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JOURNAL OF THE
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BEYOND THE CODE OF PROFESSIONAL ETHICS

By Robert L. Meserve* Member

(Presented at BSCE-ASCE Student Night, Northeastern University, November 1, 1972.)

It is a pleasure to join you on Student Night, which brings together student chapters and members of two of the oldest professional engineering societies in the country: the Boston Society of Civil Engineers, founded in 1848, and the American Society of Civil Engineers, founded in 1852. The origin of engineering and technical societies[1] can be traced back a good deal further than the middle of the nineteenth century, actually to a group of learned friends in Naples, Italy, in the mid-sixteenth century. Led by della Porta, these gentlemen formed the *Academia Secretorum Naturae* in 1560. Membership qualifications were rather exclusive: each applicant was required to have made a discovery in the realm of natural science. A few decades later, at the turn of the century, another learned society was formed in Italy by Federigo Cesi, known as the *Academia dei Lincei* – its purpose was to direct "its labors diligently and seriously to studies as yet little cultivated". One of the early members was Galileo.

In 1657, the *Academia del Cimento* was initiated by Leopold de' Medici. The members of this organization, many of whom were disciples of Galileo, managed to meet for only ten years, but their experiments were published in a lasting memento. Torricelli, the physicist, and Borelli were among the members.

These early societies formed in Italy because of the flourishing of science in the Italian universities. The seventeenth century saw a gradual shift in the location of such interest, mainly to England and France. Attempts to start a learned society in England had been made in 1616 but were not fruitful until several of Bacon's followers who had been meeting informally for fifteen years were successful in obtaining a charter from King Charles II for the *Royal Society of London*. Investigations and publications of the Society became known throughout Europe. Their serious efforts were well enough known to be parodied by an interested and caustic Jonathan Swift.

*Associate Professor of Civil Engineering, Northeastern University

In France, Louis XIV granted pensions and funds for experimental work by the *Academie des Sciences* formed in 1666. Membership included such men as Pascal, Descartes, and Huyghens. Sir Isaac Newton was a foreign member of note. Proceedings were, as usual, published in Latin.

These, then, were some of the early beginnings – the roots from which all scientific, engineering, and technical societies have blossomed. There were many others: the *Academia Naturae Curiosorum* in Spain (1657), the *Collegium Curiosum* in Germany (1672), the *Academie Imperiale des Sciences* in Russia (1725), the *Royal Academy of Sciences* at Copenhagen (1742), and the *American Philosophical Society*, founded by that most interesting and enthusiastic gentleman, Benjamin Franklin, in 1743.

All of them had rules and regulations. Some must have developed codes of conduct as well. Even the most primitive tribes of men have had at least an unwritten tradition of conduct which has governed much of their individual and group action. One of the oldest written codes is the Ten Commandments given to the world through Moses. Another ancient code is that recorded by Hippocrates in the 4th century B.C., still taken as a vow by graduating doctors of medicine .

OATH OF HIPPOCRATES

I swear by Apollo, the Physician, and Aesculapius and Health and All-Heal and all the Gods and Goddesses that, according to my ability and judgment, I will keep this oath and stipulation:

To reckon him who taught me this art equally dear to me as my parents, to share my substance with him and relieve his necessities if required: to regard his offspring as on the same footing with my own brothers, and to teach them this art if they should wish to learn it, without fee or stipulation, and that by precept, lecture, and every other mode of instruction I will impart a knowledge of the art to my own sons and to those of my teachers, and to disciples bound by a stipulation and oath, according to the Law of medicine, but to none others. I will follow that method of treatment which, according to my ability and judgment, I consider for the benefit of my patients, and abstain from whatever is deleterious and mischievous. I will give no deadly medicine to anyone if asked, not suggest any such counsel; furthermore, I will not give to a woman an instrument to produce abortion. With purity and with holiness I will pass my life and practice my art. I will not cut a person who is suffering with a stone, but will leave this to be done by practitioners of this work. Into whatever houses I enter I will go into them for the benefit of the sick and will abstain from every voluntary act of mischief and

> corruption; and further from the seduction of females or males, bond or free. Whatever, in connection with my professional practice, or not in connection with it, I may see or hear in the lives of men which ought not to be spoken abroad I will not divulge, as reckoning that all such should be kept secret.
>
> While I continue to keep this oath unviolated may it be granted to me to enjoy life and the practice of the art, respected by all men at all times but should I trespass and violate this oath, may the reverse be my lot.

Most modern-day professional societies have developed codes of ethics. Among engineers the codes are very similar, as was pointed out by Marvin Runyan[2] in the May 1972, issue of *Civil Engineering*. Of the ten codes he compared, seven contained clauses similar to the recently deleted Article 3 of the ASCE code: "It shall be considered unprofessional and inconsistent with honorable and dignified conduct and contrary to the public interest for any member of ASCE to invite or submit priced proposals under conditions that constitute price competition for professional services." (This article was deleted by ASCE in 1971 because of a Department of Justice civil antitrust suit; the society is now working on establishing guides for passage of state legislation which would prohibit competitive bidding for professional services through a Task Committee on Professional Services Selection. I should like to read quickly the remaining nine articles in the ASCE code. Engineers are expected to practice within this code or modifications of it for their entire professional careers.

ASCE CODE

> It shall be considered unprofessional and inconsistent with honorable and dignified conduct and contrary to the public interest for any member of the American Society of Civil Engineers:
>
> 1. To act for his client or for his employer otherwise than as a faithful agent or trustee.
> 2. To accept remuneration for services rendered other than from his client or his employer.
> 3. (Deleted)
> 4. To attempt to supplant another engineer in a particular engagement after definite steps have been taken toward his employment.
> 5. To attempt to injure, falsely or maliciously, the professional

reputation, business, or employment position of another engineer.

6. To review the work of another engineer for the same client, except with the knowledge of such engineer, unless such engineer's engagement or the work which is subject to review has been terminated.
7. To advertise engineering services in self-laudatory language, or in any other manner derogatory to the dignity of the profession.
8. To use the advantages of a salaried position to compete unfairly with other engineers.
9. To exert undue influence or to offer, solicit, or accept compensation for the purpose of affecting negotiations for an engineering engagement.
10. To act in any manner derogatory to the honor, integrity, or dignity of the engineering profession.

As is the case with most canons or codes of ethics, this code can lead to honest differences of opinion and various interpretations depending on an individual's viewpoint; seldom are breaches of the code cut and dried. E. Sherman Chase once wrote,[3] however, that many engineers had "elastic consciences" which permitted them to deviate periodically from a strict adherence to the code, a convenience certainly whenever the code interfered with maximum immediate benefits to such an individual.

The code obviously must be made up of broad generalizations implying specific rules of conduct. It would be an impossibility to list specifically all of the do's and don'ts important to maintaining conduct on a professional level. Further, some engineers would then feel more than they do now that any questionable practice not included in the written code must not then be considered a breach of professional conduct. Soul-searching by the professional is a frequent need in private practice, as it must be too for the doctor, the lawyer, the educator. When Dr. William C. White retired a few years ago as Vice President of Northeastern, it was said of him that throughout his career when confronted with decision-making he always asked himself "what is the *right* thing to do?" and having determined the right thing to the best of his ability went ahead and did it. The most significant part of any code is beyond the code – that portion that is unwritten – the very foundation of professional character, which cannot be legislated or regulated or licensed or coded, but which is the culmination of years of growing, maturing, comprehending, understanding and feeling. The development of professional character is a life-long process – it cannot be acquired by accepting a degree, or by passing a professional examination or by memorizing a code of ethics. It is the process by which young engineers grow into tall men.[4] Too often the drive for easy profit and prestige anaesthetizes moral fiber, and although the written code is elastically applied, the unwritten code is stretched to the yield point. That point is easier to reach the second time, and then the third, until finally the original moral fiber has no remaining strength.

This downward process, and the difficulty in getting back, seems to be appropriately illustrated in verse by Henry A. Beers:[5]

A Fish Story

A whale of high porosity,
And low specific gravity,
Dived down with much velocity,
Beneath the earth's concavity.

But soon the weight of water
Squeezed in his fat immensity,
Which varied – as it ought to –
Inversely as his density.

It would have moved to pity
An Ogre or a Hessian,
To see poor Spermaceti
Thus suffering compression.

The while he lay a-roaring
In agonies gigantic,
The lamp-oil out came pouring
And greased the wide Atlantic.

(Would we'd been in the Navy,
And cruising there! Imagine us
All in a sea of gravy,
With billows oleaginous!)

At length old million-pounder
Low on a bed of coral,
Gave his last dying flounder
Whereto I pen this moral.

Moral

O let this tale dramatic
Anent this whale Norwegian

And pressures hydrostatic
Warn you, my young collegian,
That down-compelling forces
Increase as you get deeper;
The lower down you course is,
The upward path's the steeper.

It is a paradox, perhaps, that a professional code of conduct – professional character if you will – cannot be taught but it can be learned. Many years ago, Daniel Webster Mead wrote[6] of the importance of professional character and how it is obtained:

> Character, while partially hereditary, is more largely due to the influence of family and associates, and to education and personal cultivation. That the age and thought of the times largely control character is undoubtedly true, yet no age or nation has been so degenerate that it has not developed some men of high ideals and of character creditable in any age. No high professional standing is ever attained without properly developed character; it is like the internal mechanism of an important machine, unseen but essential to the proper and correct exercise of its highest functions. It is the mainspring of . . . success and is susceptible of great modification and improvement by individual effort.

All engineers desire success – that nebulous concoction of professional accomplishment, meaningful creativity, monetary reward, recognition, exemplary performance. Maxwell Stanley listed the requirements for success[7] in his book *The Consulting Engineer.* The top five in his judgment, were:

1. Superior engineering talent
2. Scrupulous integrity
3. Skill in human relations
4. Ability in administration
5. Compatible home atmosphere.

It is interesting that three of these have largely to do with professional character. J. W. Frazier, managing partner in a Kansas consulting firm[8] also stresses character. When selecting a young engineering graduate for employment, he says that he is more interested "in his cleanliness, his character, his personality, and his professional attitude than in his collegiate grade point average or his grade on an EIT examination." In selecting an experienced engineering associate he is "more impressed by his honesty, integrity, and engineering accomplishments than in the number of 8-hr. examinations he has passed."

Although professional integrity cannot really be taught, it can be *shown.* Professors and practicing engineers must set good examples, not that they will be blindly copied, or even imitated at all, but it is from close contact with consistently exemplary attitudes that the young engineer can best learn about professional integrity and the unwritten code. The home influence, mentioned by Maxwell in connection with the fifth requirement for success, cannot be overemphasized. Too many parents are guilty of working with a double standard, one for themselves and one for their children. The same can be said for some educators who have one standard for themselves, another for students. The same can be said for some principal engineers – one standard for themselves, another for their younger employees. Gradually the young engineer may find that the boundary between ethical and unethical conduct is not clearly established. There is a gray area in between.[9] At times, apparently, since he has observed it in others, it is possible to engage in this gray area, conveniently overlooking or turning one's back on ethical behavior, to meet the moment's immediate demands in a more comfortable posture.

Such breeches in professional conduct are visible and common: political contributions by firms or individuals attempting to gain favorable position in competing for public contracts; bribes and kickbacks including the seemingly harmless variety of wining and dining potential clients; accepting gifts from contractors; cheating on examinations, time sheets, expense accounts; moonlighting at the expense of the prime employer; stealing ideas, methods, processes, and even people from other firms. Examples are endless of such unethical behavior on the part of individuals and firms who consider themselves paragons of professional virtue.

Some of the best advice for embryo engineers that I have seen recently was given in a paper by C. S. Hedges, published by ASCE.[10] "Be as particular as you can", wrote Mr. Hedges, "in the selection of your boss. In most engineering organizations, the influence of the senior engineer is a major factor in molding the professional character of young engineers . . . particularly during [the] first few years that constitute your engineering apprenticeship. No amount of precept is as effective as the proper kind of example."

