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THE DEEP FOUNDATIONS INSTITUTE AND ITS PUBLICATIONS

The Deep Foundations Institute was incorporated in 1976 as a professional society to bring together contractors, engineers and equipment and materials suppliers concerned with design and construction of deep foundations. Educators, planners, designers, specification writers, technicians and other support personnel involved in the construction process are eligible for membership. The Institute has as its purpose better understanding among the various groups involved and through meetings, discussions, publications, and cooperation with related organizations to promote more rational, optimal and safe solutions to deep foundation problems.

More than 15,000 of the manuals prepared by DFI Committees have been distributed. These include: *Inspector's Manual for Pile Foundations, A Pile Inspector's Guide to Hammers,* and *Glossary of Foundation Terms.*

Presentations at DFI meetings generally cover applications of theory, evaluation of performance, construction methods, and the many practical technical, human, legal and economic aspects of the foundation industry. This first volume of the Deep Foundations Journal contains selected papers from earlier meetings and other pertinent papers submitted for publication. It is planned to print meritorious papers on an "as available" basis. Copies will be sent to the Engineering Societies Library and other depositories for availability as permanent references.

Hal W. Hunt, P.E., Editor
Publications Committee, DFI

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History and Present Status of Piles and Their Installation in the United States

Expanded from a Presentation at the
Deep Foundations Institute Annual Meeting
Williamsburg, Virginia, December 1982

by

Hal W. Hunt, P.E.
Chief Engineer
Associated Pile & Fitting Corp.

Steel and concrete piles and power-actuated hammers have developed almost entirely within the past 100 years. Prior to that time, piles were mostly timber, driven with a drop hammer or a maul. The Engineering News Formula – for timber piles driven with a drop hammer at a factor of safety of six – was promulgated by Wellington in 1888. Steel companies supplied fabricated and rolled shapes for piles in the early 1900's. The thin shell and then uncased concrete piles originated about that time. Now leads have become sophisticated to accurately spot a pile at any required batter. Hammers have grown to phenomenal size and vibratories are being accepted at full value.

Until about 90 years ago foundation piling was almost exclusively timber, driven with a drop hammer. A few short lengths of cast iron sheet piling were produced and used in England as early as 1864. Wood piles were driven to below the water table (creosote treatment came later). Expensive masonry was installed down to that level. A forerunner of the concrete composite pile was installed by Alfred A. Raymond at Council Bluffs, Iowa, in the early 90's by driving a wood pile 6 ft below excavation, and the water table, with a timber follower 4 in. larger than the pile. The hole stayed open in the loess soil. Concrete filled the opening to complete the pile.

In 1888 A. M. Wellington, then editor of *Engineering News*, sent questionnaires to knowledgeable foundation people asking for ideas on a means of estimating pile capacity from driving resistance. Wellington, with others, averaged the information into a formula intended to have a factor of safety of six for timber piles driven with a drop hammer. This

Engineering News formula is criticized by everyone and used by many—because it is simple—despite heavier piles, more powerful hammers and many conditions that were not a consideration 100 years ago.

In 1897 a patent on a tapered thin metal shell driven with a mandrel and filled with concrete was issued to Alfred Raymond. It was 1901 when the first such pile was driven under contract for an apartment building in Chicago, probably with a drop hammer. Some steel I-sections were driven in Nebraska before 1900.

Meanwhile the organization that became McKiernan Terry was making steam drills for quarrying and mining. From this capability a light steam hammer for driving sheet piling was made in 1896. These were double-acting hammers modeled after drills. They were a forerunner of eleven hammers by 1910. In 1906 one was made with a 500 lb ram; one with a 1,000 lb ram was used to drive 12 in. pipe to 70 ft depth at No. 1 Wall Street in New York.

Vulcan Iron Works was established in 1852 as a general foundry. Drop hammers were among their products. The first single-acting steam hammer, the No. 1, was made in 1887. Vulcan gradually moved to making only pile hammers and equipment. A small sheeting and demolition hammer was made in 1916. The first differential hammer was developed in 1935 followed by the open type, 50 C in 1938.

Union Iron Works of NJ Inc. made about 15 sizes of double-acting steam hammers with a rated energy ranging from 54,900 ft-lb down to 90 ft-lb. None have been manufactured in many years but parts can be made by Superior, Lidgerwood, Mundy Corp. at Superior, Wisconsin. Eagle Iron Works in Des Moines cast drop hammers up to 8,500 lb weight. Only a few European made air-steam hammers have made their way to this country, but the British Steel Piling Co. air-steam extractor is popular.

Diesel hammers originated in Germany as military equipment for advancing troops. Only small hammers were made for World War II. Patents were taken as war reparations and opened to all. Many different diesel hammers are manufactured. Several corporate members of DFI build them: Berminghammer Corp., L. B. Foster Co., Foundation Equipment Corp., International Construction Equipment Co., MKT Geotechnical Systems, and perhaps others.

But back to development of piling. Many things were being done. In 1901 steel sheet piling was first rolled in England. Fabricated sheet piles were used in a cofferdam in Chicago in 1902. U. S. Steel rolled a ball and socket interlock steel sheet pile in 1904. Lightweight corrugated sheets were rolled by Cambria Steel Co. Lackawanna Steel Co. rolled sheet piles in 1908. Jones & Laughlin made up I-beams with an interlocking member that gripped the flanges. In 1903 R. J. Beale developed a method of driving a steel pipe either closed or open end then withdrawing the pipe.

Sheet piling of wood also advanced. In 1903 Wakefield obtained a patent on the often used three timbers bolted or spiked together to form a tongue and groove. Wood became more expensive. Steel for support piles became economical.

Raymond was overcoming troubles with light tapered shell by imbedding a spiral wire in a matching groove. The so-called standard taper shell with a taper of 1 in. for each 30 in. limited this type pile to about a 32 ft length. In 1920 a corrugated shell was introduced to extend the standard taper. In 1931 this developed into the step-taper pile in use today.

Raymond started experiments in 1939 with open, large-diameter piles spun with reinforcing in its thick walls. Conduits in the walls provide for post tensioning of generally 16 ft sections into large capacity piles of any length with first contracts in 1949.

In the early 1900's numerous types of piles were devleoped and patented. A few have survived. In 1906 Union Metal Manufacturing Co. fluted some metal to make strong porch columns. These became the Monotube pile about 1920. In economical light-gage steel they were tapered and could be driven without a mandrel. For long piles a uniform diameter top section is available. Monotube piles were recently driven to 273 ft depth at Orlando; average pile length on the project was 77 ft.

Franki piles were first installed in Belgium about 1911 and came to the U.S. in 1951. They were generally uncased with the drive tube dragged down by dropping a 7,000 lb weight 20 ft on a slug of very dry concrete then pounding out more concrete as the tube was withdrawn. The casing is now often installed with a big diesel hammer but the interior drop hammer is used to drive out a "pressure injected footing"; pounding out concrete is continued as the casing is pulled.

Pile Foundations by Chellis (McGraw-Hill) names and describes briefly at least 50 pile types, which attained some recognition. These include Caudill, Colcrete, Duocrete, Hercules, Maco, Miga, Screwcrete, Simplex, Tuba, West's and many others.

The Deep Foundations Institute can well take a moment here to pay tribute to Robert D. Chellis of Stone & Webster. He made an avocation of collecting information on all types of piles, their installation and preservation. More than that, he generously put it all in writing. His book will probably always be the bible of deep foundations. It was a monumental task, performed despite crippling arthritis.

In the early 1900's MacArthur Concrete Pile Co. drove a heavy walled pipe, usually about 14 in. diameter, with an inside core. Concrete was used to fill the pipe, which was slowly withdrawn. Weight of the core and hammer forced concrete out against the surrounding soil to produce an uncased pile. Where soil was too soft or expansive for uncased con-

crete, a uniform diameter corrugated shell was dropped into the heavy pipe to leave a cased shaft of concrete in the ground.

In 1923 Harry Thornley and Forest Jenkins broke away from MacArthur to form Western Concrete Pile Co. They used a similar arrangement with a precast end called a button-bottom. It was 17 in. dia.; a 14 in. heavy-walled pipe drove on the button. This could be an uncased pile or a 12 in. corrugated shell dropped inside. In either case, the reaction for pulling the outside casing was taken by pushing on the just deposited concrete so there was positive compression of the concrete to assure a dependable pile. Both MacArthur and Western had variations of driving out concrete to form a compressed bulb.

Engineers were never comfortable with the 12 in. cased shaft in a 17 in. diameter hole. Sometimes outside filling was done but the compressed soil quickly reformed tightly around the shaft. There never appeared to be any problem from this large opening around a shaft. A recent development is a precast bottom section into which a corrugated shell is set and driven then filled with concrete; a variation is use of a timber shaft. Depth and type of soil are limiting factors.

In the mid 40's the Cobi mandrel, which could be expanded to grip the corrugations of uniform diameter shell, became available. Charles Guild made this usable and developed it into the G-Mandrel, now widely used to make uniform diameter shell competitive.

Skid and roller mounted rigs were the usual driving equipment in the earlier days. Most were made of timbers. Crews became quite proficient at quickly setting them up and moving them around a site. A Vulcan No. 1 or an MKT 11B3 was a big hammer. Cushions were timber with an occasional try for coiled rope or cable. A steam boiler (coal and then oil-fired) provided the motive power for the hammers and the multiple drum winches. Cables raised the hammer and the pile. Cat heads on the winches were used with manila rope to move the rig around the site. Raymond first used a crawler crane to handle pile driving about 1940.

Pile driving crews, not the equipment, were remarkably efficient. Non-restricted workmen put many piles in the ground on every shift.

Structures to be supported continually get larger and heavier while available places to build new structures have more difficult subsoil conditions. Deep foundations get deeper; hammers are more powerful. Francis Pruyn, then Chairman of the Board of Underpinning & Foundation Co., recognized the problem and developed the cast steel Pruyn Point to protect the vulnerable corners of the H sections and tie flange and web into a unit. Associated Pile & Fitting Corp. has developed this into the HARD-BITE Point with integrally cast teeth.

Splicers were developed so piles could be driven deeper as conditions require. For pipe piles the splicer moved from inside, where hard driving had a tendency to split the pipe, to a tapered outside structural unit

that forced the pipe into a dependable watertight drive fit that does not require welding. A fabricated splicer for H-piles provides a joint as strong as an unspliced member, with a minimum of welding.

For a project over Tampa's lime rock HP 10 x 42 with Pruyn Points were driving to 30-35 ft depth. A 16-pile cluster went to an average length over 600 ft; one pile in the group was driven to 702 ft. This may be the longest pile ever driven. Cavernous limestone formations are a pile driver's nemesis.

Precast concrete piles have proved increasingly economical in recent years. In other countries and in a few areas in the U.S. conventional reinforcing is used. Generally piles are prestressed for economical handling in long lengths. Efficient splicers have been developed or imported for precast piles. Simplest is an adaptation of the Advance Pipe Pile Splicer, used with short ferrules of pipe in the same manner as for pipe piles. Several splicers are on the market utilizing reinforcing bars that are welded to a plate and cast with the concrete. The plates are then connected by wedges for a tight joint. Typical is West's Piling and Construction Co. Ltd.'s Hardrive Splicer. For penetration into hard strata or debris stub H ends are used on the prestressed piles with cast steel points for penetration through obstructions.

Leads have become more sophisticated. A lighter single mast lead that travels through a boom-head connector (Berminghammer Vertical Travel Leads and Foundation Equipment Corp. leads are examples). Hydraulic pistons, controlled from the crane cab, position the leads to any angle of batter, fore, aft or either side. Accessories provide for turning H or square precast piles to a desired orientation.

This makes it unnecessary to exactly level the crane as the boom can be tilted to compensate. It also provides for quick spotting at exact location. For designers and specification writers, this is a boon. Use of swinging leads can be prohibited as equipment is available to position piles far more rapidly and accurately for greater productivity. The Guild hydraulic expanding internal mandrel has greatly improved the competitive position of thin-shell, uniform diameter cast-in-place piles.

The Equipment Applications Committee of the Deep Foundations Institute is working on standardization for evaluation of cushion material for hammers. This will provide for more dependable determination of the actual bearing of every pile. (A paper by Bengt Fellenius in this issue covers Wave Equation Analysis and Dynamic Monitoring.)

Hammers have grown huge in size and driving capability. A Vulcan air-steam hammer is rated at nearly two million foot pounds of energy. Diesel hammers range to nearly 300,000 ft-lb. The larger hammers are for offshore work, handled by floating equipment. But hammers of 60,000 ft-lb rating are common on ordinary construction.

Vibratory hammers are increasingly efficient for installing non-displace-

ment H and sheet piles. DFI Corporate Members MKT Geotechnical Systems, L. B. Foster Co. and International Construction Equipment Co. manufacture them in the U.S. Engineer-contractor-manufacturer committees are working on guidelines for evaluating the load carrying capabilities of piles installed by vibratories in different soils and conditions.

Piling has not changed much since the advent of prestressing for precast concrete piles. The improvement is in splicers and tip protection for precast piles and increased use of tip protection of H, pipe, timber and sheet piles. The wave equation and dynamic monitoring are predicted to be in use on most major pile projects in this decade.

History of Timber Piles and Preservation Techniques

A presentation at the Deep Foundations Institute Annual Meeting
Williamsburg, Virginia, December 1982

by

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Timber has been used as pile support for structures for 6,000 years. Some means of preservation of wood go back almost that far. Timber piles support all types of structures – some are buried below the water table; others raise a home for protection from animals or a chief's home for prestige. Millions of wood piles have been driven for bridge support. Earlier piles were driven by hand then by ingeniously raised weights. Charring, animal fats and oils have been used for preservation. Coal tar creosote and other preservative methods started only in the early 1800's and give effective protection.

Credit for the original concept of placing a timber log vertically into soft or unsuitable soil to act as a structural support must be given to a group of Neolithic tribes who, some 6,000 years ago, migrated into what today is Switzerland, the Italian Alps, Austria, Bavaria, and Eastern France. These so called "lake cultures" erected their homes on platforms supported by wood piles. They settled in and around the various lakes, swamps, and marshes of that region from around 4,000 B.C. to around 2,000 B.C. Evidence of these structures, known as the "Swiss Lake Dwellings" still exist today. The old piles and other artifacts were protected when the lake elevations rose and submerged the villages in lake sediments and an airless environment.

The ends of the wood piles were burned to a point before driving; the platforms of the homes consisted of a lacework of tree trunks and branches. The people lived on the platforms right along with their cattle, sheep and other domestic animals.

Stilt Houses

Similar types of stilt houses have been elevated on wood piles from prehistoric times and on down through the ages in many other regions of the world. So-called primitive cultures in Polynesia built what are known as Marquesan houses. In New Guinea they are known as Sepik Men's Houses; in Indonesia, Batak Houses. The same types of elevated houses have been found around Lake Nokue in Dahomey, Africa, as well as in Sulawesi, the Philippines, China and Peru. In Japan they are known as Takayuka Huts.

In most cases, the platforms of these houses were raised to protect the inhabitants from wild animals and floods. In Polynesia, however, the height of the house is an indication of the prestige of the owner—the higher the house off the ground, the greater the prestige of its owner.

Around 1,200 B.C. another primitive culture found itself in a spot where their ability to engineer foundations for their dwellings would mean the difference in survival or extinction. The Aztec Indians had moved into Lake Anahuac in the Valley of Mexico because all of the best lands were already occupied. They constructed "Chinampas" or floating gardens, which were towed into the lake and anchored. The "Chinampas" looked like large wicker baskets. They were filled with earth and planted. Today this land is Mexico City.

Greek and Roman Construction

During the Greek civilization, at around 600 B.C., a great temple was constructed at Ephesus, in what today is Western Turkey. It is best known as the Temple of Diana. It was built in marshy soil on charred timber piles. It covered 93,000 sq ft and was 60 ft high with 137 columns. It was rebuilt 300 years later on the same foundation and was finally destroyed by the Goths in 262 A.D.

