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SYNOPSIS: The foundation failure of the 22 story, steel framed, federal courthouse in Philadelphia occurred because of an inadequate geotechnical assessment of a complex geological condition. The founding elevations for caissons were improperly determined on materials that could not sustain the design load. This condition was further complicated by the presence of groundwater and poor concrete construction practices. These conditions resulted in an extensive and costly remedial measures which included a grouting program and the replacement of 14 faulty caissons.

INTRODUCTION

This paper examines the foundation problems that developed during the construction of the James A. Byrne Federal Courthouse, a 22-story steel framed building in center city Philadelphia. The courthouse and accompanying federal office building occupy a 650 ft by 375 ft block between Sixth and Seventh Streets and bounded by Arch Street on the north and Market Street on the south.

The courthouse tower rises above a 240 ft by 270 ft 4-story podium on the southern portion of the block. The 10-story concrete office building occupies the northern portion of the block. The two structures share a common basement with an area of 320 ft by 630 ft at an elevation (above mean sea level) of about 14.0. The basement is approximately 15 ft below street level.

Both of these structures are supported by belled caissons. In particular, the courthouse tower is supported by 46 caissons which carry column loads ranging from 1200 tons to 2300 tons. The caissons were to be seated on sound bedrock (mica schist) with a bearing pressure of 40 TSF (tons per square foot). Lightly loaded columns on the periphery of the structure were to be supported on caissons with 8 TSF bearing pressures.

The original foundation report issued in 1965 recommended that steel H-piles be used to support the building. Caissons (and pressure injected footings) were listed as alternative foundations in the bid documents for subsurface work. The proposed founding grades for the caissons were based on limited core recovery data; however, the specifications detailed a procedure for verifying the adequacy of the founding material. In fact, the 40 TSF caissons were all founded at higher elevations than the proposed grades while the 8 TSF caissons were founded at lower elevations.

Settlement observations on 26 columns began in July 1970 when the first column sections were erected. Thirteen more columns were included in the program once the structural framing was completed in April, 1973. Readings on the remaining 7 columns began in January, 1974.

By late July, 1974 ten of the columns had settled at least 0.50 inches including one at 2.42 inches and another at 1.07 inches. Some cracking had occurred in the building and the structural design firm was concerned about secondary stresses developing from differential settlements.

The foundation required extensive repairs. The remedial measures, included the construction of replacement caissons, the grouting of caissons to repair defective concrete and underpinning caissons through grouting. The cost of the repairs (including damages) approached 6 million dollars (LePatner and Johnson, 1982).

SUBSURFACE CONDITIONS

Wissahickon Formation

The bedrock formation beneath the city is the Wissahickon Formation of Cambrian and Ordovician Age. The bedrock is a complex of schist and gneiss locally crosscut by granitic bodies. The schists are mica rich and are strongly foliated and fissile. Locally the schists are quartz and feldspar rich and are referred to as the "Wissahickon Gneiss" - a coarser grained rock, low in mica and thus less shistose and fissile than the typical mica-rich schists.

The Wissahickon Formation is characterized by extremely variable physical characteristics dependent upon the orientation of the steeply dipping rock beds which are crosscut by closely spaced, steeply dipping and open joints. This geometrical complexity is further complicated by the degree and nature of the weathering of the upper surface of the Wissahickon Formation. Unweathered sound Wissahickon may have an unconfined compressive strength of 200 TSF whereas highly weathered soft Wissahickon has a strength of 10 TSF or less.

The distinction between sound and unsound rock appears to have been originally based on the percentage of rock core recovered during drilling; sound rock had an average core recovery of over 90% while unsound rock ranged between 0 and 89% recovery. The borings were taken prior to the general use of rock quality designation (RQD).

This upper weathered surface has a secondary porosity and permeability (ranging up to 3 ft per day) through the weathered open joints and rock cleavage. This forms a groundwater reservoir capable of yielding a median flow of 20 gallons per minute (GPM) (Geyer and Wilshusen, 1982).

The surface of the Wissahickon Formation at the site lies at elevation between -62 and -83, the average elevation is -70. This upper rock surface has a tendency to disintegrate when exposed to moisture over a short period of time.

The Wissahickon Formation grades upward from sound rock, to slightly weathered medium-hard rock, to highly weathered soft rock into entirely decomposed rock (saprolite).

Wissahickon Formation - Saprolite

Overlying the severely weathered and eroded rock surface of the Wissahickon Formation is a layer of saprolite, usually referred to as "decomposed mica schist." The saprolite was formed in place by severe chemical weathering on the exposed Wissahickon Formation surface both prior to (by circulating groundwater) and after the deposition of the overlying Cretaceous rocks.

The saprolite beneath the site is a soft, friable, silty, sandy, thoroughly decomposed mica schist characterized by the preservation of the geologic structures and texture of the unweathered Wissahickon Formation. The saprolite beneath center city Philadelphia ranges in thickness between <10 and 70 ft. The saprolite at this site is encountered at an average elevation of -31 and ranges in thickness between 20 and 65 ft with an average of 40 ft.

