Soil and Foundation Investigations on the Patapsco Tunnel Project

ROBERT B. BALTER, Chief Soils Engineer, and
S. MURRAY MILLER, Soils Engineer, J. E. Greiner Co., Baltimore, Md.

● THE PATAPSCO Tunnel Project constructed by the State Roads Commission of Maryland is the largest public works project ever undertaken in the state. It is comprised of a twin-tube tunnel under Baltimore Harbor between the industrial areas of Canton and Fairfield of Baltimore City, together with expressway approaches connecting the tunnel with Baltimore-Washington Boulevard and Baltimore-Washington Expressway on the west; with the Glen Burnie Bypass and Governor Ritchie Highway on the south; and with Pulaski Highway and the proposed Northeastern Expressway on the northeast. Figure 1 is a location map of the Patapsco Tunnel Project.

The entire roadway is 17.6 mi long, including 1.2 mi of tunnel, and 3.5 mi of structures. The approximate cost of the entire project is $127,000,000.

The purpose of this paper is to describe the procedures followed in the soil and foundation studies for this project from the civil engineering report phase through the design and construction phases, and, finally, into the maintenance and operations phase of the facility after it was opened to traffic. No attempt has been made to discuss the many interesting aspects of the tunnel investigations and related construction features. The tunnel portion of the project is worthy of an independent paper.

CIVIL ENGINEERING REPORT

In September 1953, the State Roads Commission of Maryland engaged J. E. Greiner Company to prepare a civil engineering report. One of the main reasons for the report was to isolate problem areas and to estimate the construction cost of the entire project. The report included a preliminary set of plans and specifications for the tunnel and its approaches and indicated the scope and general character of the work. Design criteria were also prepared for the purpose of establishing uniformity over the entire project. Because of its timing with regard to the construction seasons, it was considered advantageous in June 1954, to start the design work on the tunnel section. This enabled the design engineers to prepare a complete set of plans and specifications in time to start the tunnel construction prior to the construction of the approaches. This scheduling was necessary because the tunnel was contemplated to require 32 months for construction as compared to 27 months for construction of the approaches. On October 8, 1954, the civil engineering report was submitted to the State Roads Commission.

The first undertaking in the soil and foundation studies was the investigation of available sources of data. Soil Survey Reports of the U. S. Department of Agriculture, reports of the Maryland Department of Geology, Mines, and Water Resources, and publications of the U. S. Department of the Interior were used in developing the general soil and geological conditions of the area.

To acquaint the reader with the local soils and geology, a brief description of the general area is offered.

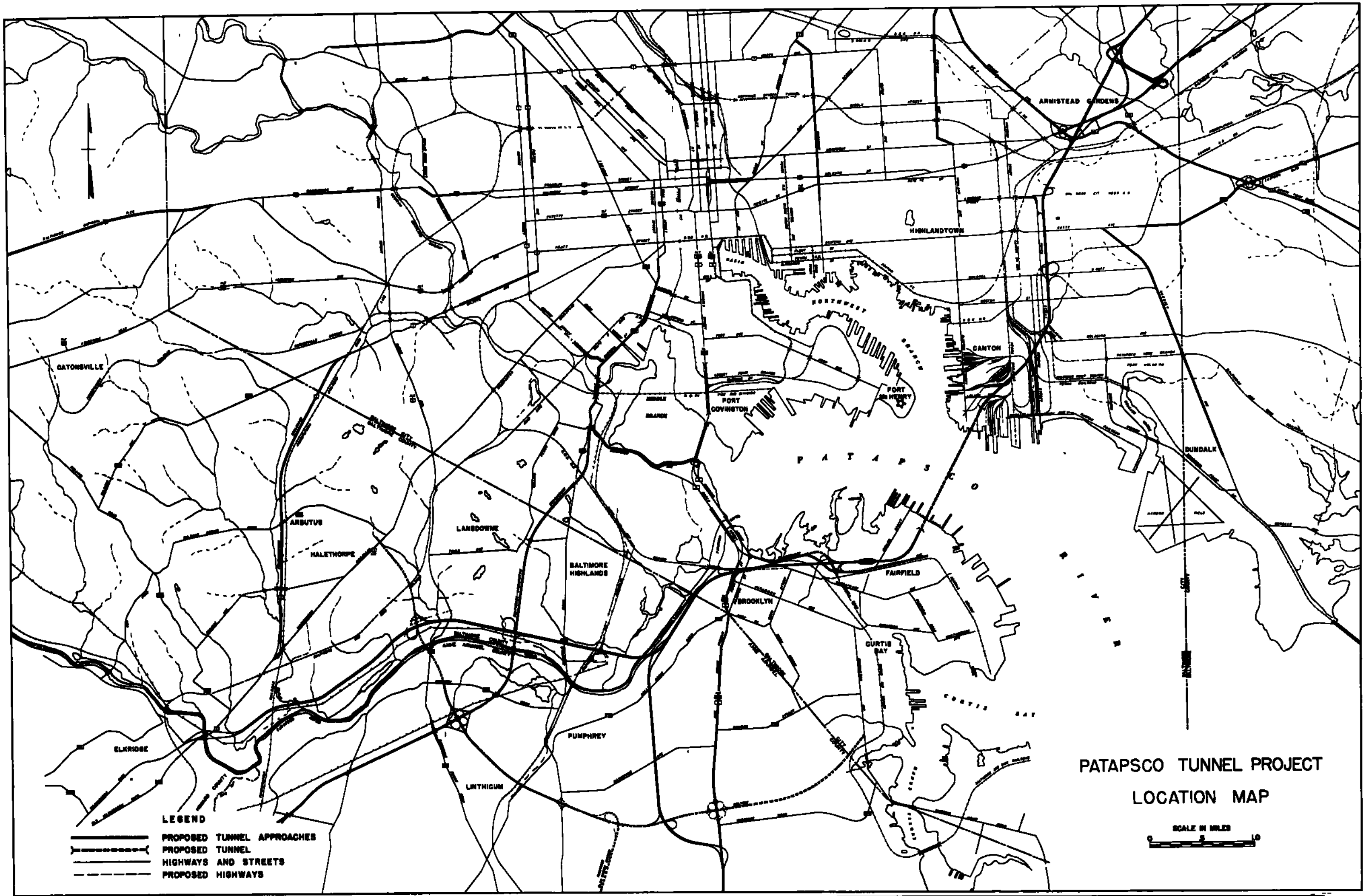


Figure 1. Location map.

The Patapsco Tunnel Project falls completely within the Coastal Plain. The predominant soil types are eroded sediments of an unconsolidated nature. Such topographic features as low hills, shallow valleys, flat plains, and numerous terraces are characteristic of this plain. Long, narrow peninsulas which extend in a southeasterly direction to the Chesapeake Bay are separated by bodies of water which have "drowned" existing valleys.

The surface soils most prevalent along the route represent formations of the Potomac Group of the Lower Cretaceous. Some recent sediments of the Quarternary are also evident.

The Potomac Group consists of three formations; namely, the Patuxent, the Arundel, and the Patapsco. These sediments rest upon crystalline rock, which, with the exception of the most westerly portion of the route near the Baltimore-Washington Boulevard, lies far below the limits of any foundation considerations.

The first deposits formed upon the rock were the coarse sediments of the Patuxent. After a period of erosive action, clays of the Arundel filled in old drainage depressions, and then increased in depth completely covering the Patuxent. This process occurred under swampy conditions as evidenced by the recovery of lignitic substances and segments of tree stumps during subsequent boring operations.

A general areal submergence of the Coastal Plain followed and resulted in the deposition by streams of sediments of a coarse nature over the Arundel. This formation is known as the Patapsco. A prolonged period of erosive action then took place.

The soils of recent age encountered in this area are gravel and clay deposits which were stream transported from the northwest during the periods of glacial action.

After all of the available information was compiled and analyzed, a field study was made, during which time the entire proposed alignment was covered on foot, observations made, and samples recovered for laboratory testing. Special care was exercised to assure that all areas of poor soil and foundation conditions, as well as areas of exceptionally good soils, were delineated.

Of the numerous areas of questionable soil and foundation conditions which were observed and described in the civil engineering report, five have been selected for discussion of their design treatments. Three of the five areas are discussed in more detail in the construction phase.