The first written record of the use of timber piles was by the Romans. A timber bridge spanning the Tiber River was built around 1620 B.C. and, with repairs, lasted over 1,000 years. In the Roman city of Ravenna, Italy, all of the buildings, public and private, were constructed on wood piles from 100 B.C. to 400 A.D.

During the heyday of the Roman Empire Hannibal transported the Carthaginian Army over the rivers in the Alps on timber bridges. In 55 and 53 B.C. Caesar led his Roman Legions over the Rhine on timber bridges constructed in ten days each. He left a detailed account of the construction methods they used. Two pairs of 18 in. dia. timber piles were driven on batters facing each other. Pile driving rammers were used to place the piles. They were then joined by 2 ft wide timbers. Bracing, buttresses, stringers, flooring and fendering were installed. The fendering was to protect the bridges from debris sent downstream by the Gauls. These bridges were about 4 yds wide by 420 yds long.

During the entire span of the Roman Empire, the Romans built around 50,000 miles of roads and 2,000 bridges. Most of their bridges were stone; two in Spain are still in use today. For the construction of their bridges, the Romans developed the cofferdam; two types of timber cofferdams were used for cases where bedrock could be reached.

For shallow water bridges, iron-tipped wood piles were driven in close formation as the cofferdam. For deep water bridges, two rows of oak piles were driven and then laced with wickerwork and filled with puddled clay. This basic cofferdam method for constructing bridge piers continued in use until 1850, when compressed air was first introduced.

Where bedrock could not be reached, bridge piers were constructed directly on charred alder, olive, or oak piles driven as close together as possible.

Roman roads were supported by wood piles when the subsoil could not be drained. Near Rochester, England, the old Roman road west of the River Medway was founded on 4 ft oak piles linked by timbers. Excavations 1,900 years later revealed the piles were still in first class condition. On the Appian Way wooden piles were driven into the Pontine Marshes to retain rock fill.

Roman legionnaires constructed the first bridge across the River Thames in London about 60 A.D. as part of the Roman plan to pacify Briton. Timber piles tipped with iron were driven and then timber frames were lowered onto the piles to form cribs.

Construction of the first masonry London Bridge started in 1176. This bridge had 19 irregular, pointed arches and it took 33 years to build. Each pier took 19 months to construct. Elm sheet piles were driven under each pier to support a timber platform. The timber platform was bolted down at the low water level. The timbers were covered with pitch to protect the bolts and to provide an even seating for the first courses of the masonry piers. This bridge lasted 600 years and was replaced in 1831. The new London Bridge that replaced it, along with the New Westminster Bridge built in 1858, used wood bearing piles loaded to 60 ton capacities.

During the medieval and renaissance centuries, wood piles were primarily used to support heavy structures and fortifications in European cities, which had extremely poor foundation conditions. Amsterdam and the majority of other cities and towns in Holland rest on wood piles. During this time it was very common to shod the wood piles with iron to facilitate penetration in hard ground and to band the piles to prevent splitting during driving.

Venice and its Foundations

In the 5th century A.D., the Veneti people of Northern Italy were facing invaders. The Roman Empire was falling and barbarians were moving into Italy, sacking and looting as they came. When Aquileia was

destroyed by Attila the Hun in 453, the Veneti took refuge on some inaccessible islands on the mud flats of the North Adriatic Sea. This was land nobody else wanted. Their situation was not unlike that of the Aztecs some 1,600 years earlier and on the other side of the world. For 200 years they tried to reoccupy their cities but the Lombards had established a very tight grip on the surrounding countryside. The Veneti turned back to the lagoon and made their permanent home there.

Today the City of Venice rests on 1,600 ft of alluvial silt and clay. The subsoil of mud, peat, clay and sand is soft to over 100 ft before a 10 ft thick layer of hard clay is reached.

The Grand Canal through Venice is two miles long with a mean depth of 9 ft. It is 228 ft wide at its broadest point and is never narrower than 120 ft. It is bordered by 200 palaces built from the 12th through the 18th centuries. Venice's total inventory of palaces and museums lists 450 structures, typically built on close driven wood piles.

A major landmark in Venice is the Campanile tower. It was built in 900 A.D.; it experienced a masonry collapse a thousand years later in 1902. It was discovered the tower foundation rested on wood piles; the piles were found to be in good condition. The 1,000 year old piles were left in place for the foundation of the new tower, new piles were driven only outside the original foundation to support the additional area required.

The Rialto Bridge over the Grand Canal is another famous Venetian landmark. It was constructed from 1588 to 1591 on 6,000 wood piles driven and capped under each of the two abutments. It is still standing, having withstood earthquake, soil compression and political intrigue. Another famous landmark, the Palace of Doges, was built in 976 A.D. on wood piles and substructure.

The Basilica di San Marco was built in 830 A.D. on the foundation of an old Roman structure. After a fire in 976 A.D. it was rebuilt during the 11th century. Today its floor shows as much as 10 in. in differential settlements. It was discovered that the wood piling under the original structure was only 7 ft long. It has been said that close to a million close-set piles were used.

Medieval Bridges and Flood Control

Some other significant bridges constructed during this time span included the Santa Trinita Bridge in Florence, Italy, and the Pont Notre Dame in Paris. The Santa Trinita was built from 1566 to 1569 using two rows of wooden sheet piles and wooden bearing piles. The Pont Notre Dame bridge was built in 1507 on timber piles and rebuilt in 1853 on the same foundations. The Seine River bridges in Paris today rest on timber piles, most of which are designed for 60 ton capacities. However, some of the timber bearing piles are loaded to 100 ton.

In Holland, the whole country has been reclaimed from the sea. Timber has been used in the construction of dams, sluices, locks and dikes. Some of the early dikes were faced with two rows of untreated wood piles. The space between the rows was filled with seaweed and reeds. In 1731, the marine borer "teredo" invaded the country and the wood facing was replaced with stone revetments.

Exposed wood piles have been used to strengthen the dikes and dunes since the advent of the treated wood pile in 1875. Several rows of 6 to 8 ft long piles are driven on the seaward face of the dune, usually at places where the sea has previously broken through.

The City of Amsterdam is built on a 50 ft layer of clay overlying 10 ft of sand and rests on oak piling driven through the clay and into the sand. The city was chartered in 1300; its major growth occurred in the 1600's, when its size was gradrupled by the digging of concentric canals. The land between the canals was drained and turned into housing blocks. Radial canals were then dug to create 100 man-made islands, which are connected by 300 bridges. Today, Amsterdam has double the canal mileage of Venice.

A major landmark in Amsterdam is the Royal Palace, which rests on 13,600 wood piles. Wood piles also support many of the roads in Holland including the "Birnnen Rote" in Rotterdam, which was built in the late 1800's on wood piles and grillage.

Researchers have recently discovered that a timber grillage system without piles was used to claim a several block area on lower Manhattan Island from the East River. This was done between 1730 and 1760 by Dutch settlers who imported this technology from Holland.

During the 1500's the Arno River in Florence, Italy, was in constant danger of flood. Leonardo da Vinci designed timber revetments, which consisted of wood piles backed by tight-fitting planking. He also used another system consisting of three rows of piles with brush woven between them. Leonardo said that for the purpose of revetting river banks timber was the preferred material because it was cheap, abundant and easy to put in place.

Ancient Pile Drivers

Up until 1600, pile drivers were operated manually by simply raising and dropping the hammer. These hardy souls drove piles with hand rams, hand mauls, hand-operated machine mauls, ratchet winch rams, treadmill drivers, water wheel drivers, and gang-operated rams. The mechanical pile driver was introduced around 1600 with the hammer being raised by two men turning cranks. The hammer was dropped by the releasing of a tripping gage.

In 1737, the book "Architecture Hydraulique" showed several methods

of pile driving, including a barge mounted pile driver with a treadmill to raise the hammer and a release by automatic trip gear.

TIMBER PILES IN U.S.A.

Timber piles have been in use in the United States since the 1700's. In 1761 the first major highway bridge in the United States was constructed by Major Samuel Sewall. It was a 270 ft long by 25 ft wide wood bridge across the York River in Maine. It had 13 spans; each bridge pier was supported by four timber piles driven with a pile driver on which the hammer was an oak beam raised by pulleys. The bridge remained in use for 175 years, with repairs.

It was deemed vital to our national security during the early 1800's that a series of impenetrable forts be built at critical locations along the coastline from Maine to Louisiana. Several of these forts had to be located at sites with extremely poor soil conditions.

To protect Philadelphia, Fort Delaware was constructed on Peapatch Island in the Delaware River from 1818 to 1826. It had to be demolished in 1833 because large cracks had opened up in it due to differential settlement in its foundation. The new fort constructed to take its place was supported by 12,000 wood piles. Early forts around New Orleans used timber grillage.

At Fort Moultrie, South Carolina, palmetto log cribs were used as foundations for rip rap breakwaters and jetties built from 1807 to 1811. Park Service employees say they can still be seen at low tide and that the jetties were the first ones used for erosion control in the U.S.A.

When Robert E. Lee graduated from West Point in 1829, his first assignment was to Cockspur Island at the mouth of the Savannah River in Georgia. When he arrived, he found a mud and marsh island. There on Cockspur Island as a fresh, young second lieutenant he supervised the soils investigation, footing excavations, and the driving of wood piles for the foundations of Fort Pulaski.

Twenty years later, as a Colonel in the Corps of Engineers, Lee was assigned to Fort Carroll, Maryland. Here again a heavy masonry fort was being constructed and wood piles were required for the foundations. Lee supervised plans for the design of a steam pile driver. Actually, the steam pile driver had been invented in England by James Nasmyth four years earlier in 1845.

In 1856 a "tower fort" was constructed in Louisiana at Proctor's Landing. Corps of Engineers' records show the government paid 12½ to 15 cents per lin.ft for cypress piles to be placed under the battery walls and end towers. Timber grillage was also used. Old Spanish forts at Puerto Cabello, Venezuela (1737) and Matanzas (1736) near St. Augustine, Florida, were also constructed on wood piles.

Pile Failures

Until around 1890 untreated wood piles were used; however, some complications began to develop:

1. At the Brooklyn Navy Yard buildings, which had been standing for over 40 years, began to settle.

2. An eight-story Chicago warehouse built in 1872 and added to in 1881 began to settle in 1918. Settlements of up to 7 in. occurred before underpinning was carried out.

3. Other foundation failures also occurred at the following buildings: Boston Public Library; New York City Criminal Courts; New York Produce Exchange; Washington, D.C., Post Office; Brooklyn Power Station; Millard Fillmore Hospital, Buffalo, New York; chemical plant in Midland, Michigan; and on a St. Paul, Minnesota project. These are just the well-known and documented cases.

In all of these cases, the cause of the failures was the lowering of the groundwater table. This exposed untreated wood piles to the well-known ingredients that cause untreated wood to rot and decay. However, in one case involving a Chicago grain elevator it was discovered, after the failure occurred, that the untreated wood piles had actually been cut off above the existing water table. In that case the structure never had a chance.

Church Committee on Water Levels

Trinity Episcopal Church in Boston was built on 4,500 untreated wood piles in 1877. The church building is still in use today, with an active congregation. It is the only Episcopal church in the United States (and probably the only church of any denomination anywhere) with a "Committee on Water Levels." The job of this committee is to climb down under the basement of the church once a week to check the level of the water table in the ground. If the water level has receded, additional water must be pumped into the ground to keep the untreated wood piles totally saturated. This practice has been carried out faithfully and on a continuous basis for the past one hundred years.

WOOD PRESERVATION

The history of wood preservation goes back to the ancient civilizations of Burma, China, and Greece, where there were attempts to preserve timber using various animal, vegetable, and mineral oils. Alexander the Great ordered that piles and bridge timbers be covered with olive oil as a precaution against decay. In Roman times timbers were smeared with cedar oils and pitch; some piles were charred. From the decline of the Roman Empire around 450 A.D. no further advances in knowledge of timber preservation were recorded for over a thousand years.

Modern wood preserving began in England in 1832 with the Kyan Process. Here wood was steeped in mercuric chloride. The Burnett Process, which involved the impregnation of wood cross ties with zinc chloride, was patented in 1838 and was used until the 1920's.

Also in 1838, the first Bethel Full Cell Pressure Process plant for injecting coal tar creosote into the wood by pressure went into production in England.

The first Kyanizing plant in the United States was built in 1848. The first pressure creosoting plant was opened by the Dighton & Somerset Railroad (which became part of the New York, New Haven & Hartford System) in Somerset, Massachusetts, in 1865 for the treatment of 700 piles for use in constructing a bridge over the Taunton River at Taunton, Massachusetts. The creosote was imported in barrels from England. In 1872 the Navy installed a creosoting plant at the Charleston Navy Yard in Massachusetts and began treating timbers for construction purposes in 1875.

In 1880 the American Society of Civil Engineers Committee on Preservation of Timber was appointed and began a thorough study of wood preserving practices. The five-year study was presented in 1885. This study served to clear up much of the fog which existed on types of preservatives, treating methods and reasons for failures. It also illustrated the importance of full penetration of the preservative into the wood cells and retention of specific quantities of preservative based on expected exposure.

The first use of creosoted timber piles was by the railroad industry; it started around 1880. Between 1879 and 1881 the Southern Pacific Railroad installed creosoted Southern Pine piles for the abutments and pier foundations on their bridge over the Atchafalaya River between Morgan City and Berwick, Louisiana. When piles were removed 88 years later, it was discovered that the original piles were about 116 ft long and were spliced at the 38 ft point. Splices were made with wrought iron plates 1/4 in. thick and 7 ft long, connected with 1¼ in. bolts. When the piles were exposed to the atmosphere, creosote bled freely for one month. Tests made later indicated the top section piles had an average retention of 31.67 PCF of creosote, and the lower section piles had an average retention of 39.34 PCF. The pile sizes varied from 21 in. to 24 in. butts at the cut off and 15 in. to 18 in. at the tips.

In 1902 Wasserman invented the first empty cell process and Rueping put it into practice. This process, now commonly known as the "Rueping Process," made creosote treatment more economical by removing excess preservative from the wood cells by vacuum at the end of the treating cycle.

At that time 15 wood treating plants were operating in the United

States. The Illinois Central Railroad began installing creosote timber piles. The Illinois Central made a check of these piles 60 years later and determined they had the same preservative retention as when they were installed.

The American Wood Preservers Association was founded in 1904. Its annual meetings became then and remain a forum for discussion and interchange of ideas by representatives of all interested disciplines and also for standardization of specifications for preservatives and their application to wood. Membership in the AWPA is open to not only the wood treating industry, but also to engineers with railroads, utility companies, government agencies, and in private practice.

In 1905 the Atchison, Topeka & Santa Fe Railroad constructed a grain elevator in Kansas City using 2,078 creosoted wood piles. Four additions were made during the next 25 years adding 21,064 additional creosoted wood piles. The original piles were checked during subsequent construction operations and found to be in their original condition.

In 1906 Lowry developed his empty cell process. This process is identical to the Rueping empty cell process, except without initial air pressure.

By 1920 there were about 100 wood preserving plants in the United States. In 1921 the American Wood Preservers Institute was formed to serve as the consulting and educational arm of the wood preserving industry.

In 1932 debarking and peeling machines for providing smoothly shaved surfaces on timber poles and piles were introduced in this country. Up until this time piles were hand peeled. Hand peeling remained a very common practice with many timber pile producers through the mid-1970's.

A new wood preservative, Pentachlorophenol, was introduced during the 1930's; it has been used as a piling preservative on a number of projects, including some 30,000 treated timber piles for the Monsanto Chemical Plant in Texas City, Texas. Pentachlorophenol is not effective against marine borers; therefore, it is not recommended for piles exposed to salt water. Pentachlorophenol for piling is normally used with oil as a carrier so its appearance and characteristics are very similar to creosote.