If the good example of a senior engineer is superimposed on the prior good example of an outstanding professor, which in turn reinforces the good example of concerned and intelligent parents, it is possible, even in an unwell modern society, that these small doses of integrity in action will lead to a permanent upgrading of one's professional character.

I think the time for a man to scan his life, to view his own personal code, is when he is a young man, when changes can be made and right attitudes chosen. The good life is made by men of substance, not by hollow men.

Certainly the time is ripe for young men of high personal integrity to become involved in the world's workings.

T. S. Eliot wrote, rather pessimistically, in 1949:[11] "Our own period is one of decline. The standards of culture are lower than they were fifty years ago . . . I see no reason why the decay of culture should not proceed much further, and why we may not even anticipate a period, of some duration, of which it is possible to say that it will have *no* culture."

In "Four Quartets" Eliot wrote:

For most of us there is only the unattended
Moment, the moment in and out of time,
The distraction fit, lost in a shaft of sunlight.
. . . or music heard so deeply
That it is not heard at all, but you are the music
While the music lasts.

As a young boy I stayed with my grandmother for extended visits. On the wall of the bedroom where I slept was an embroidered quotation, the first bit of Shakespeare I ever learned (I later learned it was from *Hamlet*[12]):

To thine own self be true,
And it must follow, as the night the day,
Thou canst not then be false to any man.

There is no better advice. It is beyond the code of professional ethics. It cannot be taught. It must be learned. If a firm, compassionate, and unselfish personal code is the foundation of one's professional character, then living with the written code of ethics *must* be a breeze.

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UNIQUE FOUNDATION FEATURES

THE BOSTON COMPANY BUILDING
BOSTON, MASSACHUSETTS

By Edmund G. Johnson,* Member

(Originally presented before the Structural Section, Boston Society of Civil Engineers, 12 February 1969)

Abstract

The Boston Company Building, a recent arrival on the rapidly growing Boston skyline, has many unusual architectural and structural features, which can be readily observed. Much less obvious, however, is the unusual and innovative foundation treatment which was developed to support this major structure.

This paper presents a description of local soil and rock conditions, a discussion of the various foundation schemes which were considered to support the unusual distributions of column loads, and a description of the unique methods used by the foundation contractor to extend the major corner piers to bearing in the local bedrock formation. The comments and observations presented herein should be of interest to those concerned with the design and construction of deep foundations for large structures in the Boston area.

Introduction

The Boston Company Building is a 41-story office building located on a 37,000-sq ft plot at the southwesterly corner of Washington and Court Streets, which is a very active and congested area in the Boston financial district. Across the street to the east is the historic Old State House, and to the west is the Annex of the Old City Hall. The Ames Building and the Veterans Administration Building lie to the north and a 12-story office building (No. 15 Court Square) abuts the property line on the southerly side. The MBTA subway (Orange Line) passes by the east side beneath Washington Street. Prior to demolition in 1967, several structures, ranging in height up to 11 stories, occupied the site.

This site slopes gradually from west to east (street grade El. 40±) and is within the limits of the original Colonial shore line of the Boston Peninsula, which was located approximately 600 ft to the east at the nearest point.[1]

The initial construction phase began in the Fall of 1967, the foundation work was essentially complete in June 1968, and the tower was completed and occupied by mid-1970.

The project owner, and the design and construction organizations are identified in the final portion of this paper.

*Principal, Haley & Aldrich, Inc., Cambridge, Mass.

Subsurface Conditions

Boring Program

During the course of the design phase a total of twelve borings were made. Of these, eight were 4-inch and four were 6-inch diameter holes, from which 3.5-inch O.D. split spoon samples were recovered. A total of 275 l.f. of rock core, both 4-in. and NX size, were drilled and the samples examined and classified in detail.

The locations of all borings are plotted in Figure 1.

Soil Conditions

The total depth of overburden soils at this site is about 90 ft, and consists of approximately 15 to 20 ft of miscellaneous granular fill over dense, natural soils. No organic soils were encountered below the fill in this vicinity. The two natural soil strata of primary importance are:

Glacial Till: Directly overlying the bedrock is a very compact glacial till (hardpan) which is typical of such deposits which mantle the bedrock throughout much of the Boston area. The total thickness ranges from 20 to 35 ft and the penetration resistance varied from 50 to 150 blows per foot. The material is, typically, a non-stratified mixture of rock fragments and minerals of all sizes, ranging from boulders to silt and clay-size particles. The rock fragments are, generally, broken pieces of the underlying bedrock material.

Sand and Gravel: Above the till is a compact to very compact layer of sand, gravelly sand, and sandy gravel, ranging in thickness from 20 to 35 ft. It is usually stratified, and is believed to have been deposited by rapidly moving streams of glacial melt waters. The penetration resistance varied, typically, from 20 to 60 blows/ft. Overlying these coarse-grained deposits, limited areas of silts and clays were found. Such material were encountered only in Borings 1 and 11 at the northwesterly portion of the tower limits.

Bedrock Conditions

The bedrock encountered in the core borings is characteristic of the major formation underlying the Boston Basin. It is known as the Cambridge Argillite which, in its fresh unaltered condition is typically a hard, blue-gray finely laminated rock resembling a slate. Occasional layers of tuff and sandstone are also typical of this formation, as well as numerous diabasic sills and dikes.

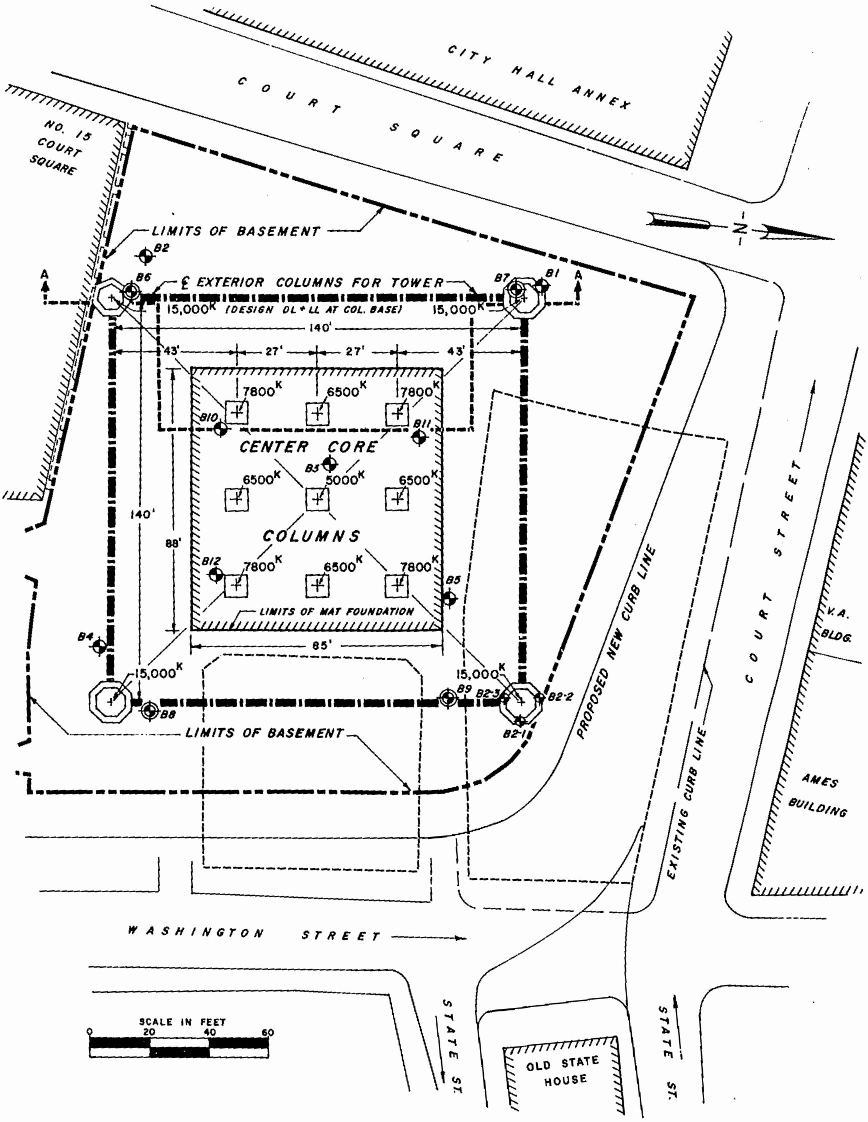

Fig. 1 Location Plan.

Four or the initial five core borings encountered the argillite and occasional tuff, whereas the fifth boring penetrated an igneous rock, believed to be diabase, occurring as either a dike or sill.

In certain zones the rock is highly altered and weakened to the point that the material can be crumbled between the fingers. This weakening of the normally sound rock formation is not unique at this location but has also been reported at other locations within the City.[2] Often fresh, unaltered strata are found interlayered with similar rocks which have undergone varying degrees of alteration. The entire system is generally steeply inclined, with individual layers ranging in thickness from a few inches to a few feet. The diabase encountered in Boring No. 1 was badly fractured and jointed, and thus has also been classified as a weak rock in-situ.

The geological sequence of events that has produced this complex bedrock system has been described as follows:

a. The sedimentary rocks were originally formed when soil particles, predominantly clays, and occasionally silt and sand sizes, were deposited under water in horizontal layers.

b. At intervals during this process, mixed, coarser-grained particles, believed to be from volcanic origin, were deposited in intermittent layers, in thicknesses up to several feet.

c. With passage of time, these layers became indurated. The clay and silt portion became the gray Cambridge Argillite and the coarser-grained volcanic material the tuff.

d. At some later stage in geologic time the entire system was folded with severe fracturing and distorting of the bedding planes occurring.

e. Following this period of folding, molten igneous intrusions penetrated up from below, filling joints along bedding planes and fractures, which formed hard, massive zones of diabase upon cooling.

f. The process by which the argillite has been altered locally to the soft weak rock noted above is uncertain.[2] In some areas the argillite, which may have been reasonably sound at one time, has become softened to the consistency of a stiff clay soil.

In view of the extreme variation in rock quality encountered in the first five borings, there was evidence to indicate that any foundation system which might require high intensity bearing pressures in the complex rock formation should not be constructed without an extremely detailed rock exploration program.

Groundwater Conditions

The groundwater level, as observed in the borings, was at or slightly below El. 0 (Boston City Base). Long-term observations of seasonal variations were not warranted for this project.

Foundation Design Considerations

Geometry of Structure and Design Loads

The general plan layout of the tower columns at ground level is indicated on Figure 1. At each column location the design gross dead and live column loads, as determined by the Structural Engineer, are shown. Of particular significance are the extremely high column loads (15,000 kips) introduced at each of the four corners of the building. This loading results from the unique framing system chosen for the tower.

The structural engineers have designed, in effect, three separate "stacked" structures which share the same four corner columns and foundation supports. The central service core accounts for about 50 percent of the total gravity loads. The balance, plus wind forces, is carried by the diagonal members and then transmitted to the four corner columns. The exterior loadings for the top 12-floor unit are transmitted to the corners at the 31st floor level and thence down to the footings. The middle (14 story) and lower (13 story) portions are similarly arranged in the total "stack". In addition, there is a two-story unit suspended from massive trusses which span the 140-ft distance across each side of the tower, immediately above the street floor.

Figure 2 shows a generalized cross section through the structure, below street level, relating the required geometry of the proposed floor levels and typical corner and central core columns. With this basic structural design information superimposed on the generalized existing subsurface soil profile (Figure 2) the task of investigating and selecting a suitable foundation scheme was undertaken.

