# JOURNAL *of the*

# BOSTON SOCIETY OF CIVIL ENGINEERS

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## 119 YEARS
## 1848-1967

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# BOSTON SOCIETY OF CIVIL ENGINEERS

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# BOSTON SOCIETY OF CIVIL ENGINEERS
## OFFICERS, 1966-1967




Editor—RONALD E. BUCKNAM
47 Winter Street, 8th Fl.
Boston, Massachusetts 02108

## ANNOUNCEMENTS

The Designer Selection Board wishes to announce that Architects and Engineers registered by the Commonwealth, who wish to be considered for appointment as designers on state Bureau of Building Construction projects, must submit to the Board, Room 2001 State Office Building, 100 Cambridge Street, Boston, Mass. 02202 (tel 727-4055) current credentials. Data must specifically include:

a. Firm name, telephone number
b. Organizational structure (Individual, partnership, corporation dba)
c. A list of principals and their Mass. registration number
d. Summary of the last five (5) years' experience
e. Scope of work - project description
f. Owners name
g. Indicate whether a new or renovation project
h. Percentage status of the project in design or construction phase
i. Total or estimated construction cost
j. Scope of responsibility
k. Name of firm principal in responsible charge
l. Present number of technical employees including principals and total clerical employees on full-time payroll
m. Name of consultant with whom a working relationship is established and who may be employed

Previously submitted brochures should be supplemented to contain data outlined in (a) through (m) above.

Basic criteria for evaluation of Designers includes: professional experience; quality of design; design originality; cost performance; schedule performance; supervision of construction; available resources; reputation requirements of a project; effective relationship with clients and contractors; economic benefit to the Commonwealth.

Established and governed by the provisions of Chapter 676 of the Acts of 1966 the Board requires designers to submit a separate letter of application for each project for which they wish to be considered. A detailed listing of the specific projects available will be released forthwith after the passage of the pending Capital Outlay Appropriation Act. Written requests for public notices and news releases will be mailed to architectural and engineering society secretaries and publication editors only. The notice will also be available and displayed in room 2001 State Office Building.

EARLE F. LITTLETON
Chairman

George A. Sampson

# MEMOIR OF GEORGE ARTHUR SAMPSON
## 1881 - 1966

George Arthur Sampson, who was a member of the Boston Society of Civil Engineers from 1910 to 1966, and was its president in 1946-47, died in Newton, Massachusetts, on March 3, 1966 in his 85th year.

He was born in Middleboro, Massachusetts, and was graduated from Dartmouth College in 1901, and from the Thayer School of Engineering in 1903.

Following his engineering training, he worked as assistant engineer to such notable civil and sanitary engineers as Leonard Metcalf, Robert Spurr Weston, and William Wheeler. He assisted Mr. Wheeler in the design of the Wheeler filter bottom for rapid-sand water filtration plants, which is still widely used. He served as assistant engineer on the Belle Fourche, South Dakota irrigation project of the United States Reclamation service from 1906-1908. In 1915 he designed one of the first reinforced concrete elevated water storage tanks, in Middleboro, Massachusetts, 162 feet of height, which is still in service. He served as Instructor in Civil Engineering, and later Acting Professor of Sanitary Engineering at the Massachusetts Institute of Technology during the years 1903-4 and 1921-2. From 1911 to 1916 he was principal assistant to Robert Spurr Weston, and in 1916 he formed the partnership of Weston & Sampson with Mr. Weston for the practice of civil and sanitary engineering, retiring from it in 1964. During this period he was responsible for the engineering design of many municipal and industrial water supplies, sewerage systems, and treatment plants for water and wastewater in the Northeast. He gave a paper on the "Sewage Disposal System at Keene, New Hampshire" before this Society, which was published in the January 1939 issue of the Journal.

He served on various committees of this and other technical associations, and contributed numerous papers. His memberships included the American Society of Civil Engineers, the American Water Works Association, the New England Water Pollution Control Association, and the New England Water Works Association, of which he was President in 1938-9.

His professional ability and conduct were of the highest order, and he was a kindly, understanding man to deal with.

He is survived by his wife, Bertha W. Sampson, a brother, H. LeBaron Sampson of Cambridge, two daughters, Mrs. George Zeller of Sayville, New York and Mrs. George L. Shinn of Morristown, New Jersey, and eleven grandchildren.

George G. Bogren

# PILE AND CAISSON FOUNDATIONS

by

H. A. Mohr*

This is one of a series of lectures given by the Society under the general heading of "Soil Mechanics and Foundation Engineering". Lest these headings mislead, the writer confesses to but little knowledge of the science of soil mechanics. He has had experience in the installation of pile and cylindrical-caisson foundations.

A reasonable exposition of this subject would fill a thick book. This presentation will be limited in scope to an explanation of the useful details of piles and cylindrical caissons as presently utilized in the United States to resist axial loads, and to a general description of the construction equipment and procedures used to install these units and the bases of payment.

The logical approach to selecting a proper type of foundation requires at least the following:

1. Reliable sub-surface information, including acceptable soil samples.
2. Knowledge of the forces to be resisted at a fixed elevation.
3. Knowledge of costs of competitive types of foundations, or a reliable source or sources for these costs.

Piles and cylindrical caissons are distinct from each other in concept. No similarity exists in their actual construction. Only where piles or caissons are needed and prove the most economical of all types of foundations should either be used.

Often the need for piles is apparent; often a doubt remains. More often than one would suspect, the logical approach is ignored. Prof. Casagrande once stated, "When in doubt, leave them out." Too many engineers ignore this admonition and, instead, distort its meaning into "Don't doubt, drive piles." Blindly resorting to the use of pile foundations seems common practice. This solution requires no study or knowledge of the behavior of soils during and after completion of the construction. It may hasten the completion of plans and contract documents. Also, it may lead to construction and end-result difficulties.

A similar situation does not apply to a caisson foundation. The engineer must know from a study of all available information, and, when in doubt, from the results of installing test caissons of the proposed type, that the construction is feasible. The cost of changes after a contract has been let is limitless.

*Consulting Foundation Engineer, Boston, Massachusetts

## Piles

Primarily, piles consist of one of three materials, or a combination thereof, namely: wood, concrete and metal.

1. Wood has been used for piles for centuries. For size, length and kind of wood, the engineer's choice is limited to a product produced by nature.

2. Concrete for piles may be cast-in-place or pre-cast. For the latter, the strands may be pre-tensioned or post-tensioned.

3. Metal for piles generally consists of steel. It may be of any section that can be driven, but commonly used are the H section or pipe. Pipe may be driven open or closed end, cleaned out when necessary, then filled with concrete.