Waterborne preservatives, starting with the Kyan Process, have experienced a tremendous amount of improvement through research and development over the past 80 years. The problem of leachability, which was so prevalent up to the World War II era, has now been overcome. Today two of these waterborne preservatives are utilized in the treatment of piling: Douglas Fir is normally treated with Ammoniacal Copper Arsenate or ACA, and Southern Pine is treated with Chromated Copper Arsenate or CCA. For the most part, the waterborne salt treated piling is used for waterfront and marine construction, especially in Southern waters, where it resists limnoria tripunctata marine borers.

Whang-Poo

Of course, no history of timber piles would be complete without mention of the series of studies on bearing capacities of friction piles carried out by the Whang-Poo Conservancy Board in Shanghai, China. Piles tested there were in unstable soil extending to depths of several hundred feet. The conclusions of the study were:

1. Failure of piles was due to shearing strength of the surrounding soil being exceeded.
2. Wood piles and piles of other materials having the same shape have the same bearing capacity per unit area of submerged part.
3. Round tapered piles have higher bearing power than straight piles of square section.
4. Safe bearing power of different shape piles can be tabulated in pounds per square foot in contact with the soil.

Information from this and similar studies led Robert Chellis to say: "It makes no difference to the soil what type of pile is in it and no difference to the soil how it was driven there. The main fact is that a pile of the same dimensions driven to the same tip grade should be capable of carrying the same load, as far as the soil is concerned."

1939 World's Fair Re-Use

No recorded rot or decay failures to pressure-preservatively treated buried timber foundation piles are known to the AWPI. Many checks have been made. One of particular interest is the Unisphere, which was built for the 1939 New York World's Fair. It is 13 stories high, weighs 900,000 lbs, and sits on a 20 ft wide base. The foundation for the base consists of 528 creosoted Douglas Fir piles 95 to 100 ft long. The piles were driven into the marshy soil of Flushing Meadows Park where they have been exposed to the organic acids of that area.

General William Whipple, Jr. (retired), Chief Engineer of the 1964-65 New York World's Fair inspected piles for reuse there 25 years later. The consulting firm of Mueser, Rutledge, Wentworth & Johnston made an independent investigation. Both reports showed the piles to be completely sound; the piles were reused for the 1964-65 World's Fair.

Dynamically Cast-in-Place Piles

by

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Consultant, Calgary, Alberta

Allowable loads on Dynamically-Cast-In-Place Piles (the Franki type) can be correlated with N values from the Standard Penetration Test. Using a constant developed by Nordlund, a conservative value for load-carrying capability of a DCIP pile is developed. The DCIP pile and method of installation is briefly described.

It has been the writer's experience that Dynamically Cast-in-Place Piles (DCIP) are generally the most versatile and suitable to adequately perform under design loading up to 400 kips. For a large variety of soil conditions and profiles they are often the most economical option.

DCIP piles can be constructed in and through soils ranging from granular to fine grained and varying from dense to loose non-plastic or stiff to soft high plastic. They are most advantageous when the required pile must be constructed through soft and/or wet soils to a more competent underlying stratum. The only rigid requirement for their use is that the soil profile provide a stratum competent to permit formation of an adequate base within a depth range specified by geotechnical design, dictated by site conditions and/or limited by the equipment used.

DCIP Pile Driving Formulas

In 1970, R. L. Nordlund, now Vice President of Franki Foundation Company, Arlington, Virginia, in an article entitled "Pressure Injected Footings" described the pressure injection of concrete into a soil profile to form a dynamically cast-in-place foundation system. His presentation was founded on a theoretical background complemented by experience. Included in the publication was the formula:

$$L = K B W H V^{2/3}$$ for allowable loads on DCIP piles based in a granular soil

Where

L denotes allowable load in kips
B denotes number of blows required to expel the last 5 cu.ft of concrete into the expanded base
W denotes weight of the hammer in kips
H denotes height of the hammer free fall in feet
V denotes total volume of concrete in the expanded base in cubic feet
K denotes a dimensionless constant

Based on experience and the review of many load tests, Nordlund found that for a granular soil, a value of 1/60 was a conservative figure for K and, as the soil becomes coarser grained, this value becomes more conservative. The writer, after considering Nordlund's report and having personal experience with the construction of DCIP piles, found correlation between the Nordlund constant K and the Standard Penetration Test N values.

Construction of DCIP Piles

To consider the capacity of DCIP piles, as determined from a pile driving formula, an understanding of the construction procedure is essential. Three procedural parts are discussed separately: 1. Driving of the Tube; 2. Construction of the Base; 3(a). Dynamically Constructed Concrete Shaft; or 3 (b). Cast-in-Place Concrete Shaft.

1. Driving the Tube – The process of installation requires first the driving of a very heavy-walled tube into the soil. The tube is driven by placing a small quantity (3 to 5 cu.ft of "zero slump" concrete, 1½ in. road crush gravel, etc.) of suitable plug material into the bottom of the soil-supported tube. A hammer weighing around 7,000 lb operating inside the tube is then dropped on the plug from increasing heights causing compaction and friction bonding of the plug to the inside of the tube. The very high force of the continued hammer drops literally pulls the tube down. When the required depth has been reached, the tube is restrained at the top and most of the concrete plug driven out.

2. Construction of the Base – Rigid restraint of the tube is continued after the plug has been driven out. Successive small charges of zero slump concrete are expelled from the bottom of the tube by applying controlled internal hammer blows of very high energy (100,000 to 240,000 ft-lbs per blow) to each charge. This expulsion of zero slump concrete densifies the surrounding soil while forming the expanded base. By closely observing the depth to which the hammer drops, as it expels the concrete, material can be added in time to maintain the compacted concrete surface just above the bottom of the tube to ensure an adequate seal against the entry of water and/or soil.

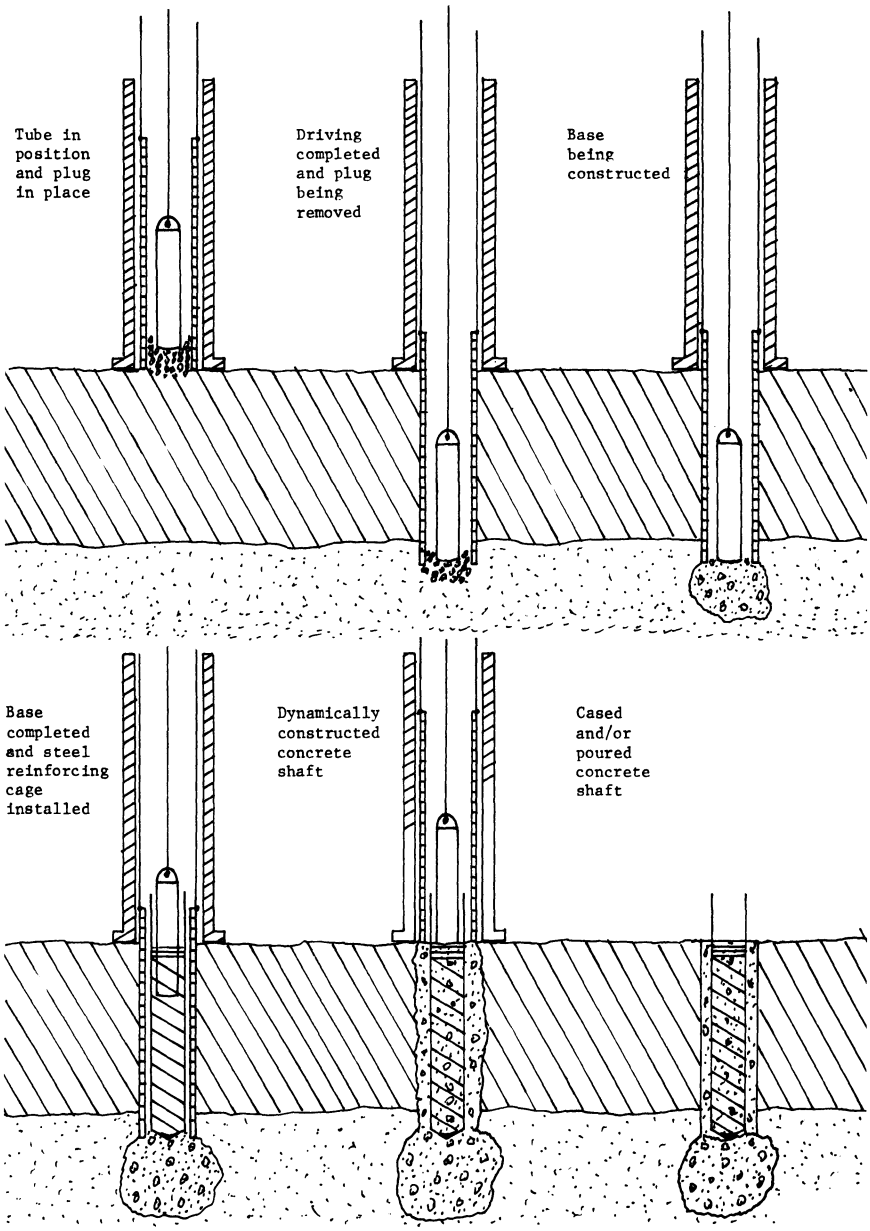

Tube in position and plug in place
Driving completed and plug being removed
Base being constructed
Base completed and steel reinforcing cage installed
Dynamically constructed concrete shaft
Cased and/or poured concrete shaft

PLATE 1

Sufficient concrete and energy are used to construct the pile base. If full length pile shaft reinforcement is required, the hammer can be removed and a steel-bar box-end, spiral-encased reinforcing cage dropped into the tube. If so, the hammer is then inserted in the cage and lowered so that the box-end bottom of the reinforcement is forced into close contact with the still fresh concrete of the expanded base. Alternatively, if only partial reinforcement is required, the steel cage can be placed part way up the pile shaft using the same installation procedure. For cold region pile construction, uplift on peripheral piles must be considered because of frost heave forces. The unreinforced tensile, shear and bending strength of both "zero slump" and "wet" concrete is only a relatively small fraction of the compressive strength.

3 (a). Dynamically Constructed Concrete Shaft — After the expanded base has been completed and, if required, the steel reinforcing cage installed, small charges of concrete are placed in the tube and the hammer rested on the top of each charge while the tube is raised slightly. Each tube charging and lifting is followed by several blows of reduced energy (25,000 to 60,000 ft-lbs per blow) to both compact and expel concrete from the bottom of the tube. The concrete is forced around any steel reinforcing bars by the surrounding soil and forms a rough surfaced shaft for the pile. Inside the tube, until shaft construction reaches the pile cut-off elevation, the concrete surface is kept just above the bottom of the pipe by controlling the hammer drop so that a seal against earth contamination of the concrete is maintained.

3 (b). Cast-in-Place Concrete Shaft — The shaft can be permanently cased through soft upper materials. After the concrete base has been constructed, the hammer is removed and a steel pipe (smooth or corrugated) is dropped inside the driving tube. The pipe inside diameter is large enough so that the hammer easily fits inside. A zero slump concrete plug is formed inside the pipe using a procedure similar to, but with a lower energy input than that which was previously used to form a plug in the driving tube.

The result is a good contact between the plugged pipe and the expanded base; in addition, the seal against contamination of the concrete from material which will surround the completed pile is maintained. The hammer is left resting on top of the internal pipe plug and the driving tube is pulled. The hammer is then removed, reinforcement that may be required is placed, and the pipe filled with plastic concrete. The result is a "column" supported by the compacted concrete base "footing."

Alternatively, after the concrete base has been constructed and a steel-bar reinforcing cage installed, the tube can be filled with a fairly high (5 to 8 in.) slump plastic concrete. The tube is then pulled from the soil allowing the concrete to flow outward and to fill the hole above the base. The result is a DCIP pile base supporting a "drilled" cast-in-place pile

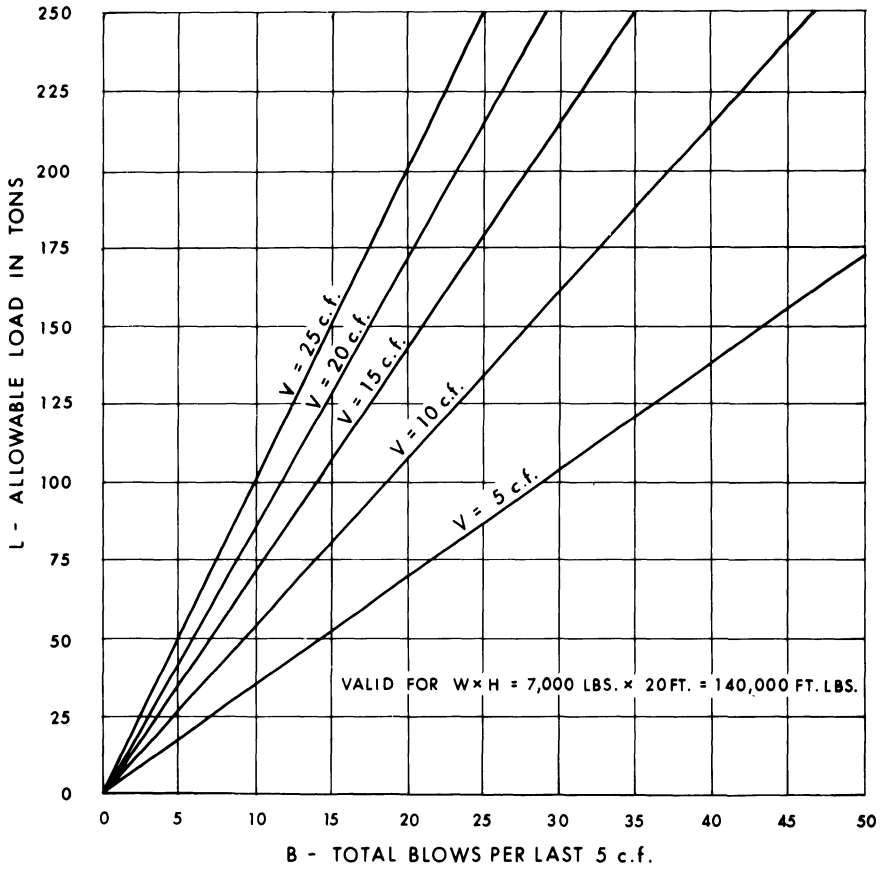


PLATE 2

shaft. This requires exceptional attention and inspection to assure that there is no earth intrusion or column separation.

Correlation Procedure

To correlate the K constant used in Nordlund's formula and the N values obtained from Standard Penetration Tests, a first assumption was made that for a granular soil with K = 1/60, the factor of safety would be greater than 2. Therefore, for an *ultimate or failure load*, K = 1/30 = 0.333 would be conservative. Based on experience and information available, a second assumption was made that for a medium dense granular soil, N = 60 would also be conservative. A third assumption was made that the pile shaft length in such granular material would not be more than 10 ft. These three assumptions were then used to determine the theoretical base *ultimate capacity* of approximately 603 kips for a pile having a 5 cu ft base which required 45 blows of a 7 kip hammer dropping 20 ft to expel the concrete. Adding a theoretical shaft friction *ultimate*

$L = KBWHV^{2/3}$

DO NOT EXTRAPOLATE.