The civil engineering report noted that the valley of Herbert Run and the Kaiser fill represented two adjacent areas of concern. Though both areas were foundation problems, the soil conditions were quite dissimilar in character. In the valley, sediments of organic silty clays to depths exceeding 10 ft were encountered. The low lying terrain was very wet and swampy. Immediately to the east of this site was a loosely placed fill which, at the time of placement, was waste material from the construction of a nearby Kaiser Aluminum plant.

The tidal marsh west of the Patapsco River was recognized during the preparation of the report as an area deserving special foundation considerations. In addition to numerous probings which revealed organic deposits continuous over sands and gravels, a number of undisturbed samples in the organic deposits were recovered from contract borings. The organic soil was found to vary from 12 ft near the shoreline to 35 ft in the vicinity of one of the many tributaries located in the marsh. The results of consolidation tests performed on the undisturbed samples indicated that settlements ranging from 1 to 3 ft could be anticipated. Eighty-five percent of these movements would occur during the construction period.

The third area discussed is the miscellaneous fill west of Potee Street which extends for a considerable distance along the eastern bank of the Patapsco River. Because of its extent and variable character, contract borings of a limited nature were taken through this reach. Organic sediments and miscellaneous dumped materials were disclosed by the borings and found to vary from 15 to 35 ft.

Compressible soils north of the tunnel were revealed by borings taken in connection with the advanced tunnel design. Intermittent soft layers of organic deposits were evident to a depth of 80 ft.

The final area to be discussed is encountered in passing through the refinery properties of Standard Oil Company of New Jersey. Numerous miscellaneous fills of shallow extent were observed. One critical area in this vicinity was the miscellaneous fill adjacent to Tank 502.

DESIGN PHASE

After acceptance of the civil engineering report by the State Roads Commission and arrangements for financing completed, six design engineering firms were selected for the actual design work of the approach expressways. J. E. Greiner Company had already been retained by the Commission as General Consultant to act as its agent in the supervision of the work. During the progress of the design work, the Commission decided to extend the project at its northern terminus. Accordingly, a seventh form of engineers was selected to accomplish the additional work.

With numerous design firms performing their own soil and foundation studies, it was considered desirable to establish a set of standard specifications for the performance of all subsurface studies. These specifications were part of an over-all engineering specification outlining the functions to be performed by the design engineering firms. They assured that adequate soil and foundation studies would be performed for the proper design and treatment of the structural and roadway aspects of the particular design section.

A master soil plan and profile was prepared for each construction section by the design engineer for that section. These drawings served a number of different purposes. They presented a complete picture of the soil conditions to the contractor for his interpretation so that full benefit could be made of the various soils encountered. They also eventually became a complete record of all of the soil and foundation studies which were performed when supplemented with information obtained during the actual construction. Where the data was too voluminous for inclusion on the master set of drawings, an additional report was prepared of such information. In an effort to simplify these plans, only one typical boring for each structure site was included. The remaining structure borings were shown on the design plans for the particular structure. Also included on the master plan and profile sheets were test data used in the development of the design plans, such as compaction test data, unconfined compression test data, and consolidation test data. Figure 2 shows a typical master soil plan and profile sheet.

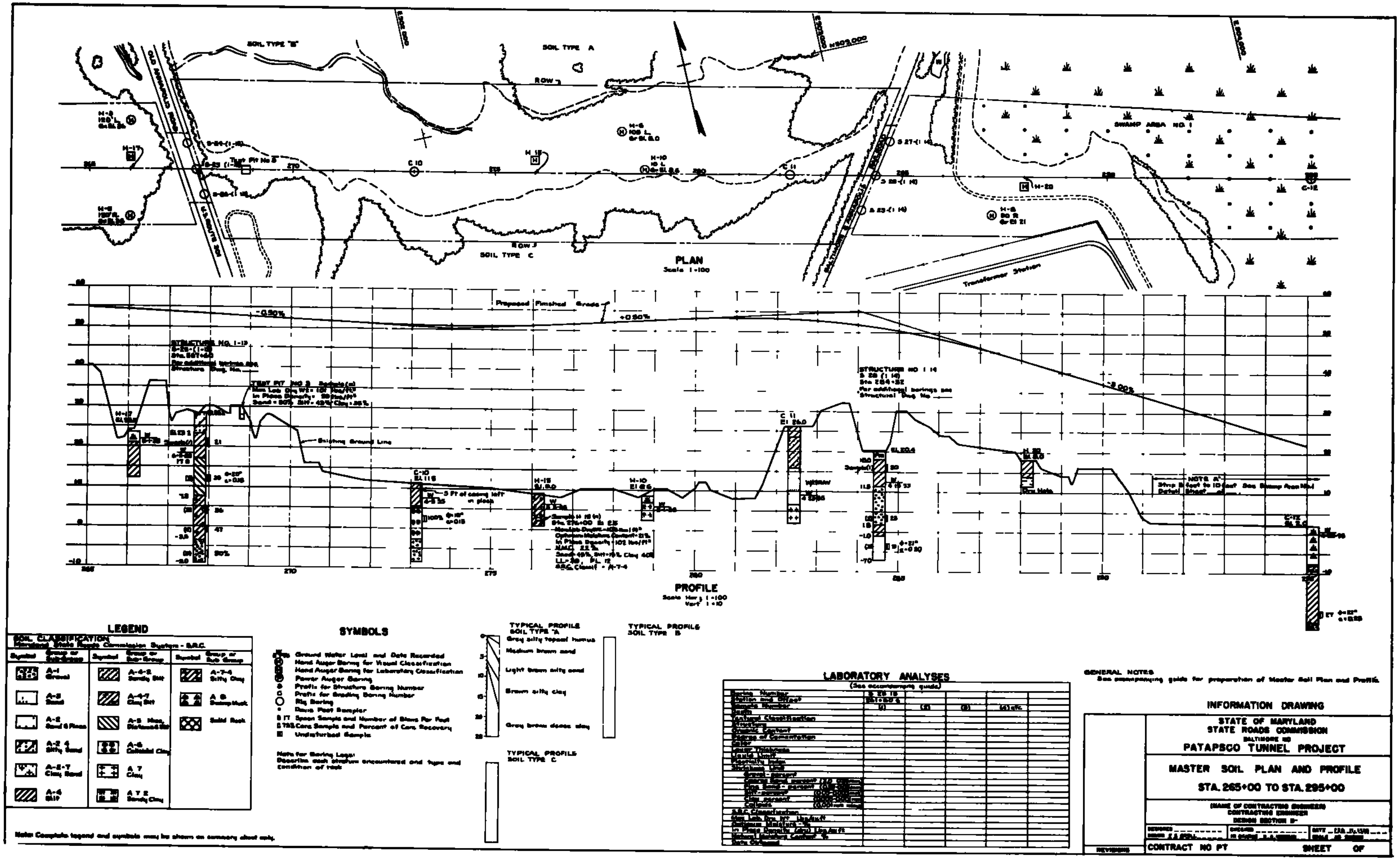


Figure 2. Master soil plan and profile.

The scales selected for the drawings were the same as those used in the preparation of the contract plans; namely, 1 in. = 10 ft vertically and 1 in. = 100 ft horizontally. The plans were kept simplified by only showing horizontal alignment, stationing along the centerline every 500 ft, locations of borings, and sufficient topographic features to permit proper orientation in the field. These studies also included interchange areas. The data obtained for inclusion on the master soil plan and profile sheets were developed from contract borings, hand and power auger borings, test pits, and special types of samples required in swampy areas.

Although the General Consultant provided general specifications for the performance of test borings, it was the responsibility of the design engineering firms to prepare the boring specifications and supervise the actual boring operations for their particular sections. The design engineers' own field personnel supplemented the contract boring data with whatever other necessary information was required to present a practical representation of the actual soil and foundation conditions.

Some of the general specifications relating to the soil and foundation studies are mentioned. Normally, one rig boring was taken at each abutment or pier location. Depending upon the structural design, borings were staggered in such a manner that the uniformity or lack thereof in the soil profile was established prior to increasing the number of borings required at a site. Contract borings were taken in cut sections when the cut was longer than 500 ft and the depth of cut exceeded 15 ft. Depending upon the length of the cut, borings were spaced at 300- to 600-ft intervals and taken to depths of at least 5 ft below profile grade line. Rig borings with undisturbed samples were also taken in areas of high embankments and questionable subsurface conditions. These samples were subjected to laboratory analyses necessary for the proper selection of the final design treatment.

