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THE NEW TECHNOLOGY [1]

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SYNOPSIS

In June of 1916, the Massachusetts Institute of Technology (MIT) moved its campus from Back Bay in Boston, across the Charles River to Cambridge. The founding of the New Technology, as the Cambridge campus was known, served as a laboratory for the development of a new technology called soil mechanics.

Although foundations which support the New Technology were designed and constructed in accordance with the state-of-the-art at the time, the buildings settled at an alarming rate for the first 10 to 15 years, prompting stories that MIT students would some day be entering the buildings on the second floor.

In 1925, President Samuel Stratton invited an Austrian engineer named Karl Terzaghi to lecture at MIT and to apply his knowledge of soil mechanics to an evaluation of the subsidence.

With Dr. Terzaghi's arrival at the Institute, MIT became a center for the development of the science of soil mechanics, which is the foundation for the practice of geotechnical engineering.

Many outstanding and dedicated alumni of MIT played important roles in the founding of the New Technology, but the real hero in this case study of the evolution of a new technology, is a remarkable alumnus of the Department of Civil Engineering, John R. Freeman, '76.

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1 This paper was presented at a Symposium entitled the "Past, Present and Future of Geotechnical Engineering" held at the Massachusetts Institute of Technology, September 24-25, 1981. Proceedings of that conference are available through the Constructed Facilities Division of the MIT Department of Civil Engineering. The authors acknowledge the generosity of MIT for permitting publication of this paper in the Journal of the Boston Society of Civil Engineers Section, ASCE.
Also presented at a 22 April 1982 meeting of the Geotechnical Group of The Boston Society of Civil Engineers Section, ASCE.

2 Respectively, President and Staff Engineer, Haley & Aldrich, Inc. Cambridge, Massachusetts.

BOSTON TECH

The Massachusetts Institute of Technology, commonly called Boston Tech for 50 years, was founded by William Barton Rogers, a geologist. First classes were held on February 20, 1865. Following a year in crowded classrooms rented at the Mercantile Library Building, located at the corner of Summer and Hawley Streets in downtown Boston, students occupied the new Rogers Building on Boylston Street. The tuition fee was set at $100 per year for first year students.

The Rogers Building was joined in 1883 by the Walker Building and together they served as the center of the campus until 1916.*

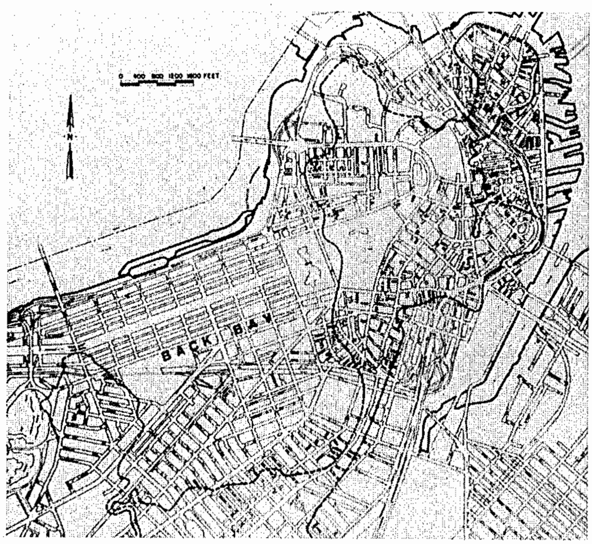

Figure 1. Boston's colonial shoreline superimposed on a modern map.

*Robert H. Richards, a member of the first graduating class in 1868, founded the Alumni Association in 1875 and served as its first president. In addition, he and other young civil engineering graduates from MIT met in May 1873 "to form a junior engineers' association", the members of which were elected to the then dormant Boston Society of Civil Engineers founded in 1848. The BSCE, now the Boston Society of Civil Engineers Section of the ASCE, continues today as the oldest engineering society in America.

The Boylston Street site for Boston Tech was located in the Back Bay, a former tidal embayment of the Charles River, Figure 1. Colonial Boston was a hilly peninsula connected to the mainland by a narrow causeway which is now Washington Street. Major filling of the Back Bay mud flats began in 1858 and construction of buildings followed close behind.

On April 10, 1861, an act chartering MIT and granting the Institute two-thirds of a block of land between Berkeley and Clarendon Streets was signed by Governor Andrew of Massachusetts. The remaining one-third of the block was granted to the Boston Society of Natural History.

A diorama of the site, Figure 2, prepared for the New England Mutual Life Insurance Company which now occupies the Boylston Street site of MIT, shows construction of the Rogers Building in about 1864. To the east is the completed Boston Society of Natural History (now housing the Bonwit Teller store). Back Bay filling is in progress west of the site and row houses on Beacon Street have been completed to about Dartmouth Street. The future site of MIT across the Charles River in Cambridge appears at the top of the diorama.

Figure 2. Diorama looking northwest across the Back Bay filling towards Cambridge. The Rogers Building is shown under construction in the foreground.

Following the Civil War, enrollment at MIT grew to 348 students in 1872-73. In that year, tuition was raised from \$150 to \$200 where it remained for 30 years. During the first decade, a third of the graduates were civil engineers.

By the turn of the century, enrollment had grown to 1300, the campus had scattered to other locations and the Institute was in dire need of additional space and the financial resources to relocate.

A 1904 map of the campus, Figure 3, shows the Rogers and Walker Buildings as well as other engineering buildings and the Lowell School located on land now occupied by the John Hancock Tower, the YWCA, the Massachusetts Turnpike Extension and the John Hancock Garage. The mechanical engineering laboratory appears in the left corner of the figure. A gymnasium was located behind the Lenox Hotel. The Institute also had an athletic field in Brookline.

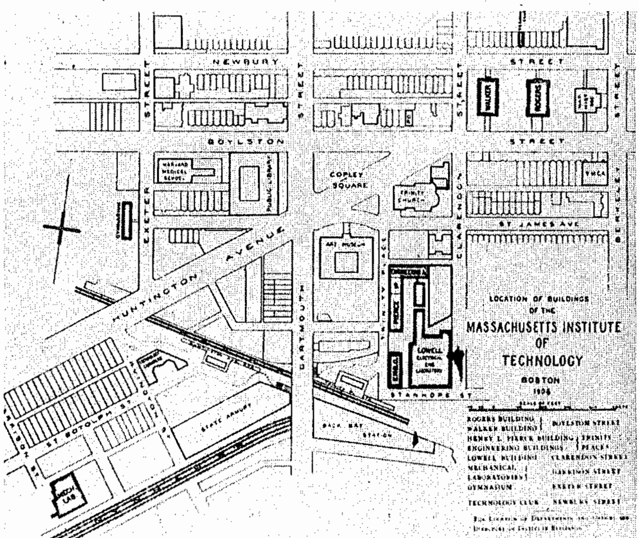

Figure 3. 1904 Map of MIT's Boston Campus.

There was a serious proposal to merge with Harvard in 1905. The faculty and the angry alumni voted against the plan, and the alumni pressured for the resignation of President Henry Pritchett who favored the merger.

Nevertheless, the Corporation voted in favor of a merger on condition the Back Bay land and buildings be sold to pay for new buildings. The Supreme Judicial Court held that MIT could not sell the land it had been granted, so MIT had no dowry and the marriage with Harvard University was called off.

At the end of 1905, Pritchett did resign to become president of the Carnegie Foundation for the Advancement of Teaching.

SITE SELECTION FOR THE NEW TECHNOLOGY

In 1906, a site committee was appointed by the Corporation to study the possible alternatives to solving the need for more land. At various times in the next five years, MIT considered a tract of land in the Fenway, a golf club in Allston near the Boston University Bridge and a site in Jamaica Plain. Several suggestions were proposed that an island be built in the Charles River.

The site that appealed most to President Richard C. Maclaurin, Figure 4, who had become the Institute's sixth president in 1909, was an area of filled land along the Charles River in Cambridge, Figure 5. When Maclaurin visited Boston in April of that year, his host was Charles A. Stone, '88. Stone pointed out the site from his windows on the water side of Beacon Street, but counseled Maclaurin that Harvard University and the City of Cambridge would be opposed to the purchase because of the great amount of tax-exempt property already held by Harvard.

In January 1911, a delegation of MIT alumni from Springfield called on President Maclaurin and offered Technology a site of 30 acres bordering the Connecticut River and overlooking the valley and the Berkshires. When news of this proposal appeared in the local press, Cambridge citizens suddenly began petitioning for the Institute to move to Cambridge. After the Cambridge City Council formally approved a move, MIT began negotiations to purchase the land.

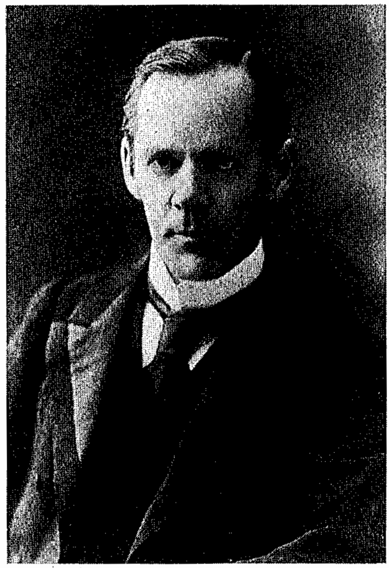

Figure 4. MIT President
Richard Maclaurin.

Figure 5. MIT's Cambridge site before completion of site filling. The new Harvard Bridge is shown.

Later in 1911, MIT had completed negotiations with 35 property owners to sell about 46 acres of land east of Massachusetts Avenue for the New Technology, Figure 6. T. Coleman duPont, '84, president of E. I. duPont, had contributed $500,000 toward the final purchase price of $775,000.

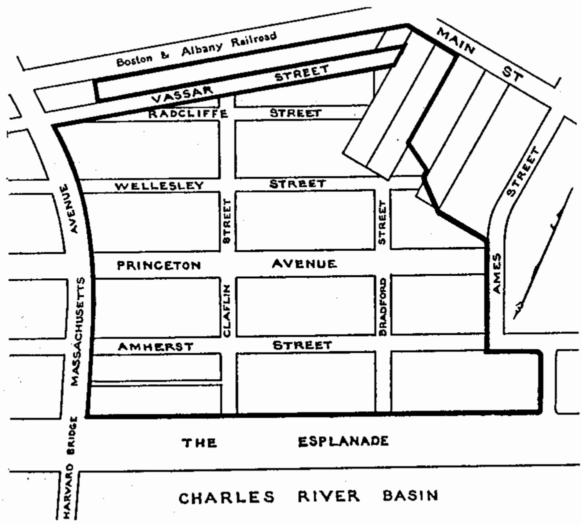

Figure 6 Map of property purchased for the New Technology.

In October of 1911, President Maclaurin boldly announced that construction would begin the following year, although at the time, he had no idea where funds for the building would be found.*

*When Boston alumni met in January 1912, Maclaurin announced alumni gifts making possible the opening of a Civil Engineering Summer School at E. Machias, ME. H. Farwell Bemis, '93, had given a 1,000-acre lot and supplied equipment for the camp. Charles W. Eaton, '85, gave $10,000 for the construction of buildings. The camp opened the following year.

On February 22, 1912, Frank Lovejoy, '94, general manager of Eastman Kodak Company, wrote to Maclaurin saying that George Eastman "would be inclined to help out". Subsequently, Maclaurin met with Eastman and he anonymously gave $2,500,000 for construction of the New Technology. Eastman was known as the mysterious "Mr. Smith" until his identity was disclosed in 1920.

TOPOGRAPHIC DEVELOPMENT OF THE SITE

The site for the New Technology had been part of the tidal estuary of the Charles River, land partially filled by dredging from the riverbed. In fact, in colonial times the entire contemporary campus bounded by Vassar Street, Main Street and Memorial Drive was primarily tidal mud flats with some bordering salt marsh, Figure 7.

Figure 7. Map showing development of Cambridge shoreline from 1777 to the present.

The proposed site for the new buildings was crossed by a tidal stream the thread of which passed through the Metropolitan Storage Warehouse and MIT's present Building 10 and Building 2. The location of this channel was to have a major impact on the construction and performance of foundations for the New Technology.

Following construction of the West Boston Bridge (site of the Longfellow Bridge) in 1793, the Kendall Square area was developed as a port with canals, wharves and warehouses which handled shipping off the Charles River.

In 1853, an embankment for the Grand Junction Railroad was constructed across the tide flats and marsh, parallel to which Vassar Street was constructed years later. Heavy industry developed around the future MIT site, stretching along the westerly side of the railroad and around Kendall Square.

In the early 1880's, the Charles River Embankment Company undertook to fill the land and construct a granite seawall between the West Boston (Longfellow) and Brookline Street (Boston University) Bridges. The company controlled a triangle of approximately 215 acres bounded by the railroad on the west, Main Street on the north and the Charles River. Promoters of the project, led by Charles Davenport, hoped to create a fashionable residential district comparable to the successful development in the Back Bay across the Charles.

Completion of the Harvard Bridge in 1890 and construction of Massachusetts Avenue along its present route east of Lafayette Square encouraged the developers. However, the Company collapsed in the panic of 1893, but not before organic soils, silt and sand were dredged hydraulically from the riverbed to partially fill the land which MIT would occupy.

With completion of the esplanade (Memorial Drive) by the Cambridge Park Department, and construction of the Charles River Dam in 1910, which stabilized water level in the Basin, one of Cambridge's most important topographic and environmental assets was assured. MIT was to become the beneficiary of that asset.

The Metropolitan Storage Warehouse was the first large building constructed on the filled land between the railroad and the Harvard Bridge. The front section was built in 1895 and the remainder some years later. The Riverbank Court Hotel (purchased in 1939 by MIT for a graduate student dormitory and later named Ashdown House) was a luxurious apartment hotel constructed in 1900 on the tip of Whittemore Point, a marshy point underlain by gravel. Shortly thereafter, in 1902, the City built an armory which is now the duPont Center Gymnasium.

East of the site for the New Technology, beyond Ames Street, the dominant structure was the International Shoe and Leather Exposition Building, a stucco-covered structure built in 1908 on the site of the present 100 Memorial Drive apartment house. Its domes foreshadowed MIT's domed classical style, Figure 8.

Figure 8. International Shoe and Leather Exposition Building.

Subsequent to the time MIT purchased the site, additional earth fill was placed. The hydraulic fill, predominantly organic silt with sand and often bearing shells, was supplemented by hauled sand and gravel and by miscellaneous fill including earth from subway construction. (The subway from Park Street to Harvard Square across the Longfellow Bridge was opened in 1912.)

GEOLOGY AND SUBSURFACE CONDITIONS

Soil conditions in Boston and Cambridge, and indeed the topography of the area in the seventeenth century, owe their origin to events which took place during the Pleistocene.

During this geologic period, there were successive advances and retreats of glacial ice followed by extreme variations in climate and sea level, all of which influenced the sediments and their engineering properties.

A generalized soil profile below the MIT campus is shown in Figure 9. Bedrock occurs at a depth of from 120 to 135 ft. below the main buildings. The rock is part of the Cambridge Slate formation which underlies most of the Boston peninsula and Cambridge. Often called Cambridge Argillite, it is a fine-grained clayey rock formed in the Permian-Carboniferous periods.

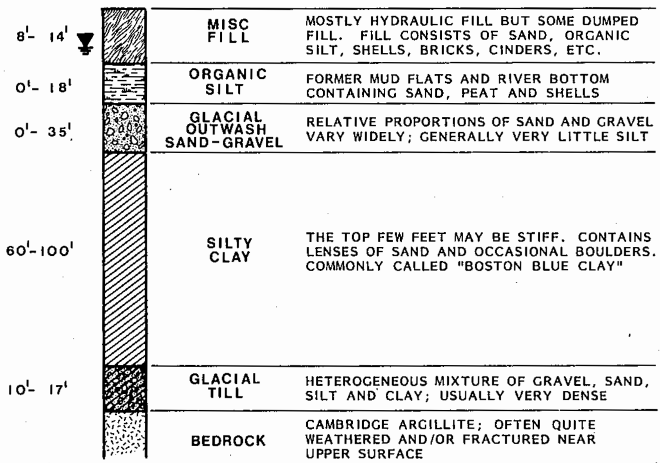

Figure 9. Generalized soil profile for the New Technology.

From 10 to 17 ft. of glacial till overlies the rock. The till, laid down at the base of glaciers during the Boston substage of the Wisconsin glacier, is a very compact heterogeneous mixture typically gray silty sandy gravel with cobbles and boulders.

A stratum of inorganic silty clay from 60 to 100 ft. thick and known locally as Boston Blue Clay, was formed when silt and clay-size particles were sorted from the till by glacial melt-water streams and deposited in quiet marine waters.

The clay, gray to olive-green in color, is generally medium to soft in consistency. The top of the stratum was subjected to desiccation and oxidation at a time when the sea was below its present level relative to the land, resulting in a stiff crust of yellowish or buff colored clay.

Some 12,000 to 14,000 years ago, following a readvance of glacial ice during the Lexington substage, medium compact to compact well-graded gravelly sand was deposited over the clay by fast moving streams. The sand stratum varies considerably in thickness, typically from 5 to 25 ft.

In recent times following the glacial age, plastic organic silt was deposited by river and tidal currents, and salt marsh peat accumulated in the tidal estuary of the Charles River due to the slowly rising sea. The topography of seventeenth century Cambridge was thus established.

The final episode in the development of subsurface deposits occurred during the late nineteenth and early twentieth centuries when man-made fills were placed over the organic soils to raise the grade a safe distance above water level.

With construction of the Charles River Dam in 1910, water level in the Charles River Basin was controlled at El. 13.2 ft., Cambridge City Base (El. 8.0, Boston City Base). Groundwater levels on campus are typically somewhat lower, from El. 10.5 to 12.

The organic soils, sand-gravel stratum and underlying clay bear especial significance to foundations for the New Technology.

JOHN R. FREEMAN'S PRELIMINARY DESIGN

In February of 1912, President Maclaurin turned to a civil engineering alumnus and member of the Corporation, John Ripley Freeman, '76, Figure 10, to study the engineering problems of building on the new site. As a loyal alumnus, Freeman was pleased to donate his time for the work.

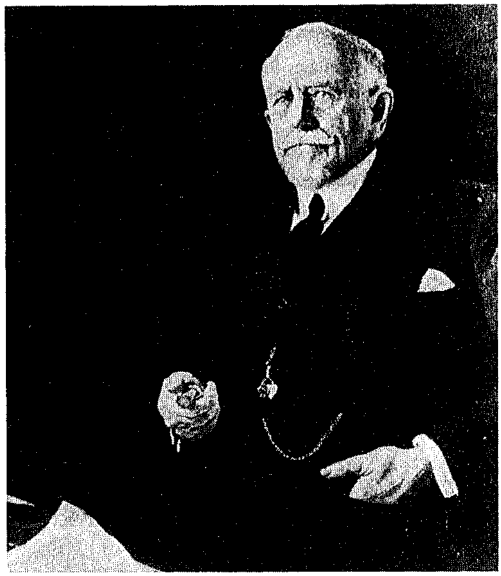

Figure 10. John Ripley Freeman.

Freeman was one of MIT's most distinguished alumni. He led a double life as businessman and engineer. He was President and Treasurer of the Manufacturer's Mutual Fire Insurance Company of Providence. This company later affiliated with others and, during his tenure, became the largest of its kind in the country.

In addition to his role as a business executive, Freeman was an eminent consulting engineer with a world-wide reputation. His specialty was large hydraulic projects.

His consulting projects included:

- Locks and dams for the Panama Canal
- Water supply systems for New York, Los Angeles, San Francisco, Baltimore, and Boston
- Regulation projects on the Great Lakes
- Water power development on the St. Lawrence River

Freeman also practiced structural and industrial engineering in association with repairs and redesign from his insurance work. He was very familiar with the state-of-the-art of industrial architecture, and had even studied the construction of large auditoriums and theaters.

He held honorary doctorates from five universities in this country and abroad. He had been president of the Alumni Association for two years, president of the Boston Society of Civil Engineers and president of the national ASCE and ASME.

Some years before, President Henry Pritchett of MIT made an "earnest suggestion" as to Freeman's "undertaking the management of this institution" (10). He declined, having "grave doubts about my possibilities in the dinner jacket end of such a job." (10).

Freeman was also one of the foremost experts in soils and foundation engineering. He was one of the first engineers to realize the importance of geology to civil engineering. He was the principal investigator for the Charles River Dam (at the present Museum of Science) and consequently was familiar with subsoils in the former Charles River estuary.

In all, John R. Freeman was one of the most qualified individuals for the job of planning the New Technology.

He initiated his study with characteristic thoroughness, dispatching teams of recent MIT engineering and architecture graduates equipped with notebooks and cameras to inspect the great colleges and technical schools in the United States, Canada and Europe.

He was convinced that the new campus presented an "opportunity for a vast improvement in the efficiency of college architecture" (5, p. 12). He believed that college buildings were usually of an inefficient monumental style in which the outward appearance, to please the benefactor, was the paramount consideration. The internal function of the buildings was generally considered secondary and sacrificed, if necessary. Freeman regarded the design of the New Technology as "one-fifth architecture and four-fifths a problem of industrial engineering" (5, p. 12).

The budget of $2.50 per square foot for 1,000,000 sq. ft. of floor area precluded consideration of a monumental style structure, which he felt was poorly suited for the Institute in any case.

Freeman proposed a structure with a classical facade comprised of narrow, interconnected wings enclosing interior courtyards, Figure 11. High ceilings and large windows were provided for light and ventilation. He provided an architectural layout and structural framing system that would allow for future expansion and renovation.

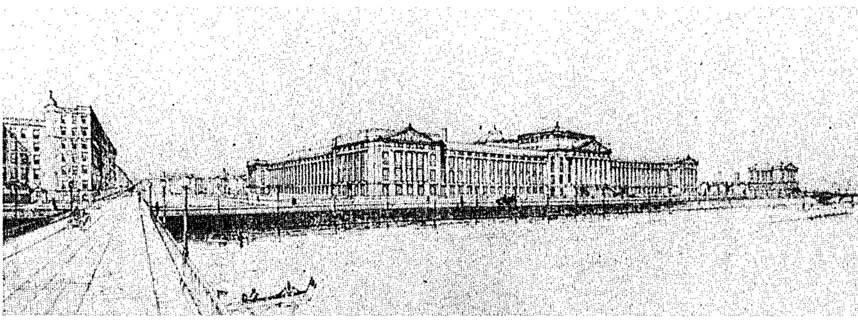

Figure 11. Freeman's proposed New Technology.

At one time, he considered moving the shell of the old Rogers Building to Cambridge for sentiment and for use as a technical museum. It was to be located at the site now occupied by the President's House.