IF N<10 AND CAN FORM BASE, USE K=0.025

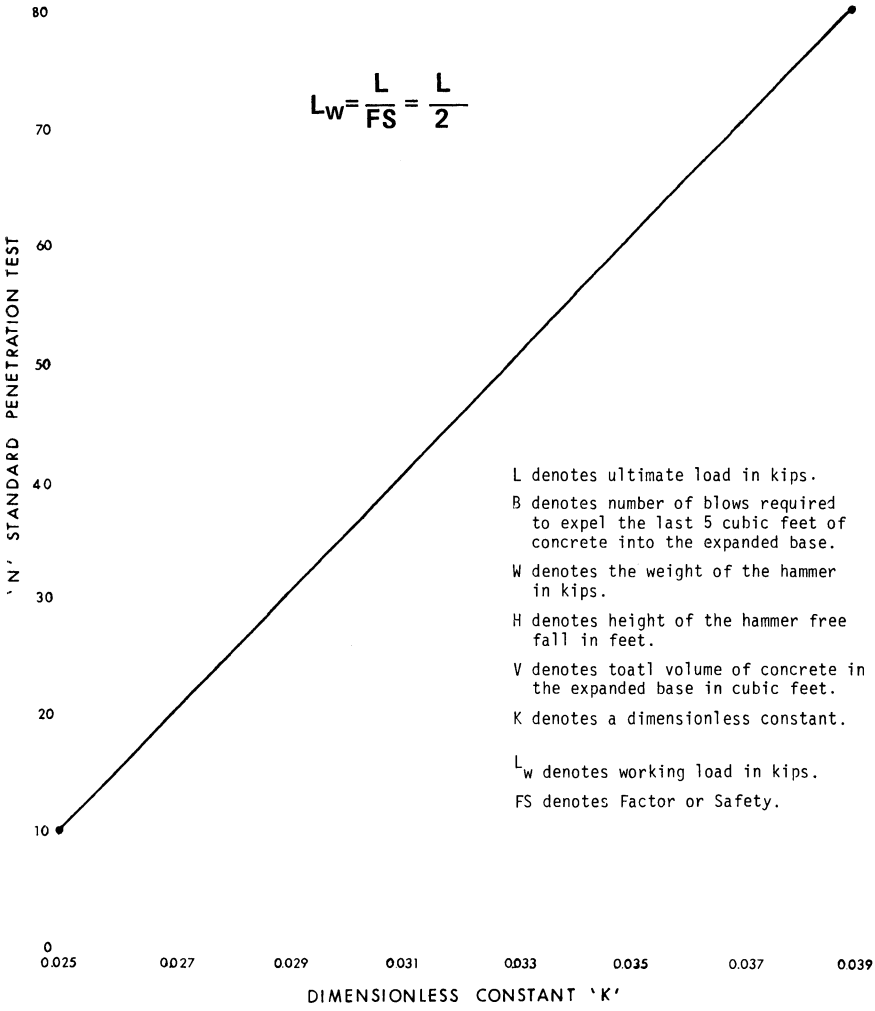


PLATE 3

capacity of approximately 30 kips gave an indicated pile *failure load* of 633 kips.

$$L = KBWHV^{2/3} + 30 \text{ kips} = 0.033(45)\ (7 \text{ kips})\ (20)2.9 + 30 \text{ kips} = 633 \text{ kips}$$

This corresponds well with the *allowable load* of 316 kips, if a *factor of safety of 2* is used, as shown on Nordlund's graph, Plate 2. Increasing the base volume to 25 cu ft and reducing the blows required to 20 (all other criteria kept the same) also provided a good correlation with Nordlund's graph. Therefore, to include some load capacity from pile shaft friction, for N= 60 a K value of 0.035 was used as a first correlated point.

By considering a number of pile load test reports, which included nearby test hole logs where Standard Penetration Tests had been conducted, and extrapolating when necessary to determine an approximate pile *failure load*, more than twenty K and the corresponding N values were obtained. The reports were from tests conducted at Edmonton, Calgary, Lethbridge, Grande Cache and Empress in Alberta and from Richmond in British Columbia. The soil types varied from clean and dirty granulars to low, medium and high plastic clays. The N values varied from 7 to 120.

A settlement of 0.6 in. was used as failure in determining the ultimate loads. Plate 3 graphically presents the best fit curve obtained from a plot of the N versus K values. As the K values were obtained without regard to the pile shaft length or any skin friction allowance, the *ultimate loads* predicted when using Plate 3 and the Nordlund formula are conservative.

Pile shaft lengths are generally a function of the parameters for the soil overlying the pile base and are determined by the depth required to reach an adequate base location. Therefore, a short shaft tends to have a higher unit skin friction than that available where a longer shaft is required. The result is a relatively consistent relationship between base construction criteria and pile load capacity for most shaft lengths. Any exception imposed due to extraneous conditions can be considered by adjusting the calculated capacity accordingly.

During the ten years since the curve on Plate 3 was first obtained, it has been used along with Nordlund's formula to estimate allowable DCIP pile loads and /or construction criteria for a large number of construction projects. It has also been used to predict failure loads for pile load tests conducted at most of the same large construction sites. None of the piles tested failed at less than the *ultimate loads* predicted. The failure criteria used was that specified by the National Building Code (Canada) or that recommended in the National Research Council publication "Canadian Foundation Engineering Manual." In each case, the predicted *ultimate load* was significantly less than the test indicated *failure load*.

For one pile load test, it was decided by others to use the much larger double design load for the project on a pile designed by the curve and

formula for a load equal to double that particular design load. The pile exhibited rapid settlement under a test load of about 70% of the project double design maximum load. The load was removed and a subsequent test to double design load for that pile was conducted with a corresponding negligible total settlement resulting.

Nordlund's formula and the K values determined from Plate 3 have also been used to provide schedules of minimum energy requirements for various base sizes and hammer weights. Subsequent pile load tests have confirmed the schedules as conservative. These schedules have been especially useful for both pile construction crews and for pile inspection teams. The procedure is also very useful in making predictions for basing depths and the relative economic analysis. The writer has found the formula and chart reliably conservative, both when used for Geotechnical Engineering and as a Pile Foundation Construction Guide.

N values of less than 10 have not been considered on Plate 3. The reason being that it would generally be difficult to form an adequately compacted concrete pile base when the Standard Penetration Test results indicated a soil condition with N = 10 or less. A maximum K = 0.039 has been used for N = 80 or more. It has been the writer's experience that Standard Penetration Test results of 80 or more are often due to a cobble located in the line of penetration with a resultant falsely high indication. Such relatively small obstructions would provide negligible resistance to the driving of a tube for a DCIP pile. No attempt at extrapolation or correlating for such high N values should be made.

Chart Adjustments

The K values on Plate 3 can be adjusted if pile load test results are made available prior to or during contract pile construction and a reasonable homogeneity of site soil has been determined. The blows (B), the hammer weight (W), drop distance (D), concrete base volume (V) and the indicated *failure load* (L) for the tested piles can be used to calculate a Kc value. The line on Plate 3 can then be parallel adjusted so that the Kc value corresponds to an N value previously determined for the tested pile base location. The formula using the adjusted K values, will then provide an evaluation for previously and/or subsequently constructed contract piles to determine their adequacy if the construction procedures vary from that of the tested piles. The improved advantage with respect to construction, inspection and economics of the Nordlund formula are also available after using site pile load test results to adjust the K values.

Base Volume Limits

Except for very light loads or where the base is constructed in a dense granular soil, more than a 5 cu ft base is recommended. The influence of

water, boulders, soil non-homogeneity, etc., at the base depth can produce misleading blow counts. If the blow counts reduce for a succeeding concrete charge, it is an indication that the soil shear strength has for some reason been reduced. Additional 5 cu ft buckets of concrete should be used, up to a maximum of five, until an increase in blow count between succeeding buckets is achieved. If unable to obtain the blow count increase and five buckets have been used, the plug should be re-installed and the pile driven deeper to a more competent soil. This monitoring method is not available when only one 5 cu ft bucket of concrete is used for pile base formation. The use of more than five 5 cu ft buckets of concrete for pile base construction is not recommended. If an adequate blow count cannot be achieved for five or less buckets, a very poor and unpredictable soil condition is indicated at base depth. The concrete could be moving in any direction, even upward, as it is expelled from the tube.

Advantages of Dynamically Cast-in-Place Piles

- By literally pulling the thick-walled steel tube into the soil, there is very little danger of contamination in the form of water, soft soil and the like entering. The tube, with a plug at the bottom, presents a closed system except at the top which is kept above ground surface. Therefore, DCIP piles can be constructed in wet or dry material, which would normally have a tendency to flow into an uncased drilled caisson pile. DCIP piles can also be constructed with a base in or on more competent material after the tube has been driven through fine grained or granular soil, which is wet or loose or has a soft consistency. The only requirement is a soil competent for the formation of an adequate base overlain by a soil mixture having sufficient shear strength to enable formation of the pile shaft (column) connecting the load to the base (footing). Any shaft friction available improves the total pile capacity.
- The application of extremely high energy blows used in forming the pile base provides a very dense concrete "footing" through which column loads are distributed to the soil. Also, while the base is being constructed, the bearing stratum is improved due to densification caused by the concrete injection.
- In most soils, additional pile load capacity is developed during dynamic shaft construction. The concrete is forced out of the tube bottom with a resultant surrounding soil densification. This increases the frictional forces of the soil which are applied against a very rough and irregular shaft surface.
- In some cases, corrugated steel pipe shaft enclosures can be used to provide an irregular surface and thereby increase the shaft friction from a soft surrounding soil.

- The effects of local variations and non-homogeneity of soil types and density at any one site are minimized due to the general upgrading of the properties at each pile location as a result of the high energy input used during construction.
- By constructing the first contract and/or test piles near logged test hole locations, and recording constant energy driving blows, a blow count per foot versus depth of penetration graphical illustration can be made to determine a good approximation of the soils profile across a site as additional piles are driven. This procedure can also provide driving criteria to ensure piles are based in the soil strata used for design.

Disadvantages of Dynamically Cast-in-Place Piles

- Most "displacement" piles will cause some movement of previously constructed adjacent piles if the separating distance is relatively small. "Fresh" concrete in adjacent piles can shatter or crack due to both vibration and uplift forces. Therefore, the inclusion of full length steel reinforcing should always be considered for DCIP pile shafts. Two piles less than nine pile diameters apart should not be constructed in the same 24-hour time period unless monitoring indicates pile movement at lesser spacing is not significant. The amount of soil displaced during the normal construction of a DCIP pile is quite large due to the procedure used.

  The effect on an adjacent pile can be two-fold. During driving of the tube, base formation and while the shaft is being constructed, uplifting forces can be sufficient to cause the adjacent shaft to separate from the base. The capacity of the pile is then reduced very significantly to only that available from skin friction for the shaft. While the base of the pile is being constructed, the soil displacement also creates uplifting forces on a nearby pile base. This can cause an upward movement of the entire adjacent pile. Such uplift should not result in a reduction of capacity and may even increase capacity due to the additional soil densification.

  The upward movement of an adjacent pile shaft due to the uplift forces resultant from pile construction can be greatly reduced or prevented by two procedures used singly or in combination. The most effective is constructing a pile, which may be subject to shaft uplift, as a "tension" pile. The procedure requires a gradual reduction of the energy input and the concrete volumes expelled as construction proceeds from base to shaft after the reinforcing cage has been firmly put in contact with a small pocket in the pile base. The result is a rough truncated cone forming the bottom of the shaft. This shape makes upward movement of the shaft very difficult. Predrilling can also be used to reduce or prevent adjacent pile uplift. The pre-drilled hole

reduces the volume of soil displaced during driving and shaft construction and thereby reduces the lifting forces on a previously constructed pile.

- Vibration caused by the construction procedure for DCIP piles can damage nearby structures and recently constructed adjacent piles. If vibrational effects on the surroundings are above tolerable levels, the input energy can be reduced, predrilling can be done or another pile type chosen.
- Reinforcement cages are fabricated by tying or tack welding the vertical bars to encircling spiral steel. Bar fracture under tension or bending loads is possible and has been observed by the writer due to underbead cracking at the welds. It is important that the bar carbon content is not too high, that the right welding rod and amperage is used, and that the welds are not allowed to "quick" cool.
- The driving equipment is bulky and cumbersome. Relatively large and open working areas are required.
- "Zero Slump" concrete is more expensive than plastic concrete. Also, the moisture content, aggregate sizing and mixture is more difficult to control. "Good Zero Slump Concrete" is worth the extra cost, time and effort required as it will produce 28-day compressive strengths much above the design requirements.

Foundations For The Sohio Corporate Headquarters

A presentation at the Deep Foundations Institute Annual Meeting
Cleveland, Ohio, November 1983

by

Clyde N. Baker, Jr., P.E.
STS Consultants Ltd.

Foundations for the Sohio Corporate Headquarters in downtown Cleveland have column loads of 11,400 kips, which increase to 15,000 kips with wind loading. An extensive exploratory program was conducted and comprehensive studies made to select the economically dependable support for the 46-story structure. A test on bond or socket friction for the caissons has usable values.

The SOHIO Corporate Headquarters project currently under construction in redeveloping downtown Cleveland consists of a 46-story tower and nine-story atrium. Maximum column loads in the tower reach 11,400 kips for dead plus live load and up to almost 15,000 kips when considering maximum windloading. Basement levels vary from El. 75 to El. 69 referred to first floor as El. 100, about sidewalk level at Euclid Avenue on the south side of the project.

This paper describes the foundation exploration program, the foundation design considerations, the different foundation systems analyzed, and the construction instrumentation program undertaken to confirm the design assumptions.

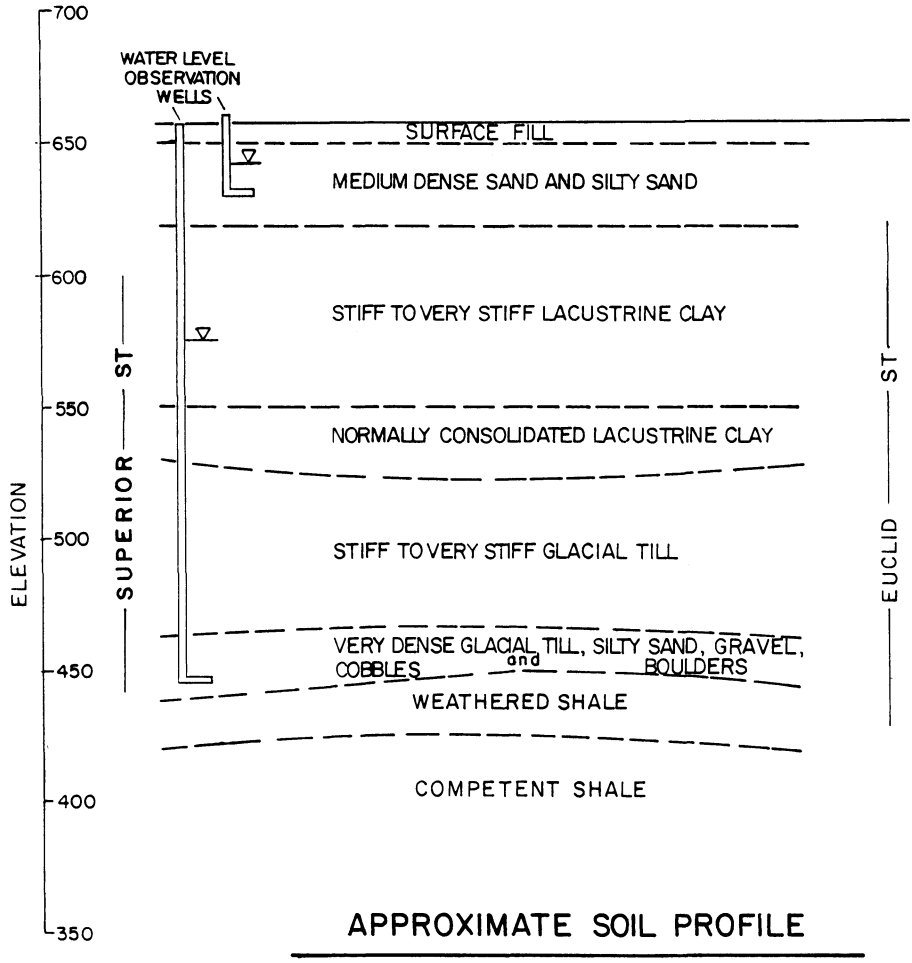
Exploration Program

The foundation exploration program was divided into two phases. The first phase was undertaken while buildings still occupied the site and consisted of five borings taken 30 to 50 ft into shale bedrock, plus two piezometer installations. The borings were performed by conventional truck-mounted drilling equipment, using casing to maintain the holes open and drilling water and drilling tools to advance the holes. Standard split-barrel and shelby tube sampling procedures were used in the borings.