Seven boring contracts were awarded by the Commission to obtain ordinary dry samples, core borings of a limited nature, and undisturbed samples for testing purposes. The aggregate amount of these contracts was $84,900, which represented 0.067 percent of the approximate project costs.

Other soil conditions were obtained from auger borings with particular attention given to transitional areas of cut to fill. These areas are quite frequently a source of pavement distress and deserved special review.

The design investigation for the valley of Herbert Runn and the Kaiser fill recognized two problems; namely, the undesirable soft organic soils in the valley and the presence of a loosely dumped, unconsolidated embankment to the east. Figure 3 is the soil profile of this area. The limits of the unsuitable material were determined by a grid pattern of 32 hand auger borings and probings which disclosed as much as 20 ft of soft organic clayey silt. Borings taken at the site of the structure over Herbert Run provided undisturbed samples for foundation studies in the valley area. These studies revealed that the weight of the 40-ft embankment across the valley would result in settlements of intolerable magnitudes requiring time of settlement far beyond the period of construction. It was decided, therefore, that the unsuitable soil be completely removed for the 800-ft valley crossing. the limits of removal of this organic soil were as shown on Figure 4, the standard section for removal of unsuitable material.

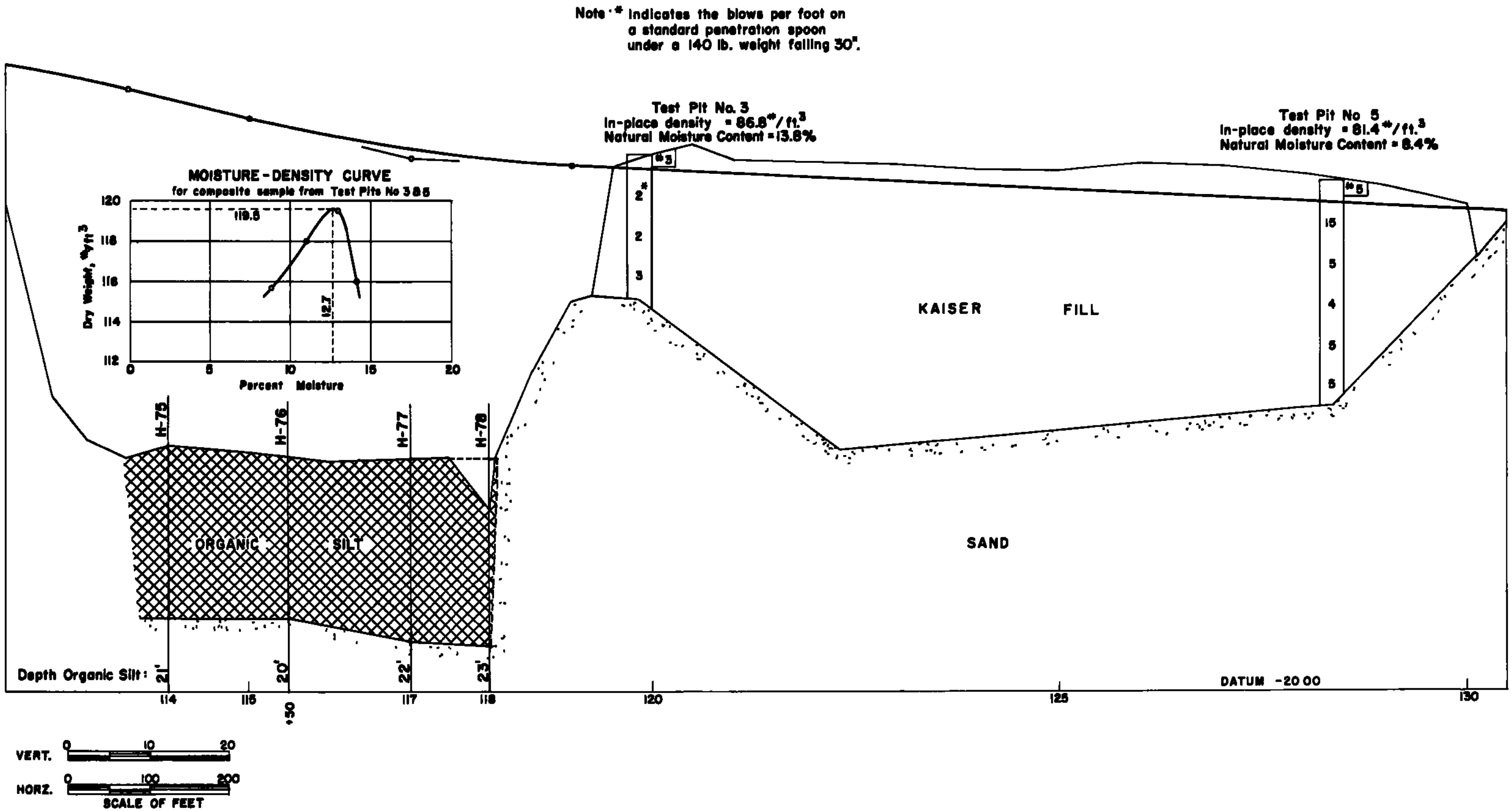
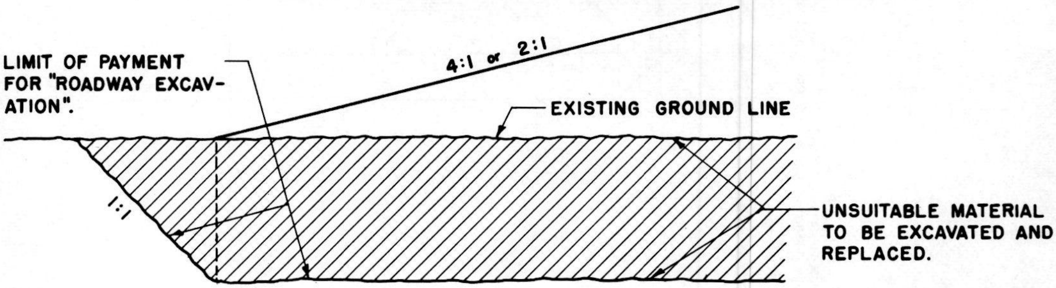


Figure 3. Soil profile—Herbert Run and Kaiser fill.



REMOVAL OF UNSUITABLE MATERIAL

Figure 4. Standard section for removal of unsuitable material.

The Kaiser fill immediately east of the valley was composed of 30 ft of granular materials, which had been placed by a bottom dumping operation. At the time of this operation, it had not been anticipated that the area would be utilized in the future for an expressway. Figure 5 is a view of the unconsolidated fill material.

Figure 5. Unconsolidated Kaiser fill.

Contract borings through this area, which extended for 1,000 ft, indicated that the very loose deposits had a resistance of 2 to 5 blows per foot on a standard penetration spoon driven by a 140-lb weight falling 30 in. This poor resistance was consistent through the entire 30-ft depth of the deposit. In-place density tests disclosed the average fill density to be 75 pcf. When compacted by the standard Proctor density test, the same soil had a dry unit weight of 120 pcf. The Highway Research Board Classification for this material was A-2-4.

Because the material was of very low density and would be subjected to frequent vibratory loads from the heavy volume of truck traffic, it was the opinion of the engineers that harmful differential settlements could occur. In the final design of this section, the removal of the Kaiser fill to a depth of 10 ft below profile grade line was considered adequate. Partial removal, rather than complete, realized substantial savings in the cost of this section due to the reduced earthwork quantities. Because the material was granular in character, it was used as underwater backfill in the valley to the west where the unsuitable organic soils were removed. After a surface preparation, the embankment up to profile grade line was constructed in accordance with the standard compaction requirements for fills.

The tidal marsh west of the Patapsco River was investigated by obtaining 15 contract borings and 22 hand auger borings and probings. It was found that unsuitable materials extended to a depth of 60 ft. These sediments were deepest toward the River and studies indicated a viaduct type structure to be most economical over these deep deposits.