Freeman's most radical proposal was the use of architectural concrete. Concrete had never been used for the exterior of college buildings. In fact, concrete technology was not available to provide architectural quality concrete. Freeman's opinion was "that it is highly probable that goodlooking, impervious durable concrete can be had if one will only study the subject faithfully, and will give a building at lower cost than any possible building material that would give equal architectural effects" (5, p. 34). He also proposed the use of precast concrete elements for most of the facade.

His preliminary study was accompanied by a companion report, prepared under his supervision, which detailed the subsidence of the land and existing buildings in the Cambridgeport area near the new campus. Data collected by Freeman indicated that ground surface in the area of recent landfill had settled two feet or more at some locations. In addition, some buildings had settled, notably the Metropolitan Storage Warehouse where settlement of 14 in. had been recorded during a period of 15 years.

Freeman identified the cause of the subsidence as the increased load on the substrata due to the building weight and the weight of 12 to 15 ft. of recent land fill. He concluded that the cause was "an extremely slow squeezing out of the water contained within the pore space of a deep and relatively soft bed of clay", Figure 12 (17, p. 1).

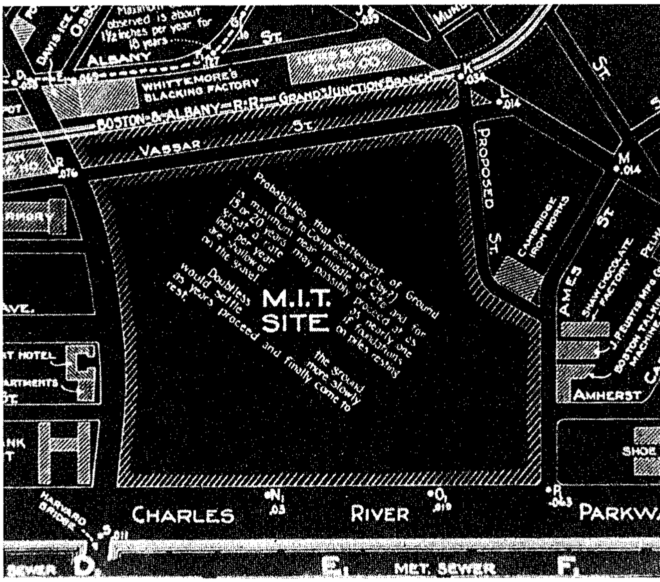

Figure 12. Figure from Freeman's report with note warning of probable settlements at the M.I.T. site.

Freeman made no mention of the possibility that compression of peat and organic silt under the weight of recent earth fill could be a contributing factor to settlement of streets and sewers. He attributed the great variations in observed settlements primarily to variations in the depth to bedrock.

Although he was unable to predict the magnitude of the settlement, he said, "Nor is there reason to expect that the settlement at any point on the Technology site may not be as great as that of the Metropolitan Storage Warehouse which has gone down at the rate of nearly 1 in. per year and which now stands 14 in. lower than when built". He also said, "obviously the motion should decrease from year to year or decade to decade and ultimately stop" (17, p. 2).

Freeman initiated a program of 22 deep test borings over the 46-acre site. He engaged his classmate and frequent collaborator, Professor William Otis Crosby, '76, to prepare an interpretative report of the geology and subsurface conditions. Both the conclusions and timing of Crosby's report would have serious consequences for the design of foundations for the New Technology.

Crosby disagreed with Freeman's conclusions regarding the cause of observed settlements in the neighborhood of the new campus. Crosby believed that the surface settlements were a result of densification of the recent fill and compression of the peat stratum. He attributed observed structural settlements to inadequate foundations rather than compression of the clay. Freeman noted these views in his preliminary study and referred to Professor Crosby's forthcoming report.

As a possible foundation system for the buildings, Freeman considered long concrete piles, bearing below the clay, but rejected the concept as uneconomical. He proposed instead to found the buildings on the sand-gravel outwash stratum. Spread footings dimensioned for a bearing pressure of 2.5 tons per square foot would be used where the top of the glacial outwash was at or above El. 0. Where the stratum was below El. 0, end-bearing timber piles would be used with a working load of approximately 8.5 tons per pile.

Freeman believed that the structures would experience settlement of a magnitude he could not predict. His solution was to stiffen the substructure to redistribute the building load as settlement occurred so as to avoid differential settlement. He termed the foundation "a floating foundation" and described it as follows:

> It is feasible to "float the entire group of buildings upon a foundation designed to withstand uplift like the bottom of a ship, and thus competent to support the entire weight of the structures, however soft the substrata might be" (5, p. 99).

Freeman proposed to support the columns on continuous foundation walls that were actually gigantic girders of reinforced concrete. The proposed girders beneath the longitudinal column lines ranged from 18 ft. to 26 ft. deep. They were intersected by 7-ft. deep girders 15 ft. on center, Figure 13.

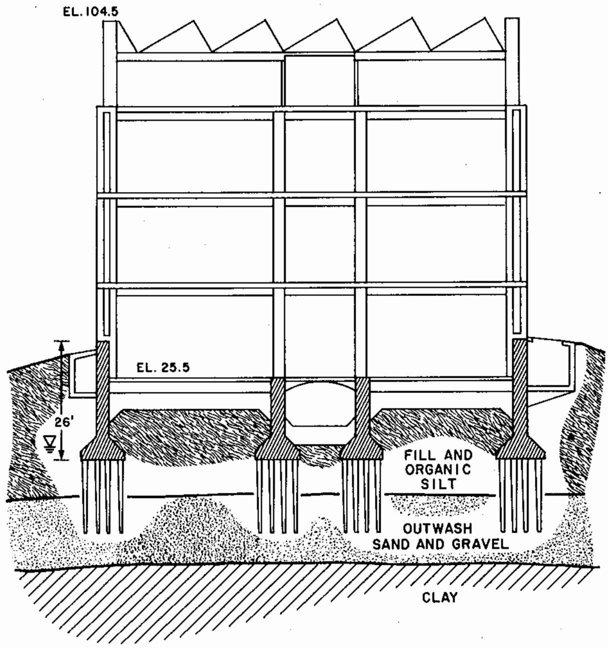

Figure 13. Cross section showing Freeman's proposed foundation. Cross hatching indicates longitudinal grade beams.

Freeman submitted his preliminary design report to the Corporation in January 1913 with an offer to prepare complete working drawings and supervise the construction.

The preliminary plans were exhaustive, as Freeman hoped they would be adopted as <u>the</u> plans for the New Technology. He provided complete architectural renderings of the exterior facade, the courtyards, the corridors, the classrooms, and even the professors' offices.

WILLIAM WELLS BOSWORTH SELECTED AS ARCHITECT

Freeman's plans for the New Technology were rejected by President Maclaurin because of Maclaurin's belief that the new Institute buildings had to be of monumental quality, in direct conflict with Freeman's design philosophy. Maclaurin was appalled at Freeman's recommendation that the facade be architectural concrete, stating that nothing less than stone would do.

On February 17, 1913, the Institute engaged an architect, William Wells Bosworth, '89, to prepare final plans under the review of the head of the Department of Architecture, James Knox Taylor, '79.

Bosworth was a relative unknown in the Boston area. He had practiced several years in New York and had acquired a reputation for skill in combining architecture and landscaping. Perhaps his most important credential was that he was John D. Rockefeller's private architect.

Freeman was furious with the choice of Bosworth. He had offered to supervise the preparation of working drawings and the construction, partly out of his enthusiasm for the project and partly due to his concern that "lack of economy would follow the adoption of ordinary architectural methods on these special problems" (11). In addition to his low opinion of architects as "beauty doctors", he considered Bosworth to be unqualified. Freeman was convinced that Bosworth's plans for the academic buildings would consume all of the available funds, leaving nothing to equip the laboratories or to build dormitories.

His fears concerning cost were well justified. In spite of Maclaurin's insistence that the cost of the buildings should not exceed $2,500,000, the monumental facade and other design changes resulted in significantly less floor area in the academic buildings than Freeman believed he could provide, no dormitories, and a cost close to double the budget.

In the months which followed, Freeman's input and influence in the architectural design and foundation engineering for the buildings would be removed, even though he attempted to intervene.

Bosworth's original plan for the New Technology, prepared in 1913, is shown in Figure 14. His design was described by Freeman as incorporating between a quarter and a half of the ideas which he, Freeman, had presented in his preliminary study.

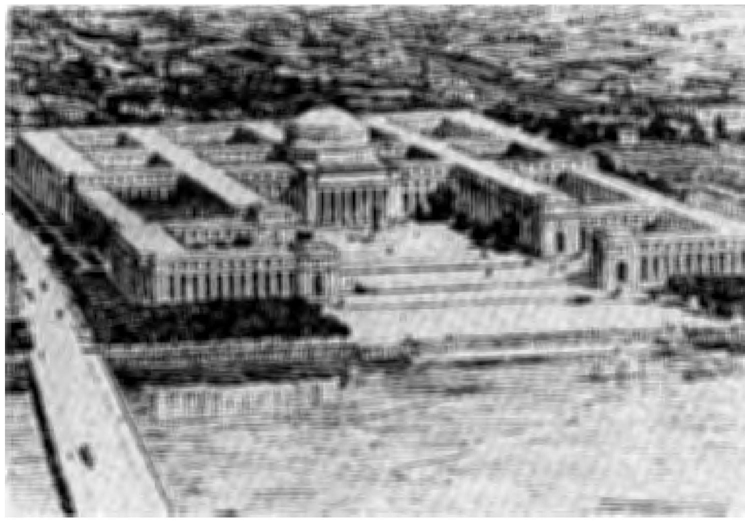

Figure 14. Bosworth's proposed New Technology.

Fundamental similarities between the two were:

° Interconnected buildings, primarily to encourage interdisciplinary work

° Standardized reinforced concrete frame with non-load bearing partitions to allow flexibility for expansion or renovations

The difference between the schemes was primarily architectural and included:

° Arrangement of the buildings to create the Great Court (now the Killian Court, named in honor of the Institute's tenth president, James R. Killian, '26).

- Indiana limestone for the facade, rather than architectural concrete made from crushed marble and white cement

- The Great Dome

The Great Dome was to have a profound effect because of the increased height and weight of Building 10, which would aggravate the settlement problem.

The prototype of Bosworth's plan, with its dome and pilastered wings enclosing a spacious open court, was Thomas Jefferson's rotunda and colonnades at the University of Virginia. It was a symbolically appropriate precedent because MIT's founder, William Barton Rogers, had come from that university.

WILLIAM O. CROSBY'S REPORT ON GEOLOGY

Professsor Crosby's report on the geology of the new site, which Freeman had referred to in his preliminary study, was submitted to President Maclaurin in July 1913. The report was accepted by the Institute as definitive.

William O. Crosby had been a professor of geology at MIT until he resigned in 1907 because of deafness. He was the first engineering geologist in the country and a leading authority on the geology of the Boston area. He and Freeman started a life-long friendship during their student days. Freeman engaged Crosby to perform geologic investigations on numerous dams and large hydraulic projects and had a high regard for his practical approach to engineering problems. Generally he placed ultimate confidence in Crosby's conclusions regarding geology. However, on this occasion they disagreed.

Crosby's interpretation of the geology of the site is essentially consistent with our current interpretation, with several significant exceptions.

In his description of the soil strata present at the site, Crosby made the following statement: "Although the drillmen make a distinction between 'soft' and 'stiff' blue clay, it is probable that in its normal condition in the ground, all of the clay is devoid of excess moisture and fairly to be described as stiff. In brief, the blue clay is undoubtedly a far more stable formation than it is commonly supposed to be." (3, p.7).

In essence, Crosby concluded that sample disturbance was responsible for the softer consistency of the clay samples from the lower region of the stratum. Crosby had held his opinion for some time. He had stated it ten years earlier in a paper on the geology of the Boston Basin.

"Superficially", Crosby stated, "the clay is usually more or less yellowed by oxidation, as the result of exposure to atmospheric influences before it subsided below the level of the sea..." (3, p.7). He was not aware, however, that desiccation had precompressed the clay to a considerable depth below its top surface, increasing its strength and reducing its compressibility.

The most curious interpretive error Crosby made was in a section of his report headlined "Stability of the Blue Clay" which reads, in part, as follows:

> "The tendency of the blue clay to yield under long-continued heavy pressure is, apparently, demonstrated by the fact that the upper surface of the clay, that is, the contact of the clay and overlying gravel, sinks as the gravel becomes thicker and rises as it becomes thinner, the clay seeming to have responded to the inequality of load. It is improbable, however, that this isostatic tendency to respond to variations of load, still exists to such a degree that the weight of the proposed buildings will cause further sensible yielding of the clay. We must suppose, rather, that, although the clay, as originally deposited, was supersaturated with water and hence soft and yielding (plastic), the long-continued pressure due to its own mass and the weight of the overlying gravel, sand and silt has squeezed the excess water out of the clay, and forced the clay and quartz particles into the closest possible relations, the remaining water acting as a cement instead of a lubricant, the mass as a whole becoming semi-dry, well compacted and relatively resistant." (3, p.11).

The logic expressed above is attractive. It was reasonable to believe that if the clay at the top of the deposit is hard, the clay on the bottom, under the added weight of the clay above, must be hard as well.

Having concluded that the clay stratum was unyielding, Crosby concluded that the surface settlements in the area of recent filling and the settlements of the neighboring structures documented by Freeman were due to densification of the recent fill, compression of peat and inadequate foundations.

Since no peat was found in any of the borings and glacial gravel was present over the clay, Crosby concluded that "... we are certainly justified in regarding the new site as virtually above suspicion." (3, p.15).

STONE & WEBSTER SELECTED AS ENGINEERS AND CONTRACTORS

On July 25, 1913, the Corporation voted to accept Bosworth's preliminary plans for the New Technology. During the same month, the Stone & Webster Engineering Corporation was selected to engineer the design and construct the buildings.

Charles A. Stone and Edwin S. Webster had been classmates at MIT and were two of the Institute's earliest graduates in electrical engineering. After receiving their degrees in 1888, they formed a partnership as consulting engineers in the then new field of electricity. At the time they were chosen as engineers for the building construction, their firm had grown to be one of the leading construction companies in the world, known for the efficiency and quality of its work.

Stone and Webster were both enthusiastic alumni and members of the Corporation. Webster had been president of the Alumni Association in 1909 and Stone would become president in 1916.

As the structural engineers, Stone & Webster was given responsibility for all of the underground work, including the design of the foundations. Stone & Webster in turn engaged Charles T. Main, '76, a classmate of Freeman's, as a consultant on the foundations.

Main was also an active alumnus and member of the Corporation. He had been president of the Alumni Association for two years at the turn of the century. In addition, he served as president of the Boston Society of Civil Engineers in 1911.

FOUNDATION DESIGN

Stone & Webster's and Charles T. Main's interpretation of the nature of the blue clay agreed with the geological report prepared by Professor Crosby. Their foundation system was intended to limit expected settlements to small values in contrast to Freeman's proposed "floating foundation" which was intended to prevent differential settlement that would lead to distortion of the building frame.

In a paper published in Technology Review in 1915 and in the Journal of the BSCE in January 1918, Charles T. Main stated:

> "It has been assumed.....that if settlement occurs, it will be in the clay, and in order to reduce this settlement to a minimum it was decided to spread the building loads over the glacial gravel as much as practicable which in turn would still further distribute the loads over the clay bed.
>
> In order to provide for a wide distribution of load on the gravel in the most uniform manner, with a resulting low pressure per square foot, it was decided to use a large number of wood piles, each sustaining a relatively small load, rather than a few heavily loaded piles." (16, p.7).

The load was assumed to be distributed at an angle of 60 degrees from horizontal from the elevation where the piles entered the glacial outwash stratum, resulting in an increase in load on the top of the clay of approximately 1500 lb. per sq. ft. which was well below the commonly used "safe bearing pressure".

The heavily loaded piles which Main referred to were Simplex and Raymond concrete piles, both installed to bear in the glacial outwash.

Pile cutoff was established at El. 13, with the bottom of the pile caps at El. 12.5. The piles were spaced between 2 ft. and 3 ft. on center. The deep girders of Freeman's design were not incorporated.

The design was very conventional, Figure 15. Timber piles bearing in the sand-gravel stratum overlying the clay supported most of the buildings neighboring the site in Cambridgeport as well as those in the Back Bay where the gravel occurs. Nevertheless, the engineering community was expressing concern, in addition to Freeman's warnings, that settlement of the clay was being observed at existing structures and must be considered in the foundation design.

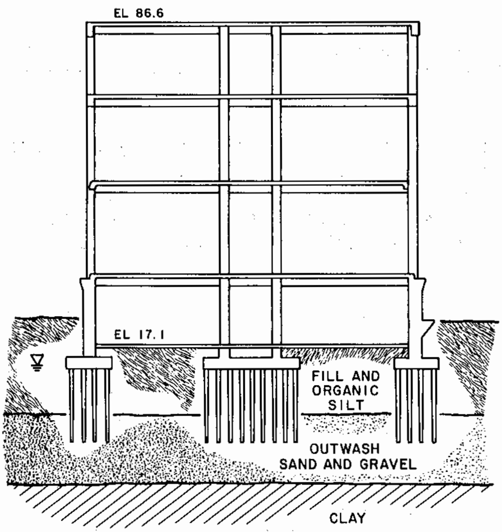

Figure 15. Cross section showing Main's foundation.

PILE TESTING PROGRAM

Although the Engineering News pile driving formula was in common use for calculating pile capacity, Charles T. Main decided to establish pile capacities on the basis of load tests. Load testing of piles was, apparently, a relatively recent innovation. Two plate bearing tests, also apparently rare, were performed to investigate the load-settlement characteristics of the glacial outwash and the clay.

The objectives of the field testing program were to:

- Verify Professor Crosby's conclusion that the clay would not settle significantly under load, and

- Compare the driving characteristics and load carrying ability of different species of timber piles at different locations over the site.

Two plate load tests, one on the glacial outwash and one on the clay, were performed by Charles Gow.

The test procedures were as follows:

° An excavation was made to the top of the glacial outwash and clay, 20 ft. and 30 ft., respectively, using a 3-ft. diameter steel casing.

° Load was transmitted to a 12-in. square cast iron bearing plate by a 10-in. square wood post. The post was surrounded by a wooden box to isolate it from friction. The annular space between the box and the steel casing was backfilled and the casing removed before the test was run.

° Load was applied by filling a box constructed on the post with concrete. Five tons per sq. ft. (tsf) was applied to the clay. Five tsf, later increased to 8.5 tsf, was applied to the outwash.

Gow reported the following results:

> "Owing, probably, to the imperfect bedding of the 12-in. cast-iron plates on the underlying soil, there was in the case of each test an initial reading of one inch of settlement when the load was applied, but no further settlement has occurred, although the load on the gravel was later materially increased and both tests have now been under observation for more than nine months."(14).

The difference between these tests and modern plate bearing tests was the time interval between the settlement readings. Initial elevations were measured before the load was applied. The next elevations were shot nine days following application of the load, missing the time dependent effects in the clay.

The plate load test suffered primarily from insufficient scale. The 1-ft. square loaded areas were inadequate to predict the influence of the proposed 60-ft. wide buildings on the clay stratum, even if the properties of the clay stratum were constant with depth. Although the designers assumed a stress distribution 60 degrees from the horizontal below the proposed pile groups, the limited zone of influence and the rapid attenuation of stress below the 1-ft. square plate were not appreciated.

A test pile program was conducted consisting of 81 timber piles, 10 concrete piles and 25 load tests. The tests were conducted to determine acceptable driving criteria for the different species of timber piles (spruce, southern pine, and oak) and to establish correlations for allowable pile load as a function of driving resistance.

The test pile driving did not go smoothly. Freeman's 22 preliminary borings were spaced across the entire 46-acre site. It soon became evident that significant local variations occurred in the thickness and elevation of the glacial outwash. As a result, an additional 106 shallow test borings were performed within the footprint of the proposed buildings.

Final design criteria for the wood pile foundation were described by Main as follows:

> "The varying conditions found made it necessary to take particular care to design the pile foundations on the basis of as nearly uniform settlements as possible. This uniformity is highly desirable in order to prevent overstressing the more or less continuous concrete floor beams and slabs used in the superstructure of the buildings.
>
> The working values have been taken at about 1/16-in. settlement, as shown by the tests, as it is believed that most piles have an initial settlement, whether noted or not, and considerable care was taken at these tests to note settlements at all times of changes as small as 1/64 in. It is also believed that the effect of a difference in settlement of 1/16 in. in the foundations can be safely ignored.
>
> A settlement of 1/4 in. was considered to be the limit of usefulness of a pile, and it was assumed that greater settlements than 1/4 in. might create conditions which would cause very unsatisfactory results.
>
> It was also considered necessary that the piling have a safety factor of not less than 2-1/2 based on the limit of 1/4-in. settlement." (16, p.23).

The test pile program revealed that spruce piles could not be driven safely into the thicker, denser portions of the glacial outwash. Many of the spruce piles, which had appeared to drive satisfactorily, failed during load tests and were found to be broken or broomed when extracted.

Main concluded that the use of spruce piles was a likely cause for the excessive structural settlements that had been reported in the vicinity of the site.

The additional shallow test borings had revealed that the sand-gravel outwash was thin or non-existent at some locations notably along the thread of the creek channel which crossed the site. Thus, some piles were driven into the clay to provide the required safe load.

A maximum load of 10 tons was allowed for spruce piles and 14 tons for oak piles. Final design loads averaged 9.0 to 9.5 tons, less than one-half the load customarily used in design of timber pile foundations today.

Criteria were established for the production piles that were intended to protect spruce piles from damage due to hard driving and to achieve as uniform settlement between the piles as possible.

The Engineering News formula, the same as in the present Massachusetts State Building Code, was used for establishing end bearing pile capacities. The formula is as follows:

$$P = \frac{2\ Wh}{S + 1}$$

where: P = allowable load in pounds
W = weight of hammer in pounds
h = fall of hammer in feet
S = penetration in inches

The load tests indicated that the coefficient could be increased to 3 for friction piles bearing in the clay.

False driving resistance in the fill was considered by deducting the formula value calculated during driving through the fill from the allowable load calculated for the final penetration resistance.

STATE-OF-THE-ART IN 1914

A state-of-the-art paper titled "Boston Foundations" by J. R. Worcester was published in the Journal of the BSCE in January 1914.

In his discussion of the geology of the Boston Basin, Worcester made the following statement:

> "Consistency of (the) deep clay formation.....appears to vary in different parts of the Boston Basin. Under the Boston peninsula it is generally fairly hard....Under a section of Cambridgeport and a part of the Back Bay the material is extremely soft, so soft in fact that it appears to flow freely from heavily loaded areas toward places where the load is less. It is not definitely determined, so far as the writer knows, whether such a flow is taking place, or the clay is gradually being compressed. It is certain, however, that widely-spread settlements have occurred." (27, p.3).