Potential Foundation Schemes Considered

1. *Support of the Central Core Columns*

   The proposed basement grade (El.23) was just a few feet above the top of a compact, gravelly sand zone which is generally 20 to 35 ft thick. However, in the northwesterly corner the sand surface dips and is overlain by a silty clay pocket. Individual footing and pier supports, extended down to bearing on or within the gravelly sand layer at an assumed gross bearing value of 5 to 6 tons/sq ft (tsf), would have required footings of such dimensions that they would nearly overlap each other. Therefore, a single, large mat area to

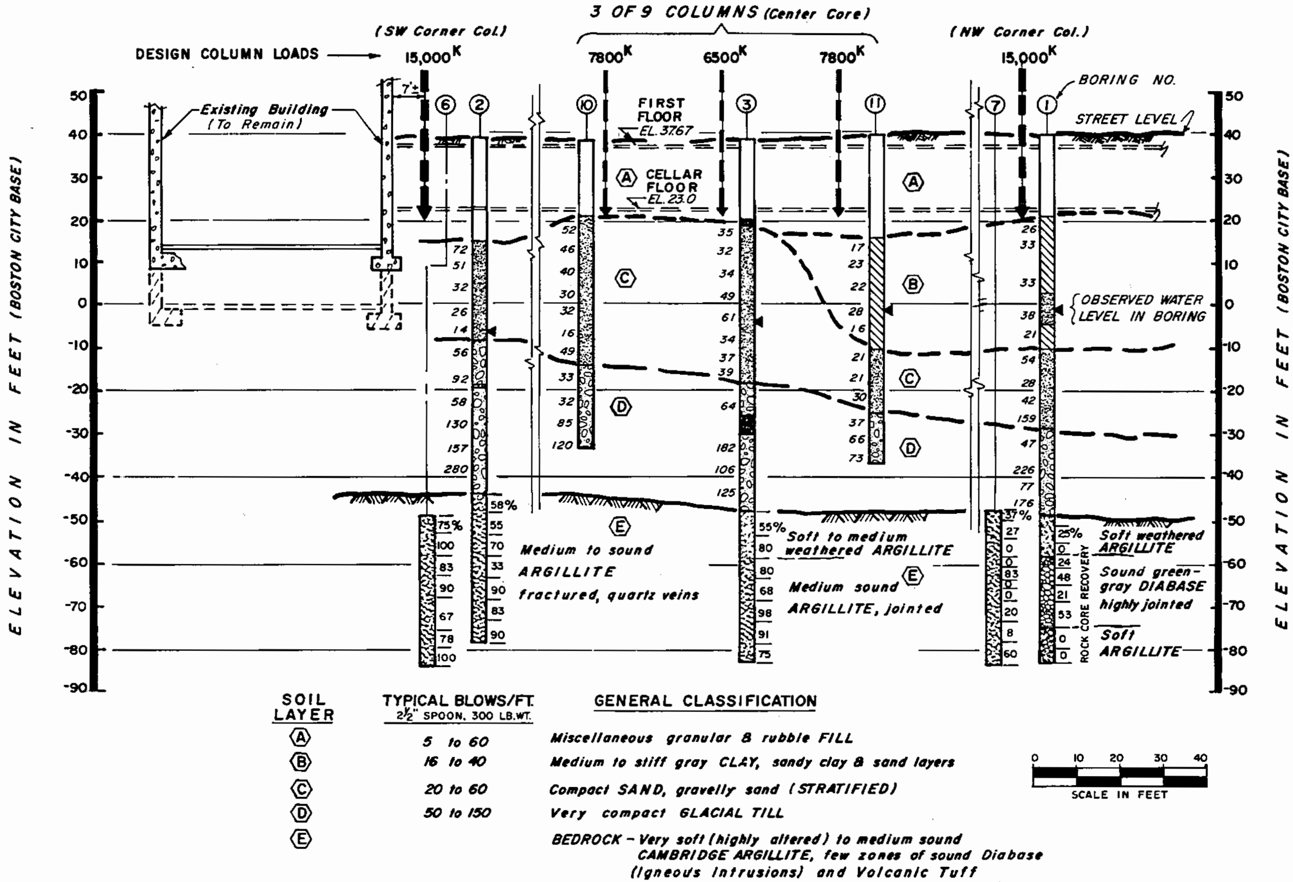

Fig. 2 Foundation Section; Subsurface Boring Information

support the nine columns appeared feasible. The fact that construction could be accomplished by open excavation methods, above the water table, was also considered favorable.

Alternate possibilities of support, such as: piles, driven to bearing within the sand layer or to the glacial till; caissons bearing on the glacial till; and even drilled-in caissons to bedrock were considered. It was determined that none of these alternate solutions was economically competitive nor technically required for support of this portion of the structure.

2. *Support of the Major Corner Columns*

There were definite space limitations which influenced the design. Near the southwesterly corner an existing building, with deep basement, was scheduled to remain. The face of this building is approximately 7 ft away from the centerline of the corner column. Along the northerly and easterly sides the locations of proposed new property lines which would form the limits of the basement were very restrictive, as seen on Figure 1. Various foundation schemes were considered, such as the following:

a. *Spread Footings Bearing on Sand Stratum (Layer C):*

Assuming a design bearing value of 6 tsf, a contact bearing area of approximately 1250 sq ft would be required per column. Space restrictions and construction difficulties ruled this out, however.

b. *Piers and Footings Bearing on Glacial Till (Layer D):*

Assuming an allowable design bearing value of 9 to 12 tsf, (allowing for depth and surcharge effects), the required soil bearing area per column would be 625 to 850 sq ft each. Again, the space restrictions at this site did not allow the excavation of 25 to 30-ft square (or equivalent) bearing areas, symmetrically below each column point. Narrower shaft excavations with possible undercutting in the till, of the dimensions required, were not considered feasible.

c. *Piles Driven to Bearing in Sand or the Glacial Till:*

Assuming the use of piles of up to 120-ton capacity, a massive cluster of approximately 65 to 70 piles would be required per corner column. The resulting pile cap size was not practical due to space restrictions and furthermore was not economical in comparison to the spread footing scheme discussed above, since the pile caps would be roughly equivalent in size.

d. *High Capacity Shafts Bearing on or in Bedrock:*

Groups of drilled-in caissons could be installed within the space limitations at each corner. Assuming the design criteria allowed in the current Boston Building Code (50 tsf end bearing and 100 psi bond stress around the rock socket perimeter) groups of 4 to 6 units, socketed 12 to 20 ft into approved rock, could develop, theoretically, a capacity of 1500 to 2000 tons each. However, although this type foundation has been used with complete success elsewhere,[3] this scheme was given a low priority for this project due to doubts and uncertainties regarding the in situ quality of the bedrock. There was sufficient evidence from the core boring explorations to indicate that the upper several feet of rock is quite variable in soundness, both laterally and with depth, due to the steeply-inclined stratification and differences in rock type, portions of which are very highly weathered and altered.

It was considered doubtful that sufficient perimeter shear strength would be developed for each of the closely-spaced caisson units. For example, if there were *no* effective bond stress developed on the socket perimeter, the total load of an individual caisson (say 1250 tons) would have to be carried by end bearing. For a 36-inch diameter unit, the end-bearing stress would be over 175 tsf, which is excessive for this material.

After reviewing these approaches, the ultimate choice was narrowed down to a foundation scheme in which one or more deep pier units would be advanced to achieve support by end bearing on or within the rock, but would not rely on bond stress or side shear in the soil or rock. If possible, a single concrete shaft, of sufficient cross section to carry the design load in compression would be advanced to the rock surface and then undercut, either in the till or the rock, to provide the necessary bearing area.

The principal advantages of this approach were:

1) The excavation would be large enough to permit dewatering and access for in situ examination of the rock bearing surface by inspection personnel.
2) Reasonably low design bearing pressures could be achieved in the relatively low-strength rock material.
3) This scheme would permit, during construction, even further reduction of the unit bearing pressures; if deemed necessary after in situ examination, the excavation could be advanced deeper and wider within the rock to effectively increase the bearing area and thus reduce the bearing stresses.

Foundation Design Criteria Established for Final Design and Construction

The following general design criteria were incorporated into the technical provisions of the construction plans and specifications. The philosophy here was to provide prospective foundation contractors with the necessary general design requirements as to allowable stresses and bearing levels, but to permit the bidding contractors to propose details of methods of installation. While the central core foundation was quite routine, the corner columns offered a major challenge to the foundation contractor.

1. *The central core foundation:* A uniform, reinforced concrete mat, bearing on the compact sand-gravel stratum at a maximum gross bearing pressure of 5.5 tsf. Where necessary, the clay soils overlying the sand to be excavated and replaced with a mass pour of lean concrete fill, to the limits specified.

2. *Four corner columns:* Each of the four corner columns to be founded on a single pier with an enlarged bearing area supported on or within the bedrock. The gross design end bearing pressure on the rock surface to be no greater than 30 tsf, with adequate provision for enlarging the bearing area (i.e., reducing bearing intensity) during construction, if necessary, to account for possible variations in rock quality.

   The minimum pier section was based on a design compressive stress of 700 psi for unreinforced concrete (current Boston Code requirement).

3. The basement floor to be designed as an earth-supported slab. This was to be taken into account in the construction procedures adopted for the corner column foundations.

Figure 3 illustrates a typical section through the foundation as constructed, and incorporates the above design criteria. Details of the construction procedures for the corner piers are described in the following section.

Foundation Construction Procedures At Corner Columns

The General Contractor received several proposals from experienced and qualified foundation subcontractors for installation of the corner piers. A number of schemes were proposed in which the excavation would be advanced to bedrock using various sheeting and shoring methods to support the shaft excavation through the very compact glacial till overburden.

A very unusual method, as proposed by the Franki Foundation Company of Boston, was finally selected. The procedure is outlined on Figure 4. The installation made use of the bentonite slurry trench method to excavate a perimeter cofferdam which, in effect, provided support for the required earth excavation for each pier, and was itself incorporated into the specified concrete pier shaft section (12 ft x 12 ft or equivalent).