Due to time constraints and construction schedule, the final foundation

SOHIO CORPORATE HEADQUARTERS

DOWNTOWN CLEVELAND, OHIO



APPROXIMATE SOIL PROFILE

FIGURE 1

design had to be completed on the basis of this first phase investigation, but with the understanding that the final confirmatory investigation would be performed after building demolition. This second phase or final exploration program consisted of 19 more borings located at major column locations and was for the purpose of determining the level of sound, unweathered shale and the final design level for caisson foundations designed to be socketed into the shale carrying their load partly in side friction and partly in end bearing. Three additional borings with in-situ pressuremeter testing were performed in the area of the low-rise structure, making a total of 27 borings for both phases.

Soil and Ground Water Profile

The soil and ground water profile is shown in Fig. 1. As noted on the figure, surface fill underlain by fine alluvial sand and silty sand was encountered to El. 620 (depths ranging from 32 to 45 ft) and was underlain by normally consolidated to moderately preconsolidated lacustrine silts and clays to depths of 140 ft (El. 520) where very stiff glacial till was noted; this continued to very dense till at elevations ranging from 464 to 479. The very dense till is 5 to 20 ft in thickness and is underlain by a deposit of extremely dense silty sand to coarse sand, gravel, and cobbles to the weathered shale surface at elevations ranging from 431 to 458. Competent shale was generally noted at El. 420 to 433.

A ground water table was noted in the upper sands at around El. 640; a separate water table was noted in the sand and gravel layer immediately overlying the weathered shale. Water level measurements in the two piezometers indicated a water head in the range of El. 570 to 590. Gas was observed in one of the five initial borings performed.

Foundation Design Considerations

Due to the complexity of the soil profile and the advantages and disadvantages of various foundation systems, a single preferred foundation system for support of the 46-story tower structure was difficult to establish. A number of factors went into the final considerations for recommended foundation systems. These include: the recent local downtown foundation construction history (including pile foundations, caisson foundations, and mats); the downtown setting where minimizing noise level is desirable; high uplift forces at certain key column locations, differential settlement criteria, particularly with regard to the high-rise and contiguous low-rise structure; and the gas condition observed at the shale level. The influence of these factors on the ultimate selection is discussed later.

The routine laboratory testing program consisted of performing unconfined compression, water content, and density tests on shelby tube samples taken in cohesive soils. In addition, consolidation tests and

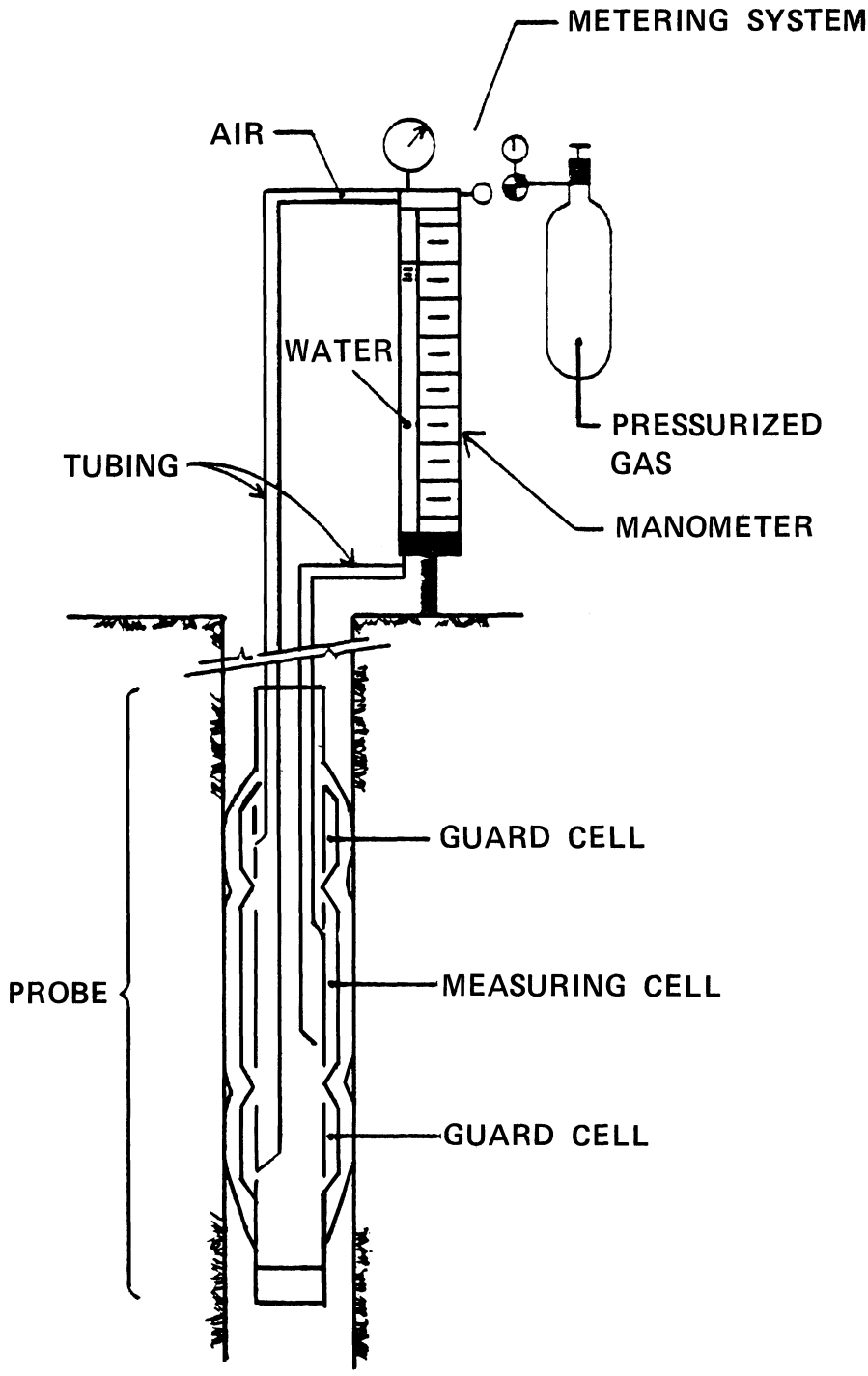


FIGURE 2 Schematic of Pressuremeter in Borehole.

liquid limit tests were performed on representative samples from selected tubes. Grain size tests were performed on representative samples taken in granular deposits with the split-barrel sampler. The grain size tests were helpful in assessing soil permeability, and the consolidation and liquid and plastic limit tests were helpful in assessing preconsolidation pressures in the clay soils relative to predicting settlement for footing and mat foundation solutions.

In-Situ Pressuremeter Testing Program

Menard pressuremeter tests were performed in the upper soils to get additional information on soil deformation for use with footing and mat foundation solutions and in the deep, very dense glacial till soils for use with possible deep foundation solutions. Fig. 2 shows a schematic of the pressuremeter, and Fig. 3 a typical test result. The pressuremeter consists of a radially-expandable probe which can be lowered into the borehole to the desired test depth. The probe is then expanded in increments against the borehole walls while monitoring both pressure and volume deformation. In this way, it is analogous to an in-the-ground load test; useful information can be obtained relative to soil modulus, soil bearing capacity, and soil shear strength. A value known as the creep pressure is also obtained. This is the point at which there is a noticeable increase in deformation under constant load between the 30-second reading and the one-minute reading. Experience has indicated that the creep pressure in preconsolidated cohesive deposits is fairly close to the preconsolidation pressure.

Special Testing Program

To assess caisson foundation solutions involving sockets into the shale a special laboratory testing program was set up to measure the bond or socket friction available between shale and concrete. Fig. 4 shows the laboratory test setup. The testing program involved simulating both the dry concrete placement condition and the wet concrete placement condition by testing core specimens that had been either air-dried or water-soaked prior to placing the concrete around the shale core. In addition to the wet/dry variable, the additional variable of pressure on the concrete during setting was simulated. This was done by taking the two extreme conditions; namely, concrete setting around the shale core with no pressure at all and the other extreme of concrete under a pressure head equivalent to 200 ft of fluid concrete.

The test results are shown in Table 1 and indicate the very considerable effect that curing of concrete under pressure can have on increasing the bond strength. The core for test No. 7 was saturated in water for six hours prior to testing and the core for test No. 8 was air-dried for one hour prior

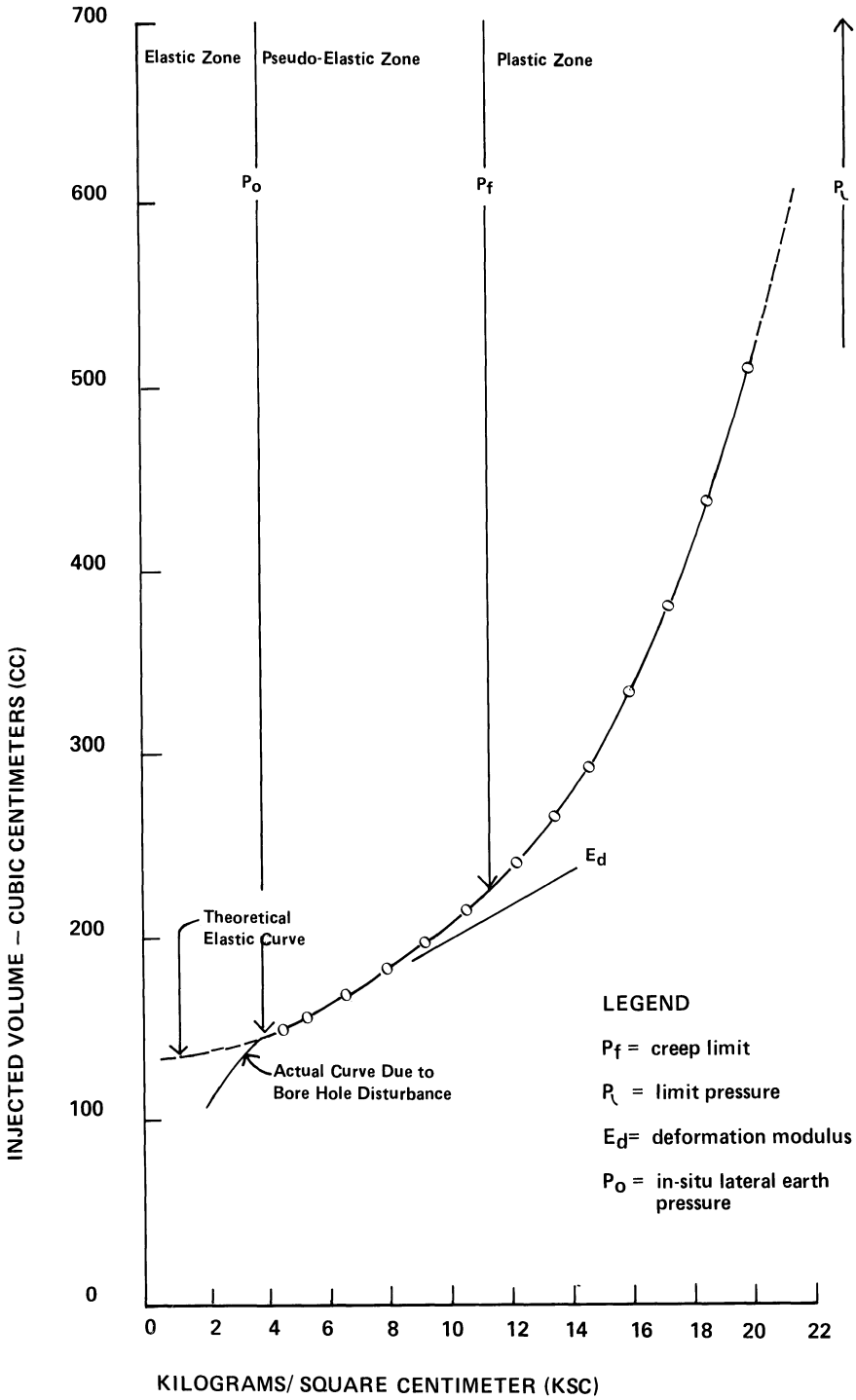


FIGURE 3 Typical result from Menard Pressuremeter test at Sohio building in Cleveland.

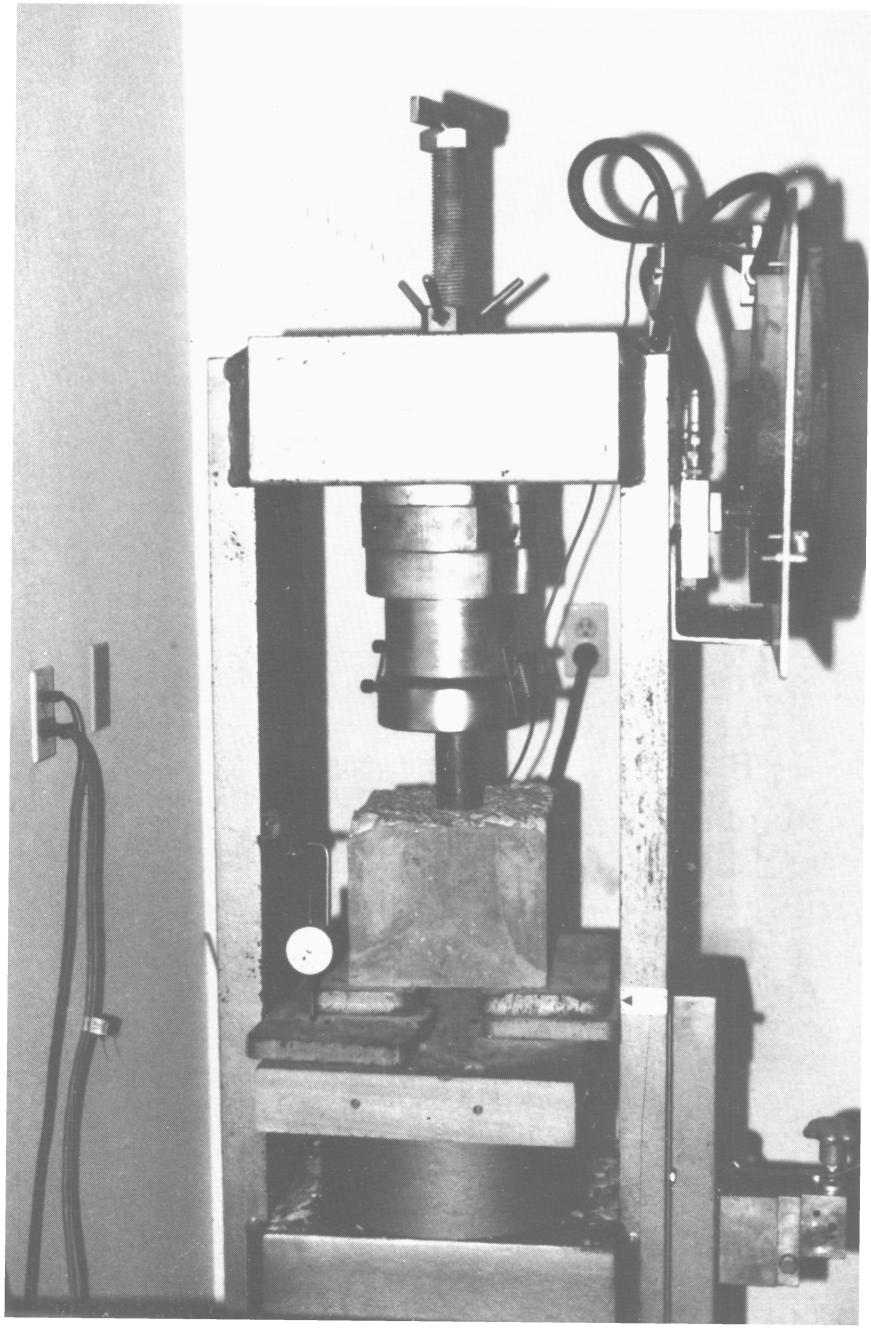


FIGURE 4 Laboratory set-up for testing the bond or socket friction between shale and concrete.