Figure 2 shows the plan view of the western limit of the marsh area in the vicinity of Station 291. The presence of unsuitable organic silt along the centerline of the expressway is indicated on Figure 6. From this information, it was recognized that portions of the westbound lanes between Stations 287 and 291 would be underlain by varying depths of organic silt. The eastbound lanes were located on firm ground. To further complicate the differential conditions at this site, the westbound lanes required a fill of 40 ft in height to bring it to grade, whereas the eastbound lanes required only a 20-ft fill. This poor condition was recognized early in the route studies, but no change in location was made because it would have been necessary to relocate an electric substation, the property fences of which were already abutting the right-of-way boundaries. Economic studies of this situation favored a complete removal of the unsuitable materials and a backfill with clean granular soils below the water level and the use of satisfactory borrow materials above this elevation.

The miscellaneous fill west of Potee Street was found to follow the Patapsco River for a considerable extent. Investigations through this reach disclosed the presence of several different types of undesirable materials. The original surface soils in this area were organic silts which varied in depth from 8 to 25 ft. Beneath the silt layer were excellent deposits of sand and gravel. Because of its poor supporting quality, this land had never been improved for either commercial or residential purposes. At different times, it had been utilized as a municipal dumping area. Twelve borings taken for the Potee Street structure and the future Patapsco Avenue structure in addition to auger borings taken at the location of fills, furnished data to differentiate between the variously constituted fills; namely, garbage fills, trash fills, and cinder fills. This is shown in part on Figure 7. The most undesirable condition existed at the location of the trash fill where the combined thickness of trash and organic silt was 30 ft. The ground water level through these fills

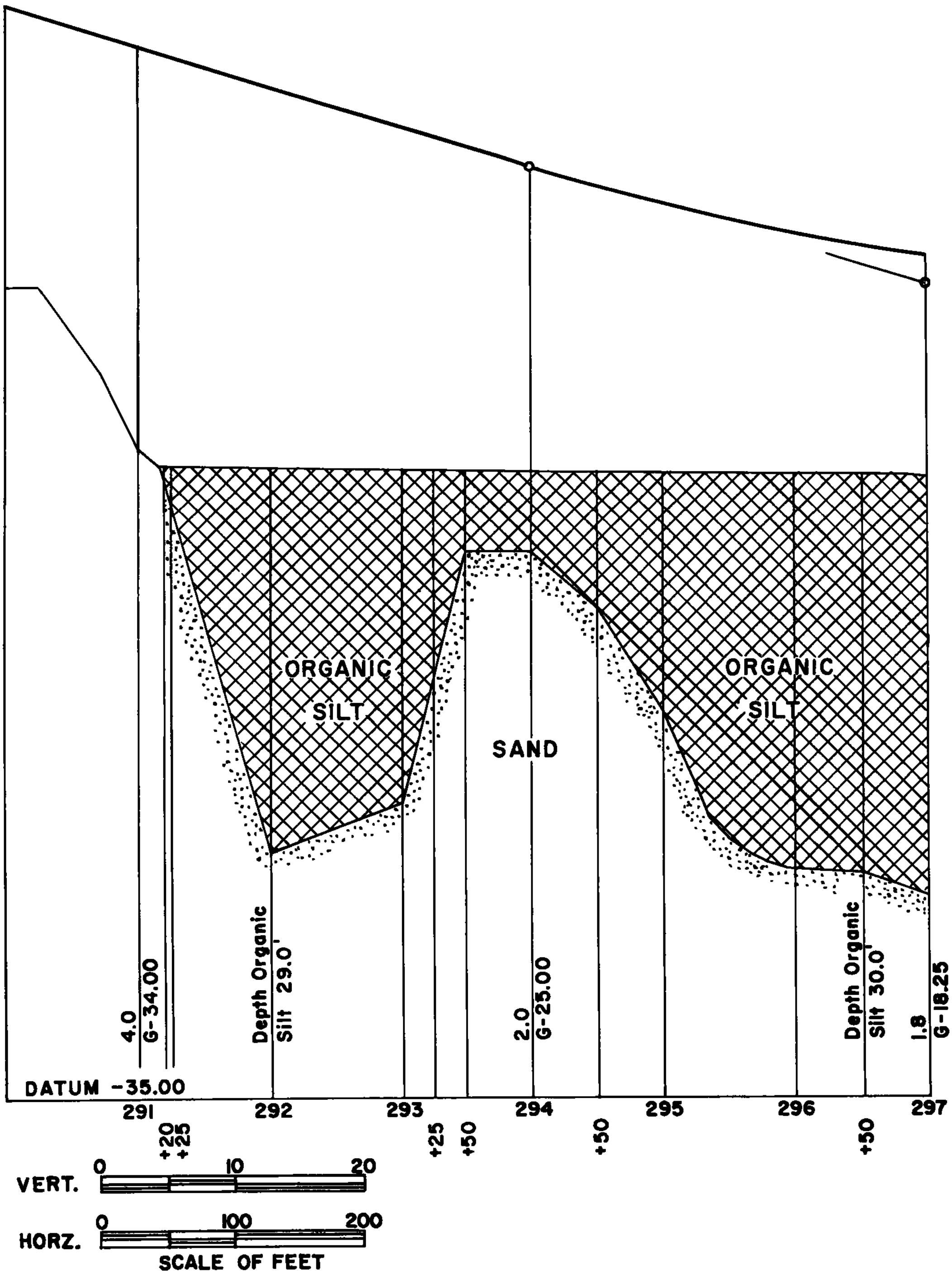


Figure 6. Soil profile—tidal marsh west of Patapsco River.

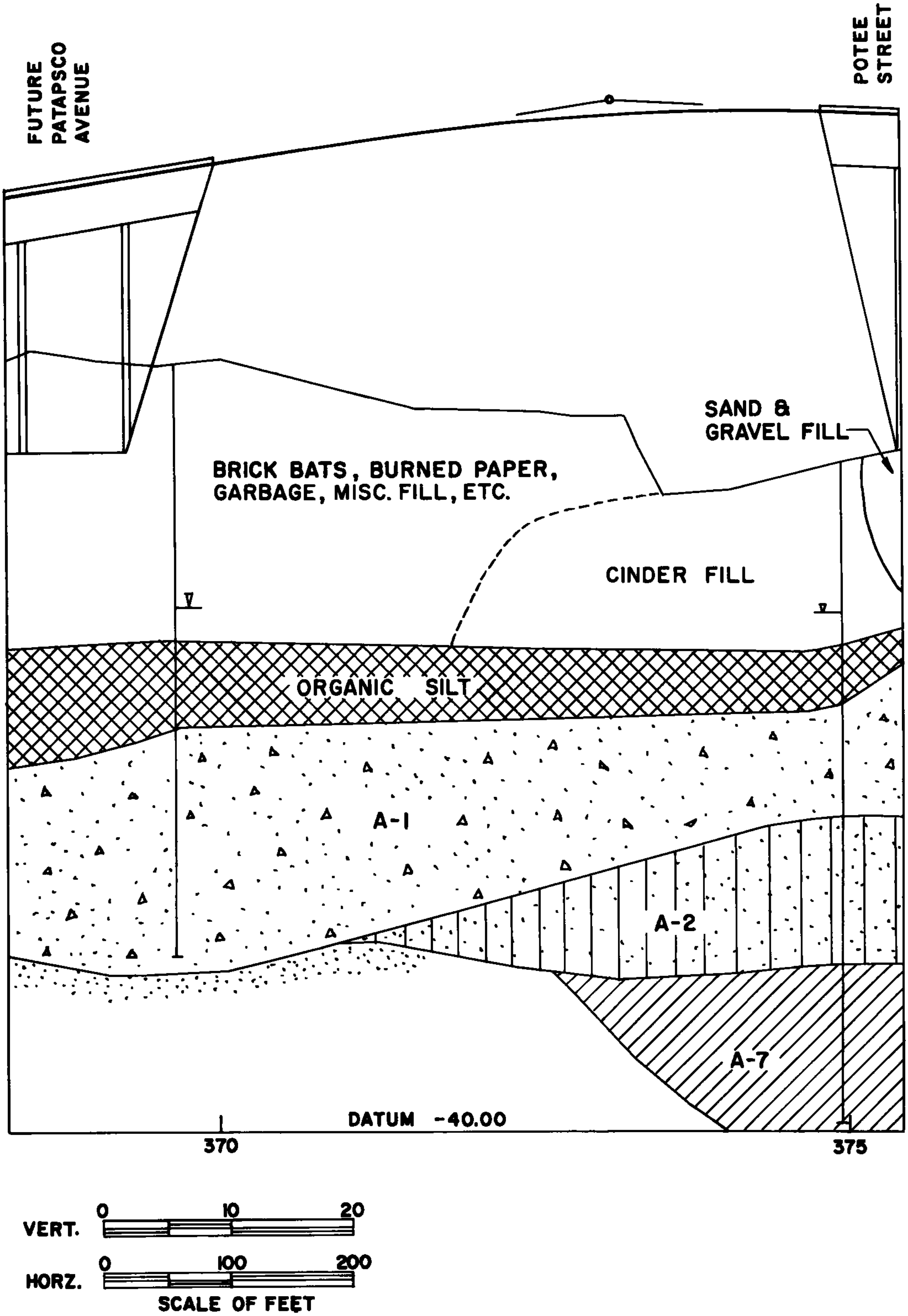


Figure 7. Soil profile—miscellaneous fill west of Potee Street.

was very close to the water level of the Patapsco River and, in many instances, was observed to fluctuate with tidal changes. This located the water level about 3 ft above the organic material.