Worcester went on to state that:

> "This tendency to settle will have to be taken into consideration...in the future. It is not enough to gain the necessary support in piles which may rest in a gravel crust, but the settlement of the crust may seriously injure important structures, as it is believed to have already done in the case of the (Boston) Public Library and The New Old South Church." (27, p.4).

Thirteen discussions were submitted for Worcester's paper. Five of the discussions comment on the observed settlements in the Back Bay and Cambridgeport and attribute the cause to either consolidation or displacement of the soft clay.

Many of the respondents expressed confusion over contradictory settlement data. The granite sea wall along the present Memorial Drive, constructed without piling, was reported to have experienced virtually no settlement, except where it was constructed over fill, whereas surface settlements were measured in the roadway behind it. The absence of structural distress for buildings reported to have settled significantly added to uncertainty about the quality of the data.

There were clear warnings, however, from some members of the engineering community that the Cambridgeport area was treacherous and that the cause of the structural settlements was the thick deposit of blue clay.

On December 19, 1917, Charles T. Main and H. E. Sawtell, a structural engineer with Main, presented a paper to the BSCE on foundations for the New Technology. At the time, data on settlement of the buildings during and immediately following construction were known to Stone & Webster and probably to Main, but the data were not disclosed in the paper.

In a discussion of Main's paper, Worcester states:

> "It will be a very pleasing surprise if a gradual settlement of the whole group (of buildings) does not develop, through the subsidence of the glacial deposit below the mud. Such a settlement might not be noticeable or injurious if it were uniform over the whole area, but the area covered by the connected buildings is so large that it would appear hardly probable that they could all move together. The outcome will be awaited with great interest." (28, p. 137).

FREEMAN QUESTIONS FOUNDATION DESIGN

Freeman had no contact with those involved in the design process during preparation of the final design until January 1914, partly because he was not consulted and did not want to appear angry about the turn of events, and partly because he was indeed soreheaded.

In a letter to Professor Crosby dated January 12, he raised several questions concerning Crosby's geological report. He asked, "Did you have any determinations made of the quantity of water contained in the blue clay or any experiments upon an undisturbed sample, by compressing it as in a testing machine to see if water could be squeezed out?" (16). He also corrected Crosby regarding the performance of the timber pile foundations of the Boylston Street buildings, noting that serious cracks had developed in the Walker Building due to unequal settlement.

Crosby replied to Freeman, "I satisfied myself by actual test in the Mechanical Eng. lab. that no important amount of water could be squeezed out of the blue clay." (7).

Freeman wrote to his friend Charles A. Stone on March 5, 1914, sending him the data he had collected regarding surface settlements at the site and the settlements of neighboring buildings. He referred to a note on his drawing which attributed the settlements to compression of the clay. He also sent a drawing of his proposed foundation scheme with the deep girders. He told Stone that he felt that Crosby's report was inconclusive.

Freeman also contacted Charles T. Main to voice his concerns, and provided him with the same documents. He gave Main "...friendly warning that while Crosby was a most excellent geologist, when he attempted to play engineer he sometimes made mistakes." (8).

In a letter to consulting engineer Allen Hubbard dated March 7, 1914, Freeman stated, "I have yet seen no sufficient assurance that there may not be the same difficulties of settlement near the middle of this site that have been experienced...a few hundred feet to the north...on ground which present indications indicate may perhaps be duplicated beneath the Architect's lofty dome."

Stone's response to Freeman regarding his concerns was to say that they had adopted the foundation scheme presented in his preliminary study although they raised the pile cutoff grade about six feet. The change in the pile cutoff grade when added to the revision incorporating a basement meant that Freeman's girder system had been eliminated.

By the beginning of May 1914, Freeman gave up. In a letter to a fellow alumnus who shared his concerns he said:

> "I believe that those actively engaged in building the new structures are satisfied (with the results of the pile load tests) and they have entirely failed to grasp the fact that tests of that kind prove nothing about the slow flow of clay or the squeezing out of water of supersaturation...I am inclined to believe that you and I are regarded as well-meaning cranks on this particular subject and are to be dealt with politely rather than seriously." (9).

BUILDING CONSTRUCTION

Constructing the New Technology was an immense undertaking. The building was to be one of the largest single structures in the country at that time.

The project was, using modern terminology, "fast tracked". Construction started as soon as the foundation plans were prepared but well before the plans for the frame were complete.

Excavation for the buildings began on September 15, 1913, and the first of 25,000 wood piles was driven on December 4. Construction started to appear above ground in August 1914. All of the buildings assumed an outward appearance of completion by November 1915.

Building materials were delivered on a system of more than seven thousand feet of spur tracks leading off the nearby Boston & Albany railroad line. A lumber yard and sawmill were set up near the Esplanade, as were machine and blacksmith shops. Concrete was batched at the site and reinforcing steel was cut and bent on site. Eight huge gantry cranes, 110 ft. tall with a working radius of 250 ft., were erected at strategic points to handle materials, Figure 16.

Figure 16. Construction of the New Technology showing Room 10-250.

One and a half million board feet of lumber were used in constructing the forms for the reinforced concrete. In all, 5,000 tons of steel, 15 million bricks and 50,000 cu. yd. of concrete were used. Four hundred and fifty carloads of Indiana limestone were used for the facades of the buildings.

The Great Dome, which rose 147 ft. above street level, was the central feature of the whole structure. The dome consisted of 3 parts - a lower drum, 120 ft. in diameter and 37 ft. high, formed by concrete columns in two concentric rings; an upper drum 108 ft. in diameter and 18 ft. high, also formed by two concentric rings of columns; and a spherical cap rising 23.5 ft. which rested on the inner ring of the upper drum, Figure 17.

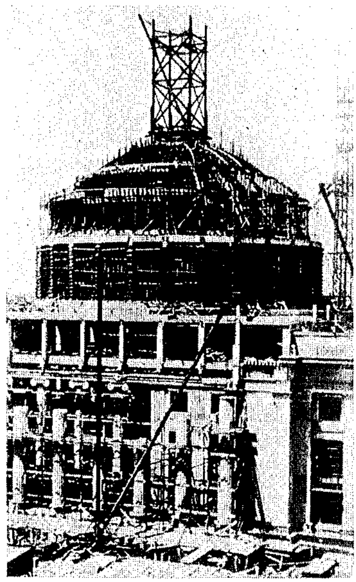

Figure 17. Construction of Building 10 showing the Great Dome.

As Freeman had predicted, the cost of construction for the New Technology exceeded original estimates by 100 percent. President Maclaurin reported frequently to George Eastman in Rochester and he made further donations to be added to those of a thousand loyal alumni and friends of the Institute to assure completion of the buildings, Figure 18.

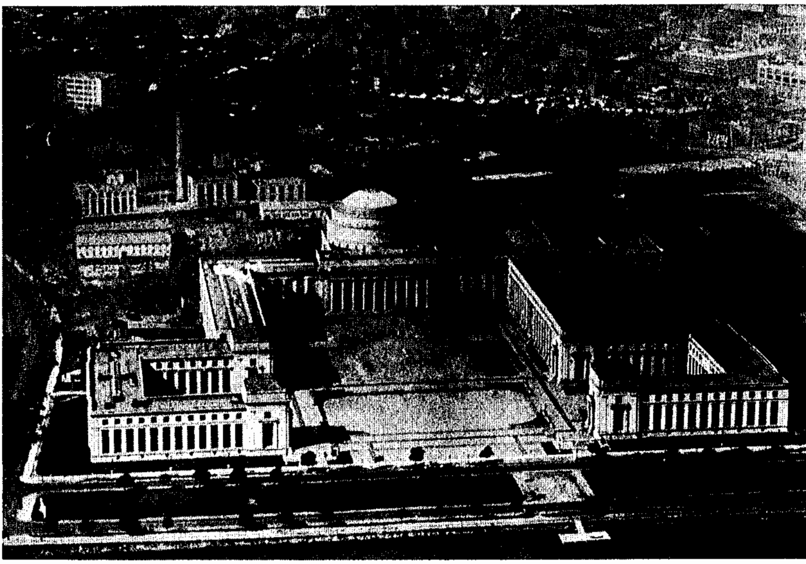

Figure 18. The New Technology; circa 1916.

The building was completed and occupied in 1916 amid great celebration and fanfare. Stone and Webster had generously contributed the President's House. Funds raised by alumni over a period of 18 years provided for construction of the Walker Memorial Building which was completed in 1917.

FOUNDATION SETTLEMENT

Engineers for the New Technology had expected settlements to be very small, less than one quarter of an inch. Nevertheless, as well-trained alumni, they inserted plugs in the basement columns of the buildings during construction so that subsidence could be measured.

Stone & Webster established initial elevations on settlement plugs during construction in February 1915. First observations were made in January 1916 and the second set of readings after occupancy in December 1916.

Both the magnitude and rate of settlement were alarming. By December 1916, the present Building 10 had settled more than 2 in., and Building 2 more than 1.5 in. Cracking began to occur in the terrazzo floors and in the plaster. Fortunately, no significant structural damage was experienced.

The greatest settlements occurred in sections of the building over the creek bed which traversed the mud flats before the site was filled, Figure 19. The outwash sand-gravel stratum was thin or absent at this location, presumably due to erosion. Unfortunately, the Great Dome (now Building 10), the heaviest structure, was located directly over the old drainage course.

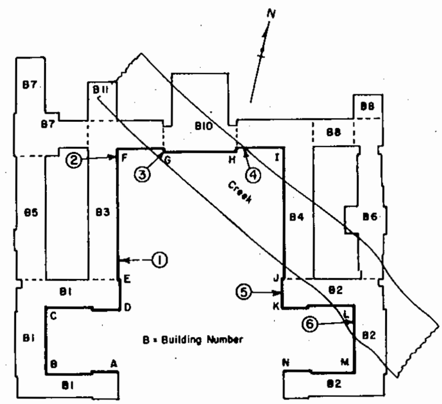

Figure 19. Plan showing location of settlement profile, shown in Figure 20, as bold line, and location of former creek channel.

Stone & Webster made two sets of readings after December 1916, the records of which were not complete nor have they been located. The next complete set of observations was made in July 1926 by Kenneth A. Smith, '27, and a party of other students. In 10 years, Buildings 10 and 2 had settled approximately 7.5 and 6.0 in., respectively. (Settlement in Building 1, Civil Engineering, was as little as 1 in.)

The Institute could do nothing but watch the buildings settle.

Figure 20 shows settlement profiles through January 1963, for the exterior building wall along a line around the inner courtyard, beginning at the left, with point A at the corner of Building 1. The upper part of the figure includes a soil profile showing the variation in thickness of the sand-gravel stratum below the building wall and locations where wood piles were driven through the outwash into the clay, noted as "long piles". These locations generally occur where the creek bed traversed the site and the outwash is thin.

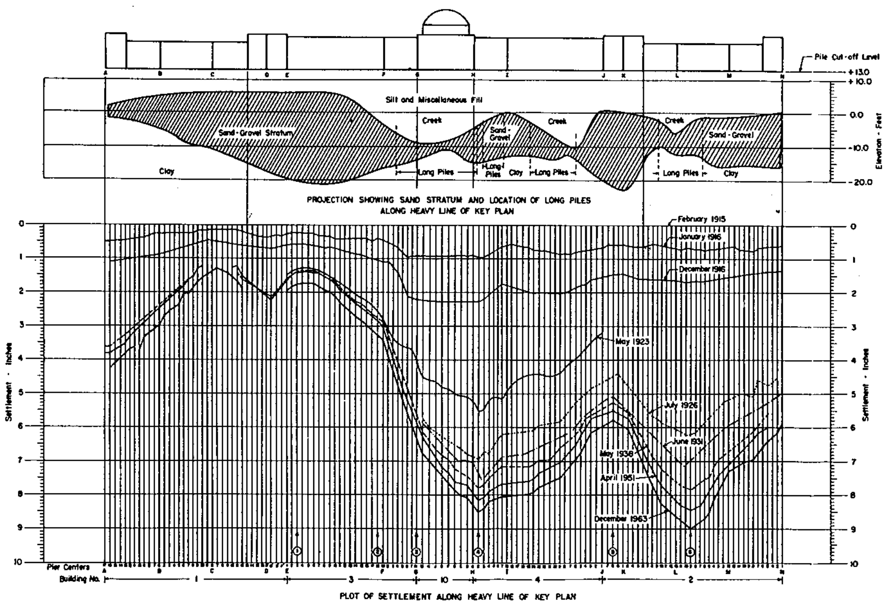

Figure 20. Settlement history of the New Technology.

Since the foundation loading for all buildings in the complex, with the exception of Building 10, is approximately the same, the extreme variation in settlement must be attributed to another source. Undoubtedly, the variation in the thickness of the clay was a contributing factor. However, the principal cause was not fully understood until years later.

Today, we recognize that settlement occurred primarily as a result of compression of the normally consolidated lower region of clay, a gradual squeezing out of the pore water in the voids of the clay, foretold by Freeman.

The long friction piles transmitted the building load more directly to the soft compressible lower region of the clay. Where the building load was carried by short piles bearing near the top of a thick deposit of sand-gravel outwash, the building load had been distributed more widely over the surface of the clay. By applying the load well above the top of the clay, the stress increase in the lower region was smaller and settlement was less.

In his 1944 BSCE paper, MIT Professor Donald W. Taylor stated, "The main reason for the difference in settlements resides in the variable thickness of a sand layer which overlies the blue clay."

Figure 21 shows the time rate of settlement, at representative points, plotted as a function of square root of time. Today, the maximum settlement of the main Institute buildings is approximately 10 in. in Building 10. Fortunately, due to the nature of the phenomenon, the rate of settlement decreased with time and is near zero today.

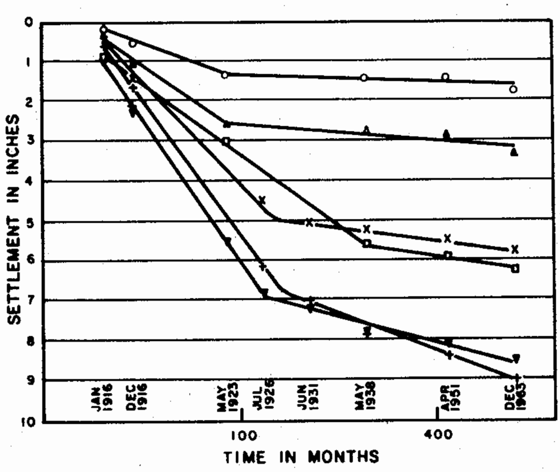

Figure 21. Time rate of settlement plotted using square root of time method.

Ironically, had Freeman's proposed foundation for the New Technology been adopted, building settlements would have been even greater than those which occurred, although differential settlement may have been less.

Freeman called his proposal a "floating foundation" only because of the rigidity of the structural concrete foundation girders. His design was not a "floating foundation" as defined in foundation engineering practice today

His plan had no basement. The ground floor was to be 4 to 5 ft. above street grade. Thus, the average net stress on the underlying clay stratum would have been even greater than that for the existing structure which has a basement.

KARL TERZAGHI COMES TO MIT

The unexpected settlements made all those involved with the design process aware of the failure of existing technology in civil engineering. Many phenomena involving the behavior of soils, earth structures and foundations were not understood.

Starting in 1925, MIT would develop a new technology called soil mechanics.

In the spring of 1925, an Austrian engineer named Karl Terzaghi, Figure 22, Professor of Civil Engineering at Robert College in Constantinople (Istanbul), Turkey, published a lengthy book in German titled Erdbaumechanik (Soil Mechanics). A result of seven years of patient experimentation in a primitive laboratory, his book established the theoretical principles of soil mechanics accepted today.

Dr. Terzaghi was 42 years old at the time, his reputation was limited and his career had reached a turning point.

Professor Charles M. Spofford, '93,* Head of the Civil and

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*MIT's first staff roster in 1865 included John B. Henck, Professor of Civil and Topographic Engineering. He was succeeded by George L. Vose in 1882 until George F. Swain,'77, became Hayward Professor of Civil Engineering and department head in 1887. Swain, possibly the finest teacher-engineer of civil engineering the country has produced, held the post of department head for twenty-two years until 1909. One of Swain's outstanding students, Charles M. Spofford, '93, left his position at Brooklyn Polytechnic Institute in 1909 to assume the Hayward Professorship. He became department head in 1911, serving until 1933.

Figure 22. Karl Terzaghi, circa 1925.

Sanitary Engineering Department at MIT and partner in the firm of Fay, Spofford and Thorndike, had heard of Terzaghi's research work, and also learned that he was to have a year's leave of absence from Robert College. At his recommendation, President Samuel Stratton invited Terzaghi to come to MIT, offering him the position of Lecturer on Foundations and Soil Mechanics and Research Associate at a salary of $2,000.

We can only assume that Stratton invited Terzaghi not only to teach but to evaluate the subsidence of the main buildings.

He is known to have written a report for the Institute concerning settlement of the New Technology, although the authors have been unable to locate a copy. Ordinarily, we would surmise that he would use his theory of consolidation, a diffusion process based on a Laplace equation, to explain the slow dissipation of pore water pressure in the clay and the resulting time-settlement curves, and thus provide some assurance that the majority of settlement had taken place. However, we find that Terzaghi in his years at MIT did not fully understand the cause of settlement.

In the spring of the year following Terzaghi's arrival at MIT, President Stratton asked him to investigate underground conditions for expansion north of the main buildings. His report dated October 13, 1926 includes some interesting observations.

He concluded that the "...test borings confirmed all the statements contained in Professor W. O. Crosby's* original report... (July 1913) except for the statements concerning the character of the blue clay...experience has shown that the blue clay did not behave as expected." (25, p.2).

He performed soil tests to determine if the clay contained "excess water" and to determine the ultimate settlement of the clay without any other loads acting on it, and finally the "ultimate bearing capacity...the load under which the clay starts to flow like a viscous liquid." For his one-dimensional compression tests, he used samples of clay which had been mixed with water into "the viscous liquid state". Undisturbed samples were used for "cube tests" to determine the bearing capacity.

Terzaghi determined that the natural water content of the clay <u>increases</u> with depth, meaning that "the percentage of excess water increases considerably with depth," and assumed that "consolidation is still going on, as a geological process."

He concluded that unequal settlement of the MIT buildings was due to three different causes:

"(a) The gradual consolidation of the clay deposit, which is a geological process, hardly affected by the presence of the buildings.

(b) The volume change (loss of water) due to the added weight of the buildings.

(c) Lateral flow of the clay, produced by the weight of the buildings.

Of the three causes, (a) and (b) seem to be of minor importance compared to (c), because the low degree of permeability of the clay prevents the volume change from proceeding with speed. Hence the settlements are essentially due to lateral flow, at fairly constant water content." (25, p. 9).

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*Professor Crosby died on December 31, 1925.

Several years later, in an ASCE paper published in 1929, he made the following statement: "...the extreme slowness of consolidation of typical clays is illustrated by the following observation. In 1915, a building of the Massachusetts Institute of Technology was erected ... The pressure exerted by the building on the clay amounts to approximately 1500 lb. per sq. ft. In 1926, a test boring was made next to the heaviest section of the building. Physical examination of the drill samples disclosed the fact that the consolidation of the clay deposit had hardly started." (26, p.278).

In fact, primary consolidation was nearly complete by 1930.

FREEMAN SUPPORTS TERZAGHI

Terzaghi's appointment attracted Freeman's attention. He obtained a copy of Erdbaumechanik which, in spite of his limited knowledge of the German language, he believed to be the most outstanding work published on the topic.

In the fall of 1925, Freeman invited Terzaghi to his home in Providence for the weekend, to become acquainted with his work. They talked for hours and got along famously. Freeman was convinced that Terzaghi was one of the brightest engineers he had met. In a letter to his good friend former MIT President Henry Pritchett he stated, "He (Terzaghi) seems to combine practical experience with a facility in mathematical analysis that is hard to find among American professors in engineering." (12).

Thereafter, he ordered more copies of Terzaghi's book and sent them to various professors, including Hardy Cross at the University of Illinois, and practicing engineers for their opinions. He made use of his personal contacts to check out Terzaghi's professional and academic reputation.

Once Freeman had convinced himself of the value of Terzaghi's ideas and abilities, he became a strong and influential supporter.

Freeman worked hard to expose the engineering community to Terzaghi's ideas. He used his influence as past president of the ASCE to arrange for Terzaghi to lecture before the civil engineering societies in Boston and New York.

TERZAGHI ESTABLISHES SOIL MECHANICS IN THE U.S.

Terzaghi needed all the help he could get. His contemporaries were confused by some of the terminology he used. Some influential engineers were insulted by Terzaghi when they sought clarification.

In November and December of 1925, Engineering News Record published eight papers he authored on the physical properties of clay and sand, and related topics. Some of his concepts were viewed with skepticism, in particular his ideas concerning capillary pressure in clay caused by surface tension during drying. Nevertheless, these papers were to establish Terzaghi's reputation.

President Stratton had asked Freeman to evaluate Terzaghi and his work, to determine if he should be asked to stay at MIT permanently. Terzaghi had made a bad impression on Stratton, who considered him a prima-donna. Freeman lobbied on Terzaghi's behalf and counseled him on how to work with the MIT faculty and administration.

In January 1926, he wrote Stratton recommending strongly that Terzaghi be retained at the Institute and that he be provided with the apparatus and space he needed for a laboratory. Professor Spofford supported the recommendation.

President Stratton accepted the recommendation, telling Freeman "I have long ago come to the conclusion that we can put up with eccentricities (of prima-donnas) in the case of real genius, but it isn't worthwhile with ordinary men. I can assure you that it is my intention to give Dr. Terzaghi an opportunity to demonstrate whether or not he is a genius."

Terzaghi's assistant during his first year at MIT, was Glennon Gilboy, '25. Gilboy would receive his Sc.D. degree in 1928 and serve on the civil engineering faculty teaching soil mechanics until 1937.

A year later, a young Austrian engineer named Arthur Casagrande joined Gilboy in assisting Terzaghi. Casagrande had arrived in the United States in April 1926, and shortly thereafter met Terzaghi in Cambridge. Terzaghi asked him to become his private assistant for the summer, working on his consulting projects in Washington, DC. By December, Terzaghi had arranged for Casagrande to be employed by the Bureau of Public Roads, serving as his assistant at MIT. Casagrande remained until 1932, during which time he made major contributions to our understanding of soil behavior and the development of laboratory testing equipment and procedures.

With Freeman's assistance Terzaghi's ideas were gradually accepted in the academic community and by practicing engineers. He became the revered father of soil mechanics.