Concrete and metals are man-made from raw materials. For piles they may be formed to satisfy any imaginable requirement. There exists no valid reason for specifying a minimum or a maximum length of pile that is imbedded in soil.

To be the proprietor of an all-purpose pile has a compelling fascination. The urge is pervasive. While the effort has not been of such duration, the chance of developing an all purpose pile is less than that of inventing a perpetual motion machine.

## Untreated Wood Piles

For permanence the pile must be continuously submerged. To function in tension the pile is debarked to raw wood, otherwise the bark is not removed. Any kind of wood that will withstand the handling and driving forces is suitable as a pile. Wood piles are easily destroyed during driving. Photographs of this destruction are in the sales-kit of every concrete and metal pile salesman. They are common property (see Fig. 1).

The design load per pile is governed by the kind of wood and the size and the limit of safe driving, but usually is equal to 12 to 30 tons. These limits are not likely to be increased due to possible damage when driving for greater loads. A hammer that delivers 15,000 foot-pounds of energy per blow is capable of destroying any commercial native grown wood pile.

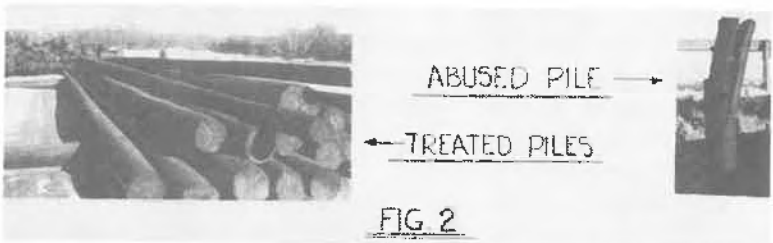
Untreated wood piles are used inland and in open water that transports no deleterious manufactured wastes or is not infested with destructive marine organisms. They are suitable where it is judged that the driving will not be destructive, where the permanent ground-water level is at or not too far below the normal pile-cut-off elevation, and

where the required length is within that of native grown trees.


Lower right: A load of peeled Western fir piles. It is cheaper to peel than to pay freight on the bark across the continent.

## Treated Wood Piles

Properly treated piles may be cut off at any elevation. They may be used in sea water exposure. The treatment deters the activity of fungi and destructive marine organisms. The treating liquid is creosote, a byproduct of the distillation of coal and the refining of oil. Except for a short distance at each end, the heart-wood will not accept treatment. To prevent later destruction, raw wood exposed during framing must be field treated to absorption refusal with hot creosote. It is impossible to frame piles properly before driving. A treated pile is more brittle than an untreated pile. It therefore will withstand less handling and driving forces. These piles will in general conform better to straightness and size specifications than will untreated piles simply because it is uneconomical to chance rejection (see Fig. 2).




Left: Selected Norway Pine. Right: Damage not caused by driving.

Only selected kinds of wood will accept treatment. Before treating, a tree trunk is debarked to raw wood. The trunk is air dried or the sap is removed by applying a vacuum in a kiln or treating drum. Then the drum is filled with creosote and the pressure applied.

The composition of the treating material; the pounds of retention; whether the process involves a full cell with no after-pressure vacuum applied or an empty cell with after-pressure vacuum applied; the duration of application; whether air or artificial drying is used and other details of the treating process are points of argument between suppliers and engineers.

The design load is not more, and often slightly less, than that allowed on an untreated pile of the same size and kind of wood. Treated piles are used for piers, wharves, trestles inland and on water and for the support of structures.

## Cast-In-Place Concrete Piles

This type of pile may be cut off at any elevation. The driven part, i.e., a mandrel or drive-pipe of metal that is always removed, will withstand harder driving than will wood or concrete of equivalent area.

When the pile is properly driven, the design load is that allowed on the area of the pile material at its critical section. Where the pile is driven to bedrock, the critical section is assumed to be one foot above the point. Where the pile is not driven to bedrock, the critical section is at the top of the stratum in which the pile develops its supporting capacity. For cylindrical piles, the critical section is constant. A tapered pile with a point area less than that of a cylindrical pile, when driven to bedrock, is allowed a lesser load. When it is not driven to bedrock, the same tapered pile with more area at the top of the supporting stratum is allowed a greater load than the cylindrical pile.

Installed shells must have the strength to withstand the external pressures created by the installation of subsequent adjacent shells. Normally an installed shell or pipe is not a pile until after it has been filled with concrete. A pipe with wall thickness sufficient to support a share of the axial load is not considered a shell. The City of Boston Building Code provides that a wall thickness less than 0.2 inches shall not be used for support of load. Ultimately, when the inside of installed shells and thin-wall pipes are free of water and clean, they are then filled with concrete.

The prevailing notion that some part of the bottom of an installed pipe or shell surrounded by soil must be visible to assure its integrity for support of load is not well founded. These shell piles, as

well as solid piles, can be bowed and much out of plan alignment and still support the design load. Any pile driven exactly in design position is a happenstance. Where the bottom of an installed shell or pipe cannot be observed, drop in a stone approximately 1/3 the size of one's fist. If the shell or pipe contains water, the sound of a splash is reflected as the stone strikes bottom. If soil is present the sound is a dull thud and if the shell is clean, the sound is a sharp reverberation. The alignment of installed shells and pipes is determined occasionally with oil well surveying instruments.

The concrete should have a slump of 4 to 5 inches and, to avoid entrapping air, be allowed to fall freely after passing through a funnel having a neck opening smaller in diameter than that of the shell or pipe. For height of free fall less than approximately 5 feet, the top 3 to 4 feet should be rodded. End bearing piles, as when the point is on bedrock or in hardpan, should first receive at least one cubic foot of 1 to 1 sand-cement grout.

In the drive for competitive advantage for cast-in-place concrete pile work, the thickness of metal often is reduced to the point of collapse due to outside pressures or from being mangled during driving, resulting in redrives or replacements, and even possibly re-design of footings and argument. The results of strength tests performed on shells and thin-wall pipes before being installed in the ground are misleading. The strength of the shell after being installed is important. Many shell corrugations are badly deformed during the driving operation, as are many thin-wall pipes, thus reducing their resistance to external pressures.

In pile-driving parlance the term re-drive may mean 1) again driving a pile, pipe or shell that has heaved, or 2) inserting a shell inside a previously installed but unsatisfactory shell and again driving the mandrel and shell to the required resistance or depth of embedment, or 3) testing a shell, pipe or pile to determine loss or gain in the original final driving resistance with time. A re-drive may be inserted in a tapered shell but not in a constant cross-section shell or pipe. Solid-bodied piles that are damaged below the ground-surface cannot be redriven.