The profile grade line through this area was approximately 50 ft above the sand and gravel stratum. This resulted in a fill of 30 ft in height. Because of the magnitude of this superimposed load, it was necessary to treat the various underlying fill materials in different manners. The garbage fill and trash fill were completely removed and granular materials used to backfill these areas. The section which was for the most part a cinder fill appeared capable of supporting the embankment and it was decided to surcharge this area rather than remove the cinders. This procedure resulted in a savings of $300,000. The design specified a 20-ft surcharge load which was to be placed by a carefully controlled field operation.

Though settlement computations were made in the underlying organic silt, it was the opinion of the engineers that the settlement in the cinder fill would be of more concern. Because of its heterogeneous character, settlement calculations were of little value in these deposits. However, the fact that the water table was at the lower extremity of the cinder material indicated the likelihood of very rapid deformations.

Another area requiring special foundation design and construction treatments occurred in the vicinity of the toll plaza which is located immediately west of the tunnel. Deep deposits of silt were encountered. The design engineers recommended the use of a surcharge loading to accelerate the consolidation of the unsuitable materials. Henry W. Janes, of Whitman, Requardt and Associates, thoroughly discusses this situation in "Deep Silt Consolidated by Surcharge Fill on the Patapsco Tunnel Approach," HRB Proceedings, Vol. 37, p. 538 (1958).

The compressible soils north of the tunnel are discussed briefly to emphasize the desire of designers to keep substructure and superstructure costs in economic balance. The roadway in this area ascends from the tunnel to the original ground surface and then to an elevated viaduct which crosses the railroad yards in Canton. Soft organic clays were the predominant soil throughout this portion of the alignment and were underlain by intermittent organic deposits to great depths. Design studies of the various foundation types indicated that retaining walls less than 9 ft in height could be founded on spread footings whereas higher walls required cast-in-place concrete pile foundations. Figure 8 indicates the open ramp north of the tunnel. The northernmost 400 ft of the walls on either side of the roadway are less than 9 ft in height.

A miscellaneous fill near the Esso tank farm was crossed as the route continued north from the tunnel. In addition to the presence of heterogeneous deposits of fill materials, other problems were encountered due to the maze of distribution lines which required relocation. These relocations were very critical because the change from one line to another had to be made during periods adjusted to satisfy the needs of the refinery. Water lines for fire fighting, crude oil distribution pipes, and safety dike reconstruction had to be coordinated into the over-all construction schedule.

A 20-ft depth of gravel, brick, and cinders was underlain by 8 ft of soft organic silt. The grade line through this area was 20 ft above the existing ground line. Examination of the fill samples indicated that the

Figure 8. Open ramp near north portal of tunnel.

consolidation of the organic silt was the main consideration. It was the opinion of the engineers that this compressible layer could be fully consolidated during the construction period.

CONSTRUCTION PHASE

The construction phase of the project became by far the most important. Situations arose which required immediate field decisions. Because judgment played an important part in these decisions, it was essential that the field personnel responsible for the soil and foundation aspects of the work be experienced and competent.

It must be appreciated that after construction was begun, the laboratory test data and office calculations were secondary in importance to actual field observations. The purpose of this data was fulfilled when used in designing foundations on questionable material. It then remained for field observations to govern final construction.

Many areas of interest were encountered during the construction phase; however, only three have been selected for further discussion in this paper.

First to be discussed is the tidal marsh west of the Patapsco River. This troublesome area was located in two design sections and, consequently, was constructed by two contractors.

The detailed studies favored the complete excavation of the unsuitable soft organic silty clay. At the western shoreline of the marsh only 3 ft of unsuitable soil was evident. This depth increased as the alignment progressed further into the swamp to an area where the maximum centerline thickness was 20 ft.

Initially, the unsuitable material was removed by a dragline in an underwater operation. The selected materials for backfill satisfied the standard specification for underwater backfills and consisted of either A-1, A-2-4, A-2-5, or A-3 soils. The material was placed by dumping it on the higher ground and then pushing it into the excavation by a bulldozer.

As discussed under the design treatment of this area, the soil conditions under the westbound lanes were much more critical than under the eastbound lanes.

During the progress of the work, a tension crack was observed along the edge of the embankment. Borings, which were taken to establish the cause for this visible indication of distress, revealed that there were areas not completely void of the unsuitable materials. A meeting was held between the design engineering firms concerned and the General Consultant to establish the course of action which should be taken to rectify this difficulty. It was the general opinion of the engineers that the probing method of assuring complete removal of unsuitable soils was defective under certain subsurface conditions and could prove misleading. To alleviate this situation, it was decided to remove the unsuitable material by dragline and attempt to lower the water table by pumping so that the actual soil conditions could be viewed during the removal operations.

Six days after the "excavation in-the-dry" operations were begun a hurricane passed through the Baltimore region and inundated the entire area as shown in Figure 9. It was necessary to use two 4-in. sump pumps on a 24-hr basis for $2\frac{1}{2}$ days to return the site to its original condition before the hurricane. Aside from the loss of this working time, no damage was evident.

Near the completion of the removal of the unsuitable soil, it was necessary to operate on an around-the-clock schedule. This continuous operation was necessary because the organic silts were unable to maintain stable slopes through the night and a great deal of time was lost each morning reestablishing stable working conditions. The lack of stability was the result of deepening silt deposits on an unfavorable cross slope away from the centerline of roadway. In addition, it was necessary to extend the standard limits for removal of unsuitable material (as shown in Figure 4) to provide a more stable condition.

It should be noted that the success of this operation of removal in-the-dry below the water level of the surrounding marsh is a direct function of the limited areas of operations. Small temporary dikes and generally two small sump pumps were sufficient to keep an area dry until the backfilling was completed. The experience here led to the conclusion that excavation of unsuitable material in-the-dry, if at all possible, is always preferable.

Figure 10 is a photographic view looking south at the completed embankment.

The area over the cinder fill portion of the miscellaneous fill in the vicinity of Potee Street was designed for the placement of a 20-ft surcharge load to accelerate the incalculable settlement. Because the