MIT established and then expanded the laboratory facilities Terzaghi needed. He was promoted to the rank of full professor in 1928, just three years after his arrival. When he left to return to Austria in 1929, a vigorous research program had been established at MIT, providing a rational basis for evaluating the behavior of soil in relation to foundation engineering and earthwork problems

Indeed, the New Technology in Cambridge had served as a laboratory for the development of a new technology called soil mechanics.

ACKNOWLEDGEMENTS

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We thank Professor T. William Lambe for making available records of settlement of the MIT buildings and related information.

If further acknowledgement is due, it is hereby gratefully presumed.

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SEDIMENTATION SINCE CAMP [1]

By Richard I. Dick [2]

Thomas R. Camp's achievements were in diverse areas. Many engineers, of course, have broad interests, but Thomas Camp made important and lasting contributions in a number of areas. The first American Society of Civil Engineers publication on "Engineering Classics" (2) featured some of the important contributions of Thomas Camp. These were categorized in such diverse areas as sewerage, filtration, distribution systems, flocculation, and sedimentation.

In this paper, Thomas R. Camp's contributions related to sedimentation are reviewed. Their significance is considered vis à vis developments in sedimentation technology since Camp's work so as to evaluate the current state-of-the art of sedimentation tank design. Such retrospective evaluations are the fate of famous men and women. It will be shown that Thomas R. Camp's sedimentation contributions withstand the test of time, but that the record of the engineering profession in making use of his work is far less impressive.

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[1] Thomas R. Camp Lecture presented to the Boston Society of Civil Engineers on March 11, 1981.

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The Importance of Sedimentation in Water Quality Control

Before evaluating Camp's work on sedimentation, it seems appropriate to justify my own choice of sedimentation as a topic for discussion in 1981. It is neither an "innovative" nor an "alternative" process[c]. In an era of concern about topics such as priority pollutants, TSCA, RCRA, BAT, and NPDES (all terms which would have puzzled Camp's audiences)[c] is it reasonable to be concerned with a mundane topic such as sedimentation? Sedimentation, after all, is the oldest of all water quality control processes - indeed, it was an old process when Camp worked on it.

The answer to the question of whether sedimentation is a pertinent, current water quality control topic is an emphatic yes! Solids-liquid separation processes continue to be the major means by which pollutants are separated from water. And of the techniques for solids-liquid separation, sedimentation continues to be the most economical and common. Not only are pollutants that exist in the suspended form removed by solids-liquid separation, but, also, chemical or biological processes are used to convert soluble pollutants to the suspended form so that they can be removed by solids-liquid separation processes. There are, of course, exceptions to this usual scheme (for example, ion exchange, reverse osmosis, and adsorption processes), but these

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[c]It is hoped that any future readers who were not yet born when this paper was written also will be puzzled by these symbols of excessive governmental involvement in environmental engineering.

processes are used for only a small fraction of the water and wastewater currently being treated.

Some indication of the economic significance of sedimentation in water quality control is available from data presented in the U.S. Environmental Protection Agency's "Needs Surveys" (37, 38). If it is assumed (conservatively) that each primary wastewater treatment facility, trickling filter installation, and activated sludge process is served by an average of at least two sedimentation tanks, then, according to the 1976 EPA Needs Survey (37), at least 20,750 sedimentation tanks were in operation in United States municipal wastewater treatment facilities. At a rough (and conservative) replacement cost of $250,000, this represents about $5.2 billion. Using the same assumption of two sedimentation tanks per primary, trickling filter, or activated sludge installation, the Needs Survey indicated that at least 17,650 new tanks needed to be constructed, and, again assuming a construction cost of $250,000 per tank, this represents $4.4 billion. The 1978 Needs Survey (38) indicated an estimated need of $14.6 for sedimentation and flocculation facilities to control of combined sewer overflows stormwater discharges as necessary to protect recreation areas. To these costs must be added the cost of sedimentation facilities for industrial wastewater treatment and for municipal and industrial water treatment. The total expenditure for sedimentation tanks in water and wastewater treatment is, perhaps, roughly in the order of $50 billion[d]. To use President Reagan's (30) analogy that a millionaire is a person with a four inch stack of 1,000 dollar bills in his hand, then $50 billion of expenditures for sedimentation tanks can be portrayed as a stack of 1,000 dollar bills over 3 miles high. These total expenditures for sedimentation facilities are in the same order of magnitude as public expenditures for placing a man on the moon, constructing the interstate highway system, or developing the space shuttle or the MX missile system. It does seem pertinent, therefore, to examine the cost effectiveness of expenditures for sedimentation tanks.

Camp's Sedimentation Contributions

Thomas R. Camp's published contributions to understanding of sedimentation spanned two decades. They began[e] with a paper in Sewage Works Journal in 1936 (6), and extended through a discussion (9) of Ingersoll, McKee, and Brooks' paper (25). These

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[d]This assessment of the economic significance of sedimentation facilities is admittedly crude - it is intended only to remind the reader that much money has been, is being, and will be spent for sedimentation tanks. Another approach would have been to observe that roughly one-third of conventional municipal water pollution control expenditures are for primary and secondary sedimentation facilities. Based on federal expenditures for construction grants for publicly owned treatment facilities in recent years, the annual federal subsidy for wastewater sedimentation tanks must be about $1 billion/yr. Note that, based on usual industrial levels of expenditures for research and development, this justifies an annual sedimentation tank research budget in the order of $35 million/yr!

[e]Many of the ideas in Camp's first major sedimentation paper were published by Camp earlier in 1936 in discussion (5) of a paper by Slade (32).

contributions drew heavily upon the early sedimentation paper by Hazen (23), but Hazen's work was clarified and extended by Camp.

Camp clearly demonstrated the relationship between the sedimentation velocity of particles in water or wastewater and the theoretical performance of sedimentation tanks. To do this, he considered a rectangular horizontal flow sedimentation tank from which all imperfections had been removed. As illustrated in Fig. 1[f], Camp considered that the sedimentation tank had an inlet zone that accomplishes uniform distribution of flow and particles across the entire cross-sectional area of the tank. A sludge zone was designated, and particles that settled into the zone were considered to be removed - resuspension was not possible. A third portion of the tank was designated as the outlet zone, and any particle entering that zone was considered to be lost in the effluent. Camp confined his analysis to the remaining part of the tank, the ideal settling zone, through which horizontal plug flow was considered to occur while particles settled at their characteristic sedimentation velocities.

To develop the relationship between the sedimentation velocity of a particle and the performance of a continuous-flow gravity sedimentation basin, Camp considered the trajectory of the slowest settling particle that would be completely removed. This trajectory is illustrated in Fig. 2 in which h_o, w, and L

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[f]Figures 1 through 6 are from Dick (12).

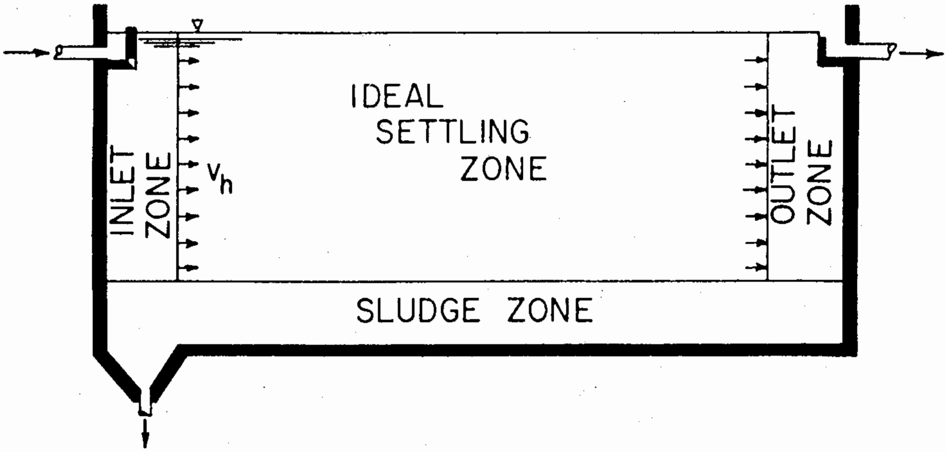

Figure 1. Definition Sketch for Camp's Ideal Sedimentation Tank.

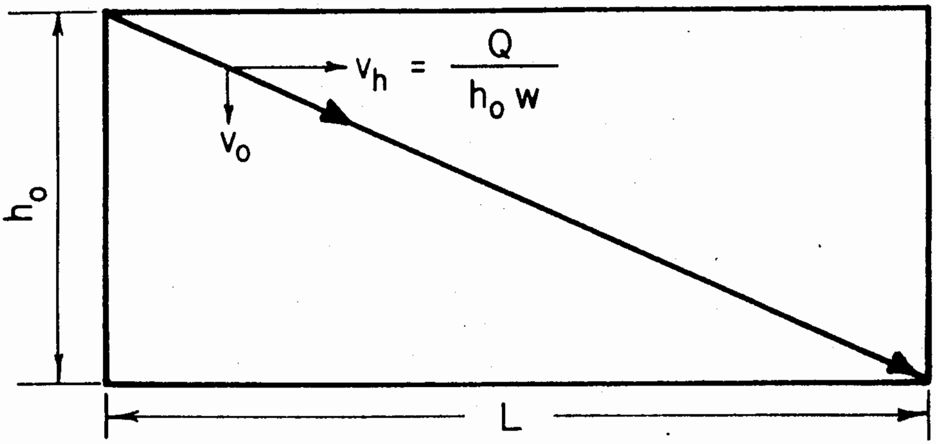

Figure 2. Trajectory of the Slowest Settling Particle Completely Removed in an Ideal Sedimentation Tank.

are the depth, width, and length, respectively, of an ideal settling tank, v_h is the horizontal fluid velocity created by flow through the tank at rate Q, and v_o is the sedimentation velocity of the slowest settling particle that is completely removed. From similar triangles:

$$\frac{v_o}{v_h} = \frac{h_o}{L} \tag{1}$$

or

$$v_o = \frac{v_h h_o}{L} = \frac{Q}{wL} = \frac{Q}{A} \tag{2}$$

The "hydraulic loading", "overflow rate", or "surface settling rate", Q/A, of a settling tank, thus, is equal to the sedimentation velocity of the slowest settling particle that is completely removed[g].

Camp showed that, depending upon the elevation at which they entered the ideal sedimentation tank, some fraction of particles that settled slower than v_o will be removed. This is illustrated in Fig. 3 in which v_p is the sedimentation velocity of a slow settling particle. Note that discrete particles with settling velocity, v_p, that enter above elevation h in Fig. 3

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[g]Camp confined his analysis to horizontal flow rectangular sedimentation basins. However, it can be readily shown that both Eqs. 2 and 3 apply to radial flow circular tanks with either center or peripheral feed. Also, Eq. 2 obviously applies to vertical flow sedimentation tanks. However, partial removal of slower settling particles (Eq. 3) does not occur in vertical flow tanks (unless it occurs by flocculation or entrapment of particles in a suspended sludge blanket).

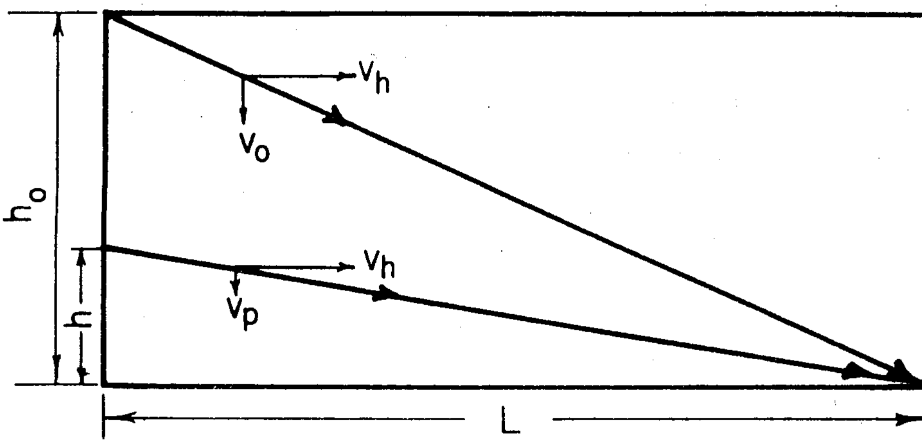

Figure 3. Partial Removal of Particles that Settle Slower than the Hydraulic Loading Rate, Q/A.

will not be removed because they will enter the outlet zone. Camp showed that the fractional removal, f, of particles that settle slower than v_o is[9]:

$$f = \frac{h}{h_o} = \frac{v_p t}{v_o t} = \frac{v_p}{Q/A} \qquad (3)$$

where t is the hydraulic residence time in the ideal sedimentation tank.

Camp emphasized the absence of retention time or sedimentation tank depth or volume in Eqs. 2 and 3, and sought means for economically increasing the effective area of sedimentation basins. As illustrated by Fig. 4, reducing the depth of an ideal settling tank by half does not alter particle removal efficiency, and, thus, the most economical sedimentation tank for discrete particles would be the shallowest tank possible. In Camp's words, "it follows from the simple theory that the most economical tank will have the least possible depth for the required overflow rate" and "for economy, therefore, the...depth should be made as small as is consistent with no scour" (7).

In his 1946 paper on "Sedimentation and the Design of Settling Tanks" (7), Camp proposed a primary sedimentation tank of

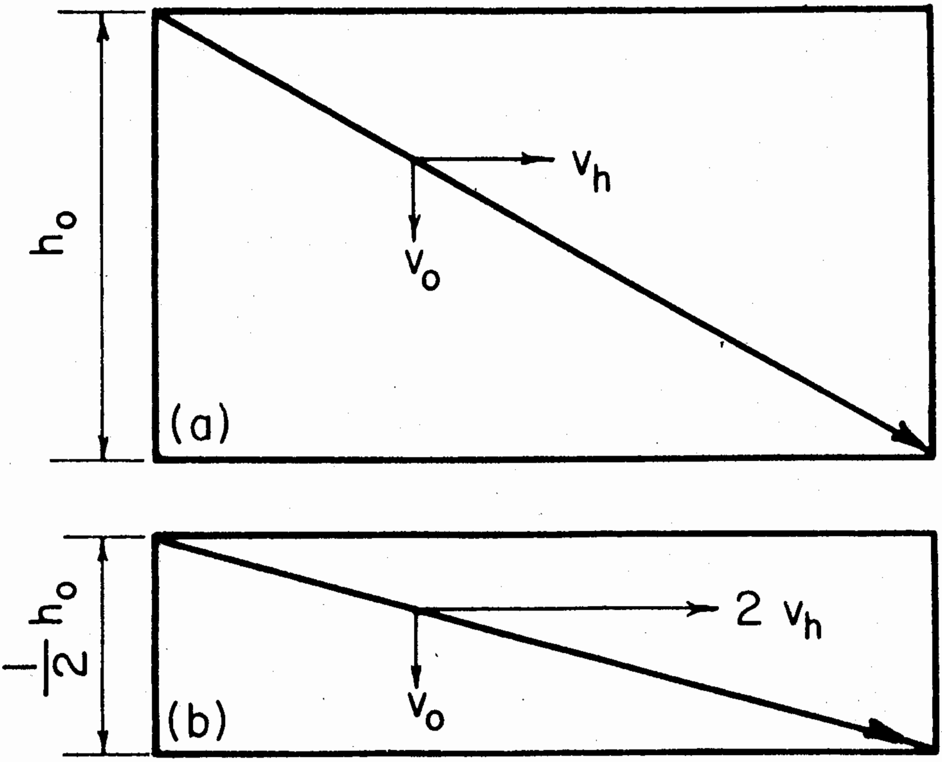

Figure 4. The Removal of Discrete Particles in Ideal Sedimentation Tanks is Independent of Depth.

normal depth with nine horizontal trays to cause a ten-fold increase in the effective capacity of the tank. Camp proposed a design for a reciprocating scraper mechanism on each of the trays that would move sludge laterally.

In a later paper, Camp (8) described implementation of his sedimentation concepts in improving existing sedimentation basins at the Cambridge, Massachusetts water treatment plant[h]. In upgrading the Cambridge facilities, two horizontal trays were constructed in the existing 16 ft (5 m) deep sedimentation tanks. Mechanical sludge collection equipment was not provided in the Cambridge basins[i]. In the same paper, Camp expressed his hope that sludge removal equipment suitable for use in tanks with shallow trays "will be perfected in the not-too-distant future." The problem of effective mechanical sludge removal from shallow horizontal trays still has not been solved, but it has been avoided by inclining the trays (or tubes) at an angle sufficient to achieve removal of sludge by gravity (21). The solution represents a compromise because only the horizontal area of the inclined trays is effective in sedimentation.

Camp recognized that flocculation of particles in a sedimentation tank complicates the analysis of settling tank

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[h] A more detailed description of Camp Dresser and McKee's design of the Cambridge, Massachusetts water treatment plant sedimentation tank refurbishments was presented by Dresser (15).

[i] The Cambridge tank was designed to be desludged periodically by taking it out of service. Nozzles were installed to assist in removing sludge from the tank (15).

performance. Fig. 5 illustrates that, if particle agglomeration occurs within a sedimentation tank, then particle sedimentation velocity will increase and a particle whose trajectory at the inlet end of the tank seemed destined to convey it to the outlet zone can, in fact, be removed. If flocculation occurs at the same temporal rate in a deep tank as in a shallow tank, then the concept that sedimentation tank depth is of no significance (which was developed by considering discrete particles) needs to be modified. This is illustrated in Fig. 6, which shows that in a shallow basin a flocculant particle may not achieve sufficient size and sedimentation velocity to be removed when tank depth is reduced.

Camp's assessment of the significance of flocculation in sedimentation tank design was a bit ambivalent. He was critical of Hazen's (23) failure to consider flocculation and, in his 1936 paper (6) indicated that "removal of suspended matter is more nearly a function of detention time than of tank surface area for most suspensions." However, particularly in his later work, he was a consistent advocate of shallow tanks or tanks designed with trays to increase the effective surface area. Camp's 1953 paper (8) contains the fascinating speculation that in primary sedimentation tanks, the use of shallower depths (limited only by the need to avoid scour of deposited sludge) was warranted to achieve an increase in turbulence in the tank. Camp's argument was that as horizontal velocity increased (due to reduced tank depth) turbulence would increase and this would cause

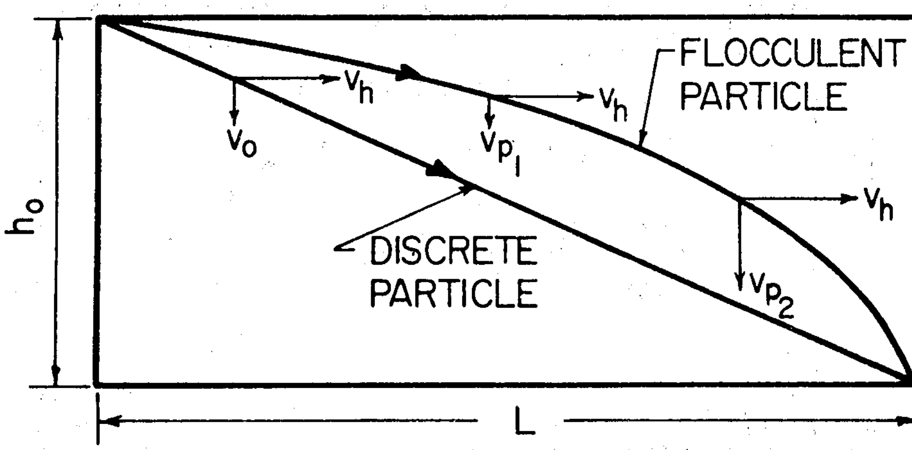

Figure 5. Particle Agglomeration can Influence the Performance of Ideal Sedimentation Tanks.

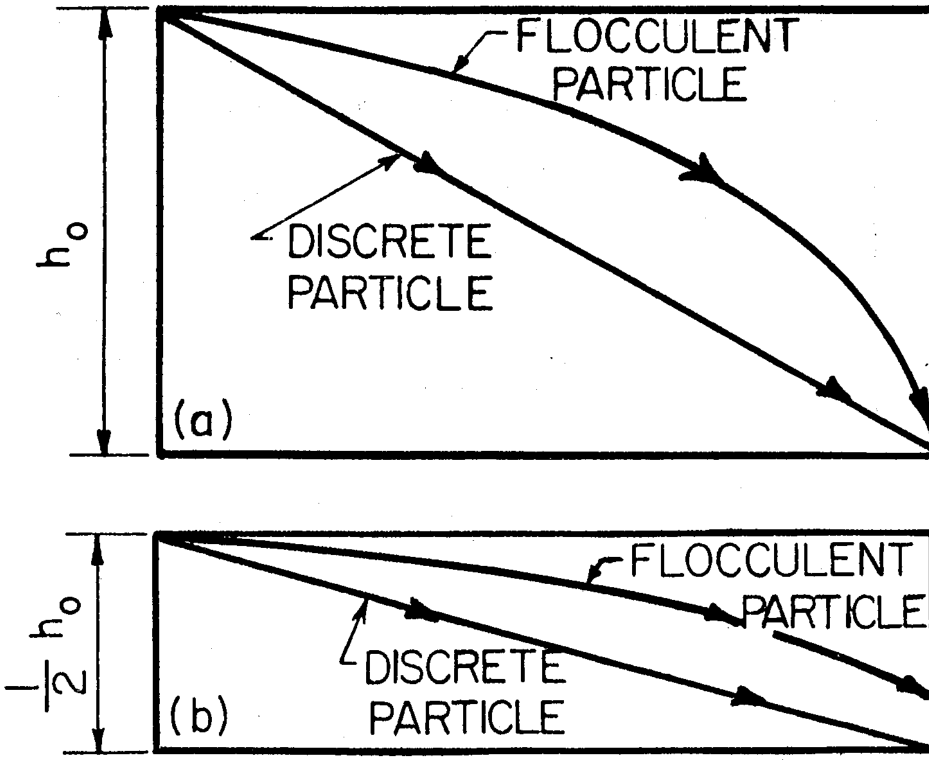

Figure 6. Sedimentation Tank Depth May Be Important With Flocculant Particles.

increased removal of particles because of improved flocculation. Camp speculated that the improved flocculation would more than compensate for the reduction in sedimentation tank performance due to increased turbulent diffusion (14). Camp's argument cannot be discounted in the absence of experimental evidence. Regrettably, such evidence still does not exist in spite of Camp's provocative assertion.

Camp apparently felt that whatever the effect of sedimentation tank depth on removal of flocculant particles, money was ineffectively spent to achieve improved flocculation within sedimentation tanks. Note, for example, that the improvements to the Cambridge, Massachusetts wastewater treatment plant (15) involving installation of trays also included provision of flocculation basins prior to sedimentation. More recently Parker, et al. (28) have argued that flocculation of activated sludge is warranted prior to its introduction into sedimentation tanks.