Assuming that a shell strong enough to withstand all external pressures is used, the developed driving resistance is not distributed by withdrawal of the mandrel. This strength assumption applies to all permanent shells and pipes installed in contact with the soil.

Cast-in-place concrete piles are suitable for inland construction, not through open water, where the driving may be reasonably hard, where the required length of pile does not exceed that of available equipment and where the load to be supported is reasonably high. Except for the Monotube and thin-wall pipe, the volume of work must be sufficient to

bear the cost of moving heavy equipment to and from the construction site.

## Cast-In-Place Concrete Shell and Pipe Piles
*(Permanent shell and pipe installed in contact with the soil)*

*Caudill (Thin-wall pipe, non-expanding mandrel Fig. 3)*

A thin-wall open-end pipe is dressed outside a thick-wall drive-pipe or mandrel. An inverted truncated, conical, steel point is built onto the bottom end of the mandrel. It is dimensioned to enter the boot and, with a few blows of the pile hammer, crimp the top rim of the boot into the pipe wall to produce a tight joint. The boot is made of steel. Its top rim, similar to a heavy wire, is cut at equi-distant points to allow for expansion. It is closed by a welded-on bottom plate.

Soil friction created during driving often causes the thin-wall pipe to climb the mandrel and wipe the crimp off the end. Providing a shoulder to engage the top of the thin-wall pipe prevents this action. The shoulder must be adjustable to engage variable length pipes, or all pipes must be of one length. This pile may be formed of variable diameter pipe sections, but usually is of constant section. Thinner wall pipes can be installed by this method more easily than by any other.

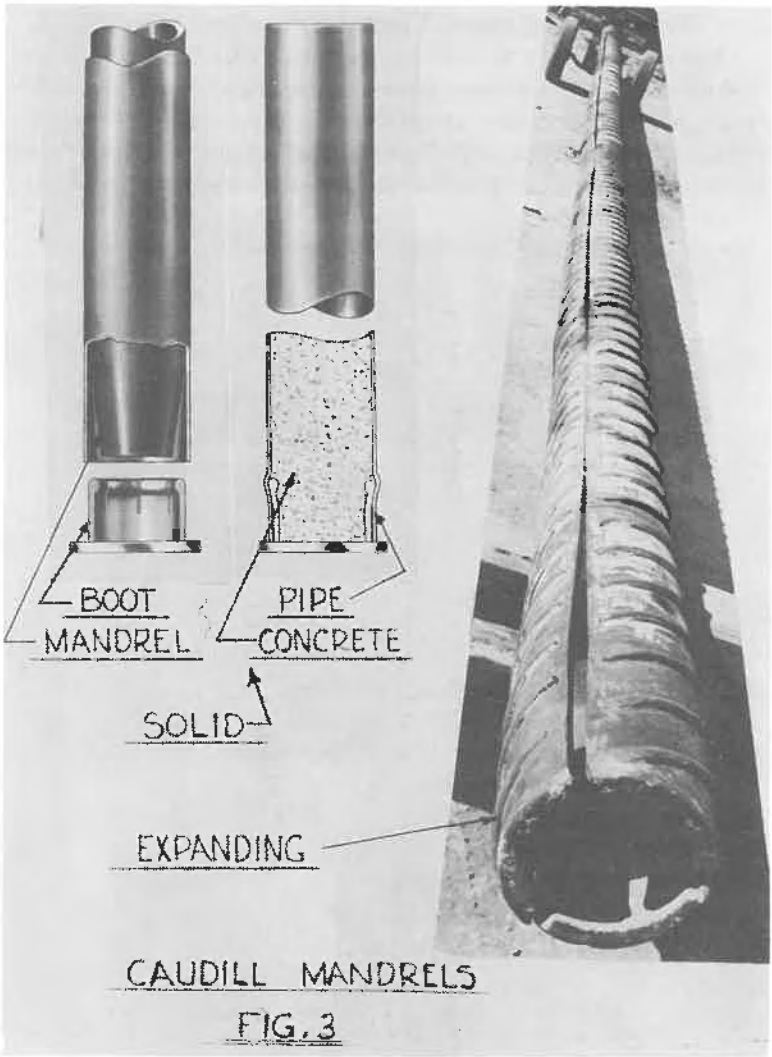
The design load is equal to the area of the concrete at the critical section times the allowable working stress.

*Caudill (Shell, expanding mandrel)*

A one piece, closed-end, corrugated shell is dressed on the outside of a three-leaf mechanically expanding mandrel. The shell corrugations are parallel helices as are the half-rounds welded on the outer surface of the mandrel leaves. The mandrel expands to support the roots of the corrugations, then is driven with the shell to the required resistance or depth of embedment.

Built on the center stem of the mandrel are the male portions of tapered bearings. When the dressed mandrel is set full weight on a timber, the center stem moves down, expanding the mandrel which locks in that position. Wear of the tapered bearings eventually results in looseness. A cylindrical mandrel and shell will not fit closer by moving the mandrel deeper into the shell. Wear of the mandrel bearings is adjusted in a threaded joint at the connection between the center stem and mandrel head. The design load on this pile is equal to the area of the concrete times the allowable working stress.

Both types of Caudill piles, being cylindrical, require either leads at least twice the length of the mandrel plus the length of the hammer, or

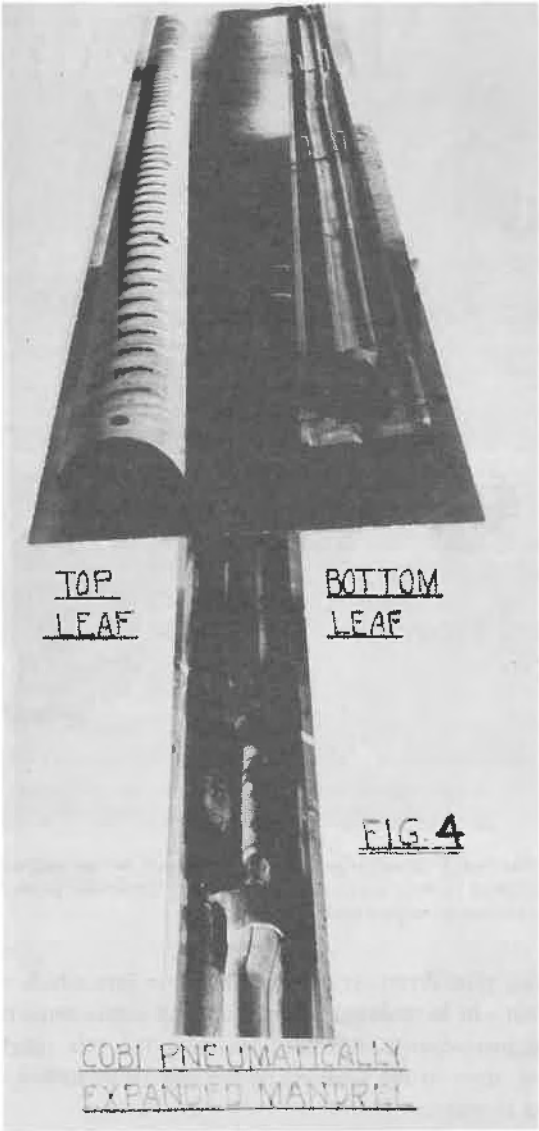



Left: Pipe-mandrel, thin-wall pipe and cast steel boot, before assembling. Center: Sketch of finished bottom portion of pile. Right: Three-leaf expanding mandrel. Outside half rounds to engage shell corrugations.