TABLE 1

SUMMARY OF ADHESION TESTS
SOHIO CORPORATE HEADQUARTERS
CLEVELAND, OHIO

Test No.	Moist Unit Weight (PCF)	Core Length (in.)	Maximum Bond Stress (PSI)	Strain At Maximum Bond Stress (in.)	Residual Bond Stress (PSI)
1	158	8.2	321	0.067	180
2	159	7.1	233	0.038	205
3	153	9.1	361	0.082	205
4	154	7.7	157	0.050	85
5	155	7.8	207	0.075	140
6	——	6.8	323	0.135	155
7*	——	10.8	760	0.09	800
8*	——	9.5	1180	0.25	1180

*Concrete cured under confining pressure equivalent to the fluid head of 200 ft of concrete.

to testing. Representative core specimens were also tested for unconfined compression and Young's modulus; results of these tests are noted in Table 2. The effect of air-drying on increasing strength and modulus is evident from these test results. In addition, representative core specimens were also water-soaked for an extended time period (seven weeks) to see if slaking occurred; none was noted.

Foundation Alternatives Analyzed

With regard to the low-rise atrium structure, footing foundations were the primary solution considered because of the obvious cost advantages. A mat foundation was considered only for the possibility of a mat under the entire tower and atrium. Since the weight of soil removed in the basement excavation exceeds the weight of the nine-story atrium structure, there is no concern for deep-seated settlement. The question is merely one of selecting the appropriate bearing pressure and considering compression within the zone of influence of the individual footings. A net allowable soil bearing pressure of 5 ksf in the medium dense sand deposit was recommended, based on the Standard Penetration Test data, experience, and the in-situ pressuremeter test results. Maximum settlement under the heaviest column footing was estimated to be one inch or less.

Under the tower portion, mats, piles and caissons were all considered. With regard to the mat possibility, a definite advantage was that there had been good experience in recent years with support of major structures on mats in downtown Cleveland, albeit not as tall as SOHIO. However, predicted long-term settlement of the mat foundation ranged from 2.5 to 4.5 in., which was greater than desirable for the proposed structure. This ruled out the mat solution.

High-capacity pile foundations were also considered with both pipe piles and H-piles analyzed. Pipe piles offered a cost advantage over H-piles, but had the disadvantage of requiring pre-drilling and, because of the pre-drilling requirement, a less assured resistance in uplift. Both pile solutions had the disadvantage of increased noise factor compared to the other systems.

The third system analyzed was caisson foundations. This was the foundation solution ultimately selected. Because of the known gas situation at the shale level, and because of past problems with caisson construction in Cleveland, effort was concentrated on developing a caisson foundation system which would not require personnel going down to the bottom of the caisson and inspecting the bearing surface. A socket friction approach was developed utilizing the test data obtained from the shale concrete adhesion tests. Based on the data accumulated, a design socket friction value of 160 psi was selected. This value happens to represent the average residual bond stress for the six adhesion tests per-

TABLE 2

SUMMARY OF CORE TEST RESULTS
SOHIO CORPORATE HEADQUARTERS
CLEVELAND, OHIO

Boring	Depth	Moisture*** Condition	Moist Unit Weight (PCF)	L/D	Unconfined* Compressive Strength (TSF)	% Strain** At Failure	Type of Failure	Youngs Modulus, E (TSF)
5	231	1	155	2.9	119	1.1	Shear	32,100
5	233	1	158	2.2	125	0.9	End	34,300
5	236	1	153	1.9	135	0.9	Shear	30,200
5	243	1	161	3.0	85	1.0	Failed on seam	20,200
5	247	2	156	2.5	713	1.4	Shear	70,000
5	251	1	153	1.5	78	0.9	Seam failed	27,300
3	238	3	155	2.8	466	1.2	Shear	60,500
3	240	3	154	2.4	426	1.0	Shear	67,000
4	237	3	162	2.8	247	0.8	Shear	51,800
4	239	3	159	3.1	414	1.0	Shear	68,500
4	243	3	156	3.4	527	0.6	Shear	94,800
4	252	3	155	3.2	457	1.0	Shear	63,000

* ASTM D-2938 ** ASTM D-3148 – 6.4.2

*** Moisture Condition (1) Core samples waxed in field. Tests performed within 30 minutes from removing wax. (2) Core sample left in core box to dry out. (3) Core samples partly coated with acrylic spray in field.

formed where the concrete was allowed to set under zero pressure. It represents about one-fifth the value obtained when the concrete was allowed to set under continuous pressure equal to the value of 200 ft head of concrete.

While no bottom inspection was planned, it was anticipated that maximum possible mechanical clean up would be performed at the bottom of each caisson. For this reason, some end bearing would likely be developed whether it was counted on or not. Taking this into account, the socket penetration into the shale was ultimately based on assuming that the load-bearing surface of the caisson would carry 25 tsf (the Cleveland presumptive bearing pressure for shale), and the socket perimeter area would carry 160 psi. While no tests were actually performed in the weathered shale, it was observed to be fairly thick (typically 20 ft or more). For design purposes, it was assumed that there would be a minimum of 10 ft of weathered shale exposed in the upper socket and that this zone would carry 50 psi in socket friction. These assumptions resulted in design caisson socket lengths into the competent shale varying from 3 to 16 ft.

Specifications were developed for tremie concrete placement and for dry concrete placement. In addition, bottom sounding procedures were developed for checking the cleanliness of the caisson bottoms. For tremie concrete placement, an additional requirement included providing 2 in. inside diameter PVC pipe to the base of the caisson to provide access for non-destructive evaluation of in-place caisson concrete integrity.

Instrumentation for Design Verification

A full-scale caisson load test was required to confirm the design socket friction assumptions. This was scheduled at the beginning of the caisson construction. Figs. 5, 6 and 7 show the caisson load test setup, instrumentation and load test results.

In the load test setup, an effort was made to isolate the test shaft through the overburden down into the weathered shale in order to maximize the amount of load carried by the weathered shale and competent shale. Instrumentation consisted of both Carlson stress and strain meters located at the top, middle and bottom of the socket and special tensioned wire extensometers. The test results indicate that the isolation efforts were not successful in that fine sand leaked beneath the top outer casing and filled the annular space between the inner casing and the outer casings up to the leak points. Because of the resulting friction, about half of the maximum test load of 1,250 tons was carried in the overburden. Fortunately, the remaining load carried by the socket was sufficient to confirm the reasonableness of the design assumptions.

As a final check on the design assumptions, one of the major caissons has been instrumented with Carlson stress and strain meters, and also with

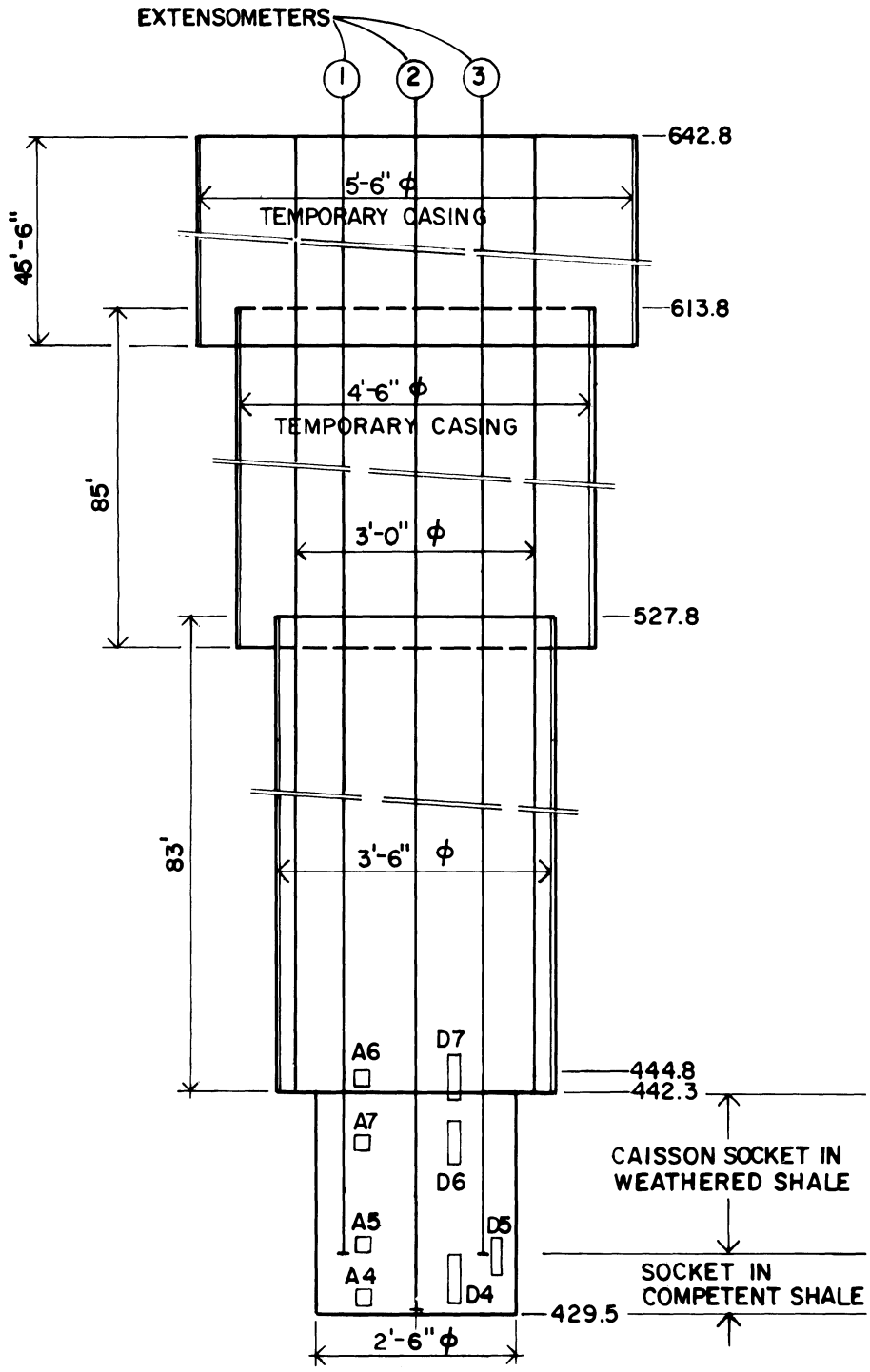


FIGURE 5 Test Caisson Instrumentation Arrangement.

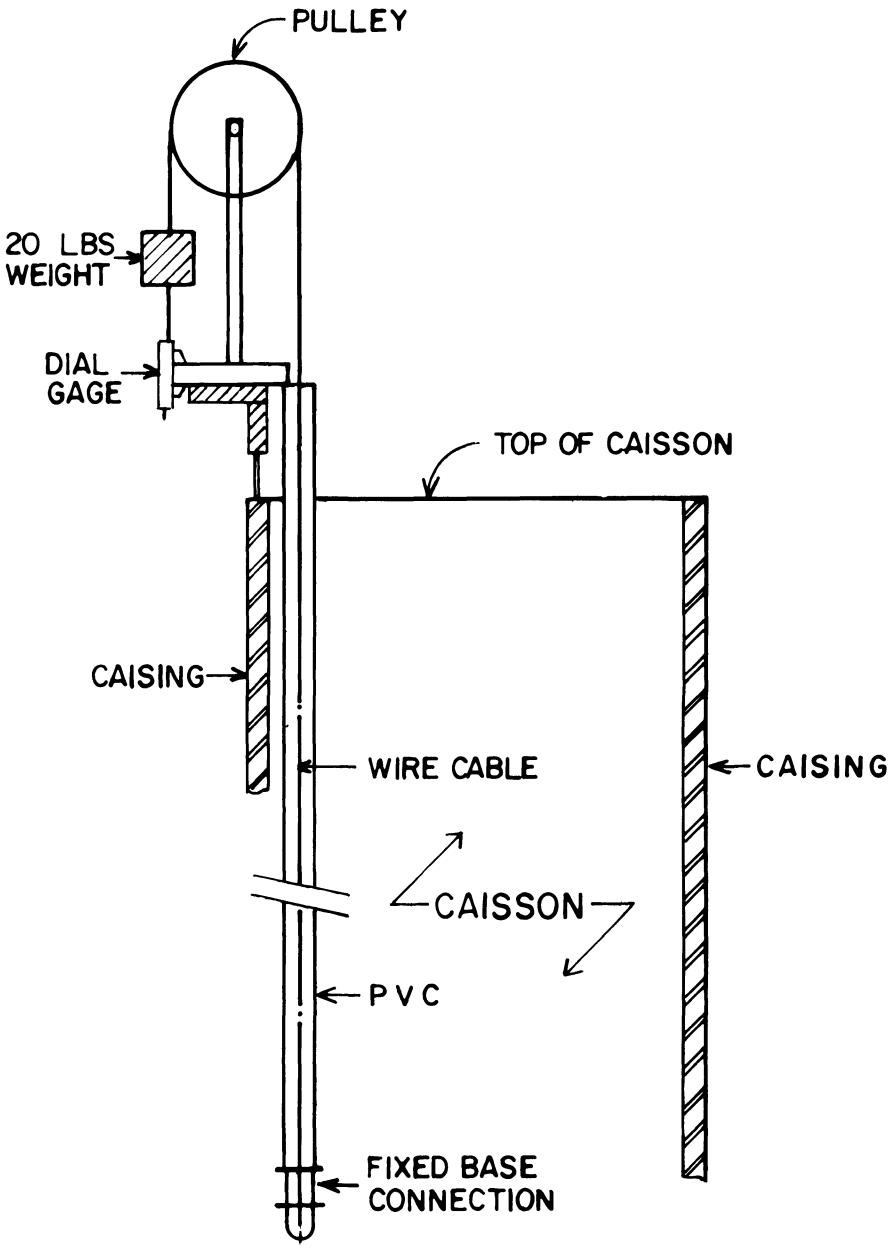


FIGURE 6 Extensometer Set Up.

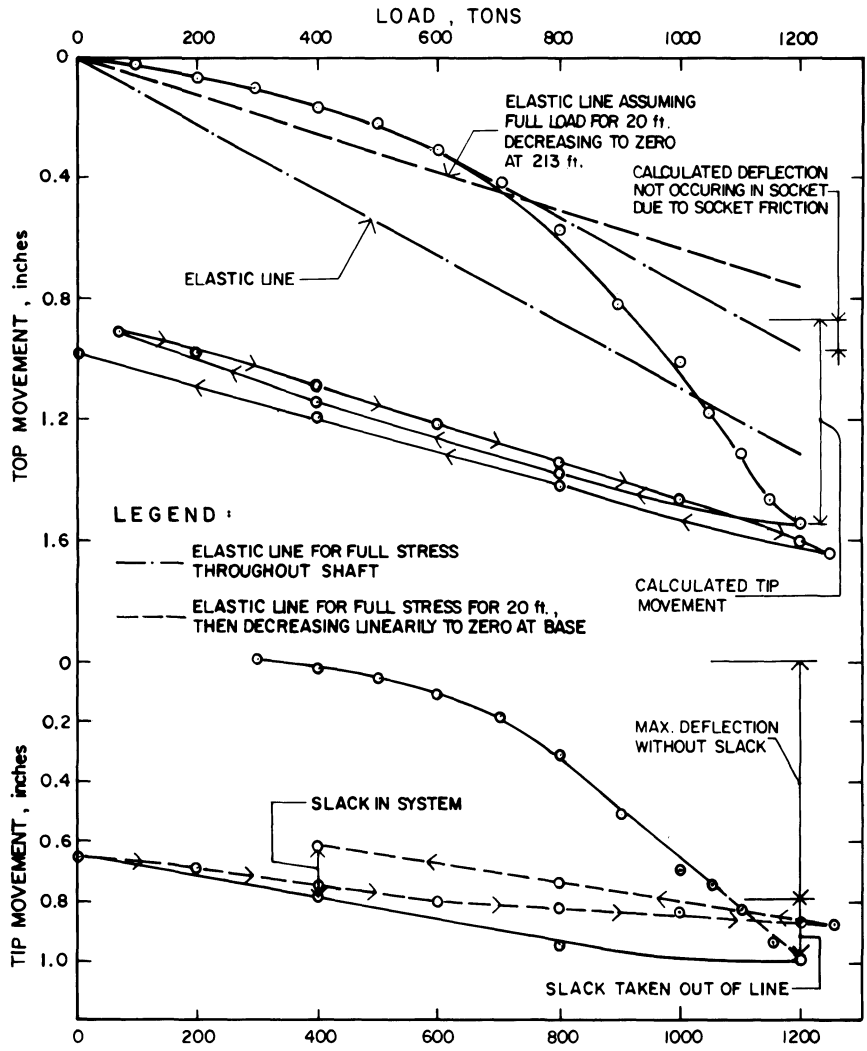


FIGURE 7 Caisson Load Test — Load vs Settlement.

vibrating wire meters for redundancy. The meters are positioned at six different levels in the caisson, specifically, near the top of the caisson, near the top of the very dense till within the bottom casing, at the bottom of the bottom casing, at the top of the competent shale, at the middle of the socket in the competent shale, and at the bottom of the socket.