The standard penetration resistance of the saprolite averages 125 blows per foot. The saprolite is less dense and more moist than the parent rock. The average saprolite moisture content is 17.5%, with a dry density of 115 PCF (pounds per cubic foot). Sound mica schist has a moisture content of 1.5%, and a dry density of 165 PCF.

Potomac-Raritan-Magothy Sequence

The Wissahickon Formation and its overlying saprolite are covered unconformably by the unconsolidated alluvial deposits of the Potomac-Raritan-Magothy rock sequence of Cretaceous Age. These stream channel and flood plain deposits consist of interbedded clays, silts, sands and gravels. At the site this sequence is found beneath the southern half of the site (beneath the courthouse) where it ranges in thickness between 4 and 17 ft, with its upper surface between -20 and -33 in elevation. The deposit consists of a lower dense to very dense gravelly sand bed (with an average standard penetration resistance of 123 blows per foot) and an upper medium dense silty sand bed. The Potomac-Raritan-Magothy sands and gravels form artesian aquifers beneath center city. The yield of this aquifer in the vicinity of the courthouse seldom exceeds 400 GPM (Greenman, et al., 1961).

Trenton Gravel

The surface of downtown Philadelphia is formed by the Trenton Gravel which completely masks the underlying alluvial deposits, saprolite and bedrock. The Trenton Gravel is composed of highly weathered gray to brown dense gravelly sands and sandy gravels with interbeds of cross bedded loose sand and silt and occasional boulders. These beds attain an average thickness of approximately 30 ft at the site.

The porosity and permeability of the Trenton Gravel deposits are both very high and may yield groundwater at rates over 1000 GPM. The water table lies within these deposits between elevations of -2 and -10.

FOUNDATION CONDITIONS

Construction Procedures

The 40 TSF caissons had shaft diameters ranging from 30 inches to 90 inches. The caissons had 60° bells so that the diameter of the bearing area ranged from 36 inches to 120 inches. The caissons were supposed to have permanent steel liners extending from the top of the bell to the top of the shaft. In many cases the liner could not be advanced to its design depth and ended as much as 20 ft above the top of the bell. The design strength of the concrete was 4000 pounds per square inch. The concrete was poured through a tremie pipe so that the free fall was limited to 10 ft. Water inflow was controlled by pumps until concrete placement. The CE noted that during the concrete placement some caissons had water rising to the top of, and flowing out of, the concrete. The caissons were essentially unreinforced - 6 ft long cages were used to tie the shafts to the caps.

Settlement

Early in 1973 engineers noted that many. columns in the courthouse were settling. By 1974 settlement measurements ranged from 0.09 to 2.42 inches.

The caisson (G-9) beneath the column that had subsided 2.42 inches was jacked up and shimmed with steel plates while the surrounding rock around the caisson was grouted.

Susbsequent studies involved the drilling of 112 test borings 39 borings adjacent to the caissons, 30 borings in the center of each four column bay, and 43 borings through the caissons. These test borings indicated that the caissons were poorly constructed and not founded on sound rock (Table I).

TABLE I. Material Beneath Caissons

MATERIAL FOUNDED ON	NUMBER OF CAISSONS
Saprolite to soft rock	31
Soft to medium hard rock	15
Sound rock	0

The caisson concrete was found to contain many voids and water bubbles and in rather poor general condition. Table II summarizes the concrete defects found in borings cored through 38 of the courthouse caissons.

TABLE II. Physical Condition of Caissons

CAISSON CONDITION		NUMBER OF CAISSONS
Honeycombed		8
Mortar segregated		4
Cold joints		2
Poor concrete		18
Aggregate segregation	(<6")	3
	(>6")	4
Mica schist inclusions		10
No recovery.		7

A plan of action was outlined in early 1975 (Table III).

TABLE III. Caisson Remedial Action

RECOMMENDED ACTION	NUMBER OF CAISSONS COURTHOUSE
No action - except monitoring	14
Repair concrete	15
Relevel columns	10
Replace caissons	16

Fourteen of the courthouse caissons were replaced by the installation of a pair of 24 inch diameter caissons on either side (2 feet from) of the existing caisson into sound rock - an average of 18 ft below the original caisson. The length to diameter ratio of the rock sockets is over 10.

CONDITIONS AND CAUSES OF FAILURE

Specifications for the Determination of Caisson Elevations

The construction contract detailed a procedure for verifying the adequacy of the founding material for the 40 TSF caissons and the 8 TSF caissons. The courthouse/office complex had a total of 186 40 TSF and 124 8 TSF caissons. The direction of the testing, and approval of the tests was assigned to the Construction Engineer (CE).