a closed-end pipe driven at a central location into which the thin-wall pipe or shell can be lowered in order to pull either onto the mandrel. The latter procedure necessitates traveling the pile machine to the driven pipe, then to the location of the pile to be driven, slowing the pile driving operation.

*Cobi (Shell, expanding mandrel Fig. 4)*

This pile is formed by dressing a one-piece, closed-end corrugated shell on a 4, 3 or 2 leaf mandrel and driving the mandrel with its shell to the required resistance or depth of embedment. The shell corrugations are parallel helices. On the surface of each leaf, to overcome the greater tendency of the shell to climb the mandrel during driving,



half-rounds are welded at gradually widening intervals, moving upward. The mandrel is expanded by pneumatically inflating a specially made full-length internal hose. This procedure forcibly supports the roots of the shell corrugations during the driving operation.

At the completion of driving (which is on the leaves, the mandrel having no center stem), the hose is deflated, permitting collapse and removal of the mandrel, thus leaving the shell as installed.

Where leads of adequate height are not provided, driving progress with this pile is slowed by the necessity of moving first to a driven closed-end pipe, then to a pile location.

The design load per pile is equal to the area of the concrete times the allowable working stress.

*Raymond Standard (Shell, expanding mandrel Fig. 5)*

This pile is formed by driving a three-leaf, mechanically-expanded tapered mandrel dressed with a set of shells to the required resistance or depth of embedment. The shells are of 4 and 8 foot lengths; they will nest or telescope into a set for each pile for handling and shipping and are manufactured of different gage metal. To resist anticipated external pressure a proper gage wire, wound on a 3 inch pitch, is crimped inside each shell section.

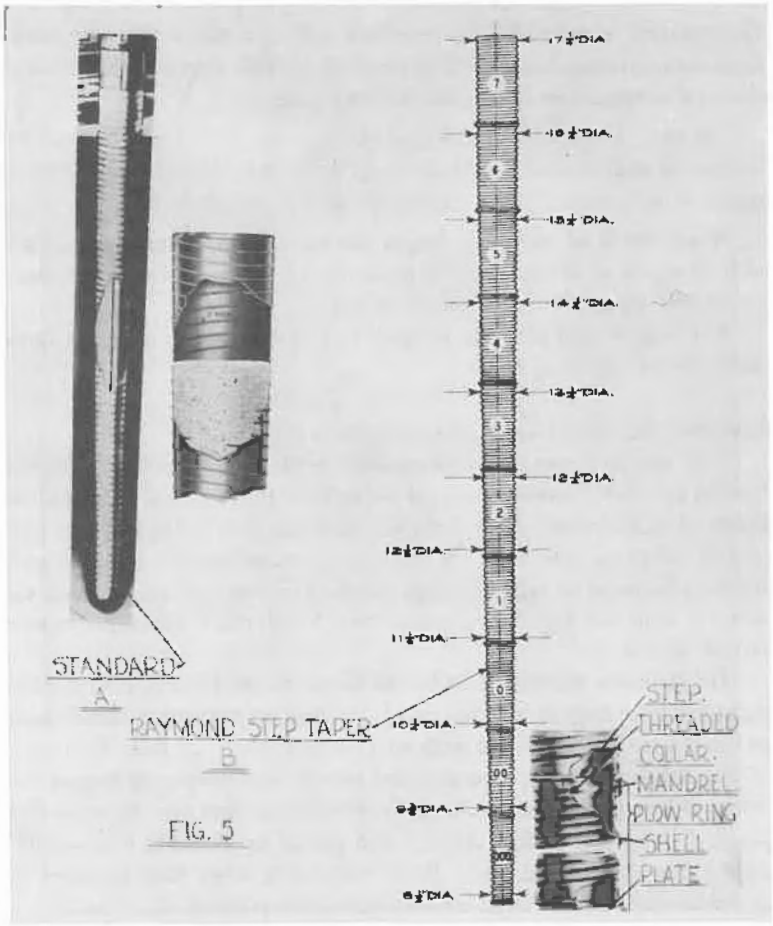
The mandrel tapers 0.4 inches in diameter per foot of length, the point diameter may be 8 inches or 11.2 inches; its maximum length with an 8 inch point is 37 feet and with an 11.2 inch point, 29 feet. With wear of the bearings this tapered mandrel merely sets deeper to engage the shells. When landed on blocking, the mandrel latches into its expanded position. The top shell is chained and pulled up and over the mandrel with a power operated line. Each succeeding lower shell is added to complete the set. All shell connections are slip joints.

The design load per pile is equal to the critical area times the allowable concrete working stress.

When driven in granular soils, a conically shaped pile will develop the required driving resistance at less depth of embedment than will a cylindrical pile of equivalent volume.

*Raymond Step-Taper (Shell, non-expanding mandrel Fig. 5)*

This pile is formed by dressing special loose-fitting, variable-diameter cylindrical, corrugated shells on the outside of a non-expanding mandrel and driving the mandrel with a set of shells to the required resistance or depth of embedment. These shells are manufactured of different gage metal to resist varying anticipated external pressures in sections that range in diameter from 8.5 inches to 17.5 inches, and in lengths from 4 to 16 feet. Each shell varies one inch in diameter from




its adjoining shells. A plow ring having an outside diameter 0.75 inches greater than its upper connecting shells is set at each joint. The corrugation is a thread which permits each set to be assembled in a horizontal position into a continuous shell. The bottom shell is closed by a welded flat plate. Even-numbered shell sections will nest, as will odd numbered sections, thus forming two bundles for handling and shipping.

At each change in shell diameter a plow-ring engages the mating shoulder on the mandrel, thus causing each shell section to be lightly pushed at the upper end and heavily pulled at the lower end as the mandrel is driven. Except where a shell is deformed, there is no other contact between shell and mandrel. The root of the corrugation is not supported.

