft deposits for highway construction should not be used for temporary roads for which speed of construction and minimum volume of fill material are basic requirements. Particularly objectionable because of the danger of slides and because of waste of fill material are: (1) the breaking up of the surface mat or crust over the full base width of the embankment, and (2) the "relief method."

3. *Construction of Low Levees Across Peaty Deposits*

The author believes that for the construction of low levees across highly organic soil deposits, a procedure similar to that described above for the construction of temporary roads would be suitable and economical.

4. *Suggestions for Further Investigations*

Among the many topics which deserve investigation, the following are of particular interest:

(a) *The necessary width of contact of the fill with the firm bottom,* to achieve (1) positive protection against sliding, and (2) settlements of tolerable magnitude. It is believed that for many conditions it would be sufficient if this contact width is made equal to the crest width of the embankment. Ideally, this question would require systematic, full-scale experimentation. However, it is likely that considerable progress could be made by investigating the in situ stress-deformation characteristics of the soft deposits combined with theoretical studies of the stress conditions at the interface of fill and displaced mass.

(b) *The use of ammonium nitrate—fuel oil* as blasting agent for settling fills through peaty deposits would deserve investigation because of several potential advantages as discussed under a preceding heading. However, the only satisfactory approach to this question would be by full-scale experimentation in close cooperation with the manufacturers of this type of blasting agent.

(c) *Installation of charges before dumping of main fill* has important advantages as compared to the placement of charges by jetting through the fill. Experience in Germany with this procedure has been very encouraging. It is much faster, simpler and cheaper, and can be used for all types of fill materials including rock fill. The only objection which has been raised against this procedure is that the electric wires may be torn by the mass displacements. On the projects on which it was used, there has been no evidence that any significant number of the charges did not fire in spite of the fact that no special efforts were made in stringing out the wires to include extra slack, or to provide such slack by spiralled wires. For a systematic investigation of the installation of the effectiveness of charges installed ahead of the filling, it is recommended that an actual peat crossing project be utilized. Dummy charges should be installed ahead of the filling, in every respect as the real charges, but with the wires connected within the dummy. After the filling operation, the integrity of the wires should be checked. Various types of wires and methods for providing slack should be investigated in this manner.

(d) *The most desirable combination of details for construction of temporary roads on peaty deposits* deserves full-scale experimental research. The procedure recommended above would form a basis for the start of such an investigation. Certainly the type of organic deposit, the thickness and strength of the mat or crust, and the character of the fill material all are important variables. A systematic investigation of these variables by full-scale experimentation would yield invaluable guiding rules for such construction operations.

Acknowledgments

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