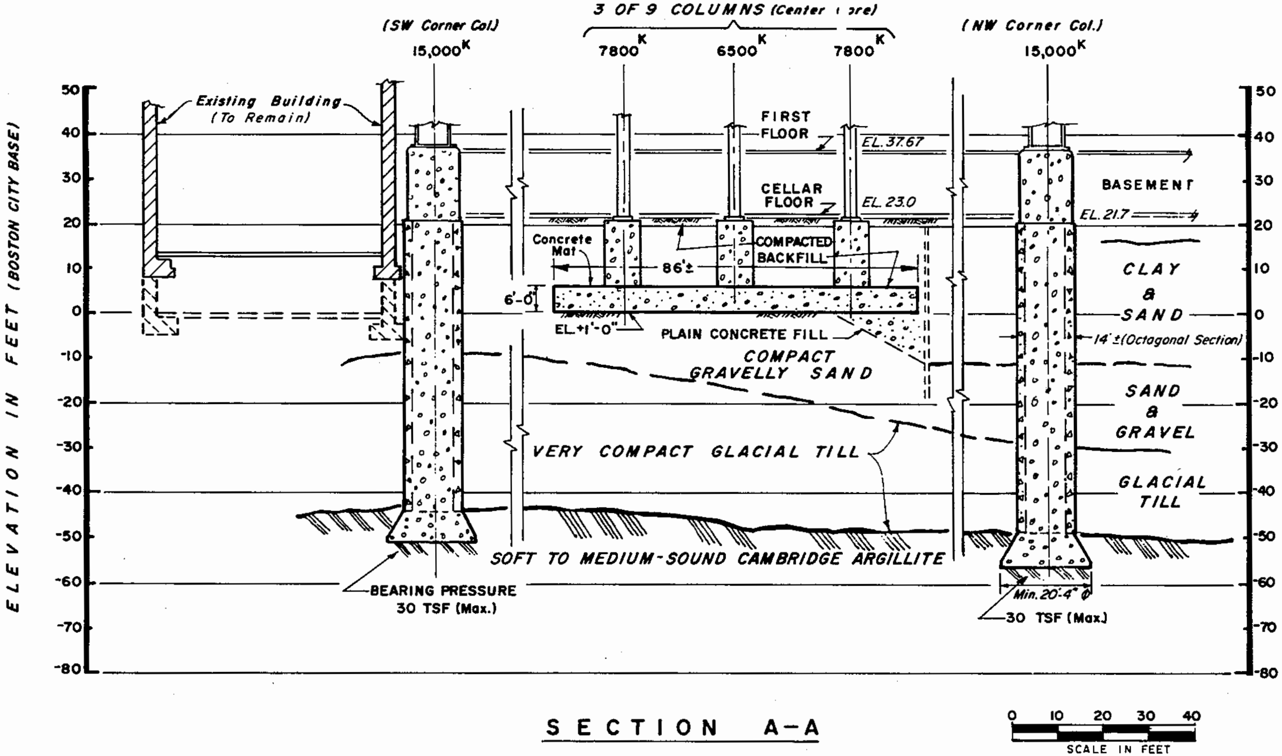

Fig. 3 Foundation Section; Typical Geometry And Column Design Loads

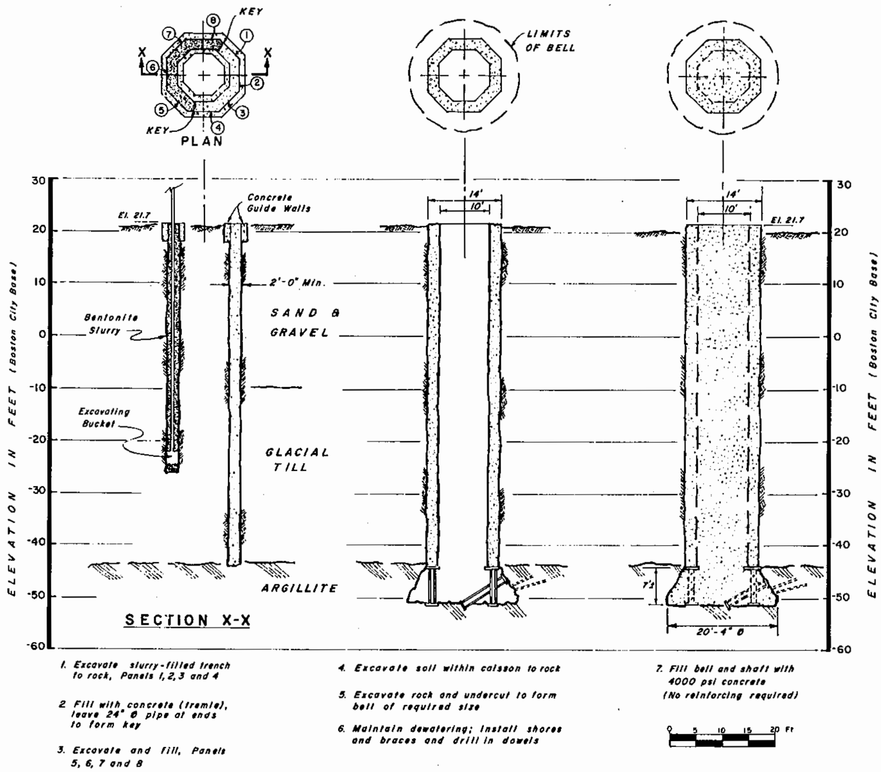

Fig. 4 Construction Procedures for Corner Piers

The initial vertical cuts were made in an inter-connected series of "panel" sections (2 ft wide by 6± ft long in plan view) which form an octagon when completed, using a specially-designed "clam shell" type digging bucket that is mounted on a vertical shaft. The equipment was handled by a heavy crawler crane. During excavation a bentonite slurry of carefully controlled specific gravity was pumped into the trenches and circulated to maintain the sides of the excavations. This liquid mixture is slightly heavier than water. When three or four adjacent panel sections were excavated the full depth to bedrock, and cleaned of loose soil, a high-slump concrete mixture was poured through tremie pipes to the bottom of the trench. As the concrete level was built up, displacing the slurry, the panels became completely filled. Steel pipes of 24 in. diameter were placed at the extreme ends of the panels to form a semi-circular "key" to interconnect with each of the subsequently excavated adjacent panels.

After allowing for a period of strength gain in the concrete, the interior of each caisson was excavated down to the rock surface by means of a clamshell bucket, a 7-ft auger, plus hand excavation toward the bottom of the very dense till. Except for minor leakage through the walls, which were sealed off, the open shaft was maintained in an essentially dry condition down to the rock surface.

For excavation below the rock surface, the subcontractor relied on various rock-tunneling methods and procedures to support and maintain the bell for a sufficient length of time to permit careful inspection of the bottom conditions prior to pouring concrete. Before excavation there was little advance indication as to the quantity of water which would be encountered or the ability of the rock to remain in place when undercut.

Typically the following methods were used:

1. The initial interior shaft diameter of 10± ft was advanced to the required minimum depth in rock by means of air drilling tools and hand excavation. Materials were hand-shoveled into buckets and hoisted to the surface for disposal.
2. The hole was expanded laterally below the underside of the 2-ft thick caisson wall and vertical steel supports installed to support each panel of the wall, extending down to the floor of the bell.
3. Each bell was expanded to the required diameter by undercutting with air tools and hand methods. Steel dowels were driven into predrilled holes extending radially into the exposed rock faces. These usually consisted of straight No. 8 reinforcing bars driven into 8± ft deep predrilled holes of the same diameter. No grouting or other special techniques were required.
4. Due to the intense jointing patterns in the rock the material broke off quite readily into small pieces (approximately 12 in. maximum size) which were handled by one or two men.
5. The quantity of groundwater inflow was generally less than 10-15 gpm and was handled in each hole by one sump pump. There was relatively little evidence of seepage from behind the concrete wall at the interface between soil and rock. Bleeding of the water along joints in the rock did, however, "lubricate" the joints and cause sliding along these planes.
6. During the course of bell excavation a number of inclined braces were installed to prevent collapse. All such supports were either steel pipe or beam sections, propped against channel sections bearing at the rock face. Most of the steel shoring members were left in place permanently when the bell and shaft were finally concreted.

7. Since there was a tendency for the softer portions of the argillite to deteriorate quite rapidly in the presence of water, the final cleanup of the bottom 6 to 8 inches was delayed as long as possible until just prior to concrete placement. Softened materials were removed by air-blasting and hand methods.

8. The procedure for concreting was to pour the bells to a level of 1 to 2 ft above the bottom of the caisson walls and to form a key. Air vents were provided to minimize the formation of voids directly below or behind the underside of the slurry wall. Dowels (#8 bars @ 5-ft spacing) were drilled into the inner face of the caisson wall. Subsequently, one to two days later, the shafts were filled with concrete to design cutoff grade.

The actual construction time per individual caisson is difficult to determine because all four units were constructed simultaneously during the various stages of progress. The initial slurry trench work was started in January 1968; three of the four pier shafts were concreted in May and the fourth pier concreted on 1 July.

In spite of many technical difficulties encountered, as might be expected when such innovative work is undertaken, the Franki Foundation Co. deserves full recognition for their efforts in the successful completion of this project.

Bedrock Conditions Below The Deep Corner Piers

Conditions Anticipated from Boring and Test Data

Prior to construction, the in situ condition and strength properties of the bedrock were evaluated from the following available data:

1) Field drilling rates and core recovery records from a total of 275 lineal ft of 4-in. and NX-size core runs made at a total of 12 boring locations; 2) Visual examination of the core samples which were recovered; and 3) Laboratory unconfined compression tests performed on selected representative samples.

As discussed previously, extremely erratic conditions were anticipated, based on this information.

During installation of the northeasterly corner pier (identified as Caisson No. 2) it was decided that three additional core borings should be made to provide further evidence of rock conditions in the immediate vicinity of this unit. This was done primarily to assist the contractor in determining the proper depth at which to terminate the bottom of the slurry-wall caisson and to plan the belling operations.

The results of these borings are shown on Figure 5. Borings 2-1, 2-2, and 2-3 were located within the slurry-filled trenches, which at that time had been advanced to within a few feet of the apparent rock surface. In plan view, the 3 holes form nearly a symmetrical triangle, with approximately 11-ft spacing. At this close spacing of borings, it was possible to interpret the general orientation of the strike and dip of the steeply-inclined, stratified materials, as shown in detail in Figure 5. Note, also, the extreme variation in percent rock core recovery, which was believed to be indicative of the differences in rock quality. The driller attempted many techniques to improve the core recovery. It was speculated that the soft, altered argillite portions were eroding in the presence of water in the drilling process, such that the reduced diameter of individual cores could not be retained inside the core barrel.

In view of the high degree of jointing in the core samples, there was little opportunity to perform laboratory unconfined compressive strength tests. From a total of four argillite samples tested, the ultimate fracture strength ranged from 2300 to 6300 psi. For three samples of tuff the strength ranged from 4600 to 8400 psi.

One additional method of field investigation was attempted. A number of Menard Pressuremeter tests were performed with specialized equipment which was supplied and operated by a representative of Geocel, Inc., Golden, Colorado. The pressuremeter is a borehole expansion device which has been developed to provide an indication of the in situ strength and elastic properties within a rock mass. The probe consists of a cylindrical assembly of three independent cells arranged in a vertical alignment. The assembly is lowered down a pre-drilled core hole in the rock. At depth intervals of 3 to 4 ft, the cells are pressurized through a tube system from the ground surface. The radial expansion within the rock, as measured by the volumetric change observed in a manometer device, can be interpreted to provide an indication of the modulus of elasticity and possibly the "ultimate" strength of the rock in the immediate vicinity of the test.

A total of 15 pressuremeter tests were performed. Several tests were invalidated due to the fact that the NX bore holes were slightly oversized, possibly due to erosion of the weaker rock during drilling, as discussed above. The results were therefore quite inconclusive, inasmuch as no meaningful tests were made in the softer rock zones. Of the successfully completed tests, however, Geocel did report that the available in situ strengths and bearing capacities, as interpreted from these data, far exceeded the requirements for a design bearing capacity of 30 tsf.

Observed Conditions During Construction

Each of the bells for the four major caissons were successfully excavated and undercut to the required dimensions. Close examination of the exposed rock faces generally confirmed the predicted variability and characteristics of the rock

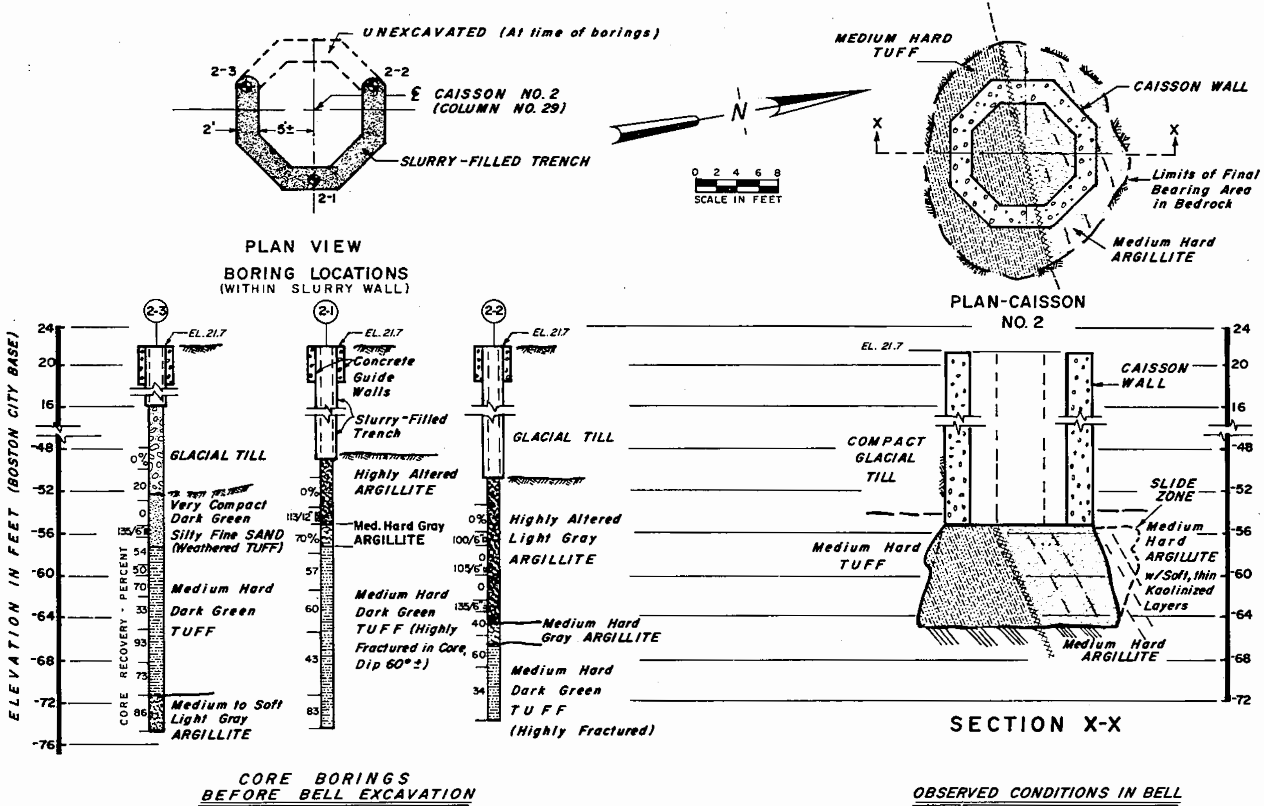

Fig. 5 Details of Bedrock Conditions, Northeast Corner

materials. In no case was it believed necessary to further reduce the design bearing pressure of 30 tsf by enlarging the bearing areas, a contingency which was anticipated earlier.