Because Camp's description of the relationship between the sedimentation velocity of a particle and the extent to which the particle is removed in sedimentation basins were so clearly presented, they were readily accepted by the profession. The analysis of the expected performance of an ideal sedimentation tank is a standard part of most water quality control process texts, and design standards of regulatory bodies [for example, see "Ten State Standards" (19)] have come to include the surface settling rate, Q/A, as a basic sedimentation design parameter (whereas, in the absence of Camp's work, they probably would have emphasized hydraulic residence time).

Comparisons of Observed Performance With Predictions Based on Camp's Work

In assessing the state-of-the-art of sedimentation tank design and the impact of Camp's work on sedimentation practice, it is appropriate to compare actual sedimentation tank performance to predictions founded on Camp's work. If the conditions assumed by Camp are fulfilled, predicted results should be obtained with mathematical inevitability.

Fig. 7, adapted from Ingersoll, et al. (25) provides a basis for determining the theoretical performance of sedimentation basins. The upper curve represents the sedimentation velocity distribution of particles in the influent to a sedimentation tank and the lower curve represents the expected effluent particle size distribution. The lower curve is obtained by applying Camp's principles[j]. By considering different values of Q/A, the expected relationship between the hydraulic loading and sedimentation tank performance can be determined for a particular suspension. Fig. 8 shows the results of such computations for a particular suspension [indeed, the one considered by Camp in his 1946 paper (7)].

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[j]Thus, all particles with settling velocity greater than Q/A should be removed, one half of the particles with settling velocity equal to Q/2A should be removed, etc. Hence, all particles represented by the cross-hatched area in Fig. 6 should be removed.

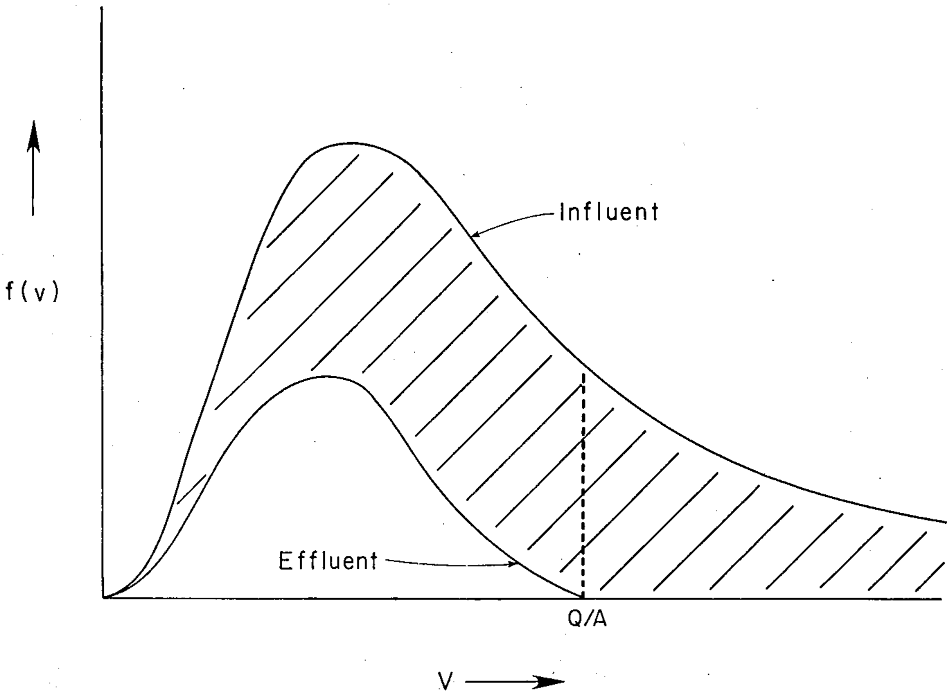

Figure 7. The Anticipated Performance of an Ideal Sedimentation Tank Receiving Particles with a Spectrum of Sedimentation Velocities [after Ingersoll, et al. (25)].

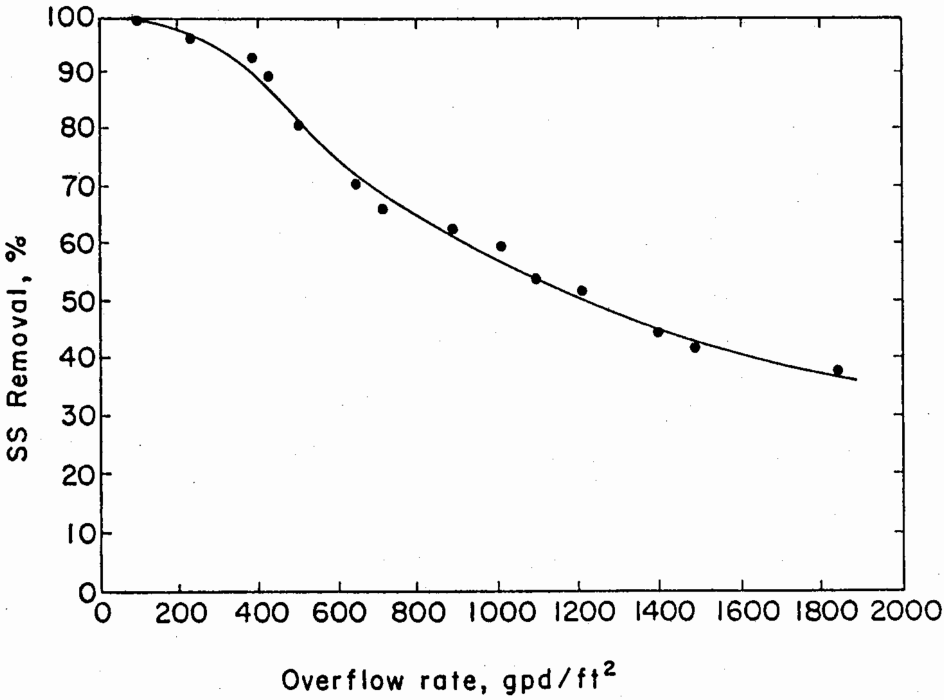

Figure 8. The Expected Performance of an Ideal Settling Tank Receiving a Particular Suspension.

Clearly, it was Camp's intention that the performance of sedimentation tanks would be evaluated by comparing observed removal to theoretical removal such as plotted in Fig. 8. Regrettably, this approach has not been adopted by the profession. Only in very few cases [for example, El-Baroudi (16)] have comparisons with ideal settling tank performance been made.

In the absence of data to allow rigorous comparison of theoretical predictions and actual performance, some assessment of the conformance of sedimentation tank performance to theoretical predictions can be made by observing whether or not the general shape of the percentage particle removal versus hydraulic loading curve is like the expected curve shown in Fig. 8[k]. Regrettably, the most abundant source of data on the relationship between the sedimentation tank hydraulic loading and particle removal is small-scale laboratory and pilot studies. While small-scale, continuous-flow sedimentation studies often have been conducted to model full-scale performance, the hydraulic similitude problems are, in fact, very severe. I regard such data as being interesting results from miniature sedimentation basins that can be closely regulated and easily modified - not as useful bases for scale-up.

[k]The reason this approach is unsatisfying is that a settling tank achieving, say, 50 percent removal of suspended solids could be of inferior design to one removing, say, 25 percent removal of suspended solids. This would be true if the tank achieving 50 percent removal was, according to the analysis illustrated in Figs. 7 and 8, expected to achieve 90 percent removal while the one achieving 25 percent removal was theoretically expected to be capable of removing only 30 percent of suspended solids.

Data presented by El-Baroudi (16) are illustrative of small-scale studies and are shown in Fig. 9. His sedimentation tank was 24 in (61 cm) wide and 58.5 in (149 cm) long with a water depth of 14.5 in (37 cm). A dispersing agent was used in El-Baroudi's research to control particle flocculation. It is seen that hydraulic loading exerted the expected effect on particle removal. Other evidence of the conformance of the results of small laboratory and pilot-scale settling tank data with expectations based on Camp's analysis of the performance of the ideal sedimentation tanks are available [Tebbutt (34), Villemonte (39), Baumann, et al. (3), and Hayden (22)]. Data from full-scale sedimentation tanks are, however, far more pertinent. As shown in the paragraphs that follow, these data are also less reassuring.

Data from a 94 ft (29 m) diameter primary sedimentation tank at Sao Paulo, Brazil as presented by Bradley (4) are shown in Fig. 10. The data are daily values obtained over a four-month period. While it is not surprising that the data are more scattered than those from closely controlled, small-scale tanks, little hint of a relationship between Q/A and performance is indicated by the data[1]. Heinke, et al. (24) conducted extensive

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[1] Note that the authors who presented the data shown in Figs. 10, 11, 12, and 13 are not responsible for the sedimentation tank designs that caused the data to be as they are. Bradley (4) and Dallas (10) fit curves to the data shown in Figs. 11 and 12, respectively, and Lin and Liao (27) derived an equation from the data in Fig. 13. I have elected not to suggest any cause-effect relationships.

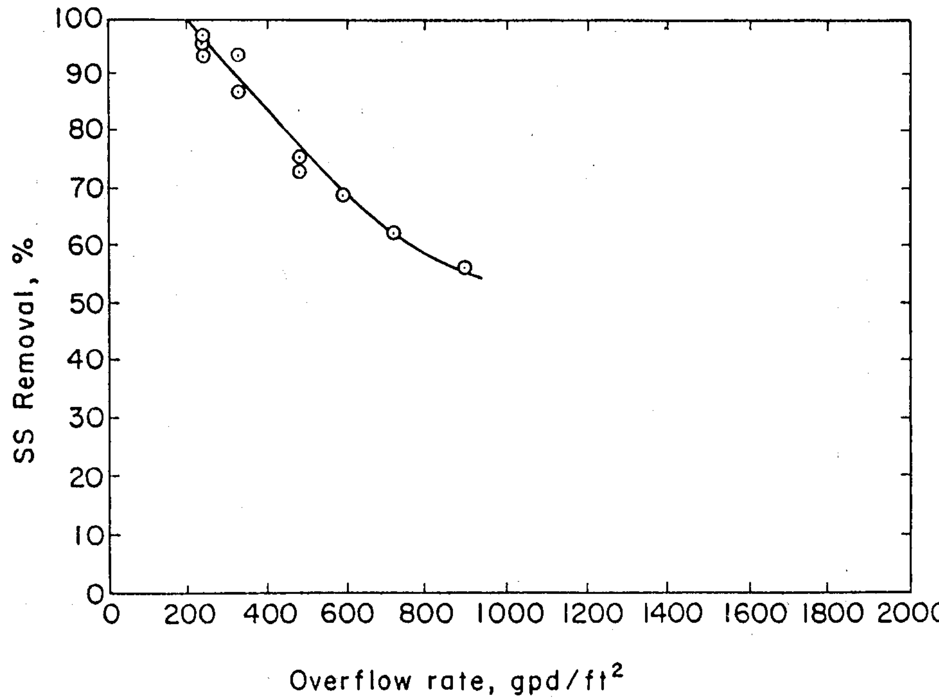

Figure 9. The Performance of a Closely Controlled Laboratory-Scale Settling Tank Illustrating Conformance with Expected Influence of Hydraulic Loading [from El-Baroudi (16)].

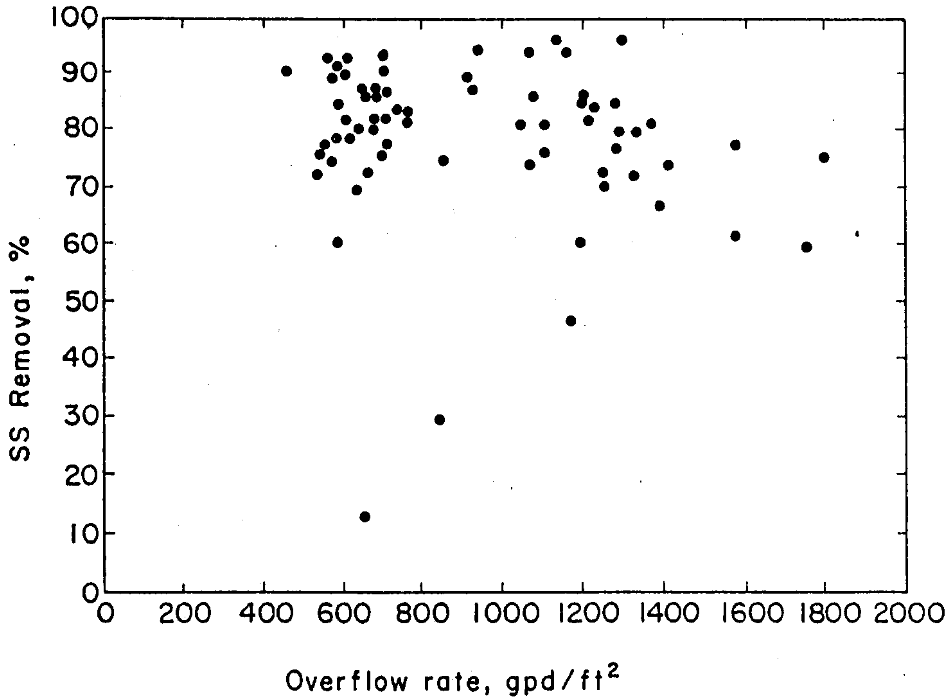

Figure 10. Independence of the Performance of a Full-Scale Primary Sedimentation Tank from Hydraulic Loading [data from Bradley (4)].

studies with two full-scale primary sedimentation tanks in Canada that were operated with and without chemical coagulants. They concluded that, within the range of hydraulic overflow rates considered in the studies [up to about 2,000 gpd/ft^2 (imperial) (100 m^3/m^2/day)], hydraulic loading had no significant effect on effluent quality. Thus, at loadings up to two to three times normal, factors other than Q/A (the parameter that limits ideal settling tank performance) controlled actual primary sedimentation tank performance. Similarly, Hamlin (20) presented results from a 80 ft (25 m) long experimental primary settling tank that showed "that there is very little change in tank performance with increasing overflow rate".

Activated sludge final sedimentation tank performance data (Figs. 11 and 12) also do not indicate a relationship between hydraulic loading and clarification performance. Data in Fig. 11 are for a 60 ft (18 m) diameter final settling tank at Dallas, Oregon (10)[1] while those in Fig. 12 are for a small [11 ft (3.4 m)] tank serving a salmon processing wastewater treatment plant as reported by Lin and Liao (27)[1]. As in the case of primary sedimentation tanks, it is difficult to sense any relationship between hydraulic loading and suspended solids removal from the data presented in Figs. 11 and 12.

Data on the effect of hydraulic loading on the performance of final sedimentation tanks following first and second stage trickling filters developed from results presented by Pierce (29) are shown in Fig. 13. These data are from a variety of full-scale

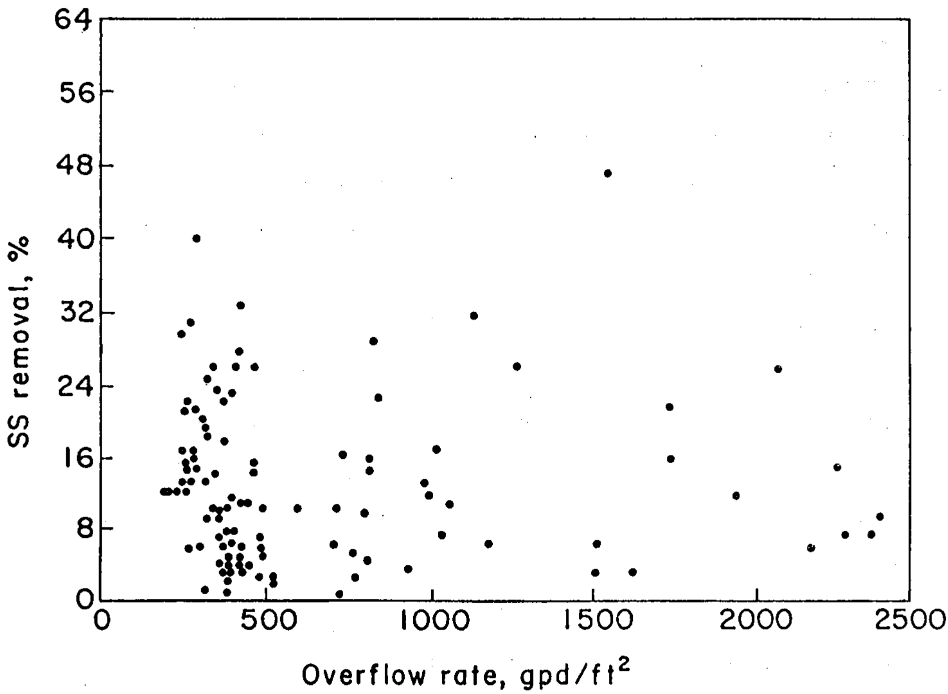

Figure 11. Absence of Influence of Hydraulic Loading on Performance of a Full-Scale Activated Sludge Final Sedimentation Tank [data from Reference 10].

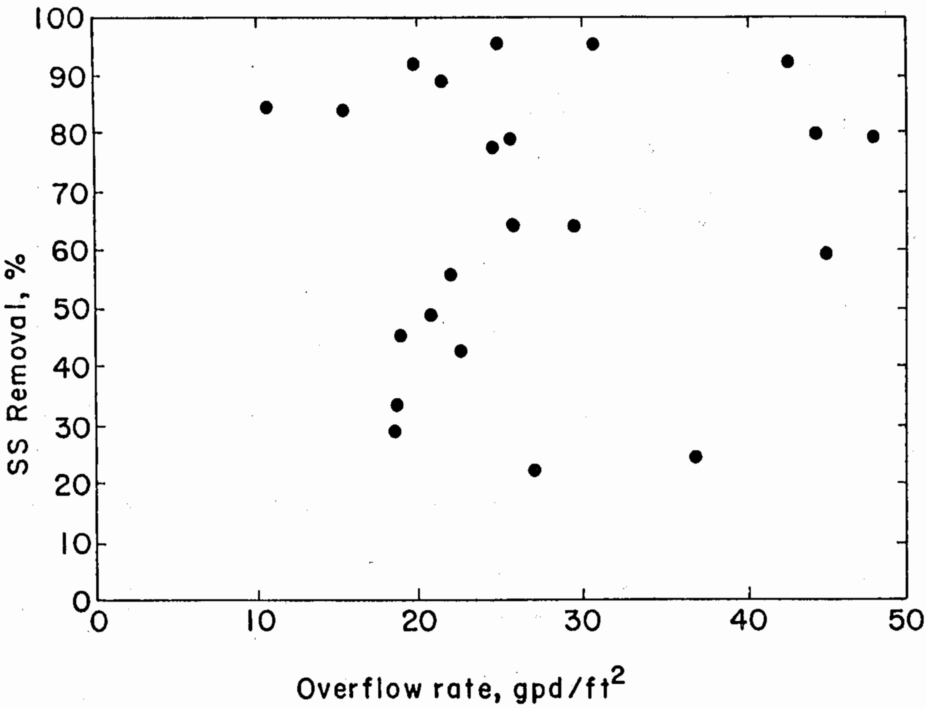

Figure 12. Lack of Effect of Hydraulic Loading on the Performance of an Activated Sludge Final Sedimentation Tank [data from Lin and Liao (27)].

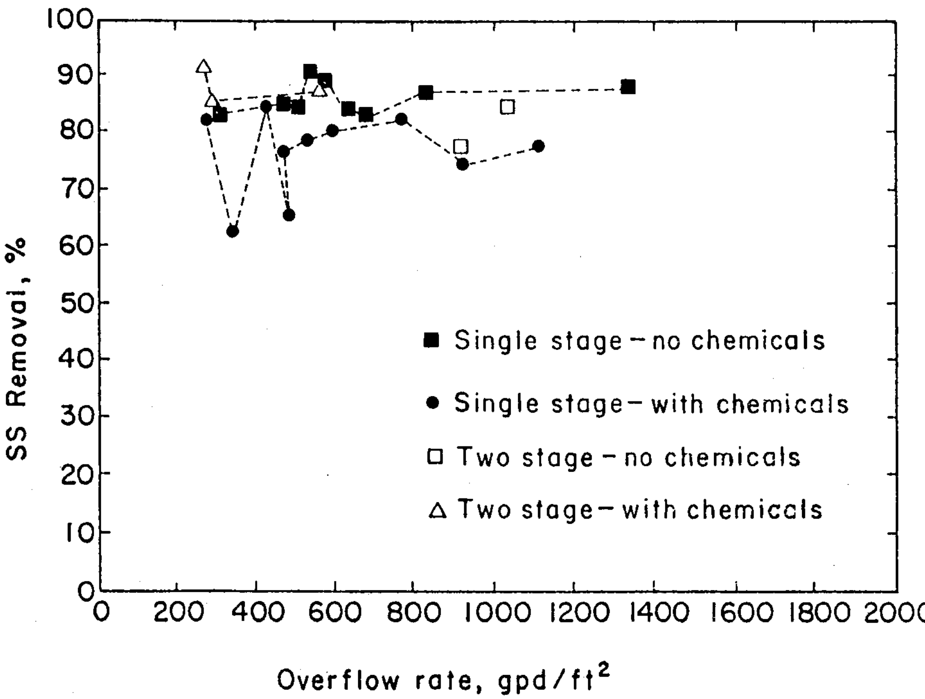

Figure 13. Absence of Influence of Hydraulic Loading on the Performance of Sedimentation Tanks Following Trickling Filters [data from Pierce (29)].

sedimentation tanks operated with and without chemicals for improving suspended solids removal. Again, factors other than the hydraulic loading per unit area apparently governed performance[1].

What's Wrong

Given Camp's convincing identification of the hydraulic loading, Q/A, as the principal factor controlling sedimentation tank performance and the long acceptance by the profession of hydraulic loading as the major sedimentation tank design parameter, the actual performance data shown in the previous section are distressing. Camp's work provides a basis for suggesting causes for the discrepancies.

Camp assumed that all particles that reached the bottom of the ideal settling zone were irrevocably removed. Common sedimentation tank design practices limit horizontal fluid velocities to avoid scour of particles, but the possibility of resuspension due to locally high velocity gradients at the sludge zone is not eliminated. Also, work since Camp has led to the realization that solids can be transmitted upward from the sludge zone by mechanisms other than scour. If the solids handling capacity of the sludge zone is less than the rate of application of solids, then solids propagate upward. This is most likely to occur in sedimentation tanks with high solids loadings such as final sedimentation tanks in the activated sludge process (13).