Periodic readings of the instrumentation are planned during construction of the tower to monitor load shedding characteristics in the ground. It should be noted that on the production caissons there was no inner casing and the bottom two outer casings are permanent. There was a

design shaft size corrugated liner inside the top temporary casing. The annular space between the top temporary casing and the corrugated liner was backfilled with sand. The outer casing was pulled after concreting. Thus, full side friction through 200 ft of overburden was available, although conservatively not included in the design.

Wave Equation Analysis and Dynamic Monitoring

A presentation at the
Deep Foundations Institute–McGill University Symposium
Montreal, September 1983

by

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The conventional wave equation analysis without control of the representativeness of the parameters to the actual conditions does not provide accurate results. However, the representatives can be vastly improved by means of using the results of monitoring the dynamic response in a pile during driving. Dynamic monitoring displays the interaction of independent strain and acceleration measurements having directly proportional response to impacts, but inversely proportional response to reflections from the soil. Examples are given and the CAPWAP method used to calculate from the measurements the dynamic pile and soil parameters and mobilized soil resistance is outlined. The dynamic monitoring technique and analysis has been accepted internationally and been incorporated into Building Codes. The prediction is offered that in five year's time, no major piling project will be undertaken without dynamic monitoring and analysis.

The advent of the wave equation analysis was a quantum leap in modern foundation engineering. For the first time, the full complex of pile driving could be considered; that is, velocity dependent aspects (damping), soil deformation characteristics, soil resistance (bearing capacity, resistance distribution–shaft and toe), hammer efficiency, cushion and capblock properties, and others.

However, while the wave equation is theoretically correct, when used without factual data, the results are only qualitatively correct, not quantitatively. Each input datum is really a variable with a certain range of possible values. As there are many input data, the combined effect is that the result (for instance, a bearing graph) is only somewhere near the quantitatively correct value. Therefore, one should never just be satisfied with one analysis (computer run), but use several runs to find the most

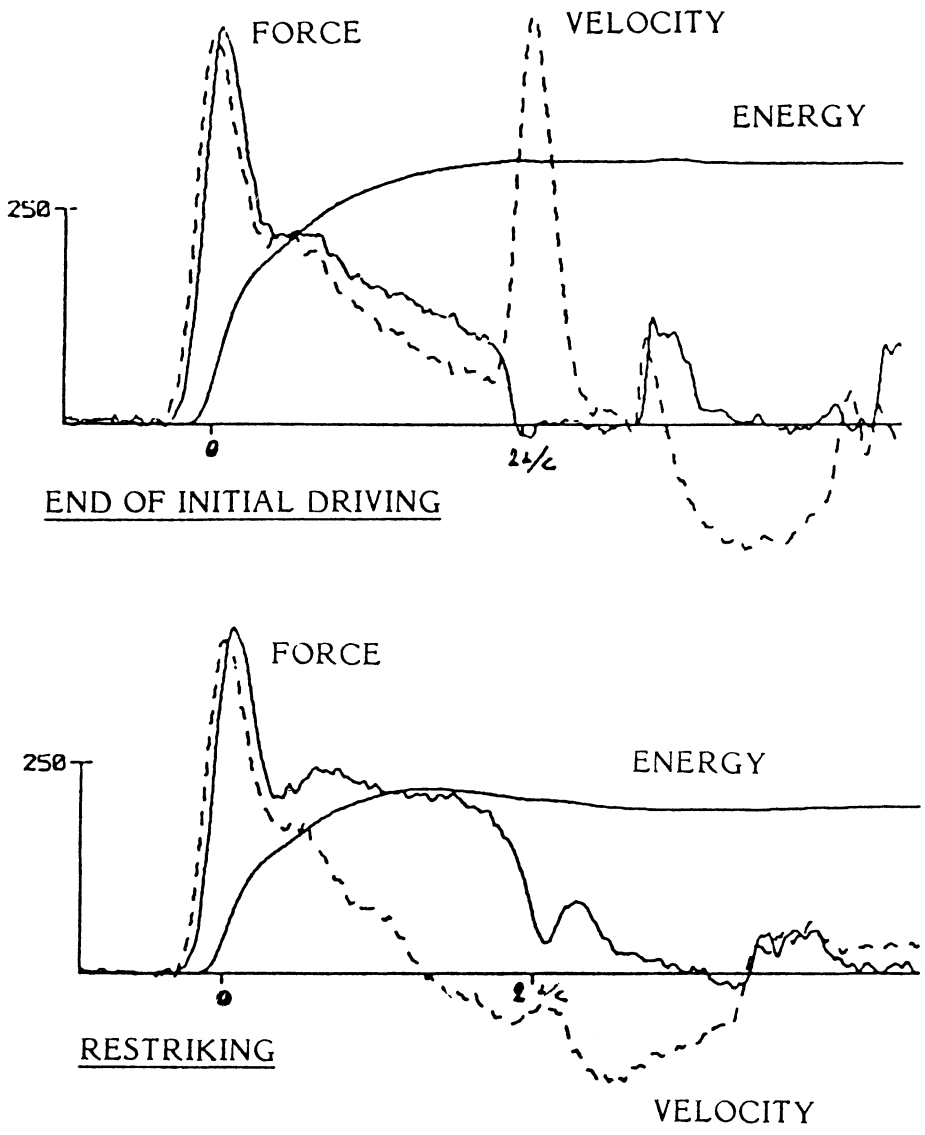

FORCE
VELOCITY
ENERGY
250
0
2L/c
END OF INITIAL DRIVING
FORCE
250
ENERGY
0
2L/c
RESTRIKING
VELOCITY

FIGURE 1

probable conditions by using ranges of values. The results are then represented not as one well defined curve, but as representative areas or bands.

The bearing graph (Fig. 1) is an example of the recommended way of presenting wave equation results (WEAP). For this actual case, a back-calculated project, a range of "refusals" had been obtained. By means of a WEAP analysis, the potential utlimate capacity of the piles was calculated as a function of the penetration resistance blow count. Note, that the band approach increases the range of potential capacity. As indicated, there is a wider and narrower width of the confidence band affecting the precision of the results.

Occasionally, the bearing graph can be paired with other experience information and "engineering judgment" to adequately answer the problem at hand. Usually, though, once one has realized the band necessity of the wave equation analysis, one must improve the accuracy of the results, because **the wave equation does not give single line or simple answers. It is a tool that cannot be used alone.**

The full power of the wave equation analysis is first realized when combined with dynamic monitoring of the pile during driving. Dynamic monitoring was developed in the USA by Dr. G. Goble and co-workers at Case Western University in the early 1970's. It has since been accepted, adopted really, all over the world (as often happens, the full acceptance has been slower in coming in the United States). The dynamic monitoring consists in principle of clipping gauges onto the head of the pile to measure and record force and velocity induced in the pile by the impact. (There are numerous papers describing the actual system.)

The results are usually presented in the form of "wave traces," which show the measured force and velocity drawn against time, Fig. 2. At first, force and velocity are proportional by the so-called acoustic impedance, a material constant (equal to AE/c, the product of cross sectional area and elastic modulus over the wave progation velocity). Therefore, when plotted to scale of the ratio of the impedance, force and velocity at first plot on top of one another.

When the strain wave travels down the pile meeting the soil, soil resistance will cause reflections of strain, which travel back up the pile. These will result in an increase of the stress at the measuring point and a simultaneous decrease of velocity (the pile slows down), visible as a separation of the two curves. The greater the separation, the greater the resistance. At the pile end, if there is no toe resistance, a reflected tensile wave will result, which will be measured as the pile head gauges as a loss of stress and an increase of velocity; that is, the force trace dips and the velocity trace peaks.

The above is illustrated in Fig. 2 showing traces taken from the driving of a 140 feet 18 inch octagonal concrete pile. Conventionally, the force scale is given in units of L/C, that is, the time it takes for the strain wave

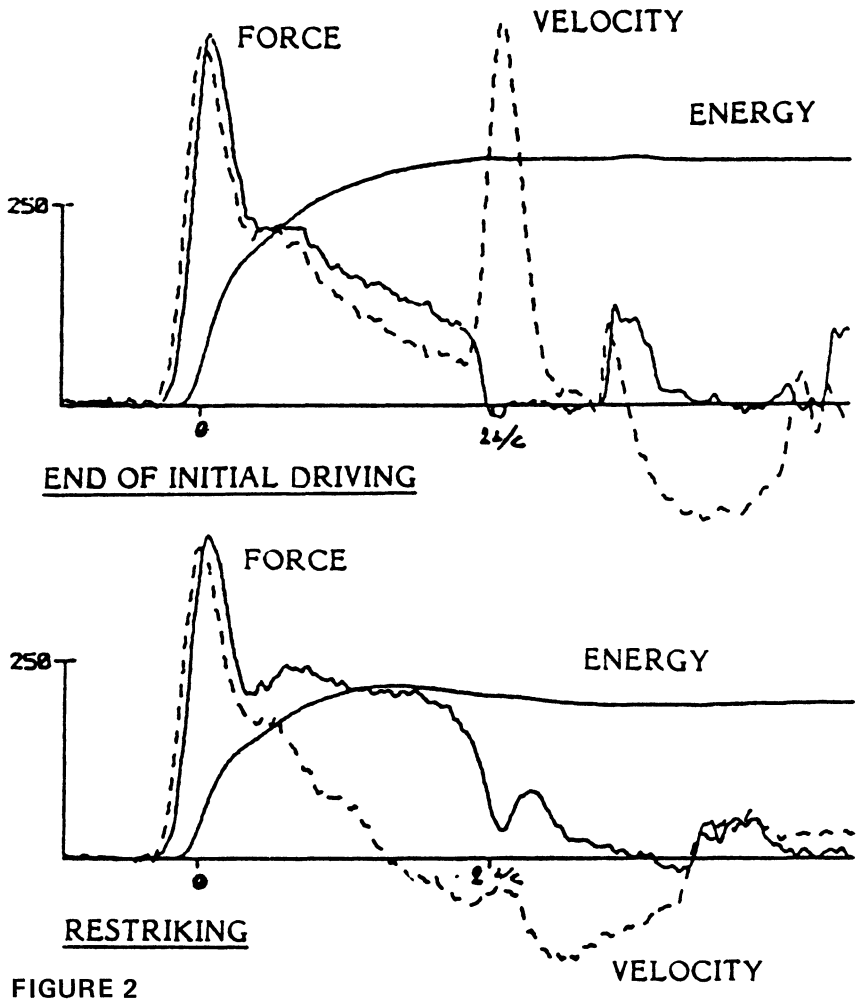


FIGURE 2

to travel the length of the pile. At 2 L/C, therefore, the traces show the reflections originating from the pile toe. The peak at zero time, zero L/C, is defined as impact. However, because of the wave action the hammer impact force is transmitted to the pile before this time as well as after.

The graph shows traces from easy driving at a depth of 78 feet and when approaching refusal at 95 feet. The small separation of the force and velocity traces indicate that only a little shaft resistance is occurring. The velocity peak at 2 L/C in the upper set of traces indicates that there is little or no resistance at the pile toe. The lower set shows, in contrast, a small force peak indicating that the pile toe is now meeting some toe resistance.

The visual study of the wave traces provides very important information to the engineer performing the monitoring in the field. Blow by blow is displayed as the pile is driven. The data are routinely stored on a recorder and can be played back in the office for renewed study.

Fig. 3 shows traces taken from a pile at the end of initial driving and at restriking. The force and velocity traces have been supplemented with traces of measured energy. It should be obvious also the inexperienced that the pile has been subjected to a capacity increase due to soil set-up, although the restriking was performed with a smaller force and energy. The tremendous value of having such records in an actual case is evident.

The data can be treated analytically, of course. The two traces, force and velocity, are mutually independent records. By taking one trace, say the velocity, as an input to a wave equation computer program called CAPWAP, a force-trace can be calculated. The shape of this calculated force trace depends only on the static and dynamic soil parameters used as input in the analysis and it only remotely resembles the measured force trace. However, by adjusting the input data, the resemblance—the match—can be improved. Ultimately, after 5 to 10 computer runs, the computer trace is made to agree well with the measured trace. The condition for the match is that the soil data—quake, damping and ultimate shaft and toe resistance values—are those of the soil in which the pile has been driven. In other words, the CAPWAP match has calibrated the wave equation analysis. Thereafter, also with changes of pile lengths, hammer and pile size even, the wave equation can be used with much greater confidence and the resulting band width can be narrowed considerably.

In the CAPWAP process, the static capacity of the pile has been determined. It is rarely identical to the statically tested ultimate capacity of a pile. Consider that the ultimate capacity of a test pile as evaluated from a load-deformation curve can vary by 20 percent with the definition of failure load applied. Also, only very few tests can produce load values within an accuracy of 5 percent. Usually, the inaccuracy is around 15 percent or more.

A CAPWAP analysis performed on good measurements taken when a pile penetrates at about 5 to 8 blows per inch will provide values of ultimate capacity, which are reliable and representative for the static behaviour of the pile at the time of the driving. Provided the static test is equally well performed (not always the case) the two values of static capacity are within a 15 percent to 20 percent deviation. For all practical engineering purposes this can be taken as complete agreement. Building codes in Europe and Canada recognize this and have accepted the dynamic methods as equal to routine static pile testing. As the costs of one conventional static test equals ten to twenty or more dynamic tests and analyses, the savings can be considerable.