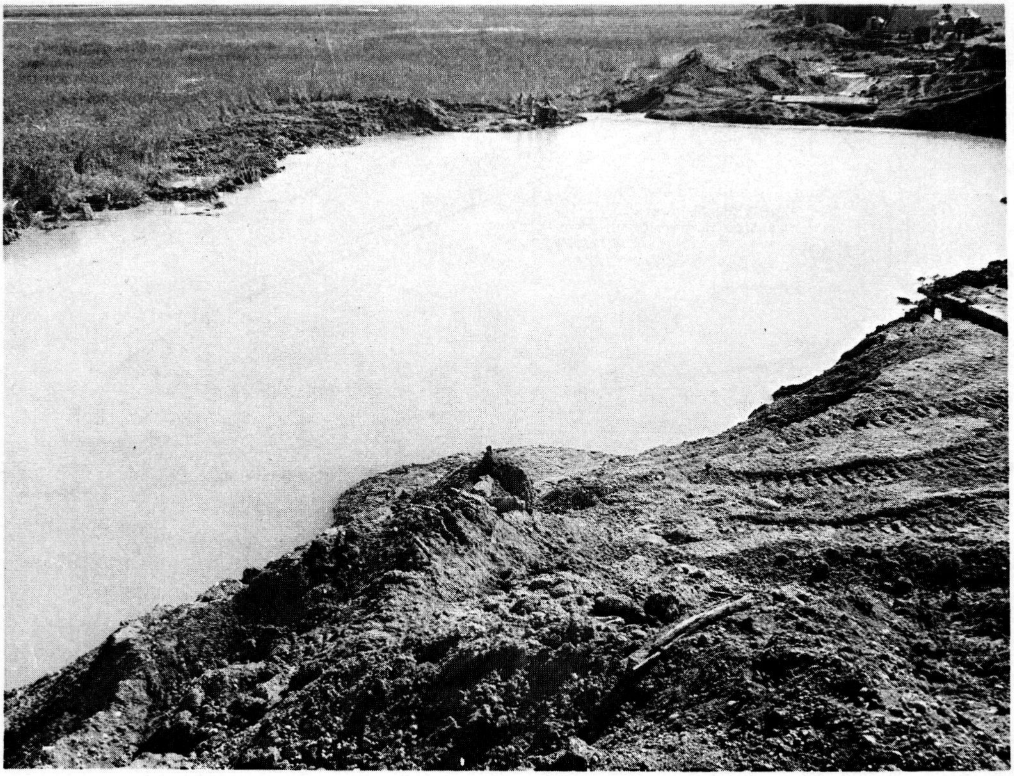


Figure 9. Station 287—area of unsuitable material.

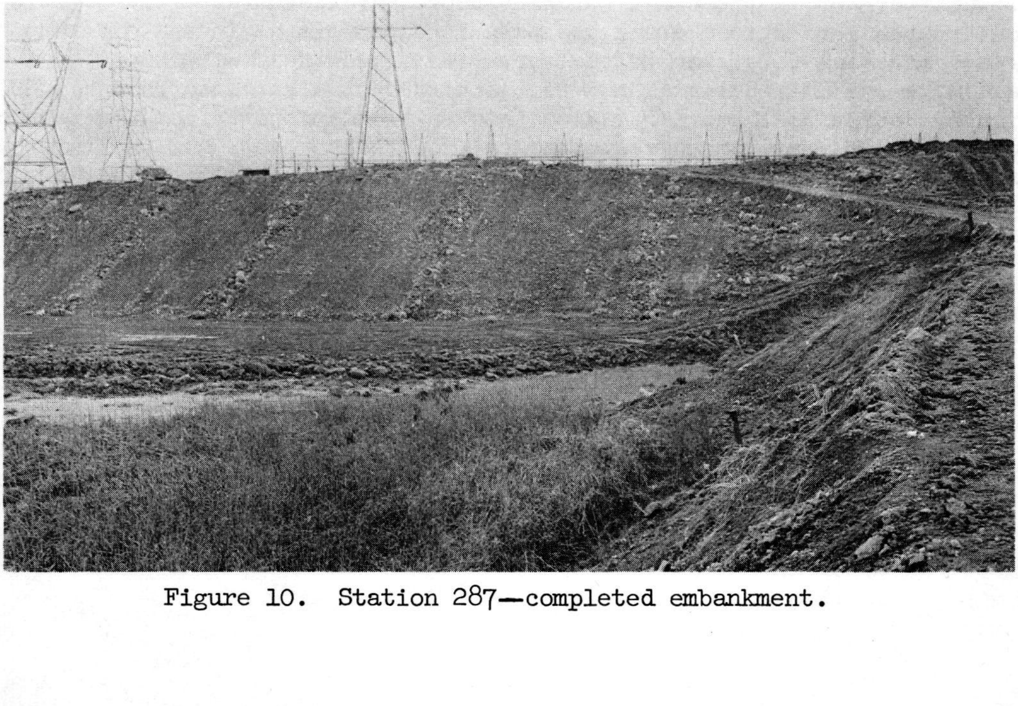


Figure 10. Station 287—completed embankment.

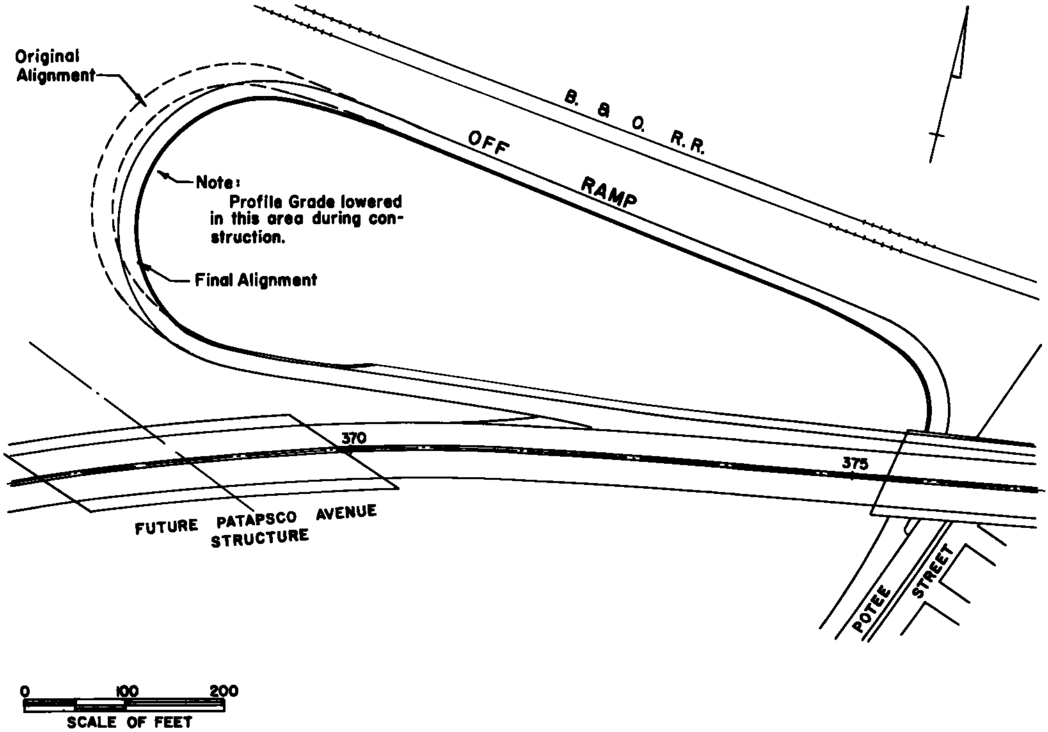


Figure 11. Station 375—main line and off-ramp plan views.

fill material overlying the relatively thin layer of organic silt was variable in character it was not the subject of settlement computations. Figure 11 shows the plan of the site.

In March 1956, construction was begun on the main line and off-ramp between the Potee Street and future Patapsco Avenue structures. Settlement plates were placed along the main line as well as in the off-ramp area. Abutting this ramp to the north was a two-track embankment of the Baltimore and Ohio Railroad and to the northwest a railroad bridge. Obviously detrimental settlements to this railroad property could not be tolerated. The new embankment, the railroad embankment, and the railroad bridge were in an area of questionable foundation subsoils, therefore, controls were established on one rail of the double track, the toe of slope of the railroad embankment, and the abutment and first pier of the railroad bridge so that settlements could be carefully observed and construction operations controlled to avoid difficulty.

Because the initial settlement readings were most important, two level runs a day were made. Shortly after construction was begun, rapid settlements were observed adjacent to the main line roadway embankment. Some heaving was also observed in the loop area. At this time the construction operations were halted and additional controls placed along the original ground surface on both sides of the main line fill. Thereafter, limited lifts of fill material were placed and their effect on the underlying compressible soils carefully observed prior to the placement of additional lifts.

Figure 12. Off-ramp slide failure.

The operations continued for the off-ramp according to plan until the elevation of the fill was within 2 ft of the profile grade line. At this time tension cracks were noted in the fill and a slide occurred toward the Patapsco River. The earth mass continued its movement for about 2 hr after which time it reached a stable condition. Figure 12 is a view of the failed embankment. Because of a forecasted rain storm, the General Consultant and the design engineer immediately decided to have the contractor grade the slide material to obtain proper drainage of the storm flow toward the river. The contractor was also requested to remove some of the undisturbed embankment to relieve the pressure on the underlying stratum which had sheared. This material was not wasted but utilized by the contractor in other areas which required borrow material.