The following observations were noted which are of interest:

1. It was possible to examine directly the in situ argillite in zones where little or no core recovery had been achieved during the previous exploratory drilling. The rock was found to occur in a steeply inclined (60° to 80°) layered system of medium hard argillite, with occasional bands of medium to soft, clay-like altered zones of 1/2 to 2 inches in thickness. It is concluded that perhaps the softer layers may have eroded during drilling, thus the cores tend to separate along the softer layers, resulting in little or no recovery.
2. The sounder portions of the argillite were highly jointed. The steep bedding planes tended to become highly lubricated with seepage, and often unstable. The instability was an advantage to the contractor during most of the excavation, since the rock tended to break out in flat or cubical shapes of up to 12+ in. maximum size. Shoring and rock bolting was required in many areas.
3. As noted on Figure 5, the bell in Caisson No. 2 was nearly bisected by a contact zone between the argillite and volcanic tuff. The tuff is hard, well indurated, relatively sound, but is highly jointed, although bedding planes are absent. The contact between the tuff and argillite was very irregular along one side of the bell and smooth along the opposite side. Both of the materials were quite stable during the excavation, except for a collapse zone noted in the argillite as it tended to break off along the weaker, lubricated bedding planes.
4. The bell at Caisson No. 1 also encountered both the argillite and the tuff. Bedding in the argillite dipped an average of 55° north, with an east-west strike, and was highly jointed. Very thin, kaolinized beds (1/4 in.±) were noted within the relatively sound rock mass.
5. The bells of Caissons 3 and 4 were excavated totally within the argillite, which contained, typically, many high angle joints and an almost slatey cleavage. A few minor clay-filled joints were noted.
6. In all caissons, the seepage inflow of groundwater was negligible. Minor flows usually developed near the interface between the overburden soil and rock, where exposed. In Caisson No. 4 there was significant flow out of joints along the north side and from behind the slurry wall. The estimated pumping requirement here was 10 to 15 gpm.

7. Since the softer argillite portions tended to soften in the presence of water, the bottom of each bell was cleaned of any loose or softened areas by hand methods immediately prior to placing concrete.

Summary

Foundation Design

1. The foundations for this 41-story office building required deep foundation piers located at each corner, capable of supporting total design loads of 15,000 kips each. At each corner, single concrete piers extend to end-bearing in the bedrock, some 90 ft below street level, at a gross design bearing pressure of 30 tons per sq ft.
2. The central service core is supported on an independent soil-bearing mat foundation at a shallower depth.

Bedrock Conditions

1. The rock core samples recovered during the design state indicated the erratic nature of the Cambridge Argillite formation. The widely varying sample recovery rates and visual examination revealed:
   - alternating zones of very soft, weathered argillite and sounder, but highly fractured, argillite and/or volcanic tuff.
   - the stratification is steeply inclined, at 60 to 80 degrees with the horizontal.
   - occasional dikes and sills of diabase intrusions.
2. The zones of weathered or altered materials followed the orientation of the inclined strata, rather than occurring as a surface condition of limited depth. Thus, soft rock cores were sometimes encountered *below* zones of relatively sound rock.
3. The overlying overburden soils are a very dense glacial till. The interface between the soil and rock is quite level, resulting from the glaciation process, and does not reflect the steep bedding orientation found in the underlying rock.

Corner Pier Construction

1. The foundation subcontractor elected to install the deep piers by very unique procedures, making use of slurry wall construction techniques to form an octagonal "caisson" enclosure through the overburden soil at each location. The interior soil was then excavated down to rock and an enlarged base formed by undercutting in the rock to a typical diameter of at least 20 ft at bearing level.

2. The extreme fracturing pattern and the alternating weak strata made excavation and undercutting, by hand methods, quite difficult and hazardous. Many joints were, in effect "lubricated" by clay and water. Rock bolting techniques worked quite satisfactorily, provided they were placed immediately after excavation.
3. The rate of water seepage into the rock excavations was not significant. The maximum observed inflow was approximately 10 to 15 gallons per minute in one of these caissons.

General Comments

1. The selection of these large pier foundation units, designed for end-bearing in the argillite at relatively low gross bearing pressures, is believed to have been most feasible for the conditions at this site. In view of the erratic nature of the bedrock, this design assures that the bearing areas bridge across intermediate soft zones, where present.
2. Alternate schemes which were considered, such as drilled-in-caissons or even end-bearing piles, require a much higher load concentration to be economical. However, such schemes risk the danger of overstressing localized soft areas in the rock formation.
3. The construction method which was adopted permitted direct examination of the rock in situ by geotechnical personnel. The provisions were such that if observed conditions in the rock were unsatisfactory, and a further reduction in bearing pressures was warranted, this could be accomplished by deepening and widening the bell excavations.
4. The program of core borings or other investigations of the in situ bedrock conditions is extremely important in the argillite formation. The following approach is suggested, when major foundations in the rock are considered:
   - During the initial boring program, extend a number of holes at least 30 to 40 ft into the rock to determine whether erratic stratification, such as observed at this site, does occur. NX-size holes are preferred.
   - Require the best possible drilling equipment, techniques and personnel to achieve best possible core recovery. In spite of precautions, there may be unavoidable zones of little or no recovery.
   - When a final design approach is developed, and the locations of major column loads established, make a number of additional core holes in very close proximity to these locations. Include groups of 3 borings, closely spaced, to determine the strike and dip of the strata.

- Reliance on strength tests of samples which are recovered may not be adequate, since samples of the more critical low strength zones will probably not be available.

Acknowledgments

The project owner and the design-construction organizations which participated in this project are as follows:

Owner Developer:	Cabot, Cabot & Forbes Co., Boston
Associated Architects:	Pietro Belluschi, Boston Emery Roth & Sons, New York
Structural Engineer:	The Office of James Ruderman, New York
Soil and Foundation Consultant:	Haley & Aldrich, Inc., Cambridge
General Contractor:	Aberthaw Construction Co., Boston
Foundation Subcontractor:	Franki Foundation Co., Boston
Boring Subcontractor:	American Drilling & Boring Co., Inc. E. Providence, R.I.

The major contents of this paper were originally presented before the Structural Section, BSCE, on 12 February 1969. At the same meeting, lectures were also given as follows: "Structural Design Aspects" by Mr. Leo Plofker, Partner, The Office of James Ruderman; and "Foundation Construction" by Mr. P. A. O'Neill, President, Franki Foundation Company.

The author wishes to acknowledge the review comments and contributions to this paper received from the above gentlemen, plus those of Mr. Joseph Day, Project Manager, Aberthaw Construction Co. He also appreciates the cooperation of Cabot, Cabot & Forbes Co. in granting initial permission to present this information.