The dynamic monitoring does not replace static testing and conventional inspection methods, it compliments them. Commonly, instead of, say, three static tests, only one is performed and combined with dynamic

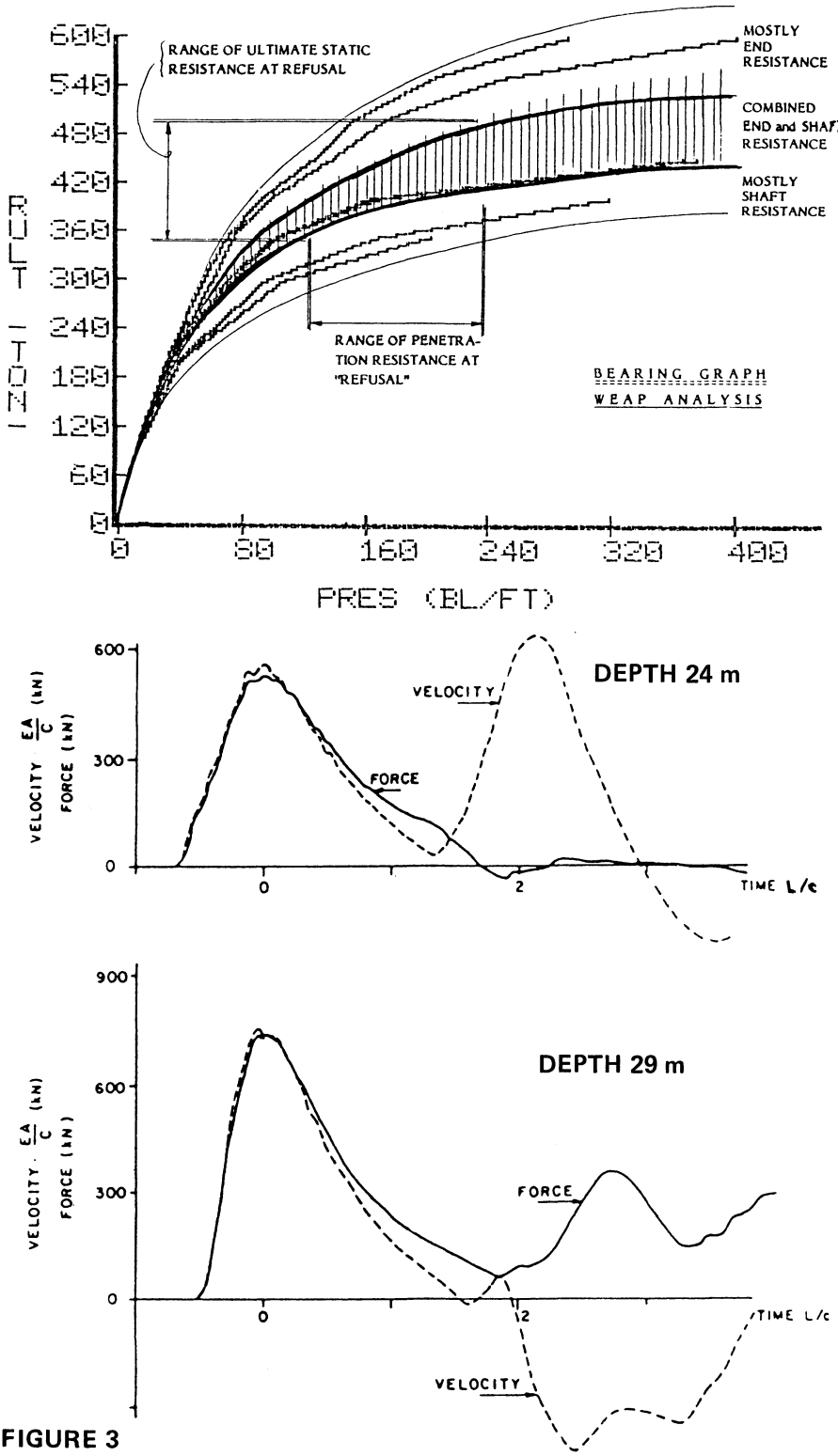

RANGE OF ULTIMATE STATIC RESISTANCE AT REFUSAL
MOSTLY END RESISTANCE
COMBINED END and SHAFT RESISTANCE
MOSTLY SHAFT RESISTANCE
RANGE OF PENETRATION RESISTANCE AT "REFUSAL"
BEARING GRAPH
WEAP ANALYSIS
600
540
480
420
360
300
240
180
120
60
0
0
80
160
240
320
400
PRES (BL/FT)
DEPTH 24 m
VELOCITY
FORCE
VELOCITY · EA/C (kN)
FORCE (kN)
600
300
0
0
2
TIME L/c
DEPTH 29 m
900
600
300
0
FORCE
VELOCITY
TIME L/c

FIGURE 3

monitoring and analysis. Thus, money and time is gained along with an improved confidence.

Dynamic monitoring has the reputation of a "black box technique" and a "cure-all". Consequently, there are many skeptics and many critical voices. However, the knowledgeable and experienced piling man is not replaced by the technique. Instead, he has become even more needed, because, without him, the technique is just a fancy tool best left in the shed. Today, the number of people who understand both the advantages of the dynamic monitoring and its limitations—the latter is most important knowledge—are few. However, the numbers are rapidly increasing as recognition of the value and need for dynamic monitoring develops. It is a fairly safe prediction that, in a few years, no major project involving driven piles will be undertaken without dynamic monitoring and analysis.

Effective Dewatering Is More Than a Pump in the Hole

A presentation at the Deep Foundations Institute Annual Meeting
St. Louis, Missouri, November 1980
by
William J. Rothschild, President
General Dewatering Corporation

Ground water at construction sites can be controlled by properly placed wellpoint dewatering systems. Care must be taken in planning cofferdams to have adequate space for and access to pumps, headers and the points. A most important part of dewatering is protection of adjacent structures from movement. A checklist gives items to consider in bidding.

Ground water can adversely affect the construction of many structures founded in pervious soils below the water table. Uncontrolled ground water can reduce the stability of foundation soils making them unsuitable to support the structure being built.

Proper control of ground water can greatly facilitate construction of sub-surface structures by intercepting seepage that would otherwise enter the site from the sides or bottom of the excavation; it can increase the stability of excavated slopes and prevent loss of material from the slopes to the bottom of the excavation. Dewatering can reduce the lateral loads on a cofferdam. In tunnel construction, dewatering can reduce the need for air pressure to keep water out of the tunnel. Dewatering can improve the excavation or back fill characteristics of soils. Dewatering can also reduce the possibility of an artesian effect when piles are driven through an impermeable layer to a permeable layer.

The method used to dewater depends upon the size and depth of the excavation, the location of the excavation and, of course, the soils above the structure, the soils adjacent to the structure, and the soils below the structure. One should consider not only how to take the water out of the ground, but also what will happen when the water is taken out of the ground.

Some of the methods of dewatering are: open pumping, sand drains, deep wells, wellpoints, ditches or underdrains. Open pumping can be accomplished by placing a pump in the excavation or in a sump, which is



"Franklin Medway Sewage Treatment Plant", Medway, MA.
Note the three-stage Wellpoint System.

a deepened hole to one side of the structure to be built. Underdrains are sometimes installed for permanent dewatering under a structure after its completion. An excavation can be isolated from ground water and pumping eliminated. The site and excavation can be isolated from water by a cutoff wall formed by: sheet piling, a slurry wall, a grout curtain, or a cutoff trench. The site may be isolated by freezing the soil. Each of these tools has to be evaluated as to its effectiveness and cost.

Very often, the approach to dewatering is principally determined by the damage one might do to adjacent property. Dewatering firms frequently engage the services of a soils consultant to predict the effects, good or bad, to the adjacent structures, as well as the new structure to be built.

Soil borings are important to site dewatering considerations. Borings should have been taken deep enough for dewatering tool selection. Borings can help determine whether a cutoff with a sheet pile cofferdam is possible, thus minimizing pumping. Boring samples should normally be taken at 5 ft intervals. If the soil conditions vary, continuous undisturbed samples should be taken. Very often, a soils consultant will be retained to describe the soil, run grain size curves and present geologic sections.

If subsurface information is inadequate, additional borings should be taken early in the planning stage—before a cofferdam is designed, before bracing is designed, before tiebacks are considered and before the dewatering method is decided upon. If tiebacks are considered, borings should be taken outside a cofferdam to determine the difficulty of installing the tiebacks. Often their use requires permission from peripheral owners.

Engineers, contractors and consultants should consider the specifications for dewatering. Assume the structure is to be supported on end-bearing piles. The dewatering specifications might be modest, in that no fines are to be removed, and ditching or sumps will be adequate for dewatering. If, however, there is a possibility of boiling, by virtue of driving a straight pile as opposed to a tapered pile, the specification should state that the water level in a piezometer or observation well is to be lowered to a certain elevation before the piles are introduced into the ground.

If the structure is not pile supported but has a slab on grade, the foundation consultant must decide whether he wants undisturbed soil; and, if so, it should be clearly stated in the specifications. If the contractor feels that the specified procedure can be substantially improved, he might engage a soils consultant to assist with a "value engineering" proposal. Perhaps he can "sell" a change in specifications to more favorable conditions. The integrity of the structure must not be jeopardized. Time will be required to get approvals.

In designing a wellpoint system, the first consideration is the method of installation and the amount of water that has to be pumped to dewater the excavation. This will definitely affect the number of wellpoints and

SUGGESTED COST ITEMS IN BUDGET FOR A WELLPOINT DEWATERING SYSTEM

Installation and Removal of the Wellpoint System in Manhours
Crane – Days
Front End Loader
Trucking
Rental of the Wellpoint System
Jet Pump Rental
Fuel for the Jet Pump
Sand for the Wellpoints
Rain Gear
Tools for Installation
Power for Operation
Lost Equipment
Filters & Oil Changes
Holidays
Wood Blocking for the Header
Supervisor
Laborer (Days to Keep Sumps Clean)
Laborer (Evenings to Assist Operator)
Rental of a Submersible Pump
If Selected – Electricity for Submersible Pump Operation
Electrical Service by an Electrical Contractor
Condition Survey of Surrounding Structures
Labor to Read Piezometers Daily
Labor to Read Piezometers Weekends
Recharging if Required
Winter Protection
Sheds Over Pumps
Mis. Service of Pumps Such as Generators, Starters, etc.
Pump Platforms
Supports for Header & Discharge Pipe
Permits for Discharge into Harbor or Sewer
Filter for Discharge Water
Ramps Over Sidewalk
Relocation of Dewatering Equip. as Construction Progresses
Grout for Sheeting
Piezometer Installation
Rental of Street Usage for Piezometers
If Recharging is Considered, Use of City Water
Rental of Steet Usage for Recharging Pipe
Installation of Recharging System
Chemicals to Treat Recharging Water
Miscellaneous & Contingencies

their spacing, the size of the risers, the size of the header, and the size of the dewatering pumps and the time for installation. A dewatering outfit will consider not only how high the water must be lifted, but how far it must be pushed.

Inside a cofferdam the header elevation is critical since water can be lifted only as pushed by atmospheric pressure. As a general rule, the header elevation and the eye of the impeller of the pump should not be more than 14 ft above subgrade. The reason for this is that water must be lowered an additional 5 or 6 ft at the perimeter so that the center of the excavation is dry. If the structure is large in size, it may be necessary to lower the water further at the perimeter so that the center of the work area is dry. The header and pumps will then have to be lowered.

The position of the header has to be such that a man can monitor and adjust the system. Very often the header can conveniently be placed along the top of a waler that is part of the support framing of a sheet pile cofferdam. Contractors often find it convenient to place the wellpoints in the web area of the sheet piles back of the wale. This is not the best location as there may be less water moving to the wellpoints in this area. More points may be needed when pumping starts and it will be difficult to install them.

If a cofferdam is not large enough for additional wellpoints, there is a problem. General Dewatering Corp. recommends that wellpoints be installed in front of the walers so the number is not limited to the number of spaces between pairs of the sheet piles.

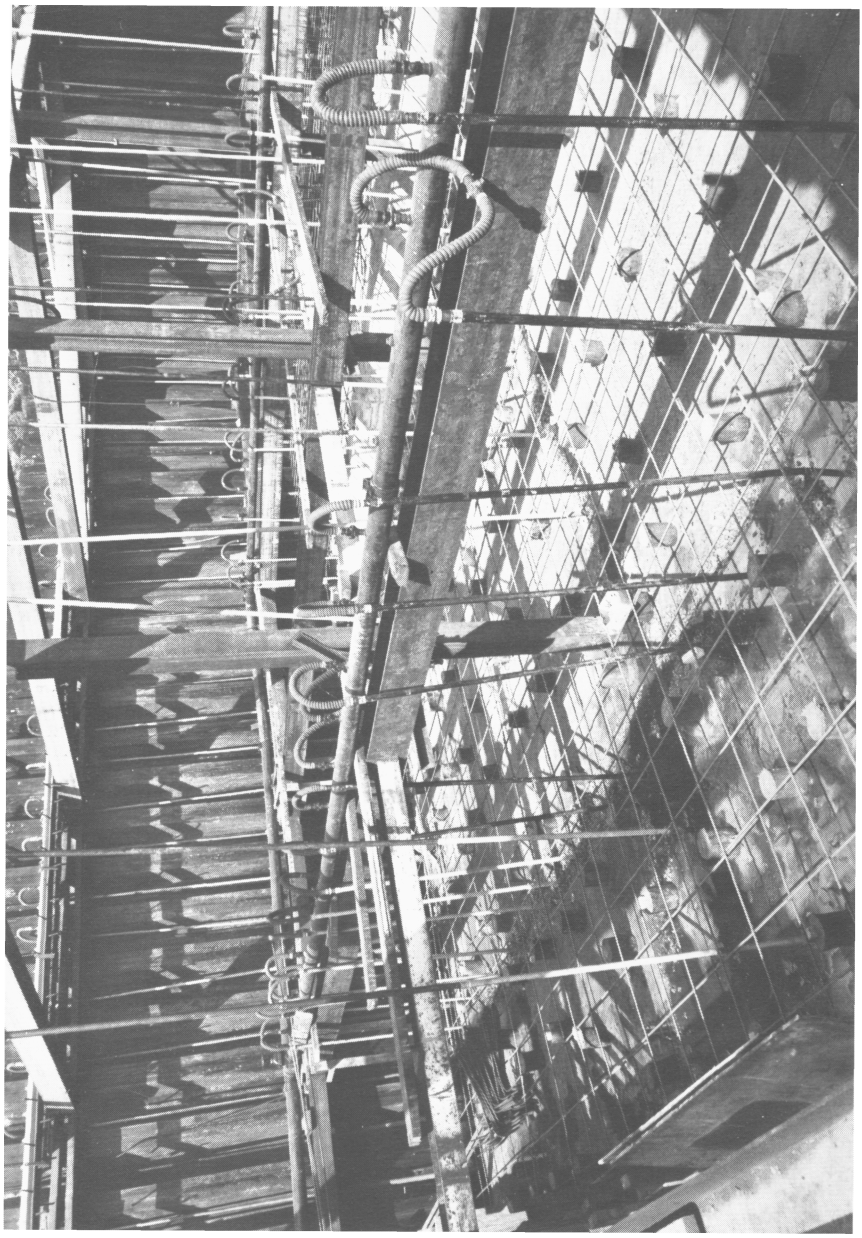
Space will have to be allocated not only for the pumps, but also for servicing and maintenance. Normally, a dewatering operation needs 3-4 ft all around the wellpoint pump for proper maintenance.

There is a problem of the system stopping unexpectedly. Will the structure float? Have provisions been made for openings in the walls so that the structure will fill before it floats? Should provisions be made to flood the structure if holes cannot be left in the walls? Should gravel be placed at subgrade so that if pumping is stopped the water level will rise in a way not to cause damage by movement of fines?

How is a dewatering system installed? Very carefully! How does one monitor the system? All the time!

Whether piezometers or observation wells are used,they must be installed before the dewatering system is installed. Piezometers should be placed inside the cofferdam. If there is concern for structures outside the cofferdam, piezometer should be installed outside as well.

The exposure and liability of the contractor and engineer for the effect of the dewatering system to the outside structures must be considered. A soils consultant might be retained to determine the effects of dewatering outside the cofferdam. Before dewatering begins, one should determine how the structures outside the cofferdam were designed and, more



"Tide Gate Cofferdam, Shinnecock Inlet", Hampton Bays, NY.

importantly, how the foundations were designed and built. Will damage be inflicted upon an adjacent structure which is on untreated wood piles?

There is a question of whether the exposure of the untreated piles to air will cause deterioration. If such is the case, the engineer or consultant may wish to consider protecting the untreated piles before dewatering is accomplished. A recharge system to protect the wood piles might be employed.

If the structure outside the cofferdam is a modest non-pile supported structure with a slab on compressive soil under which is a pervious soil, the question arises as to whether the soil will compress when removing the buoyancy of the water upon implementing the dewatering system. In this case, recharging may be considered, or underpinning may be required. One may have to consider not only the adjoining structures, but also the utilities in the streets, as well as the streets themselves. These cautions are not limited to cofferdam installations, but apply to any deep excavation. The point is that if the exposure to damage and liability of the contractor is great, the damage should be considered before the dewatering system is designed and installed.

Every wellpoint that is installed must be tested. It is tested to pump clean water only, and to record capacity. The number of wellpoints installed must lower the water level in the piezometers or observation wells to the criteria established by either the design engineers, the soils consultant or the dewatering firm used by the contractor.

The monitoring and the use of piezometers is extremely important, whether the excavation is sheeted or open cut. Piezometers must be read daily. Monitoring of observation wells must be continued throughout the entire phase of dewatering. Should the qualities of the water be such that any encrustation will build up on the screens, the water levels will slowly rise. This can be determined only by constant and daily monitoring of the piezometers or observation wells.