In developing the principles that have come to serve as bases for sedimentation tank design, Camp assumed that uniform velocity distribution was achieved at the inlet to the ideal settling zone. Data demonstrating that common inlet design practices achieve this condition are not available. Data from small-scale studies [such as by Villemonte, et al. (39), and Baumann, et al. (3)] serve to illustrate the importance of inlet design. The data of Villemonte, et al. [from a 3 ft (1 m) by 3 ft (1 m) by 14 ft (4.3 m) rectangular sedimentation tank] and those of Baumann, et al. [obtained using a 5 ft (1.5 m) circular tank 16 in (0.4) deep] are shown in Figs. 13 and 14, respectively[m]. The results confirm the importance of inlet design, but it is not clear how the inlet designs used in the experimental facilities can be scaled up so as to be applied in full-scale sedimentation facilities.

Camp's development also was based on the assumption of adequate sedimentation tank outlet design. Conventionally, outlet conditions have come to be designed by limiting the hydraulic loading per unit length of overflow weir to some arbitrary value [such as 15,000 gpd/ft (190 m^3/day/m) (19)]. Graber (18) used potential flow analysis to show that "weir loadings are of no

[m]Note that, like the results in Fig. 9, data from carefully designed and operated small-scale settling tanks shown in Figs. 13 and 14 show the expected form of the relationship between hydraulic loading and suspended solids removal.

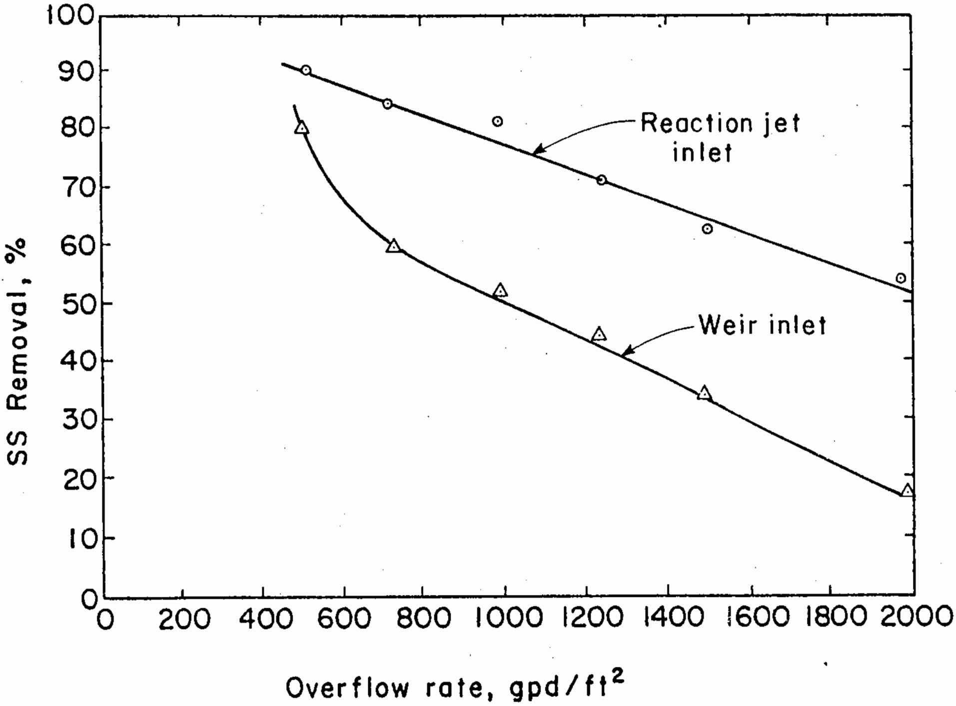

Figure 14. Effect of Inlet Design on the Performance of a Laboratory-Scale Rectangular Sedimentation Tank [from Villemonte, et al. (39)].

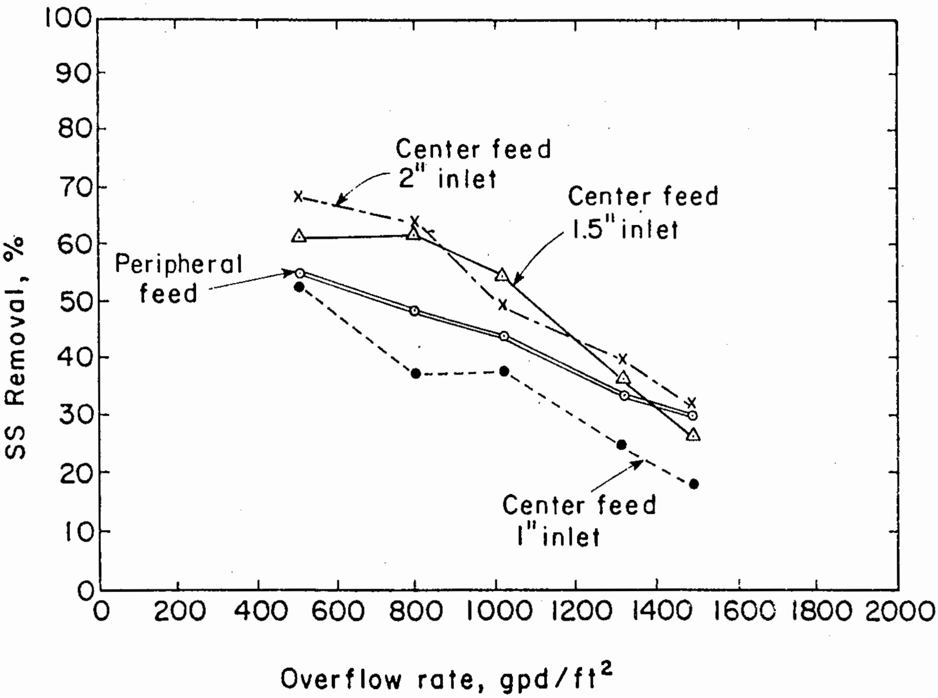

Figure 15. Effect of Inlet Design on the Performance of a Laboratory-Scale Circular Sedimentation Tank [from Baumann, et al. (3)].

direct consequence in primary settling tank analysis." This is in contrast with an earlier similar analysis (33) that may have served as part of the justification for weir loading restrictions. I am not aware of data that demonstrate the validity of common weir loading limitations, but contrary data are available. Theroux and Betz (35) plotted suspended solids removal data from 17 wastewater treatment plants as a function of weir overflow rate [up to 300,000 gpd/ft (3,700 m^3/day/m)] which showed little suggestion of a cause-effect relationship. They also conducted full-scale experiments in which effluent weirs were blocked to increase loadings from 40,000 gpd/ft (500 m^3/day/m) to 70,000 gpd/ft (880 m^3/day/m) and found "no obvious advantage, as far as suspended solids removal is concerned, in using longer weirs." Johnstone, et al. (26) reported on studies in which clarification effectiveness was improved by a change which drastically increased the hydraulic loading on weirs. Johnstone and his co-workers improved the performance of a full-scale sedimentation tank equipped with an inboard launder with weir plates on both sides by blanking off the outboard weir (and, thus, more than doubling the hydraulic loading per length of weir). They concluded that "the use of weir loading as a design criterion is of little relevance for this type of tank."

Results from tracer studies on sedimentation tanks are another source of data that demonstrate the ineffectiveness of conventional sedimentation tank design techniques. Regrettably, again, most such data are for small laboratory or pilot scale facilities.

In recent decades there have been a large number of studies of turbulent dispersion in sedimentation tanks [for example, by El-Baroudi (16), El-Baroudi and Fuller (17), and Thirumurthi (36)] and on means of anticipating the expected performance of tanks with nonideal characteristics [for example, by Alarie, et al. (1), Cordoba-Molina, et al. (11), Heinke, et al. (24), Tebbutt (34), and Shiba, et al. (31)]. Little progress, however, can be reported concerning improved design of sedimentation tanks.

State-Of-The-Art

One to two generations of water quality control engineers have received academic training on rational analysis of sedimentation tank performance. These principles they have been taught are based largely on the work of Camp.

When graduates enter professional practice, they find established sedimentation tank design practices to be seemingly compatible with their fundamental academic training. The surface area of sedimentation tanks is sized on the basis of hydraulic loading per unit area, and this is basic to Camp's analysis. Minimum hydraulic retention times are sometimes used, but graduates are aware that hydraulic retention time may be important with flocculant suspensions. Hydraulic loading on effluent weirs is restricted, and this seems compatible with the need emphasized by Camp to provide adequate outlet facilities.

Additionally, attention is given to inlet design and to high average horizontal velocities. Everything seems copacetic.

In fact, as demonstrated in previous sections, actual hydraulic design is inferior to that presumed by Camp in his analysis. Concepts based on his analysis are, thus, not applicable, and performance of settling tanks is inferior to that which might occur with improved design practices.

Manufacturers of sedimentation equipment, for their part, have provided the profession primarily with empty style changes since the time of Camp's work[n]. Improvements in materials of construction and means for removing sludge have occurred, but rational and well documented improvements in the clarification efficiency of settling tank equipment have not been offered.

Summary

Thomas R. Camp's rational assessment of the performance of sedimentation tanks was readily and widely adopted by the profession. His concepts have guided the design of tens of billions of dollars worth of sedimentation tanks and continue to serve as the foundation for current design techniques.

The profession failed, however, to heed the stipulations set forth by Camp as prerequisites for application of his

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[n]In presenting this paper, I showed a collage of advertising artistry and slogans used by manufacturers of sedimentation equipment to be considered vis à vis the actual records of performance of sedimentation tanks. Discretion dictated its elimination from published version of the paper.

concepts. Analysis of data from full-scale sedimentation tanks indicates that they do not perform in accordance with predictions based on Camp's work. Thus, major expenditures for water and wastewater treatment are being ineffectively spent.

In spite of Camp's monumental sedimentation contributions, in one sense he did not succeed in achieving his objectives. The first sentence of Camp's first paper on sedimentation (6) was:

> "It is the purpose of this paper to describe briefly some of the factors influencing clarification by sedimentation in an effort to stimulate interest in discussions leading to improvements in the methods of design of settling tanks."

Perhaps Camp presented his arguments too convincingly, for they were embraced too eagerly by the profession. The discussion and improvements that Camp sought to stimulate never occurred and the design of the most commonly used process in water and wastewater treatment still is not performed effectively. It is to be hoped that future developments will lead to an ability to design real settling basins that perform like the ideal basins accurately described by Camp.

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GEOTECHNICAL INVESTIGATIONS FOR TRIDENT DRYDOCK

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ABSTRACT: The results of geotechnical investigations at the Trident drydock site are presented. Site studies disclosed complex geologic conditions. Compact soil deposits ranging from sands and gravels to hard silts and clays were encountered. Piezometric conditions were disclosed within an aquifer sand and gravel stratum which had a major impact on design studies for the cofferdam and drydock. In the vicinity of the drydock, the piezometric elevation ranged from about 25 to 35 ft (7.6 to 10.7 m) above mean sea level. Soil engineering properties of the major strata are summarized and aquifer piezometric head contours are shown.

The site investigation focused on defining groundwater conditions in the aquifer and determining its response characteristics to pumping. Specially designed, remote reading, dual-sensing piezometers, were installed to monitor piezometric heads in the aquifer over water. Their installation is summarized and instrument details are included. The use of heavy-weight drilling mud to control artesian flow during drilling and sampling is described.

A 12-in. (30.5 cm) diameter test well was installed on land near the shore, opposite the drydock location. Two pump tests were conducted. With a moderate pumping rate from a single well on shore, significant artesian pressure relief was achieved within the aquifer throughout the future drydock area. Data on aquifer piezometric levels before and during the pump testing are shown. Computed permeability and transmissivity are presented, along with plots of drawdown with time and distance.

It is concluded that the glacial till and underlying aquifer sands and gravels have high strength and low compressibility, providing excellent foundation soils for the drydock. With the ability to lower the aquifer piezometric levels demonstrated by the pump tests, it is established that drydock construction at this geologically challenging site is technically feasible.

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Introduction

This paper describes the results of geotechnical field investigations undertaken for the design of a drydock for Trident class submarines at the Naval Submarine Base, Bangor, Bremerton, Washington. The drydock is part of a major construction program undertaken in connection with the development of a refit and training facility to support the U.S. Navy's Trident class submarine. The site is located on the Hood Canal in northwestern Washington, as indicated in Figure 1.

The site investigation program included test borings, field instrumentation and testing, pumping tests and geophysical surveys. Earlier test borings at the site had disclosed an aquifer with unusually high artesian head conditions, which caused concern for the feasibility of designing and constructing the proposed drydock. The field investigations gave particular emphasis to defining the groundwater conditions and the response of the aquifer to pumping. A special piezometer was developed and utilized for measuring piezometric heads offshore.

This paper is one of a series written on the geotechnical engineering aspects of the Trident drydock design and construction. Cellular cofferdam design, construction and performance for the facility, are reported elsewhere (2 and 3).

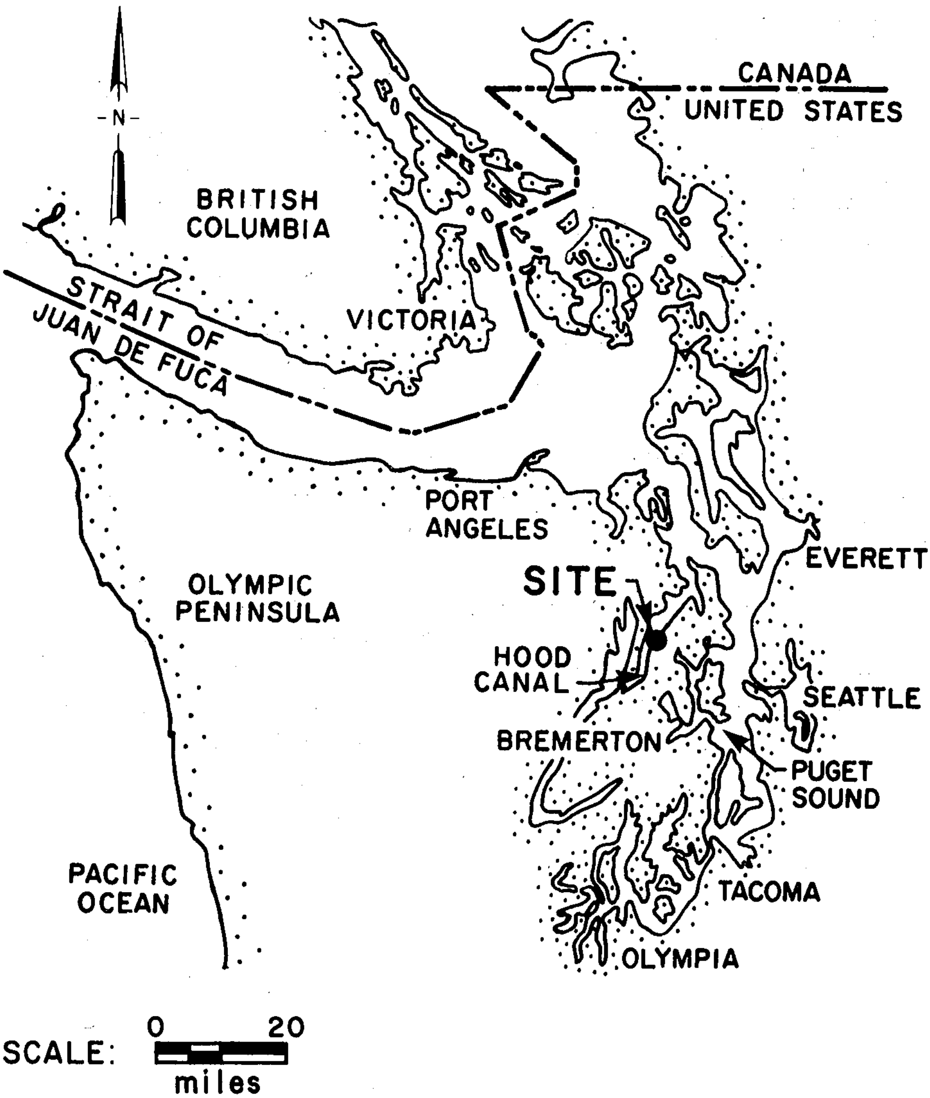

Fig. 1. Project Locus

Facility Description

The drydock, together with the refit piers, form what is termed the Trident Refit Delta, as indicated in Figure 2. Access from land to the Refit Delta is provided by two trestles, one each at the north and south ends. The drydock, which is oriented approximately parallel to the shoreline, is located from 400 to 700 ft. (122 to 213 m) offshore.

As shown on Figure 2, the existing ground surface within the Hood Canal drops in a westerly direction over the drydock site and varies from El. -25 ft (-7.6 m) to El. -62 ft (-18.9 m). Project datum, El. 0, is MLLW. (Refer to Appendix II for explanations of symbols and abbreviations). EHW is at El. 14.6 ft (4.5 m) and MTL at El. 6.4 ft (1.9 m).

The drydock was designed to be constructed at this offshore location in-the-dry within a deep steel sheetpile cellular cofferdam. The subgrade elevation for the drydock floor ranged from El. -61 to El. -64 (-18.6 to -19.5 m) which is approximately 75 ft (22.9 m) below the Hood Canal water surface at MHHW. The required excavation depth for the drydock floor ranged from 2 to 36 ft (0.6 to 11 m) below the ground surface of the Canal.

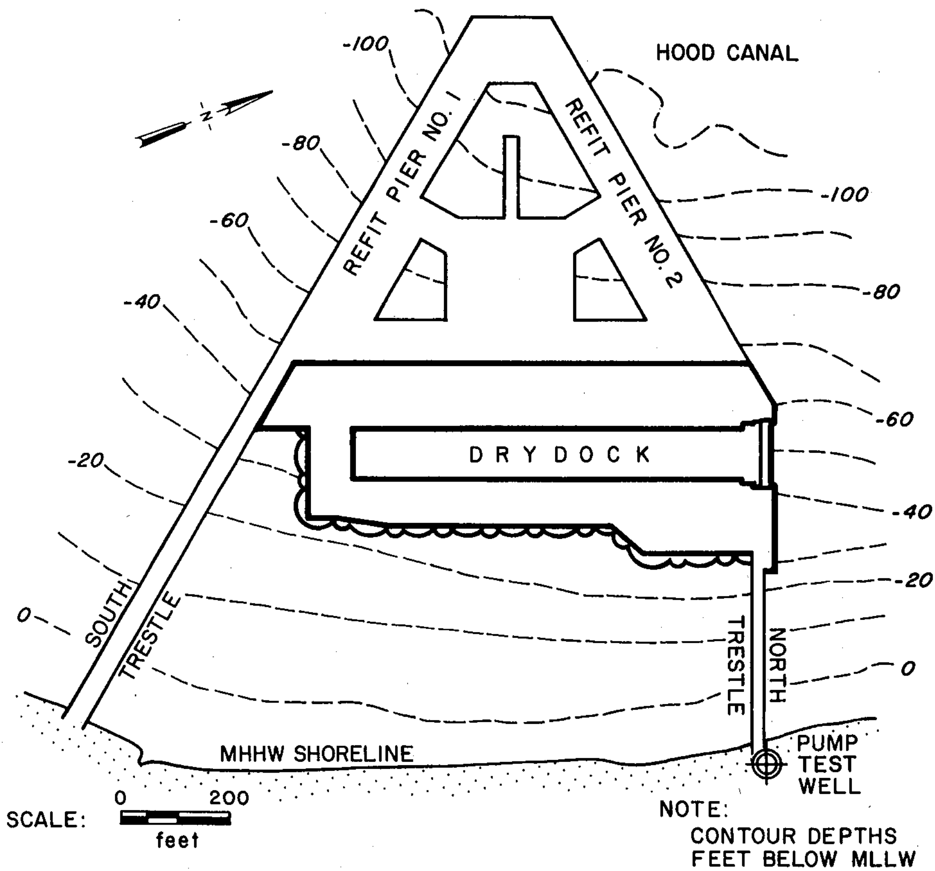

Fig. 2. Site Plan

SITE GEOLOGY AND PLANNING FOR EXPLORATIONS

The typical subsurface soil profile in a direction perpendicular to the shoreline is shown in Figure 3. For the project, the most significant strata are the glacial till and aquifer sands and gravels. The glacial till and other near surface, relatively impervious strata on land provide confinement for high artesian heads in the underlying aquifer. The artesian aquifer was revealed by earlier test borings, which encountered elevation heads over 50 ft (15.2 m) above the low tide level (MLLW) in the Hood Canal. Under artesian pressure, soil and water were blown up the drill casings, in a continuous shower. An example of cobble-sized materials which were washed up on to the drill barge through the drill casings is shown in Figure 4.

Due to these high artesian conditions, there was concern regarding the feasibility of design and construction of a drydock at the site. Would it be possible to reduce pressures within the aquifer sands and gravels to permit site dredging, drydock floor subgrade excavation and construction of the drydock in-the-dry? Could a permanent pressure relief system be designed for in-service drydock conditions to limit uplift pressures on the facility and to provide for adequate seismic stability of the offshore slope upon which the Refit Delta was to be built? What effect would temporary and permanent pressure relief have on water supply wells which were installed within the aquifer at locations on land?

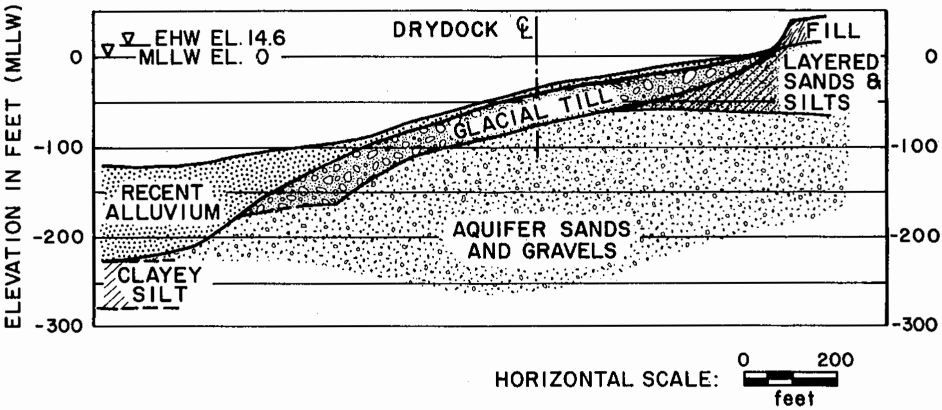

Fig. 3. Generalized Soil Profile Perpendicular to Shoreline

Fig. 4. Cobbles Washed Up Drill Casings By Artesian Flow

A major emphasis of the drydock subsurface exploration program was the definition of piezometric elevations and groundwater flow characteristics within the aquifer in addition to providing other soil and groundwater information required for project design. Along with a comprehensive test boring program, pumping tests were conducted to determine the permeability, transmissivity and other characteristics of the aquifer related to groundwater flow and technical feasibility for artesian pressure relief.