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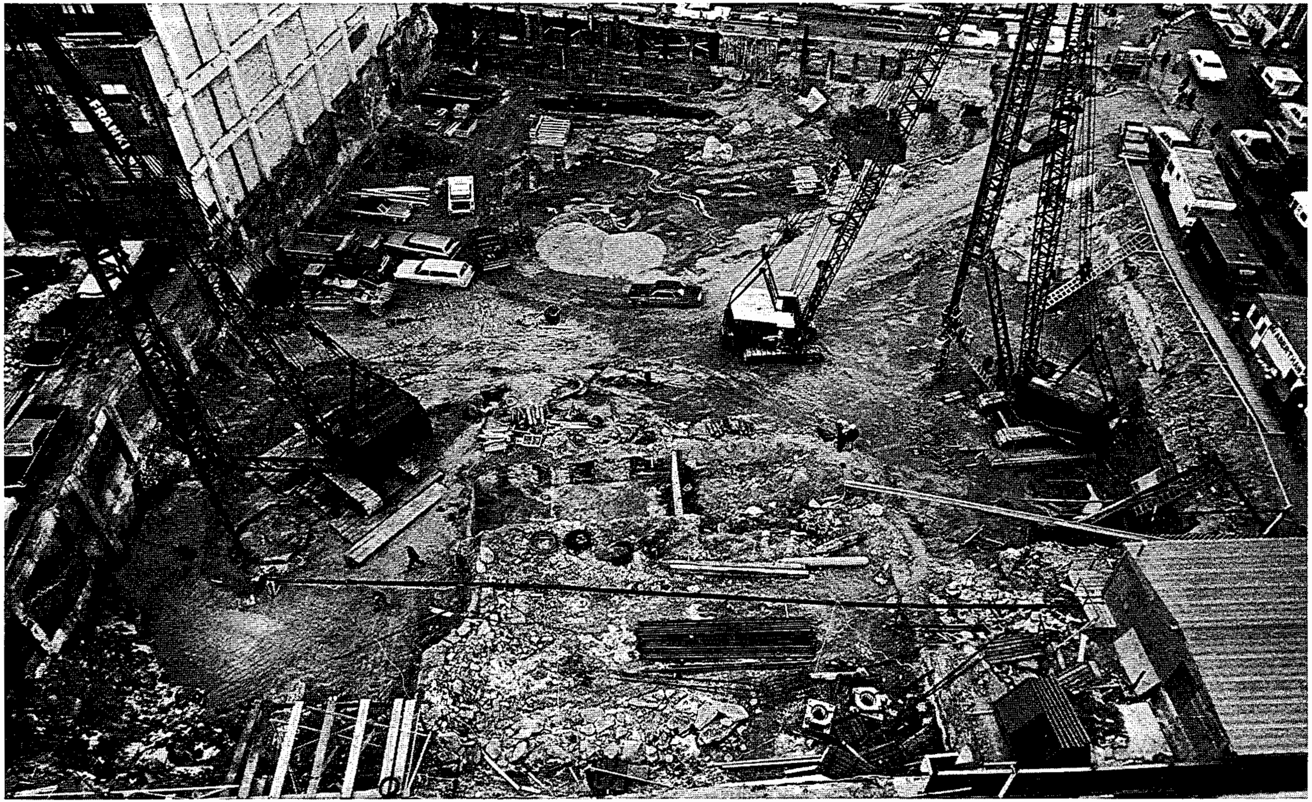
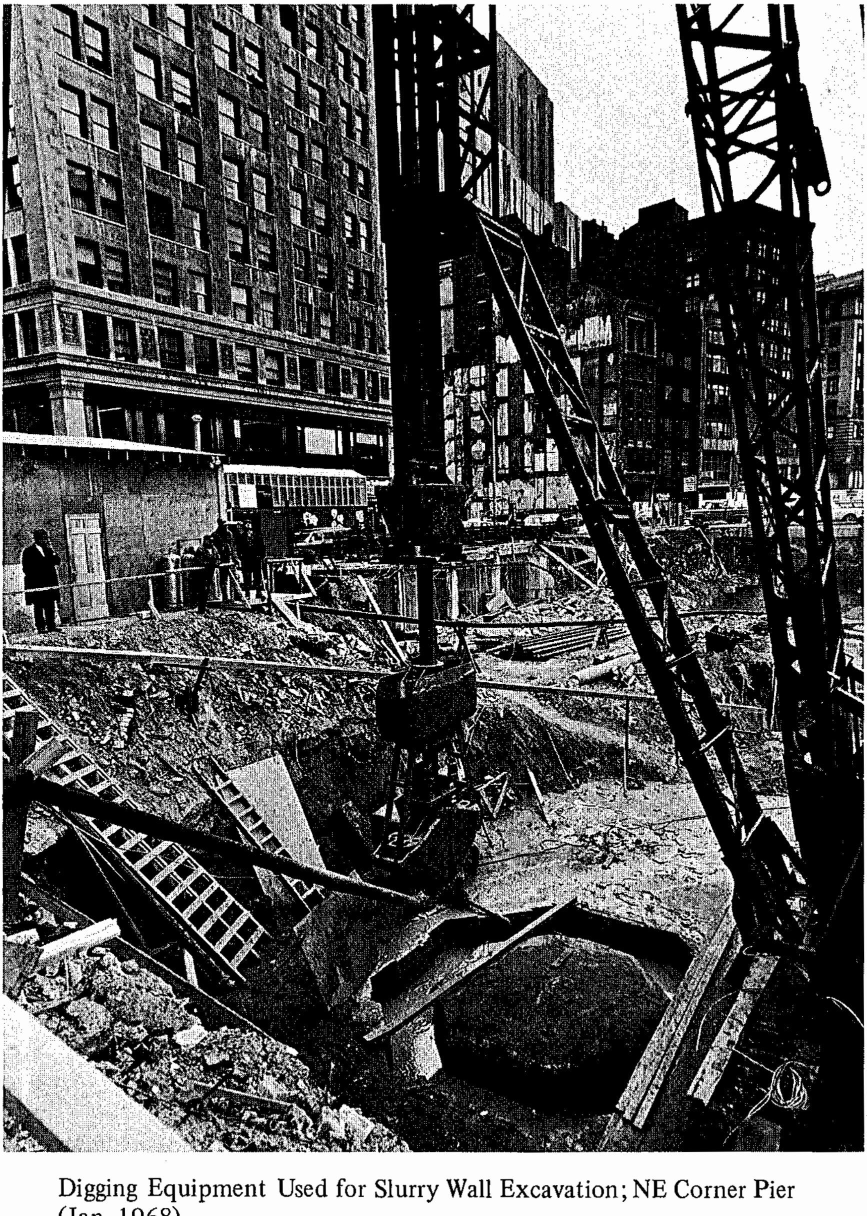
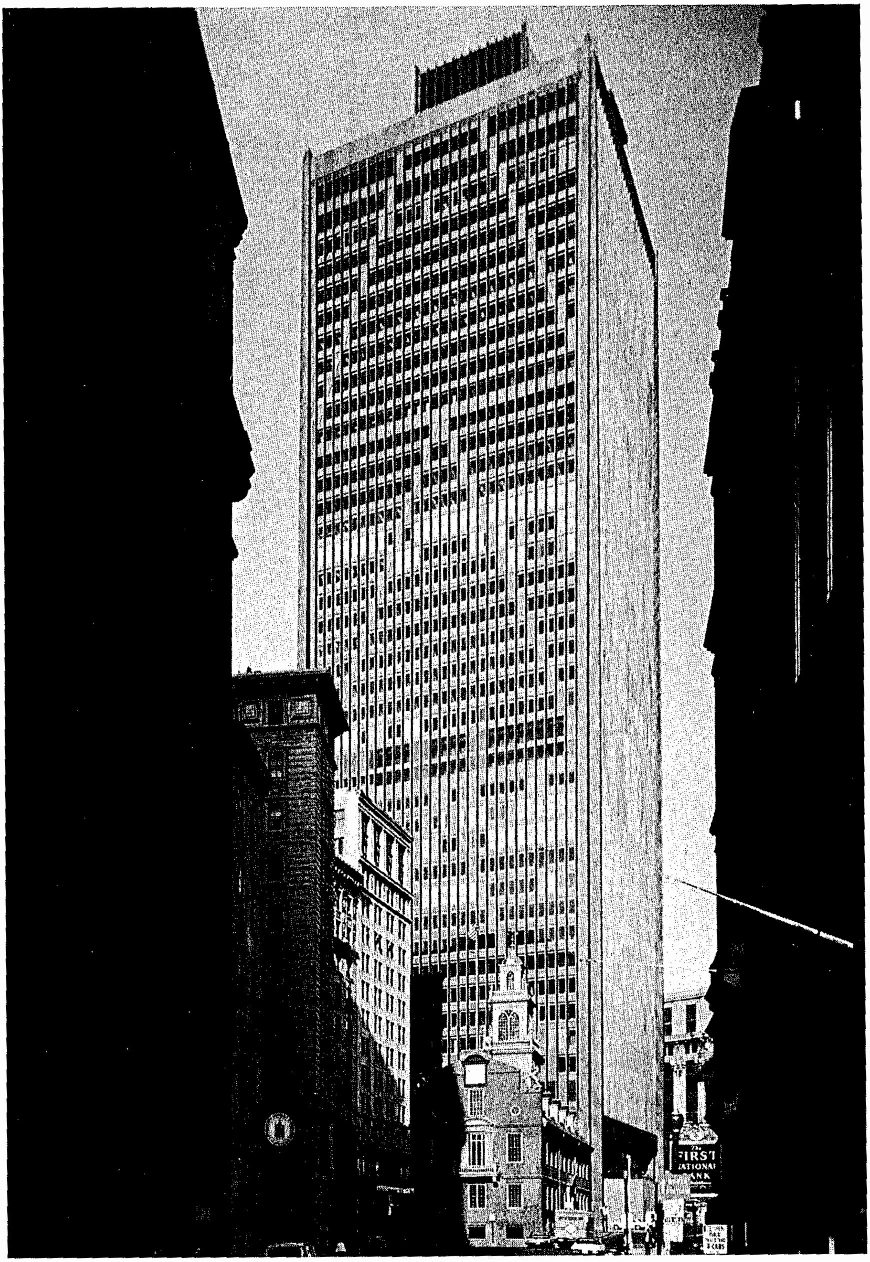


General Excavation to Basement Level; Construction of Slurry Walls for NE and SE Corner Piers in Progress (Jan. 1868)



Digging Equipment Used for Slurry Wall Excavation; NE Corner Pier (Jan. 1968)

Typical Condition of Rock Undercut in Enlarged base of Pier



View of Completed Building from State Street

HYDRAULIC MODEL INVESTIGATION OF SAM LAE PUMP STATION

by
Dr. Norbert L. Ackermann,* Mr. Prida Thimakorn**
and Mr. Howard A. Eriksen***

Introduction

The Sam Lae pump station is part of the new water supply facilities designed by Camp Dresser & McKee International Inc. for Metropolitan Bangkok, Thailand, and scheduled for construction starting in 1973. The station will pump water from the Chao Phya River into a raw water canal for delivery to a new water treatment plant. The pump station will be located 40 kilometers north of central Bangkok at the east bank of the Chao Phya River (see Fig. 1).

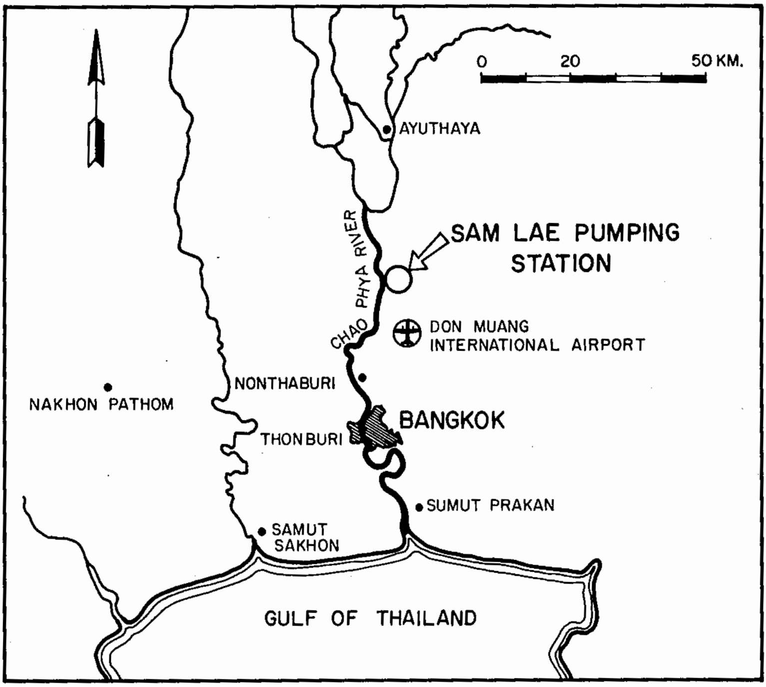


Fig. 1 Location Map

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The river stage at Sam Lae is affected by the tides in the Gulf of Thailand. In order to ensure proper pump performance at all river stages, a study of the river hydraulics was conducted to determine critical combinations of river stage and discharge considering the effects of the tidal cycle and the river fresh water discharge. Of special concern was the flow condition associated with low river stage. The hydraulic design of the approach channel and pump intake did not lend itself to a theoretical evaluation. A model study of the pump station was therefore undertaken.

The model was constructed in the hydraulic laboratory of the Asian Institute of Technology in Bangkok. Various hydraulic conditions that will be expected to occur in the prototype were simulated in the model and the intake design was modified to obtain the best hydraulic performance.

Project Description

The new pump station will be built across the existing raw water canal, adjacent to the existing raw water pump station (Fig. 2).

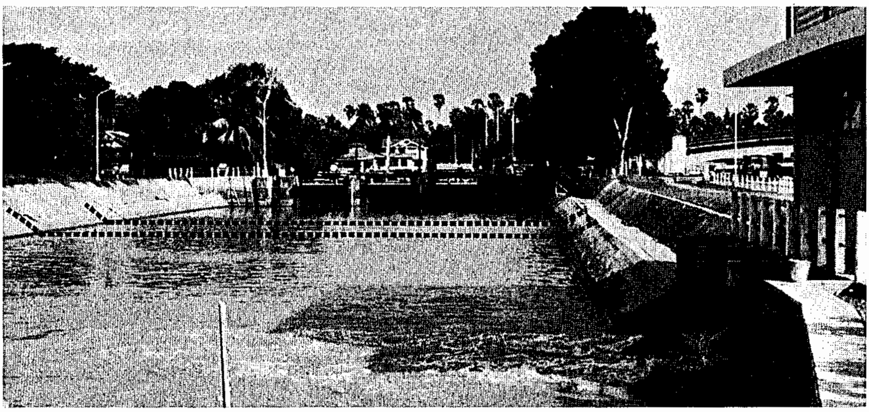


Fig. 2 Existing facilities looking toward river showing present pump station to the right and control structure in background. Location of new pump station shown in dashed lines.

The existing raw water pump station has a capacity of 7 cms and is located on the north bank of the canal. In the present system the water can flow by gravity at high river stage and must be pumped at low river stage. A gated control structure at the upstream end of the canal regulates the gravity flow from the river and is closed while pumping.

The approach channel to the new pump station will be built by expanding the present raw water canal intake.

The new pumping facilities will be built in stages. The first-stage pump station will house 5 pumps, each pump rated at 6.6 cms at a head of 3 meters. Ultimately the station will be expanded to house 12 pumps (2 spare) for a total discharge capacity of 66 cms which, with the existing station, will supply the projected water demands in the year 2000.

The approach channel will also be built in stages. In the first stage the existing channel will be deepened and the existing control structure will be removed. In the ultimate stage the approach channel will be widened to match the total length of the pump station.

The proposed facilities for the initial and ultimate stage are shown in Fig. 3a through 3c.