Mandrels are hollow forgings of special steel. Specified point dia-

meters are obtained by sectionalizing the bottom portion of the mandrel. These sections are connected by sockets and tenons secured by tapered elliptical wedge pins.

Under identical soil conditions and with the same equipment, these shells can be installed faster than any other type by lowering an assembled shell into a previously installed shell. The machine does not move to "shell-up", but merely swings or moves directly to the next pile position. Redrive shells have no plow-rings, therefore when they are driven into a damaged or unacceptable set either some of the original corrugations disappear or all the sections below the point of damage are compressed until there are no corrugations. Some engineers object to this condition, but the writer knows of no instance where such a completed pile has failed to function acceptably.

The design load is equal to the critical area times the allowable working stress on the concrete.

*Union Metal or Monotube (Shell, no mandrel Fig. 6)*

This type of shell has longitudinal flutes or corrugations. It is driven without the aid of a mandrel. It generally is shipped and handled in one piece for each pile. Since the shell is light in weight it can be handled and driven with less powerful equipment than ordinarily is needed to handle a heavy mandrel. To withstand anticipated pressures and sometimes to support part of the design load, it is manufactured in metal gages ranging from No. 11 through No. 3.

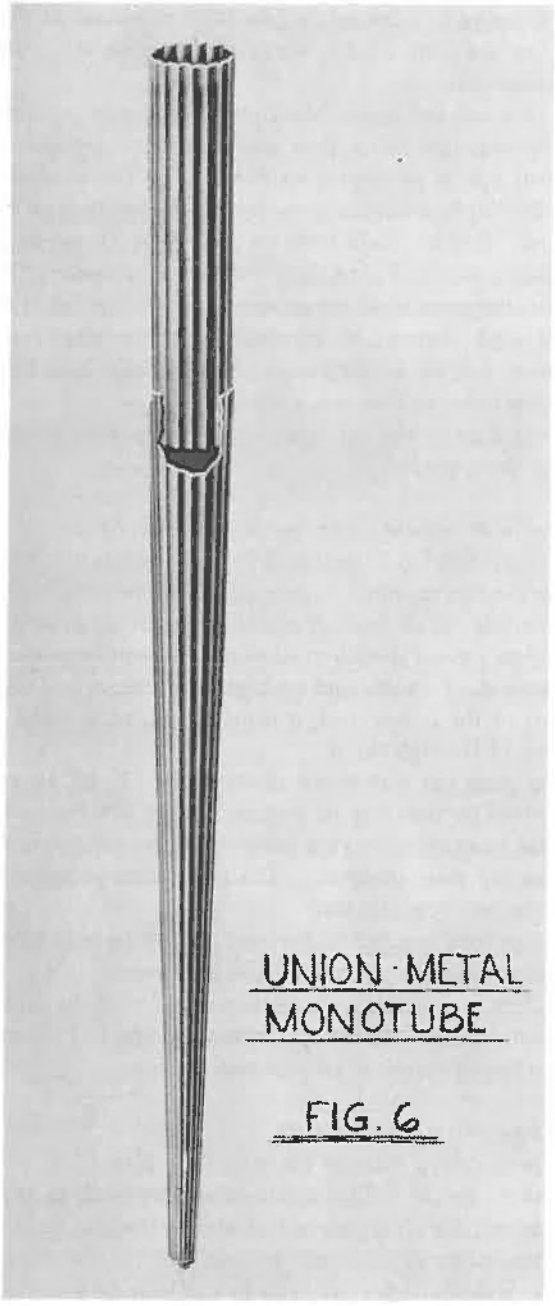
The top stem has a constant diameter in 12, 14, 16 and 18 inch sizes. The lower portion may be conical, starting at 8 inches in diameter or larger and tapering at varying fractions of an inch per foot of length to meet the top stem diameter. The taper is advantageous when the shell is driven into granular soils.

The design load is equal to the area or areas of pile materials at the critical section times the allowable stress or stresses.

Monotubes are used in all instances where shells and thin-wall pipes are used. In cases where the steel supports part of the axial load, it is subject to loss of metal, as are pipe and H piles.

*Open-End Pipe (No mandrel Fig. 7)*

The pipe is driven without the aid of a mandrel. It is driven with an open end to permit drilling and cleaning out inside to gain depth or, where necessary, the chopping out of obstructions to let the pipe pass. To prevent possible damage, the bottom end is reinforced by a relatively thick, wide outside steel band in the form of a cutting edge that is securely welded at the top and bottom and with numerous plug welds


UNION · METAL
MONOTUBE
FIG. 6

to prevent the band from being disconnected when the pipe is driven hard. Normally, the fully driven pipe is cleaned inside with compressed air and water or by other means, and then is filled with concrete.