Field Investigations

Test Borings

A total of 57 test borings were drilled at the site from November 1974 through May 1975 by Warren George, Inc. of Jersey City, New Jersey. The borings included 17 drilled on land and 40 drilled within the Hood Canal throughout the Refit Delta area in water depths ranging from 10 to 120 ft (3 to 37 m). Field explorations for design of the two pile supported refit piers and the south trestle were performed by others and are not reported herein. Four types of borings were conducted for the drydock design studies:

- Type A - Borings designed to obtain pore pressure and flow rate data in strata exhibiting artesian pressures. These borings were cased full depth and drilled with water. A total of 31 such borings were made. Substantial soil disturbance was anticipated in these borings owing to artesian flow up the drill casings. However, in order to obtain the necessary information on the aquifer, blow count data and sample quality were deliberately sacrificed.

- Type B - Uncased borings designed to obtain good quality soil samples and blow count data. Heavy-weight drilling mud, ranging from 90 to 105 lb. per cu. ft. (1.44 to 1.68 kg. per cu. cm), was used to counterbalance artesian pressures and to prevent borehole collapse and soil disturbance. Drilling mud consisted of a mixture of bentonite or attapulgite clay and barium sulphate, used to control the unit weight. Drilling mud returning from the borehole was processed through a screening device ("shale shaker") to remove sand and gravel particles. Bentonite was used for the borings on land where fresh water was used to mix the mud. Attapulgite was used offshore where salt water was used in the mix. A total of 20 Type B borings were drilled.

- Type C - Conventional shallow, cased borings drilled with water and terminated in or slightly below the glacial till deposits which overlie the aquifer. Falling head permeability tests were made in these borings. Five such borings were made.

- Type G - Uncased borings drilled on land, with steel casings grouted into the completed boreholes for use in geophysical seismic testing. A total of six such borings were made.

Soil samples were normally obtained by driving a 2-1/2 in. (6.4 cm) I.D. split-spoon sampler with a 300 lb. (136 kg) hammer falling 30 in. (76.2 cm). The larger than normal split-spoon and relatively heavy hammer were used due to the dense and coarse nature of soils at the site.

All completed test borings were sealed with cement grout to prevent flow of artesian groundwater.

Soil Instrumentation

Soil instrumentation consisted of piezometers installed within completed boreholes, primarily in the aquifer sands and gravels. In addition to providing a measure of static groundwater levels, the piezometers were used to monitor the effects of test pumping. A total of 24 piezometers were installed on land and over water.

- "Dual-Sensing" Piezometers Within the Artesian Stratum

  Over water, 11 dual-sensing piezometers were installed within the aquifer sands and gravels. The dual-sensing piezometer is a unique instrument developed especially for the project. The design allowed all offshore piezometers to be read remotely from a central location on shore. A typical installation is shown in Figure 5a. These piezometers utilized a specially designed and fabricated head assembly to provide for two independent methods of measuring water pressure. The head assembly, attached by a diver to the piezometer riser pipe near the mudline, included both a pneumatic pressure transducer and a simple hydraulic tubing connection. A detail of the dual-sensing head is shown in Figure 5b. Use of the sensing head at the mudline was possible because of the permeability and high flow rates within the aquifer.

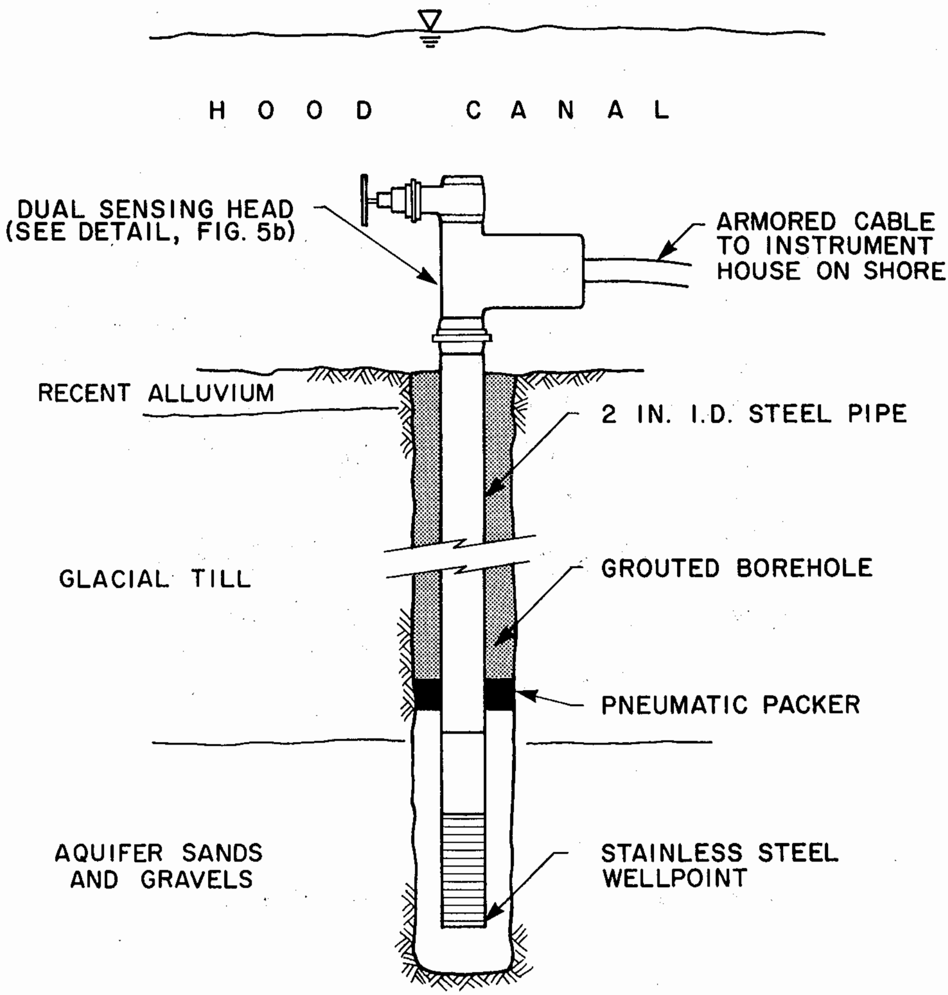

Fig. 5a. Dual-Sensing Piezometer Installation

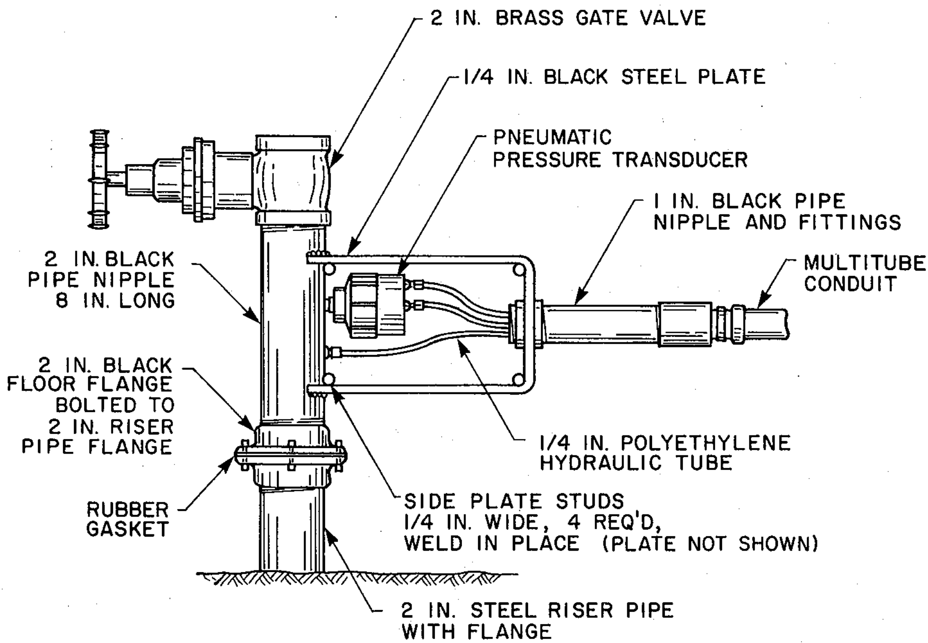

Fig. 5b. Detail of Dual-Sensing Piezometer Head

The instrument readout cables, encased in armored conduit, were laid on the bottom of the Hood Canal and connected to the instrument house on shore. Cable runs of up to 1200 ft (366 m) were required.

The piezometers were installed in Type "B" holes, which were advanced using heavy-weight drilling mud. This installation is highly unusual, but was made possible by taking advantage of the artesian groundwater conditions. At each location, the wellpoint, riser pipe and packer were installed into the mudded borehole and grouting performed. After grout set-up, the wellpoint and riser pipe were purged with water until artesian flow was generated. The resulting continuous flow cleansed the piezometers of the drilling mud and all units performed satisfactorily. After purging, all remaining drilling tools and casing above the mud line were removed. A diver then threaded the drill casing stickup at the mud line and affixed the bottom flange. The dual head assembly was then attached. The valve at the top of the unit shown in Figure 5b was opened during the attachment process to facilitate its assembly under active flow conditions. After assembly the valve was closed.

In addition to providing two semi-independent methods of measurement, the piezometer unit was designed to permit relatively simple replacement of the sensors if malfunction occurred. If necessary, the entire head assembly or components could be replaced by a diver. This resulted in considerable cost savings to the field work since the drilling contractor did not have to include contingencies for drilling supplemental boreholes to replace conventionally installed piezometers if malfunctions occurred.

All 11 piezometers operated satisfactorily throughout the 5-1/2 month period of use. Only one temporary malfunction occurred when a drilling barge spud became entangled in a cable and dislodged one of the dual head assemblies. The unit was quickly replaced by a diver and functioned properly thereafter.

In each case, the pneumatic and hydraulic piezometer readings showed excellent agreement. During portions of the pumping tests described later, water levels were drawn down below the level of the readout station and readings were provided by the pneumatic sensors only.

- Pneumatic Piezometers in Glacial Till

Three pneumatic piezometers were installed over water in shallow boreholes terminated within or just below the glacial till stratum. The piezometers were installed within conventionally drilled boreholes and sealed with bentonite pellets.

- Hydraulic Piezometers in the Artesian Stratum

Hydraulic piezometers were installed on land at ten borehole locations. Packers and cement grout seals were provided in a manner similar to that shown for the dual-sensing piezometers.

At locations where the piezometric head was above ground surface level, readings were made by means of a pressure gauge mounted at the top of the riser pipe. At other locations, or when heads were drawn down due to pumping, water levels were determined by soundings.

Measured elevation heads at all piezometers, both on land and over water, were influenced by tidal changes. Piezometric data were adjusted to a base corresponding to tide El. 0, using correlations established separately for each instrument. Piezometer fluctuations as a fraction of tide variation ranged from nearly 1.0 for the pneumatic units installed in the glacial till over water, to an average of about 0.6 for the aquifer piezometers over water, to about 0.3 for the hydraulic piezometers on land nearest the shoreline.

Pump Tests

The high artesian pressures within the aquifer sands and gravels were recognized as having significant impact on the design and construction of the proposed drydock. Accordingly, a test well was installed on land into the artesian stratum in order to conduct pumping tests. The well consisted of a 12-in. (30.5 cm) diameter casing with a 40-ft (12.2 m) long, 10-in. (25.4 cm) diameter wellscreen installed within the aquifer. The well location is shown on Figure 2 and a profile of the test well is shown in Figure 6.

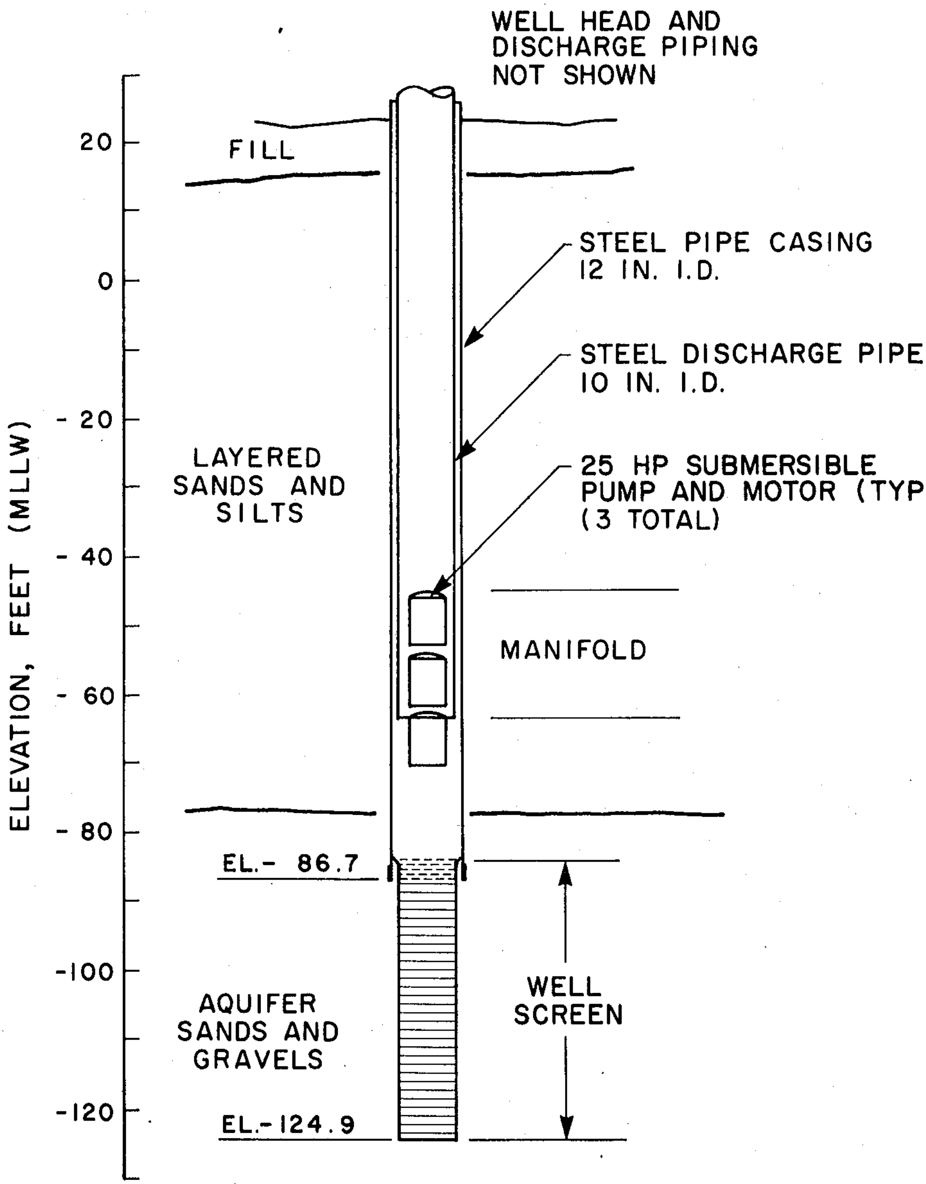

Fig. 6. Profile of Pump Test Well

Geophysical Survey

Geophysical measurements were made to develop information on in-situ soil properties needed for possible dynamic analyses of the drydock structure. The program included the measurement of both compressional and shear wave velocities of subsurface materials on land, by crosshole and uphole techniques, using steel casings grouted into the Type G boreholes. Seismic refraction and reflection measurements were also made over-water within the drydock area for purposes of correlating soil properties on land and over water.

Soil Conditions

A complex sequence of soil types and stratigraphy exist at the site. Distinct boundaries between adjacent soil zones are often not present. Although relatively simplified titles have been given the major soil zones, a zone may represent a complex interbedding of various soil types. Soil conditions at the site are illustrated in the generalized profile in Figure 3. Following are comments on geologic origins and descriptions of the major soil zones, in order of increasing depth below ground surface. Selected properties of the glacial till and aquifer sands and gravels are listed in Table I and grain size curves are shown in Figure 7.

- Recent Alluvium

  Recent alluvium forms the uppermost stratum of soils underlying the Hood Canal. The alluvium consists of very loose to medium compact gray sands and gravels with traces

TABLE 1

SELECTED CLASSIFICATION AND ENGINEERING PROPERTIES
OF MAJOR SOIL STRATA

PROPERTY	GLACIAL TILL	AQUIFER SANDS AND GRAVELS
Water Content, percent	6 - 21	-
Liquid Limit, percent	15 - 25	Non plastic
Plastic Limit, percent	9 - 17	
Penetration Resistance, blows per foot	50 - 200 (1)	70 - 400 (1)
Permeability, x 10^{-4} feet per minute	2 - 67 (2)	150 (3)
Angle of Internal Friction, degrees	-	43 (4)
Compression Wave Velocity, feet per second	-	6500 - 7500 (5)
Shear Wave Velocity, feet per second	-	1300 - 2200 (5)

NOTES:

(1) 2-1/2 in. I.D. sampler, 300-pound hammer, 24 in. drop.

(2) From borehole permeability tests.

(3) Based on pump test analysis.

(4) From triaxial tests on reconstituted samples.

(5) Cross-borehole seismic refraction tests.

(6) 1 ft = 0.3 m

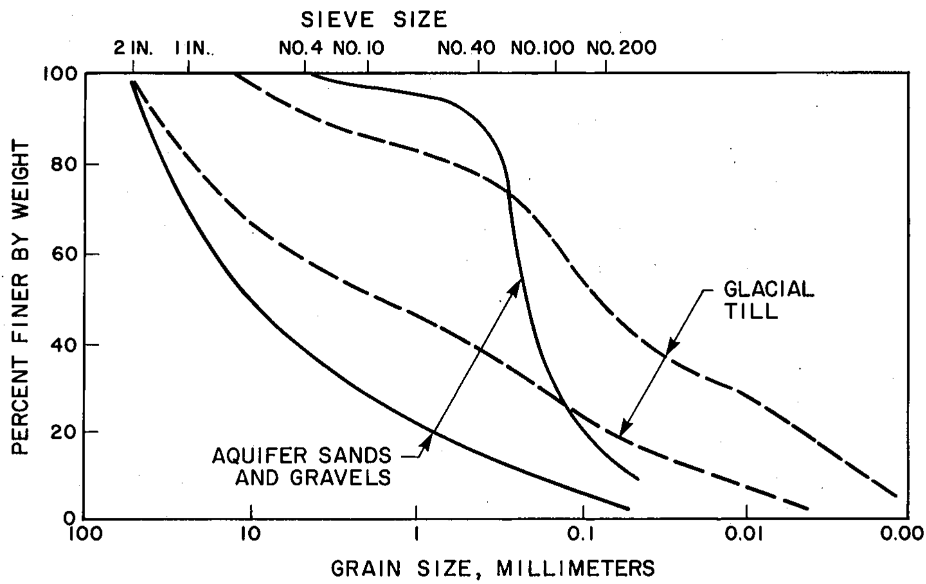

Fig. 7. Range of Grain Sizes: Glacial Till and Acquifer Sands and Gravels

of silt, shell fragments, decomposed wood, and occasional small cobbles. Generally, the alluvium is encountered as a layer of fine sand with an underlying, thinner deposit of sandy gravel. Throughout the immediate area of the drydock, this stratum does not exceed a 10-ft (3.0 m) thickness. Its thickness increases toward the west, and over 100 ft (30.5 m) of the deposit was encountered beyond the apex of the Refit Delta. Penetration resistance in the recent alluvium generally ranges from 2 to 30 blows per foot.

- Glacial till

Glacial till was encountered directly below the recent alluvium offshore. The glacial till provides confinement for artesian groundwater within the underlying aquifer sands and gravels offshore. The till occurs primarily as a very compact gray silty coarse to fine sand to sandy silt, with trace to little coarse to fine gravel and traces of clay. Pockets of silt, sand, or gravel were frequently encountered within the stratum. Penetration resistances in the till ranged from approximately 50 to 200 blows per ft (0.3 m). Obstructions to drilling were encountered in only one boring in the stratum, indicating that it was relatively free of cobbles and boulders.

Throughout the drydock area and most of the Refit Delta, the till layer is relatively uniform in thickness, ranging from about 20 to 40 ft (6.1 to 12.2 m). As shown on the soil profile, the glacial till reduces markedly in thickness and pinches out beyond the limits of the Refit Delta. Also, the glacial till layer thins out in a direction approaching the shore from the drydock location. The absence of the glacial till in areas is attributed to past wave erosion in the Hood Canal during post glacial periods of lower sea level.

- Layered Sands and Silts

This zone is the most variable soil unit and consists of interbedded deposits of medium to very compact sands and gravels and stiff to hard gray silt with layers and lenses of silty fine sand and traces of organic material and shell fragments. Together with the glacial till, the silt layers form the aquiclude to the major artesian aquifer on land. Pervious strata within the deposit also exhibit artesian pressures. The layered sands and silts range from about 80 to 180 ft (24.4 to 54.9 m) in thickness within the limits of the boring program on land. The thickness of the sands and silts decreases in a westerly direction, and beyond about 300 to 400 ft (91.4 to 121.9 m) from shore, the stratum is not encountered. Penetration resistances are variable and range from 30 to over 100 blows per ft (0.3 m).

- Aquifer Sands and Gravels

Aquifer sands and gravels, confined beneath the glacial till and layered sands and silts strata are relatively pervious granular soils which conduct groundwater with high artesian pressures. Artesian conditions within the study area are the result of groundwater flow from higher elevations on land. The aquifer is recharged from infiltration, where the confining strata are not present. This deposit is the water supply aquifer at the base.

These materials consist of interbedded, very compact gray sands and gravels, with occasional lenses and layers of silt and traces of organic material, clay and shell fragments. The aquifer likely contains pockets and layers of openwork gravel and cobbles.

Gradation tests showed that the sands and gravels have a relatively significant content of fines, with the silt content generally ranging from 5 to 15 percent. Penetration resistance typically varies between 50 and 150 blows per ft (0.3 m) and is influenced by the coarse gravel and cobbles present.

Only a limited number of test borings were sufficiently deep to penetrate through the aquifer. It is believed that near the Refit Delta, this stratum varies from about 200 to 250 ft (61 to 76 m) in thickness, with the bottom of the aquifer at approximately El. -270 ft (-82.3 m) to El. -310 ft (-94.5 m).

- Clayey Silt

It appears that a clayey silt stratum forms the lower boundary of the aquifer sands and gravels. Due to its depth, the material was encountered in only a few borings located far offshore near the apex of the Refit Delta. The deposit consists of interbedded hard silts and clays with occasional layers of sand and gravel. The top of the silt was encountered at approximately El. -300 ft (-91.4 m).

The depth to bedrock at the site, extrapolated from the map of Hall and Othberg (1), is in the order of 2,000 to 2,400 ft (610 to 731 m) below sea level. The deepest borings known to have been made on the Navy base, which are test wells for water supply studies, extended to approximately El. -1000 ft (-304.8 m) without encountering bedrock.