After reviewing the original plan and profile for the off-ramp, it was the opinion of the engineers that a realignment of the ramp within the standards set forth in the design criteria of the project would allow a safe and economical rehabilitation of the area. This was done as shown in Figure 11. Certain advantages were gained from this design change. The horizontal realignment placed the roadway and shoulders on embankment not influenced by failure planes, and the lowering of the grade line made it possible to consider as surcharge, soils which initially were below grade.

Considering the fact that some heaving was experienced in the off-ramp loop area and the initial settlements were appreciable, it was de-

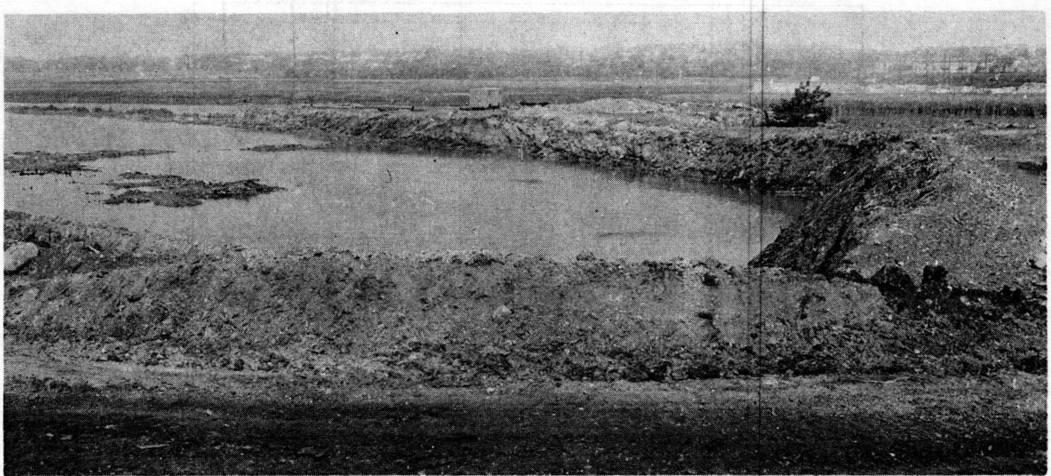


Figure 13. Station 360—area of unsuitable material.

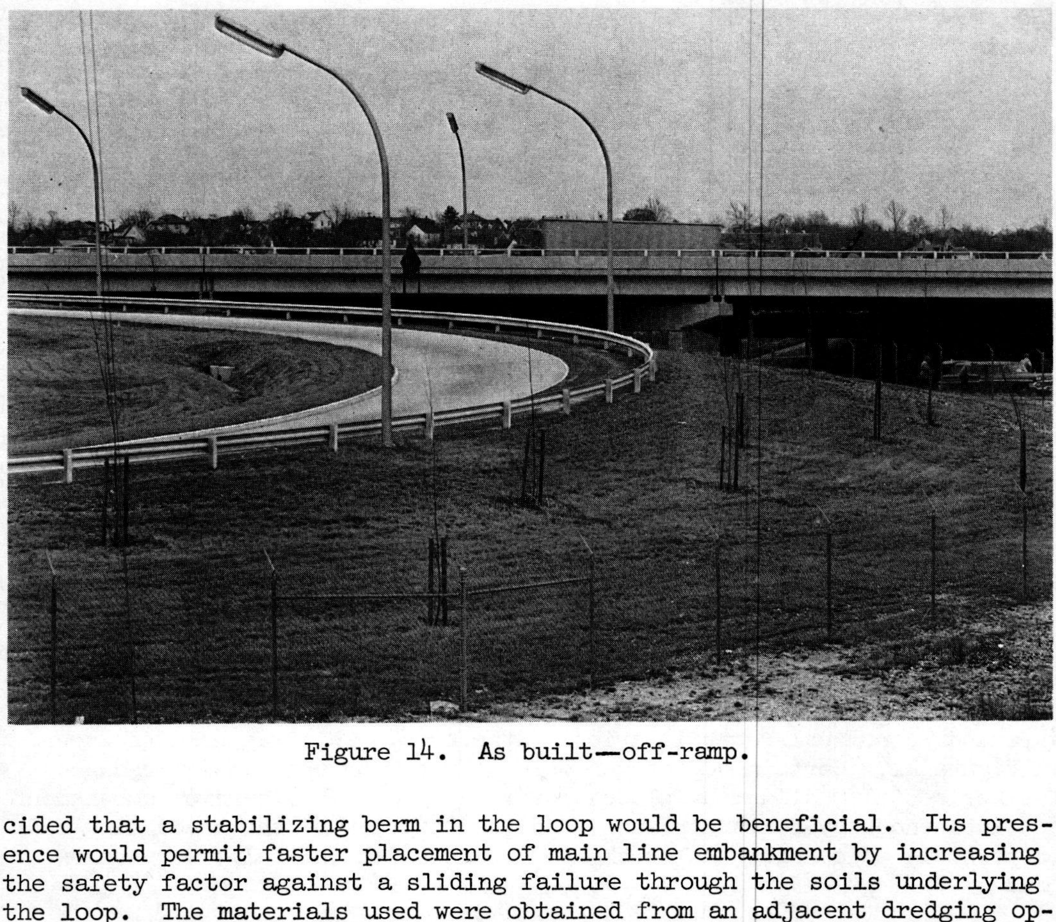


Figure 14. As built—off-ramp.

cided that a stabilizing berm in the loop would be beneficial. Its presence would permit faster placement of main line embankment by increasing the safety factor against a sliding failure through the soils underlying the loop. The materials used were obtained from an adjacent dredging operation as shown in Figure 13. Figure 14 shows the loop area as it appears at present.

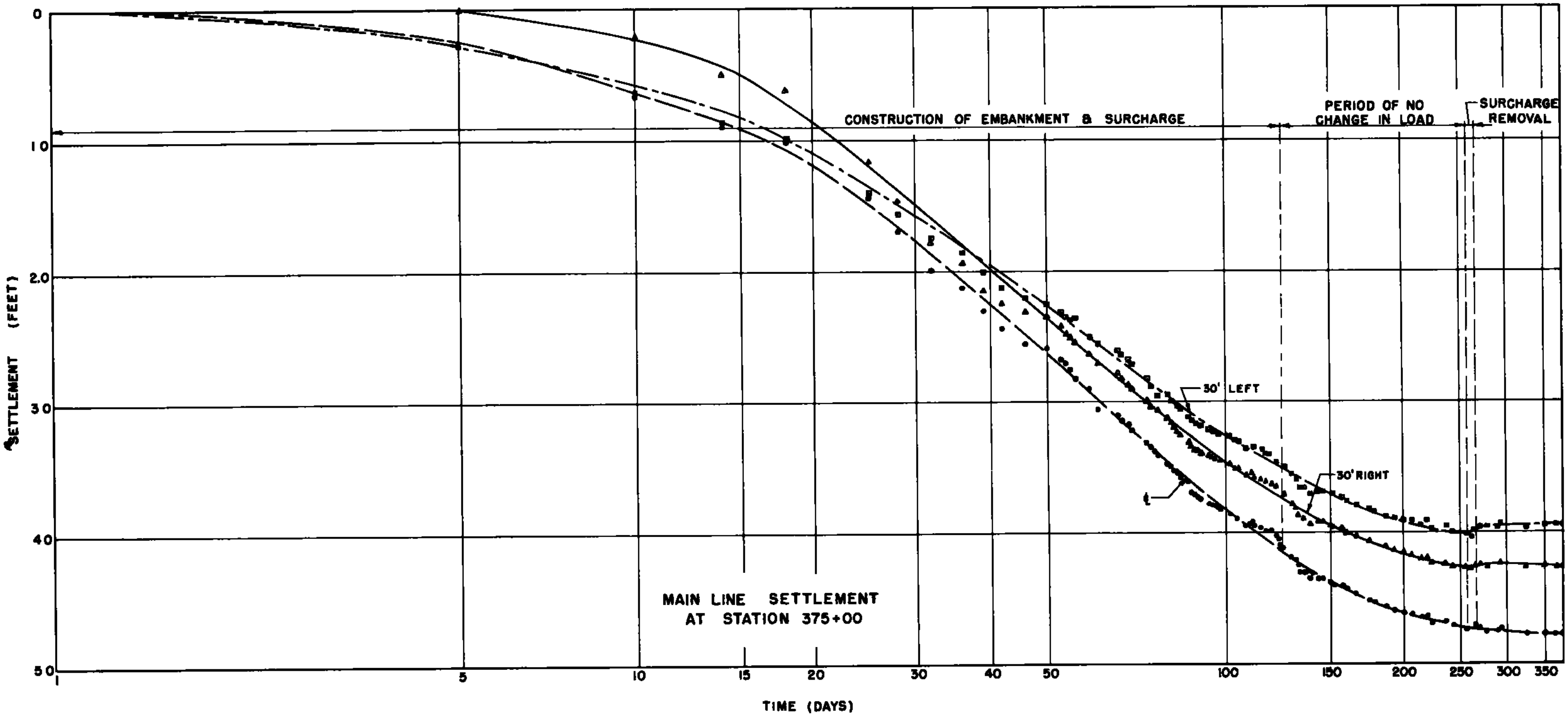


Figure 15. Station 325—main line—settlement curves.