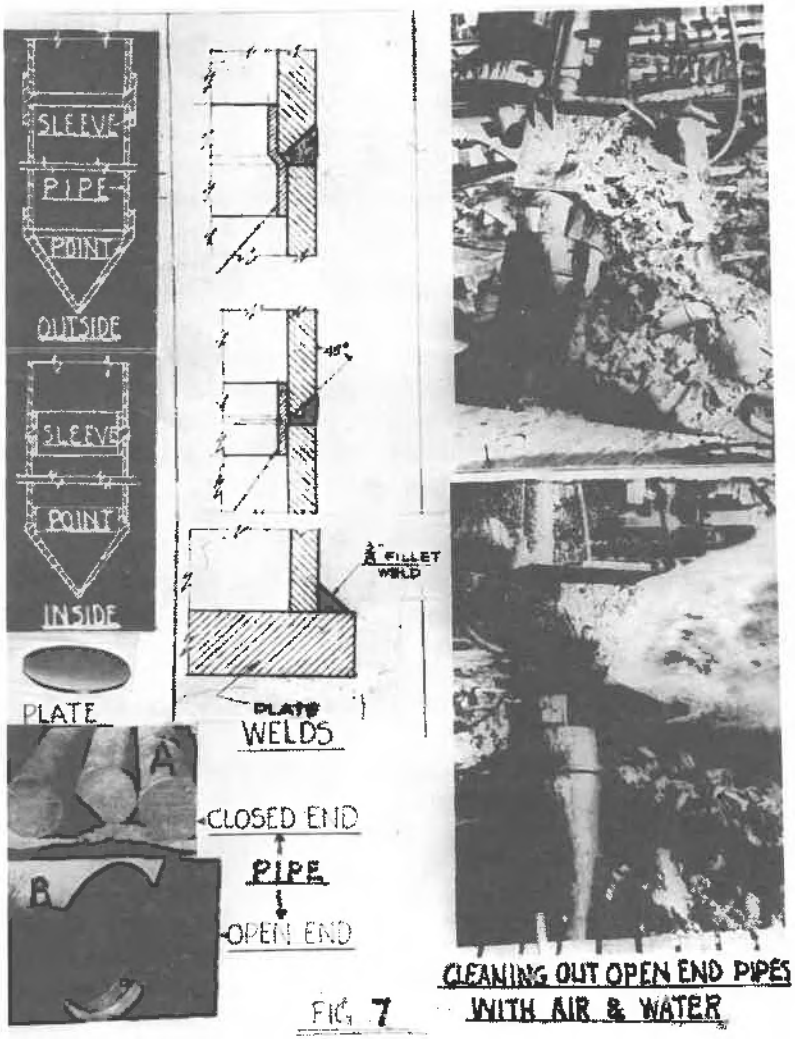



FIG. 7

Closed-end and open-end pipes are lengthened by inserting inside or outside sleeves or by welding. The writer much prefers welded joints and flat bottom plates.

Pipes are driven to support heavy loads. With the proper equipment

they will withstand very hard driving to considerable depths. They are used both inland and in open water, and are subject to loss of metal from corrosion in both locations.

The design load is equal to the area of the concrete and the effective area of the steel times the corresponding allowable stresses.

Commercial pipe for piles, driven either closed or open-ended, varies from 8.625 inches O.D. by 2 inch increments, through 24 inches O.D. The wall thickness may vary from a minimum of 0.109 inches to a maximum of 0.5 inches. To prevent shortening during hard driving, spiral jointed pipe must be fusion welded.

The writer knows of no closed-end pipes that have been driven for piles larger than 18 inches O.D. or open-end pipes larger than 24 inches O.D. They were driven to support design loads of 200 and 330 tons respectively. Hammers of sufficient capacity to drive pipes properly for heavier loads are scarce.

It is important to note that all work required to complete the previously identified shells and pipes for piles, except the placing of the concrete and cutting off to grade, had been finished and that the final developed driving resistance is not disturbed by removing an inside mandrel. A pipe is left as driven.

*Shell-Less Cast-In-Place Concrete Piles*

*(Outside drive-pipe is removed after concrete is placed)*

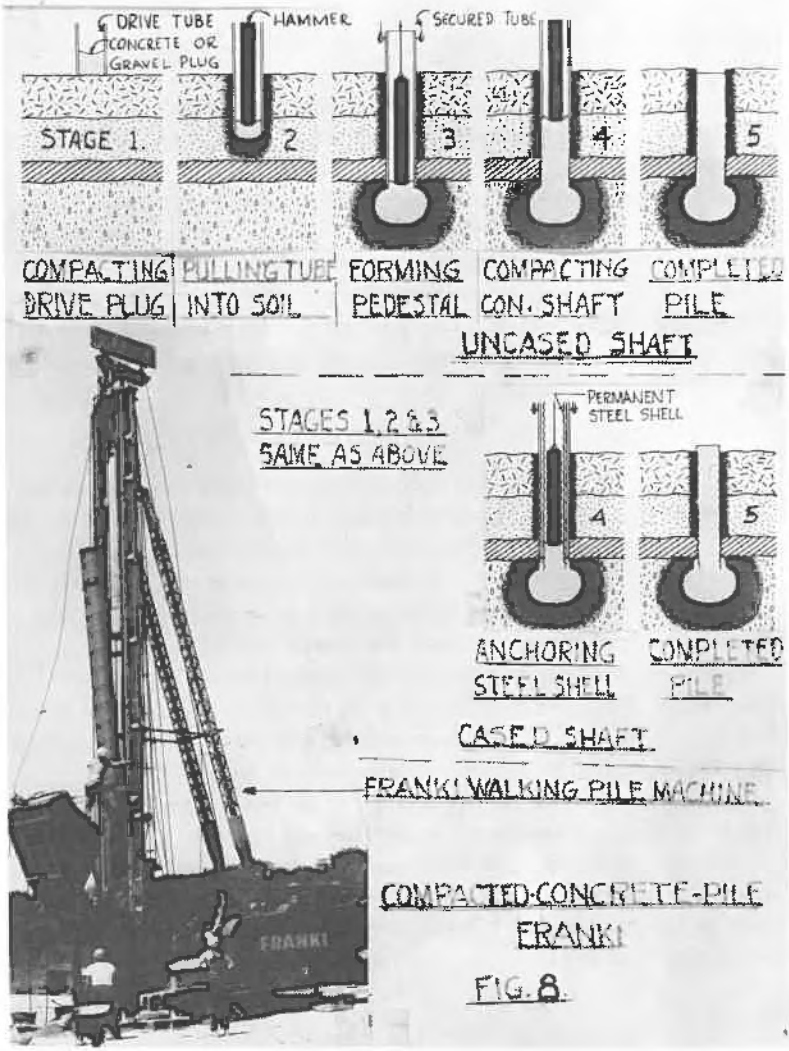
After completion of the driving and concreting of each shell-less concrete pile, the drive-pipe must be removed. Removing this pipe destroys all side friction originally developed during driving. Below the bottom of the pipe, as it is pulled, the fresh concrete and surrounding earth move laterally to meet. The writer believes that the unfavorable feature of the shell-less concrete pile, i.e., the necking of the column of concrete, has in the past been caused in part by the wrong placement or the use of too dry or too wet concrete.

*Franki (Compacted concrete pile Fig. 8)*

The Franki pile departs radically from the usual concept of the installation of a concrete pile. A thick wall drive pipe with a reinforced bottom end is set on the ground at a pile location. Inside the pipe, but sufficiently smaller in diameter to permit gravel, stone or concrete to pass through the annular space, is operated a 7,000 pound or 10,000 pound specially made drop hammer. To permit rotation its cable connection is a machined swivel.

Three to four feet in depth of screened gravel, crushed stone or harsh, no-slump concrete are deposited in the drive pipe. By manipula-

tion of the drop hammer this harsh material is formed into a plug by means of which the pipe is pulled down to the required position with massive blows of the hammer. At this point the pipe, held by heavy rigging, is raised slightly, the plug is driven out and a pedestal or bulb is formed by driving in relatively small batches of no-slump concrete until further acceptance, under a given number of blows of specified energy, is refused. As the drive-pipe is intermittently pulled, the stem is formed by driving out of its lower end relatively small batches of no-slump concrete.




COMPACTED-CONCRETE-PILE FRANKI

FIG. 8.