Groundwater Conditions

Aquifer piezometric elevations within the site area are presented in Figure 8. Contour data shown were obtained on a specific date and are representative of the piezometric levels measured throughout the period of observations. The contours indicate a relatively steady decline in elevation head in a direction from the land toward the proposed drydock. The artesian heads range from El. 30 ft (9.1 m) to El. 40 ft (12.2 m) over almost the entire refit delta complex and then drop rapidly to insignificant levels near the apex of the delta. These data reveal that groundwater is flowing within the aquifer across the site, in a direction approximately perpendicular to the shoreline, with essentially complete venting near the apex of the Refit Delta.

Although there is considerable variation within the data from the Type A borings due to localized soil conditions and testing influences, there is a definite trend of increasing piezometric level with depth at individual borings as shown in Figure 9.

Measured elevation heads within the mid-depth of the glacial till at the three pneumatic piezometers over water, when adjusted to correspond to tide El. 0, ranged from El. 5 ft (1.5 m) to El. 9 ft (2.7 m). This indicated that most of the head loss due to upward seepage occurred near the bottom of the till.

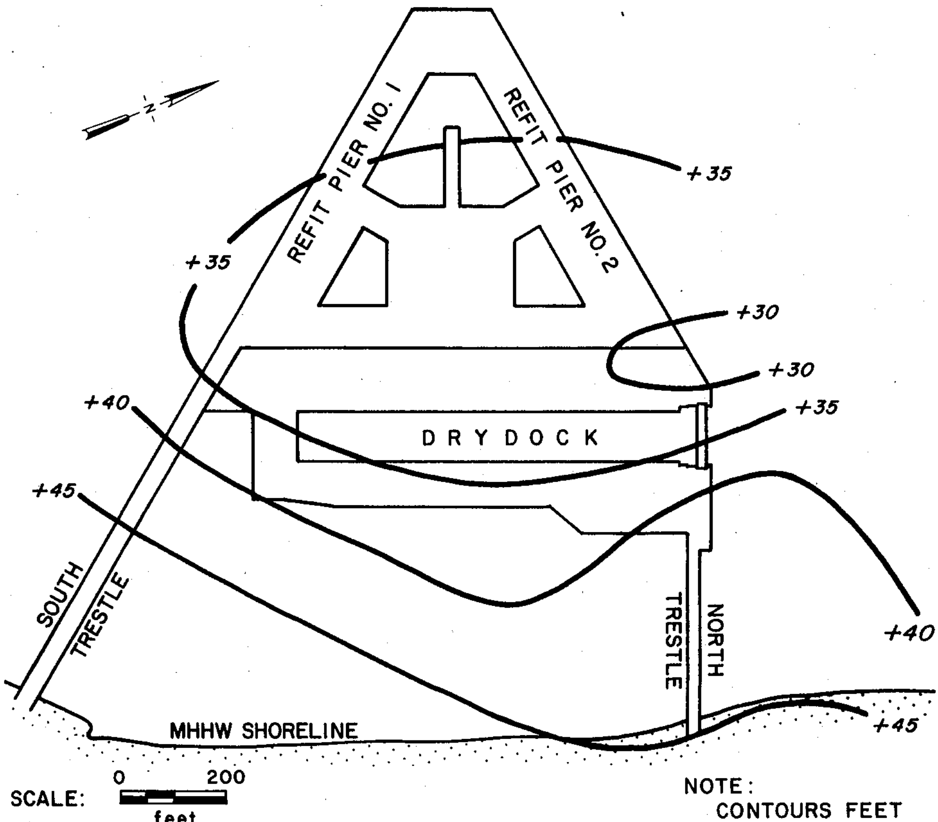

Fig. 8. Aquifer Piezometric Elevations Prior to Pump Test

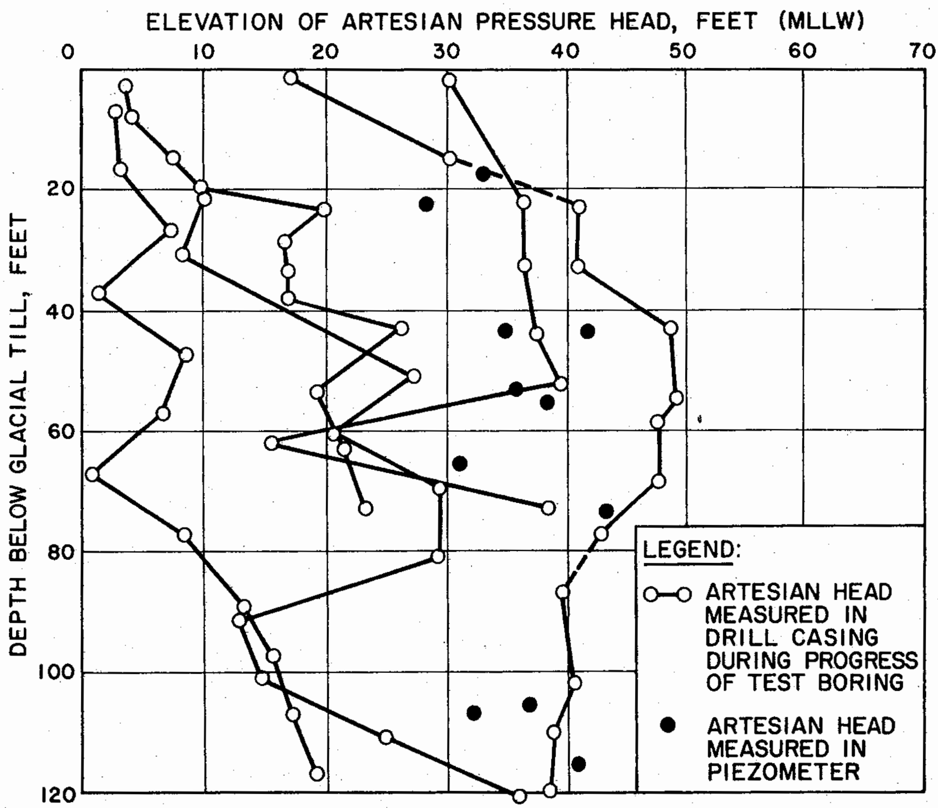

Fig. 9. Piezometric Level with Depth Below Glacial Till

While drilling Type A borings within the aquifer sands and gravels, water flowed continuously from the boreholes under artesian pressure. Flow rates varied and were a function of the working elevation from which the drilling was conducted, local stratification and characteristics of the aquifer at the borehole, tide level and pumping activities at the site. Measured flow rates generally varied from 1 to 100 gal per min (3.8 to 378 liters per min) and ranged up to a maximum of 350 gal. per min (1323 liters per min) from 6 in. (152 mm) diameter casings.

Results of Pump Tests

Two pump tests were performed during the field work. The first, test 1A, was conducted during a 5-day period in December 1974. It was performed to obtain initial data on the feasibility of aquifer pressure relief after the first few piezometers were installed offshore. The more comprehensive test, 1B, was of 15-days duration and performed in April 1975. This test is described below, and summary data for both tests are provided in Table 2.

By the time of test 1B, 11 piezometers had been installed within the aquifer over water throughout the Refit Delta area. Numerous other piezometers and observation wells, located on land, were also monitored.

Aquifer piezometric conditions at the completion of pumping are shown in Figure 10. The aquifer piezometric levels within the drydock area were reduced from initial values ranging from El. 30 ft (9.1 m) to El. 40 ft (12.2 m), to

TABLE 2

SUMMARY DATA FOR PUMP TESTS

ITEM	PUMP TEST 1A	PUMP TEST 1B
Duration of pumping, days	5	15
Average flow rate, gallons per minute	1,200	1,865
Maximum well drawdown, feet	48	96
Average aquifer transmissivity, sq. ft. per day	6,550	5,350

1 gal = 3.78 l

1 ft = 0.3 m

1 sq. ft = 0.093 sq. m

values less than El. 5 at the northeast corner and about El. 10 ft (3.0 m) near the southeast corner. Thus, with one well pumping onshore, the piezometric head at the drydock was reduced by magnitudes ranging from approximately 30 to 40 ft (9.1 to 12.2 m).

Figure 10 shows that an asymmetric distribution of piezometric head developed about the well during the test. The lower piezometric heads at given distances from the well in the southwesterly direction reflect a zone of apparently higher average permeability than elsewhere. The piezometric head distribution to the east of the test well reflects the higher initial heads in that area and recharge effects resulting from the normal, westerly flow of artesian water within the aquifer. Likewise, boundary effects due to natural discharge of water from the aquifer to the west are indicated.

Data on drawdown and recovery (residual drawdown) as a function of time for selected piezometers are presented in Figures 11 and 12, respectively. The data were chosen to illustrate the piezometric effects that occurred throughout the region, varying from positions close to the well to the extreme south-westerly corner of the drydock. Drawdown versus distance relationships for piezometers over water are presented in Figure 13.

It is seen, in Figure 11, that very rapid piezometric head reductions occurred. For example, almost 8 ft (2.4 m) of pressure relief developed at a piezometer 280 ft (85.3 m) from the well, within six minutes from the beginning of

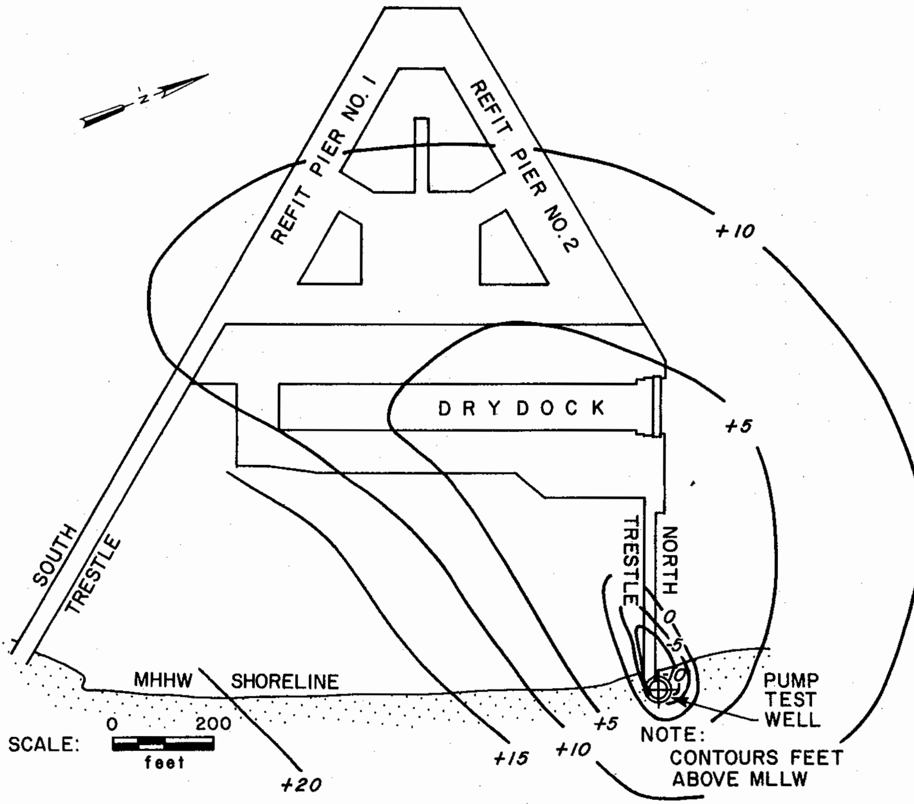

Fig. 10. Aquifer Piezometric Conditions
During Pump Test

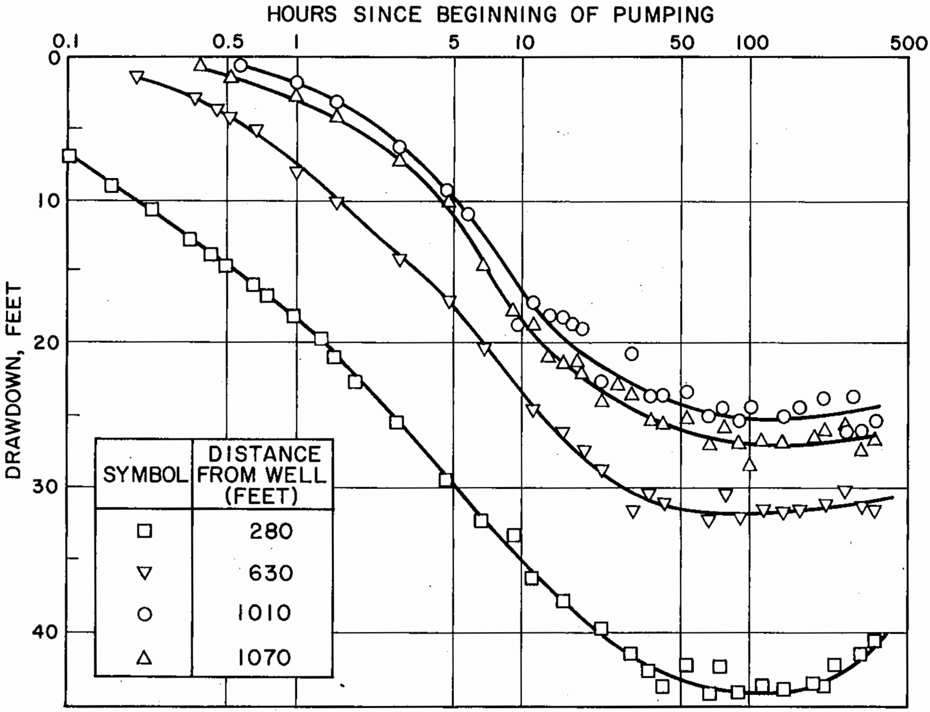

Fig. 11. Drawdown Versus Time During Pump Test

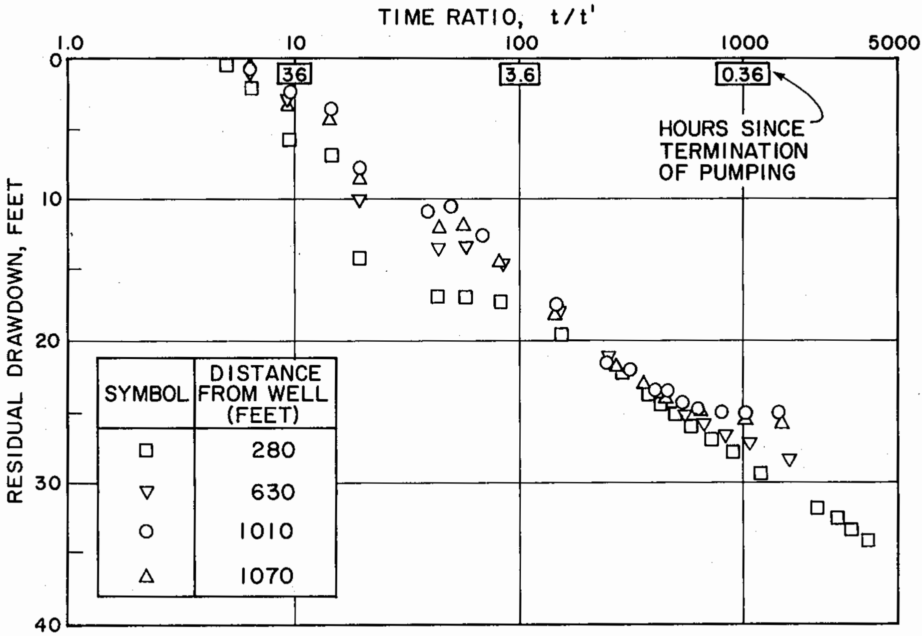

Fig. 12. Residual Drawdown (Recovery) Versus Time Ratio After Completion of Pumping

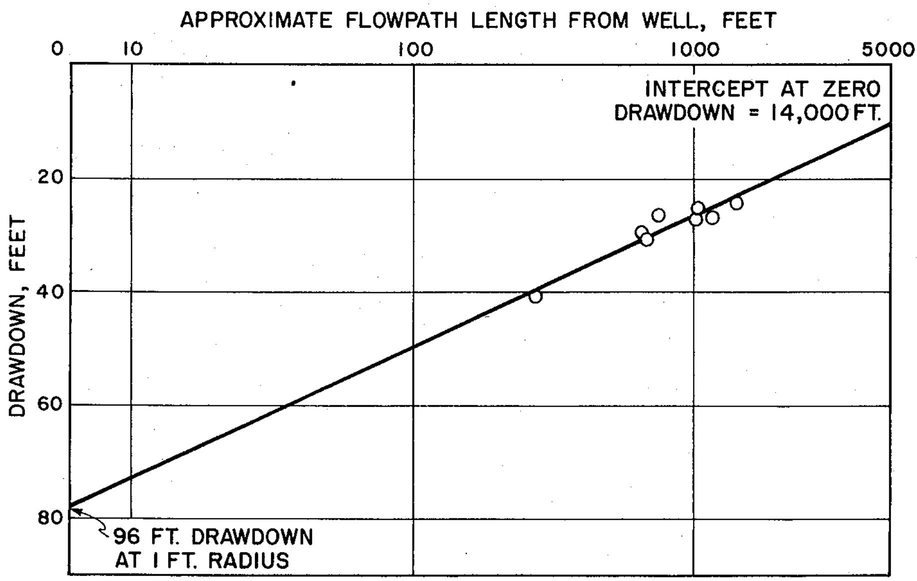

Fig. 13. Drawdown Versus Distance

pumping. This same pressure relief developed within three hours at piezometers over 1000 ft (304.8 m) from the well near the southwest corner of the drydock. Within approximately 50 hours, the four piezometers achieved their maximum drawdown.

Figure 12 shows that piezometric heads at the four piezometers recovered very rapidly. By t/t' = 10, which was 36 hours after completion of the pumping, the residual drawdown had decreased to less than 4 ft (1.2 m) for the piezometers 630 to 1070 ft (192 to 326m) from the well and to approximately 6 ft (1.8 m) at a distance of 280 ft (85.3 m) from the well.

The drawdown-distance relationship presented in Figure 13 was prepared on the basis of the approximate flow path length of the indicated piezometers from the well rather than straight-line distance. This was done to account for the significantly asymmetric distribution of flow toward the well, which is illustrated in Figure 10. The theoretical "radius" of influence in the southwesterly direction at the completion of pumping is 14,000 ft (4,267 m).

Computed transmissivity values from the test ranged from a low of approximately 3,200 to a high of about 8,000 sq. ft. per day (297 to 743 sq. m. per day). An average transmissivity of 5,350 sq. ft. per day (497 sq. m. per day) was chosen for design. Assuming an aquifer thickness of 240 ft (73.2 m) resulted in a calculated average permeability of 150×10^{-4} ft. per min (76×10^{-4} cm per sec.). This corresponds to the permeability of a very fine sand. Thus, even though the stratified aquifer contains significant amounts of coarse sand and gravel, its overall characteristics from the standpoint of seepage and flow are that of a relatively fine-grained cohesionless deposit. Computed aquifer storage coefficients ranged from 0.0004 to 0.0010, which are indicative of artesian conditions.

Conclusions Relative to Feasibility of Artesian Pressure Relief

Pump test 1B disclosed that, with a moderate pumping rate from a single well on shore, significant artesian pressure relief could be achieved within the aquifer throughout the delta area. Pressure reductions occurred rapidly throughout the aquifer both on land and over water. With two exceptions, all piezometers within the aquifer over water stabilized within approximately two days after commencement of pumping. Two piezometers stabilized within 5 to 7 days. Analyses of the test data disclosed that, from the standpoint of flow, the soils within the artesian aquifer exhibited an overall average permeability corresponding to that of a very fine sand.

Pump test 1B confirmed that it would be technically feasible, for both construction and in-service conditions, to relieve the artesian pressures at the drydock site. Unpublished studies by others showed that construction and in-service dewatering would not have an adverse impact on the base water supply.

Conclusions Related to Other Geotechnical Aspects of Design

The field and laboratory data obtained for the glacial till and aquifer sands and gravels disclosed both deposits to be very compact and to exhibit high strength and low compressibility. It was concluded that the soils were excellent foundation materials for the drydock and for the cellular cofferdam required for drydock construction. While being very compact, the glacial till was not considered so dense in-situ as to require blasting for dredging. The apparent limited quantity of cobbles and boulders within the till was also considered favorable from the standpoint of sheet pile driving for the cofferdam.

Summary

Geotechnical investigations at the Trident drydock site disclosed complex geologic conditions. Compact soil deposits ranging from relatively clean sands and gravels to a silty sand to sandy silt glacial till to hard silts and clays were encountered. Piezometric conditions were disclosed within an aquifer sand and gravel stratum which had a major impact on design studies for the drydock. In the vicinity of the drydock, the piezometric elevation ranged from about 25 to 35 ft (7.6 to 10.7 m) above mean sea level.

In addition to defining soil conditions, the site investigation focused on defining groundwater conditions in the aquifer and determining response characteristics of the deposit to pumping. Specially designed and fabricated dual-sensing piezometers, utilizing both pneumatic and hydraulic sensing methods, were installed to monitor piezometric levels in the aquifer over water.

A 12-in. (30.5 cm) diameter test well was installed on land near the shore, opposite the drydock location. Two pump tests were conducted. With a moderate pumping rate from a single well on shore, significant artesian pressure relief was achieved within the aquifer throughout the future drydock area.

Data from the borings disclosed the glacial till and underlying aquifer sands and gravels to be of high strength and low compressibility, thus providing excellent foundation soils for the drydock. With the ability to lower the aquifer piezometric levels demonstrated by the design phase pump tests, it was established that drydock construction at this geologically unique site was technically feasible.

Acknowledgements

The Trident Drydock facility was planned and constructed for the Department of the Navy, Commander, Naval Facilities Engineering Command through the Office in Charge of Construction, Naval Facilities Engineering Command Contracts, Trident. The project design was by Fay, Spofford & Thorndike, Inc., Boston, Massachusetts. The geotechnical consultant was Haley & Aldrich, Inc. Cambridge, Massachusetts. Warren George, Inc., Jersey City, New Jersey drilled the test borings and installed the instrumentation. Moretrench American Corporation, Rockaway, New Jersey installed the test well and conducted the pumping tests. Weston Geophysical Research, Inc. of Westboro, Massachusetts performed the geophysical work.

The authors wish to express their appreciation to the engineers of the Office in Charge of Construction, Naval Facilities Engineering Command Contracts, Trident for their contributions toward the successful completion of this phase of the project. Special thanks is given to Mr. Max D. Sorota of Fay, Spofford & Thorndike, Inc., Messrs. Robert G. Lenz and Patrick Powers of Moretrench American Corp. and Mr. Frank Gregory of Warren George, Inc.

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Appendix II — Notations

EHW	-	Extreme High Water
El.	-	Elevation
MLLW	-	Mean Lower Low Water
MHHW	-	Mean Higher High Water
MTL	-	Mean Tide Level
t	-	Elapsed time since beginning of pumping from the test well
t'	-	Elapsed time since end of pumping from the test well

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