Figure 16. Station 325—embankment and surcharge.

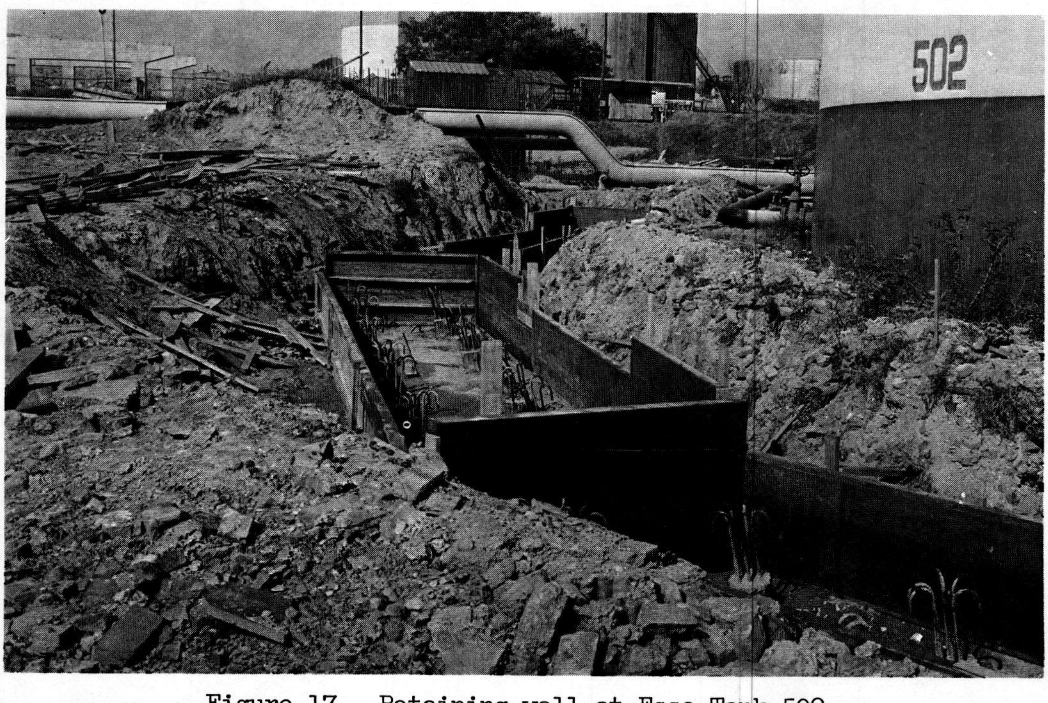


Figure 17. Retaining wall at Esso Tank 502.

The fill placement along the expressway was watched very carefully after the difficulties encountered in the area. It was decided on the basis of field observations that 13 ft of surcharge would be adequate to obtain the desired results rather than the 20 ft originally thought necessary. No further difficulties were experienced during or after the construction at this location. Figure 15 shows the curves obtained from the readings made on three settlement plates during the construction of the embankment and surcharge, the period of no change in surcharge, and the removal of surcharge. The embankment and surcharge are depicted in Figure 16. From field results obtained within the construction schedule, it was evident that rigid pavement could be used in conformance with the standard pavement section for the project.

The final subject to be discussed is a miscellaneous fill near the Esso tank farm. The problem here arose during the excavation for a retaining wall which was to contain the fill around Tank 502. This wall, which attained a maximum height of 11 ft, was designed for a spread-footing foundation, but the excavation for the footing revealed the presence of unsuitable fill material.

Cast-in-place concrete piles with an average depth of 25 ft were required to support the wall after this disclosure. It should be noted that Tank 502 had experienced some differential settlements in the past and the location of the retaining wall around this rank required, therefore, that all precautions be taken so as not to aggravate this situation. This is shown in Figure 17. An additional boring study of the site was made which disclosed that the southbound lanes passed over satisfactory soils while the northbound lanes were over unsuitable materials. The presence of bedsprings and brickbats combined with soft clays made the sampling operations quite difficult and it was decided that actual removal of these materials should be undertaken. With careful and continuous field supervision, the entire area of unsuitable soil which was found to be of limited extent, was removed by bulldozer and dragline. The removal of this material was most important because the fill over this area was approximately 20 ft in height and differential movements would have been quite serious.

MAINTENANCE AND OPERATIONS

The maintenance and operations phase of a large facility such as the Patapsco Tunnel Project normally does not gain full significance until a few years after its opening. Thereafter, the costs of maintaining and operating the project are expected to show an increasing trend.

In a strict sense, the problem mentioned in this section is incorrectly categorized because only recurring difficulties from wear are usually considered as maintenance and operations. However, this particular problem did occur after the roadway was opened to traffic.

During the fall and winter of 1957 an abnormal amount of rain and snow fell in a relatively short period of time. The unusually wet conditions caused a number of surface slides in cut sections and fill sections over 10 ft in height which were on a 1 on 2 side slope. The grass had not had sufficient time to establish its root system and consequently could not hold the 4-in. thick seeded topsoil layer to the side slopes. At one area located near the top of a cut, special drainage measures were necessary to control the flow of surface water.

At a number of other locations of distress, it was found that the topsoil was placed to greater depths than required by the plans and specifications. The general type of failure was the sliding of topsoil at the interface of topsoil and underlying fill. The basic problem, which has yet to be resolved, is a method whereby topsoil can be placed so as to become an integral part of the over-all fill.

SUMMARY

In October 1954, J. E. Greiner Company submitted a civil engineering report to the State Roads Commission of Maryland describing the Patapsco Tunnel Project with an estimate of project costs totaling $130,000,000. Construction was started in April 1955, and the tunnel and its approaches were opened to traffic in November 1957. The approximate project cost for this was $127,000,000.

J. E. Greiner Company of Baltimore, Maryland, was the General Consultant for the State Roads Commission and in this capacity was responsible for the preparation of the civil engineering report and the over-all supervision of the design and construction phases of the project. Eight design engineering firms were retained by the State Roads Commission to perform the actual design and field supervision of construction for the various sections of the work. A list of these design firms follows:

Design Section	Design Engineering Firm	Design Section	Design Engineering Firm
1	Green Associates, Inc. Baltimore, Md.	4	Singstad and Baillie New York, N. Y.
2A	Louis Berger & Associates Orange, N. J.	5	Joseph K. Knoerle & Associates, Inc. Baltimore, Md.
2B	Clarkson Engineering Co. Boston, Mass.	6	William H. MacFarland Binghamton, N. Y.
3	Whitman, Requardt, and Associates Baltimore, Md.	7	Rummel, Klepper and Kahl Baltimore, Md.

The design of the project in troublesome areas was based upon information available from borings, laboratory tests, and field tests. Certain areas were designated for elaborate field controls and careful field observations. However, all of the problems encountered during construction could not be anticipated at the time of design. The final treatment in certain areas was the result of field observations during the construction when some subsurface conditions became more clearly defined.

The importance of the revisions due to the field observations cannot be overemphasized as they affected the final costs and the construction schedule of the project. The results of these decisions stressed the importance and value of assigning competent field personnel to supervise the soils and foundation aspects of the work. Their presence assured a continuous reevaluation of the soil and foundation conditions as work proceeded.