The height of hammer fall during driving and forming of the bulb and stem is judged by the appearance, at the top of the drive-pipe, of telltales woven into or marked on the cable attached to the hammer. This height, as a maximum, may be the distance between the top of the landed hammer and the underside of the head block. Enormous energies per blow can be developed.

This pile is driven to support heavy loads. Since its pedestal is often formed to take advantage of an upper granular stratum, its allowable load is determined by the strength and compressibility of the soils supporting the pedestal. The strength of compacted-concrete in the shaft is not a determining factor for design-load support. The strength of concrete poured into a permanent shell, where one is used, determines the shaft design load. A dropped-in-pipe with wall thickness enough to support load or an increased concrete area may be used to build the shaft strength to the capacity of the pedestal.

Accurate soil data and authentic samples are indispensable for determining the feasibility of this type of construction. It is the most positive of all shell-less, cast-in-place concrete piles. This pile is driven on not less than 4'-6" centers. It may be driven tangent to an existing object. The recently adopted Part 29 of the City of Boston Building Code outlines very well the conditions under which the pile may be used and the provisions for its installation.

*MacArthur (Fig. 9)*

A closed end pipe inside a thick wall pipe of equal length is set on a bottom pan at a pile location and driven to the required resistance or depth of embedment. Then the inside pipe is removed, the outside pipe is filled to the desired height with concrete, the inside pipe and hammer are set on the plastic concrete and allowed to settle as the outside pipe is pulled. The design load varies with the area of the pulled pipe.

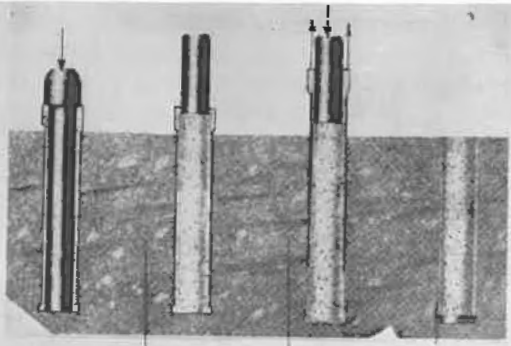
For re-use the outside pipe must be removed before the concrete has time to set or the surrounding soil grips so tightly that removal is impossible. Removing the pipe leaves a fresh column of concrete exposed to pressures created by the driving of subsequent piles.

By driving concrete out the bottom, as the pipe is raised slightly,the first pedestal-base concrete pile in the U.S. was created.

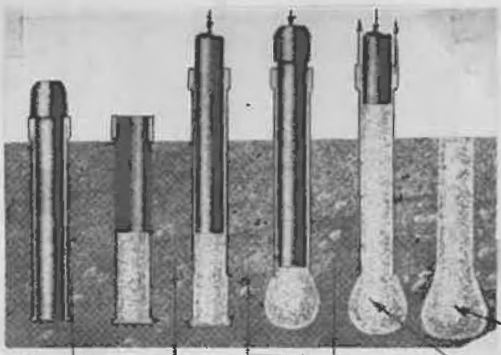
Attempting to drive these piles and subsequently described shell-less concrete piles on too close centers and in soils unsuitable for this type of construction has resulted in necking of the concrete so serious that engineers fear their use.

*Simplex (Fig. 10)*

A thick-wall pipe with an expendable bottom pan or point is driven



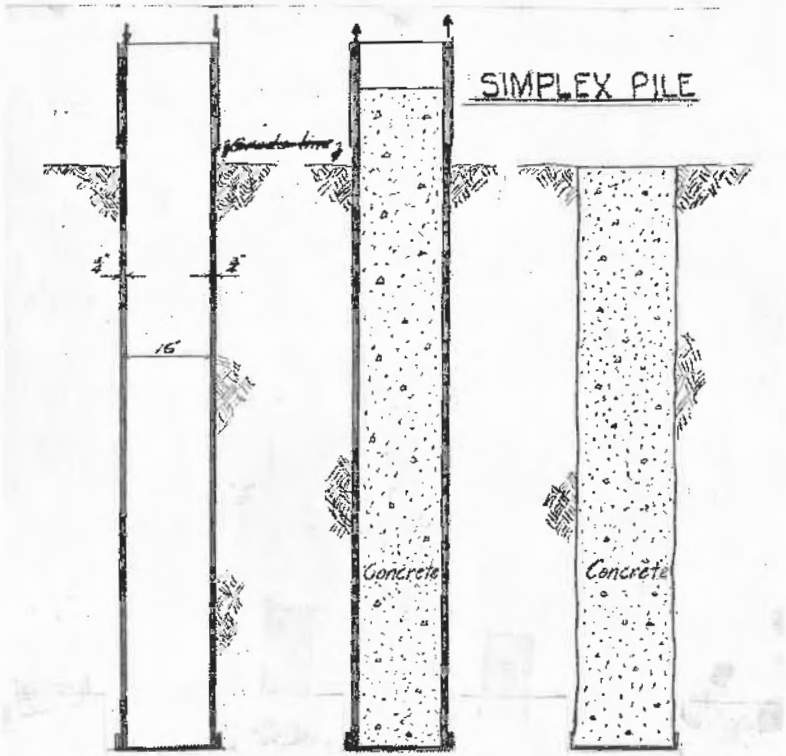

MACARTHUR SHELL-LESS CONCRETE PILE




MACARTHUR SHELL-LESS PEDESTAL PILE

FIG. 9

to the required resistance or depth of embedment. The pile hammer is raised to uncover the top of the pipe which then is filled to the desired height with concrete. Without an inner pipe or mandrel this cycle of operation offers no chance to form a pedestal or compress the concrete. The pipe is pulled for re-use and introduces the possibility of necking the outlined under "MacArthur". To reduce the chance of discontinuity, this pipe is driven on not less than 3-foot centers. The Simplex pile is occasionally used inland in the New England area for footing support in non-plastic soils.