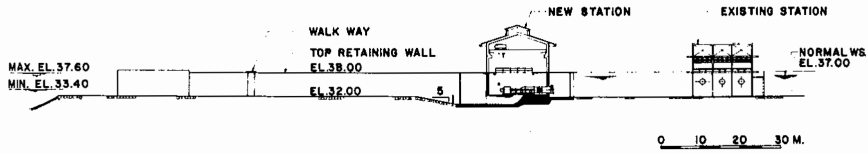


Fig. 3a Station Profile, Initial and Ultimate Stage

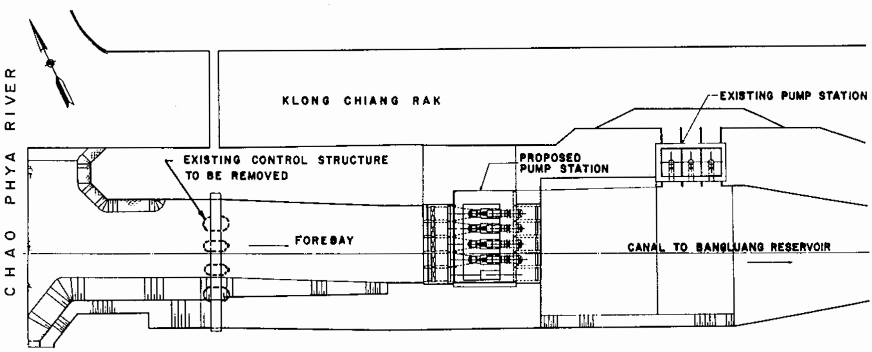


Fig. 3b Plan, Initial Stage

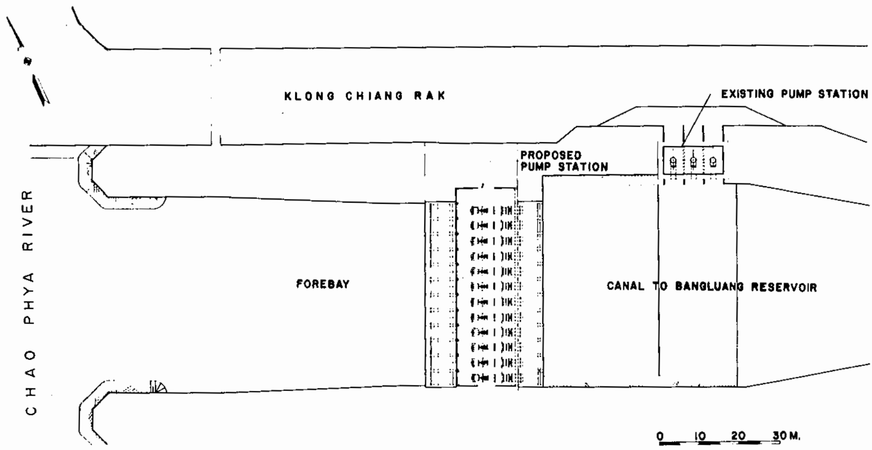


Fig. 3c Plan, Ultimate Stage

Hydraulic Design Considerations

At low river flow, a significant portion of the total flow is diverted at Sam Lae. The effect of diverting this flow may be to reduce the river's water surface elevation and hence the water level throughout the forebay to the pumping station. An understanding of the hydraulic characteristics of the flow throughout the forebay is therefore contingent upon knowledge of the water surface elevation and flow characteristics which would be maintained in the river at Sam Lae. Since the Sam Lae station is within the tidal reach of the Chao Phya River, the minimum water surface elevation of the pumping station would depend not only upon the mean daily river discharge but also upon the tidal variations of the water surface in the Gulf of Thailand. The characteristics of the river flow at Sam Lae would most readily be obtained by mathematically simulating the unsteady flow conditions in the Chao Phya River for the boundary conditions imposed by the tidal variation and mean river flow which would produce the minimum possible water surface elevations at the pumping station.

Because of the river regulation provided by existing and planned storage reservoirs on the tributaries of the Chao Phya River the minimum mean daily discharge at Sam Lae would never be expected to be less than 140 cms in the near future. After withdrawing 66 cms as provided by the pumping demands at the Sam Lae station, the mean daily river discharge would be reduced to approximately 74 cms. Because of the tidal variations in the Gulf of Thailand, however, the instantaneous stage and discharge hydrograph at Sam Lae would vary throughout each tidal cycle. A mathematical model of this dynamic response was constructed using a finite difference solution of the momentum and continuity equations describing the unsteady flow in the river. The mathematical model was calibrated using available hydrographic data of the Chao Phya River.

In order to determine the minimum water surface elevation which might possibly occur at Sam Lae for the reduced river discharge of 74 cms, the 30-year record of the lowest water levels ever recorded in the Gulf of Thailand was used as the downstream boundary condition in the mathematical model of the river. The record of the downstream stage which was used was that which occurred on August 7, 1964 at a station one kilometer from the mouth of the Chao Phya River.

The stage and discharge hydrographs at Sam Lae corresponding to flows with these boundary conditions are shown in Fig. 4. The computed low water surface elevation at Sam Lae was 33.41*m at an instantaneous discharge of 1100 cms.

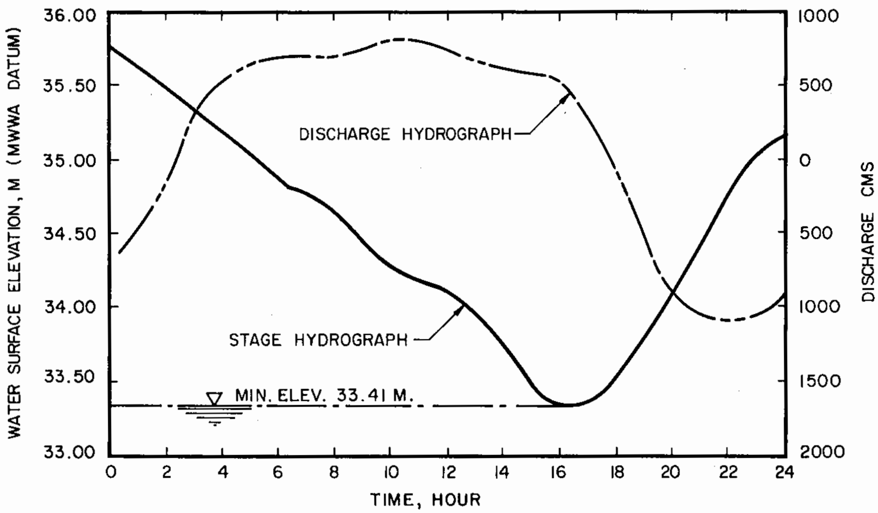


Fig. 4 Stage and Discharge Hydrographs of the Chao Phya River at Sam Lae. Mean Flow = 74 cms

*Bangkok datum, Mean Sea Level = 35.03

Hydraulic Modeling of River Channel and Pumping Station Forebay

A 50:1 scale model of the river channel and forebay to the pumping station was constructed for both the initial and final stage design. This model was operated to produce similarity between the Froude numbers of the prototype and model. Fig. 5 shows the 50:1 scale model of the initial stage design.

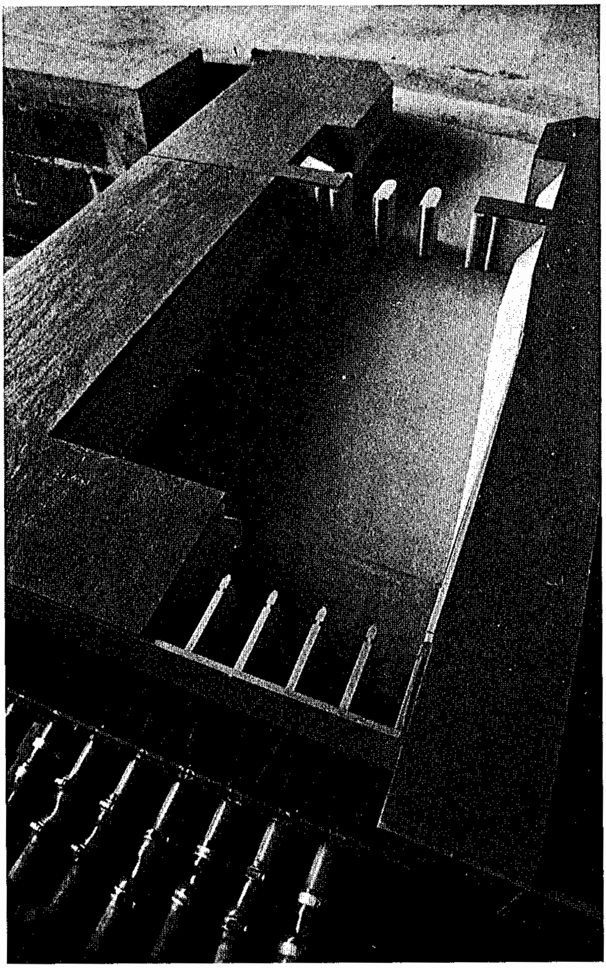


Fig. 5 The Initial Stage Model

The flow characteristics at the pump intakes were investigated in more detail using 25:1 and 15:1 scale models of the intake structure. Flow conditions to be maintained as hydraulic controls in the 25:1 and 15:1 scale models would be obtained from measurements made in the 50:1 scale model of the entire forebay and adjacent river reach.

The four flow conditions modeled (corresponding to flood and ebb tide for the initial and final stage design of the forebay) were each studied under steady state conditions corresponding to the instantaneous stage and discharge selected as the model operating criteria.

Any tendency for surging or resonance as a result of the river's rising or falling hydrograph was studied by simulating in the model several representative river hydrographs scaled for the appropriate Froude number. No resonance could be detected in simultaneous measurements of the water surface elevations in the river and forebay. Resonance conditions were also investigated which might be produced as a result of instabilities caused by vortex shedding from the piers obstructing the flow in the initial stage design (the piers were removed in the final design). Resonance conditions were not found and hence the model tests were conducted at steady state conditions equivalent to the instantaneous river elevations and discharges that were considered to constitute critical flow conditions.

Modeling Vortex Formation at Pump Intake

Vorticity in free surface flows entering the suction side of a pump is difficult to model because of its strong dependence upon viscous, gravity, and inertia forces. For dynamic similarity between these forces, Reynolds as well as Froude similarity must be maintained. It is impossible, however, to satisfy both criteria simultaneously without using a different fluid in the model from that in the prototype. When using the same fluid in both prototype and model, the velocity required to produce Reynolds similarity in a 50:1 scale model must be approximately 353 times greater than the velicity required to produce similarity for Froude number modeling.

When operating the 50:1 scale model at Froude scale velocities the surface tension forces are 2500 times greater than they should be for Weber number similarity.

Conflicting model testing criteria are suggested in the literature in order to simulate hydraulic conditions at pump intakes where vortex formation is an important consideration.[1, 2, 3] Opinions are unanimous, however, in agreeing that operating a geometrically similar model at prototype velocities provides a conservative basis to study the formation of air entraining vortices. When testing the model at prototype velocities, exaggerated representation of the water surface profiles is produced because Froude numbers are much larger than would occur in the prototype. Large scale models are therefore advisable to minimize this effect. Because of such considerations 25:1 and 15:1 scale models were constructed of the pump intake in order to study the formation of air entraining vortices.