<u>Forty TSF Caissons</u>: The use of 40 TSF bearing capacity for <u>sound</u> mica schist has become common practice in Philadelphia. The specifications at the site used an unconfined compressive strength of 160 TSF as the criterion for distinguishing sound rock. The specifications did not, however, require a laboratory test at each each caisson. They required the contractor to take rock samples from the bottom of 10 caissons selected by the CE. The samples could be either cored (2 inch diameter by 4 inch long) or cut (2 inch square by 4 inch long). The samples were to be properly oriented geologically and tested in unconfined compression. The strength of each sample was compared to the penetration rate of 1 inch diameter, 90 pound pneumatic drill at the same caisson. The purpose of the program was to establish a maximum penetration rate for 160 TSF material. The maximum penetration rate, i.e., minimum time to drill 5 ft, served as a criterion to evaluate the founding grade of caissons which had no strength tests.

<u>Eight TSF Caissons</u>: The specifications required that the founding stratum for 8 TSF caissons have a standard penetration resistance of at least 150 blows per foot. These caissons were generally founded in dense gravelly sands.

Test Borings

The difficulties at Caisson G-9 appear to define the entire problem and resulting failure. One of the original 47 test borings (38-B) was taken at the site of G-9. Although the G-9 was supposed to be founded on sound rock at an elevation of about -70 ft, boring 33-B was completed at an elevation of -40.9 ft. In fact only 17 of the original 47 test borings encountered rock. Therefore the caisson foundations had to be tested.

Twenty seven of the original borings were bottomed at depths of exactly 70, 71, or 71.5 ft - well above the rock and in two cases above the saprolite. The remainder of the borings were between 100 and 115 ft in depth - with two exceptions one to 97 and the other to 123 ft.

Two old test borings adjacent to the site are reported in the literature (Greenman, et. al., 1961) one to a depth of 71 ft at Sixth and Market the other to 72 ft at 5th and Market. Both report 'mica rock' at an elevation of -50 (actually saprolite). This may be the precedent for the depth of the borings.

The depths of the test borings, and the pattern of drill sites appear to be geotechnically illogical. An additional 112 test borings were necessary to correct the lack of information provided by the original borings.

Determination of Caisson Elevations

Obviously the cause of settlement of the courthouse was the failure to found the caissons on sound rock.

Admittedly the mica schist of the Wissahickon Formation is problematical because of the highly irregular contact between it and the overlying saprolite (from -62 to -82 elevation on the site). The very irregular vertical gradation in this 20 foot interval from completely weathered mica schist (saprolite) through varying degrees of rock weathering (high-moderate-slight) and rock durability (sound-hard- medium hard-soft) further complicates the problem.

The rock itself changes rapidly in geological character and hence in engineering character both vertically and laterally. An increase in quartz content in mica schists increases rock strength and durability - while an increase in mica decreases the rock strength and forms inclined weakness planes within the rock mass. This micaceous foliation constantly changes its orientation - changing the orientation of weakness planes and thus changing the rock strength and relative stiffness.

Many of these rock characteristics can be determined and evaluated by careful visual inspection of rock cores - particularly if the RQD is used in their evaluation. The RQD was not developed in 1965 when the original test borings were taken. The rock quality was roughly estimated by the percentage of core recovery. In general the higher the percentage of rock recovery and the longer and more intact the pieces of core recovered the higher the quality of the rock.

The system used at the courthouse to evaluate the soundness of rock by drilling time rates is theoretically useful. If the laboratory tests performed on the samples from the 10 selected caissons are accurate and representative of the rock mass as a whole and the rock mass is reasonably uniform and the drilling time rates are accurately measured the evaluation system is relatively useful. None of these premises are met.

Later legal investigations indicate that the laboratory tests were performed on 2 inch cubes rather than specimens 2 inch square 4 inches deep. This of course gave false readings of rock strength because as the aspect ratio (length/diameter) decreases the apparent compressive strength increases. Furthermore, it appears that very little attention if any was paid to the proper orientation of the foliation in the test samples. Construction records indicate that the penetration rates used to verify sound rock were remarkably similar - a rather interesting outcome considering the variability of the founding material (Table I).

Because the top weathered surface of the mica schist and the overlying saprolite slake rapidly when wet and because they lie beneath the water table and an artesian aquifer the presence of water in the caisson excavations probably caused serious deterioration of the mica schist. In fact the CE admitted there was difficulty keeping the excavations dry and that 166 of the caissons were filled with concrete before testing information was delivered to him. The basis for pouring the caissons was the drilling time rates. Additionally the caisson inspector only inspected about 25% of the caissons because he was busy conducting and recording the drilling time rates.

CONCLUSIONS

The original subsurface investigation appears not to have thoroughly tested or interpreted the geology and engineering characteristics beneath the site. The important factors that should have been verified were;

1. the presence, amount and affect of groundwater
2. the elevation of the top surface of sound rock.

The plan for establishing drilling time rates to determine the unconfined compressive strength to establish sound rock was not workable.

The slaking effect of groundwater on the soft, friable saprolite and highly weathered mica schist was not appreciated.

The caissons were poured hastily to avoid ground-water problems and in the process poor concrete conditions contributed to the settlement.

Decisions were not made on the basis of sound geotechnical information by persons with little or no geotechnical education or experience.

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