Water Surface Elevations and Flow Conditions in Forebay

Fig. 6 shows the water surface elevation and flow conditions existing in the forebay of the model of the initial stage design during the ebb flow in the river for a discharge of 1100 cms and a water surface elevation of 33.40 m. The

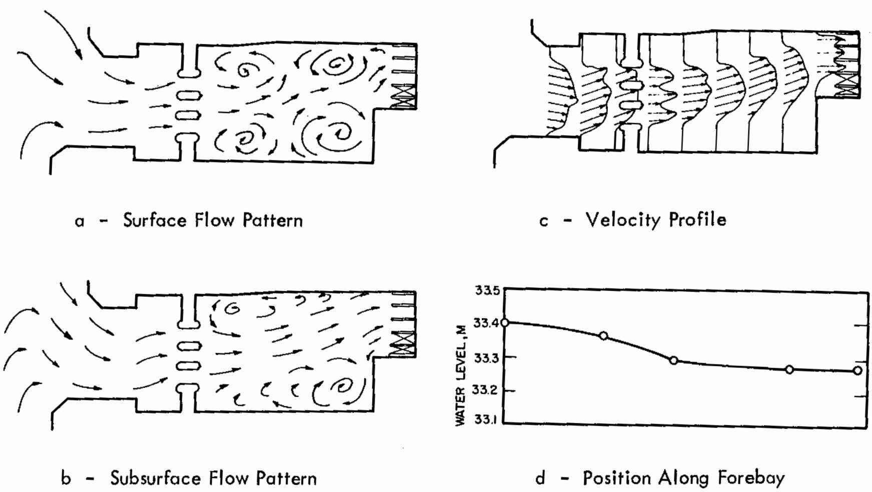


Fig. 6 Flow Condition at the Pumping Station Forebay (Initial Development) during Maximum Station Discharge and Minimum River Stage

surface streamline patterns shown in Fig. 6a are not representative of the conditions throughout the depth of flow. These surface flows represent conditions which exist over a small depth and in the prototype structure could be influenced by external factors such as wind. Such noticeable contrast between the surface and subsurface flow pattern is often found upstream from control structures which discharge from sluice gates or other submerged outlets. Fig. 6b shows the subsurface flow patterns while Fig. 6c shows the distribution of the average velocity over several cross sections in the forebay. These flow conditions were produced when 3 of the 5 pump intakes were operating at their designed capacity of 6 cms. (In the final design the pump capacity was increased to 6.6 cms. A few tests were run at this increased discharge and the hydraulic condition was found to be acceptable.) The proposed design produced nonuniform approach flow conditions at the pump intake structures – an undesirable condition prone to producing flows with considerable swirl at the pump intake. Numerous combinations of pumping and standby units were tried to determine which combination produced the most nonuniform conditions to the pump intake and in the channel forebay. All combinations revealed approximately the same degree of nonuniformity in the approach flow conditions with the pump adjacent to the standby pump exhibiting the most adverse flow conditions compared to all others. Local velocities in the forebay never exceed 1.3 meters per second, a value which was considered to be acceptable. When modeling with Froude number similarity there was no indication of air entraining vortices being formed. The water surface levels in the forebay for the initial stage design are shown in Fig. 6d. The water surface elevations at the pump intake structure for both ebb and flood flows are approximately 33.31 m and 33.28 m respectively. Corrections for differences in friction losses between prototype and model indicate that the water surface elevations at the pump intakes for both ebb and flood flows in the river are 33.32 m and 33.29 m respectively.

A similar experimental program was conducted to determine the hydraulic characteristics of the second or ultimate stage design. A difference was also observed between the surface and subsurface flow pattern. Local velocities in the forebay did not exceed 0.9 meter per second, and at identical Froude numbers as the prototype, air entraining vortices did not form. Nonuniform flow conditions at the approach to the pump intakes were also observed.

Model of Pump Intake

A 25:1 and a 15:1 scale model of a portion of the forebay were constructed to study the detailed flow conditions in the vicinity of the pump intakes. Three bays of the pumping station were modeled with discharge possible through only the center bay. In this way flow conditions were simulated which were similar to those produced at a pump operating adjacent to a standby or non-operating pump. Discharge under these conditions produced the most adverse or nonuniform approach conditions at the pump intake as found from tests of the 50:1 scale model of the entire station forebay. By obstructing the flow through different portions of the gravel screen, where the flow first entered the model, the mean values of the velocity distributions in the approaching flow could be produced to simulate conditions measured in the 50:1 scale model. In both the 25:1 and 15:1 scale models the walls of the model and pump intake were constructed of transparent plastic to enable a visual record of the flow phenomenon to be obtained.

Fig. 7 shows the relationship between the water depth and the velocity of the pump intake for conditions that would produce air entraining vortices in the flow entering the suction side of the pump. In both the 25:1 and 15:1 scale models, air entraining vortices were produced when the model was operated near prototype velocities. Surprisingly, when operating the forebay of the 50:1 scale model at prototype velocities a continuous air core was not formed. At the unrealistically high Froude numbers produced at this condition considerable surging at the pump intake was found to occur. This unstable flow may have been responsible for inhibiting the vortex generation. Another factor which might have prevented the air entraining vortices from forming in the 50:1 scale model is the increase in the effect of the surface tension forces resulting from the use of such a small scale model.

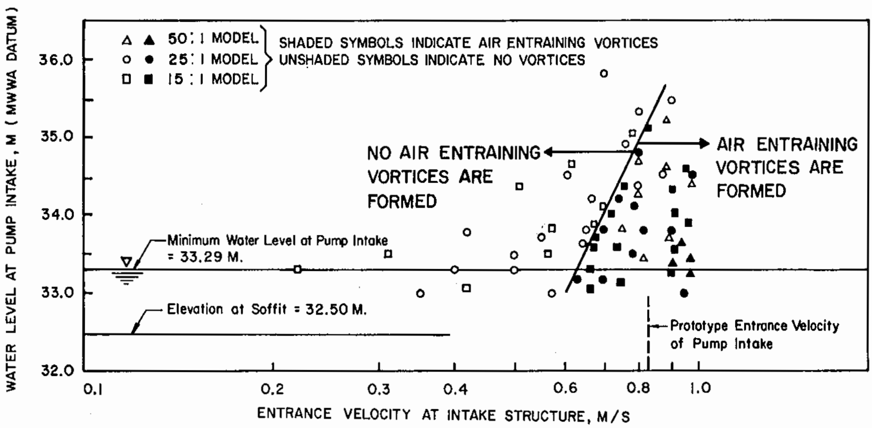
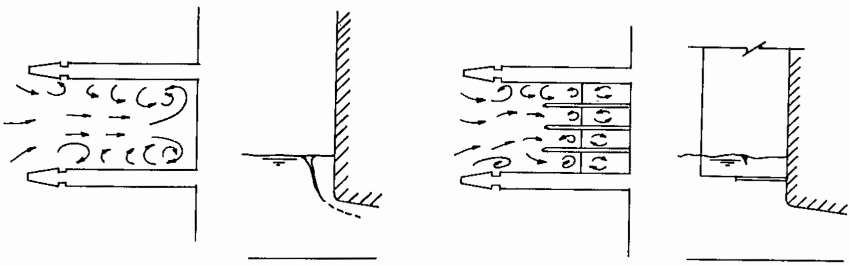


Fig. 7 Relationship between Water Depths and Entrance Velocities without Modifications to the Pump Intake Structure

Pump Intake Modifications

Structural modifications to the bays at the pump intakes were considered necessary to eliminate the possibility of forming air entraining flow leading to the suction side of the pumps. Fig. 8 shows the combination of horizontal sill and vertical vanes which were found to successfully prevent air entraining flow at the pump intakes when operating at prototype velocities.



a. - Original Design

b. - With Horizontal Sill and Vertical Vanes

Fig. 8 Flow Pattern at the Pump Intake Structure

Fig. 8 shows surface flow patterns that existed in the pump intakes before and after structural modification. Vortometer measurements revealed that some local surface vorticity was increased by the structural additions; however, the depth to which this vorticity persisted was greatly reduced. These measurements were made for a large range of model discharges and the above conclusions were found to apply uniformly for all conditions tested.

In order for the horizontal sill and vertical vanes to operate successfully as an inhibitor to forming air entraining vortices, little or no opening should exist between the horizontal sill and the vertical gate which controls the intake to the suction side of the pump.

Subsequent to the completion of the experimental investigation, the detailed design was available of the gate which controlled the flow into the pumping station. Fig. 9 shows the gate in its fully opened position. The hood, consisting of the vertical vanes and a horizontal sill is shown positioned immediately adjacent to the gate. A removable cap having an inclined face is positioned on the upper face of the sill. This cap has an inclined surface sloping away from the gates, thus preventing debris from settling into a position which would interfere with the gate operation. The cap is removable to accommodate inspection.

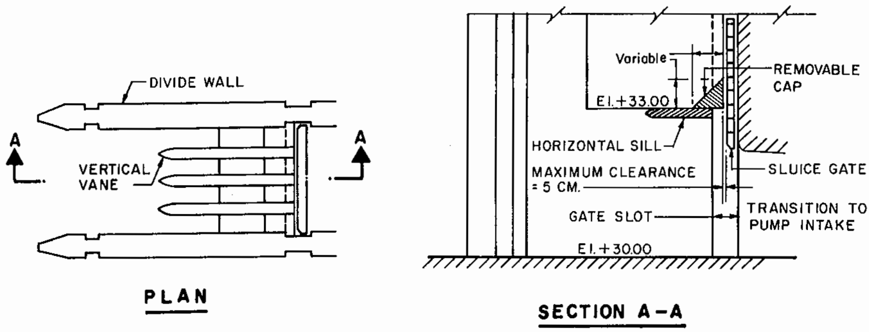


Fig. 9 Structural Modifications to the Pump Intake

Discussion of Results

With the exception of modifications suggested at the pump intakes, the design proposed for the initial and final stages of development of the forebay to the Sam Lae station, as shown in Fig. 3a, b, and c, provided hydraulically suitable flow conditions for the discharge produced at their respective design capacities of 24 and 66 cms.

The model operating criterion of the water surface elevation in the river of 33.40 m was the minimum possible level that could be obtained using a combination of extreme events of a mean river daily discharge of 74 cms and low-low water levels in the Gulf of Thailand produced from tidal effects. Water surface elevations only 20 to 40 centimeters greater than this elevation have frequently been recorded at Sam Lae, however. This is because year to year variations are small in the low-low water levels in the Gulf of Thailand produced by tidal conditions, and because these tidal effects have a dominant influence on the water surface elevations during low flow conditions at Sam Lae.

At the mean daily discharge of 74 cms the instantaneous flows were on the average much larger than the mean flow. The average absolute value of the discharge, at the low flow conditions used for the operating criteria, was approximately 520 cms. Even when considering low-low flows in the Chao Phya this average discharge of 520 cms was approximately 8 times greater than the design capacity of the Sam Lae station during its final stage of development. Therefore it should be expected that the Sam Lae station would have little or no effect on the hydrodynamic aspects of the river regime.

The velicities in the river induced by the withdrawal of water at the Sam Lae station were negligible a short distance before the entrance to the station forebay. Therefore the river acted much like a large reservoir supplying the pumping station.

Because of the small kinetic heads produced in the river as a result of the flow diverted into the pumping station, transient phenomena such as movement of sand bars or long term changes in bank geometry would have little influence upon the hydrodynamic characteristics of the approach conditions of the flow entering the station forebay.

The model operating criterion of using prototype velocities for determining the formation of air entraining vortices at the pump intakes was extremely conservative. Air entrainment did occur near this limiting condition for both the 15:1 and 25:1 scale models of the pump intake structures as designed in the proposed initial and final development plan. This is a necessary but not sufficient condition for insuring similar vortex formation in the prototype. Due to the lack of any other reliable design criteria, modifications to the intake structure were therefore recommended.

Conclusions

In general the approach channel as visualized in the original design was proved to be adequate. The velocity distribution with and without the existing control structure for the initial and ultimate design showed acceptable hydraulic flow patterns. The approach channel in the ultimate design was originally planned to be 2 m deeper than the initial design. However, as proved in the model study, the approach channel invert elevation used in the initial design will also be adequate in the ultimate design and this modification will result in a considerable saving in future excavation.

The modeling of the pump intakes showed that freedom from air entrainment could not be guaranteed if the intakes were built according to the original design. The simple and inexpensive system of piers and baffles as arrived at in the model study would eliminate potential air entrainment problems. It was therefore decided to incorporate the anti-vortex devices in the final design.

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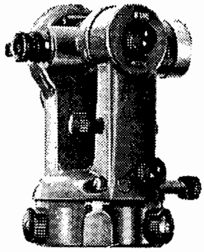

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