THICK WALL PIPE MANDREL AND BOOT DRIVEN THE REQUIRED DEPTH

MANDREL FILLED WITH CONCRETE

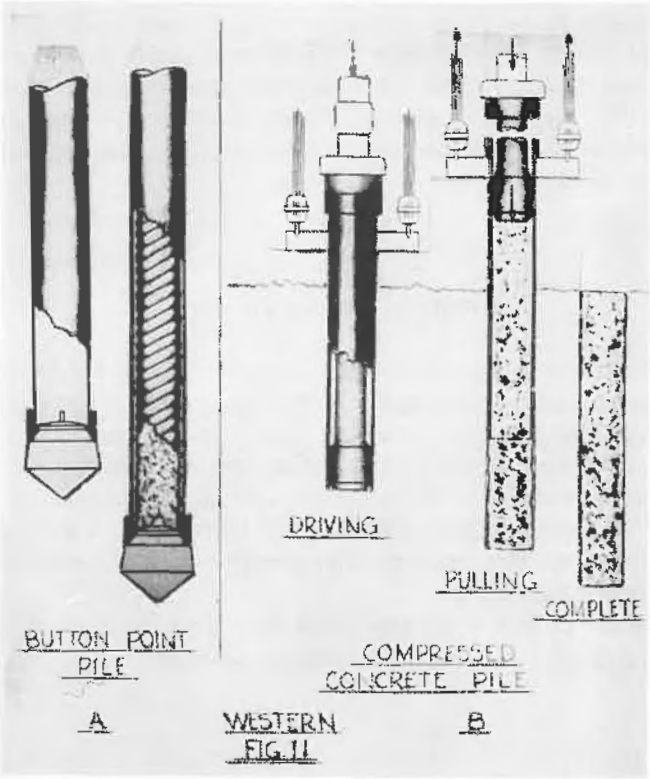
MANDREL WITHDRAWN LEAVING CONCRETE IN PLACE

FIG. 10

The design load is governed by the inside diameter of the drive pipe times the allowable concrete stress.

*Western (Fig. 11)*

Basically this pile is the original MacArthur concept. Over the years a proliferation of fanciful refinements, especially at the lower end of the pile, have been developed.




Note: the force required to pull a properly driven pipe-mandrel using the fresh concrete as the reaction will not be enough to force the concrete deeper as is shown in the columns above "pulling" and "complete". The same comment applies to MacArthur pedestal, Fig. 9. Both are exaggerations and simply not true.

Western's compressed concrete and button pointed piles are the two most used at present. The compressed concrete pile is installed by landing the inside pipe on the fresh concrete placed in the drive-pipe, which is used as a reaction for pulling the outside pipe with special block and tackle. The concrete is forced out instead of moving out under its own weight and that of the hammer, as would be the case as the drive-pipe is pulled when forming the basic pile. The button pointed pile consists of a point made of special concrete. Cast in its top center is a coarse-thread-

ed pin. The point is driven on the bottom end of a thick-wall pipe. Then a closed-end corrugated shell, with a center hole in the bottom plate, is set on the concrete point. A special made tool for rapid accomplishment is used to screw a nut on the pin. Concrete is placed and the drive-pipe removed. (See dropped-in-shell pile.)

Space does not permit the presentation of other types. Those interested should write Western Foundation Company for brochures. An outside thick-wall pipe is driven in every case. It must be pulled for re-use. The pipe is driven on not less than 3' centers. Its design load is governed by the inside diameter of the pulled pipe, and the allowable concrete stress.

## Dropped-In Shell
*(Cast-in-place concrete piles)*

For numerous reasons it often is required with piles formed by driving a closed-end thick-wall pipe that a permanent internal shell be installed before pulling the drive pipe. Some concrete must be deposited to prevent leakage at the bottom and the shell from moving up as the drive-pipe is pulled. As the drive-pipe is pulled, the surrounding earth moves in to meet the shell. (See Western's button pointed pile). Adoption of this procedure ostensibly is an attempt to meet the advantages of the permanent-shell pile.

The design load is the area of concrete times the allowable stress. This type of pile is used for footing support inland and not through open water.

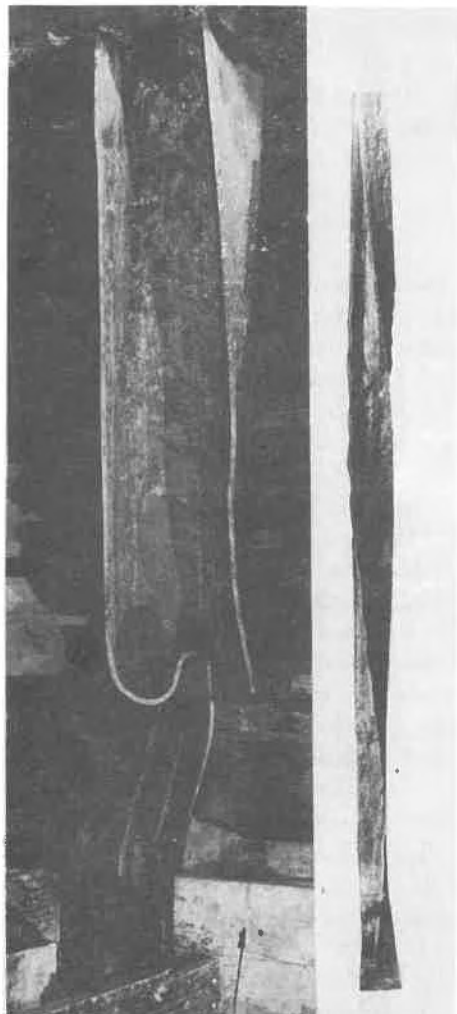
## Steel H Pile

Steel H piles are driven directly. To hold the pile and hammer centrally during driving, an anvil base, or follower, built especially for the purpose is attached to the base of the hammer.

The "H" is a solid bodied pile. After it is driven there exists no opportunity to determine its actual position or condition. Yet, with in-place shells and pipes these determinations are feasible and form the basis for untenable demands for accuracy in final position and condition.

For lengths required in excess of those delivered, the pile is spliced by welding. Many types of splice plates have been used. With first class equipment and workmanship the full strength of the beam can be developed by welding together butt and scarfed ends without plates.

To develop the same frictional resistance, the H pile will drive longer than other types of piles. The H pile, being made of steel, is subject to loss of metal by corrosion. The bottom end, especially in light weight sections with thin flanges and web, will deform, split and curl during driving almost as readily as a wood pile (see Fig. 12).



ONE THAT FAILED BETWEEN ITS ENDS

STEEL H PILE FIG.12

The H pile is used in all classes of construction, even where it does not belong in seawater exposure. Its design load is controlled by the unit stress allowed by the authority under which it is being installed. This stress may vary from 8,000 to 15,000 p.s.i. Allowance usually is made for possible loss of metal.

The damaged piles, Fig. 12, are shown not to detract from the usefulness of the steel H section, but rather for the benefit of many engineers who proceed on the thoughtless basis that its adoption solves all pile problems. It does not. It is not a universally applicable pile any more than is any other pile. It has its place.

## Pre-Cast Concrete Piles

To date, except in localized areas, the pre-cast concrete pile has not proven competitive with other piles on inland construction.

The design load is governed not by the strength of the pile materials, but by the type of deck and superstructure, by the depth of water and soil conditions, by the deck live load and the docking of ships, by the ability of the concrete to withstand driving, and so forth. In no event will these piles withstand as hard driving as will steel.

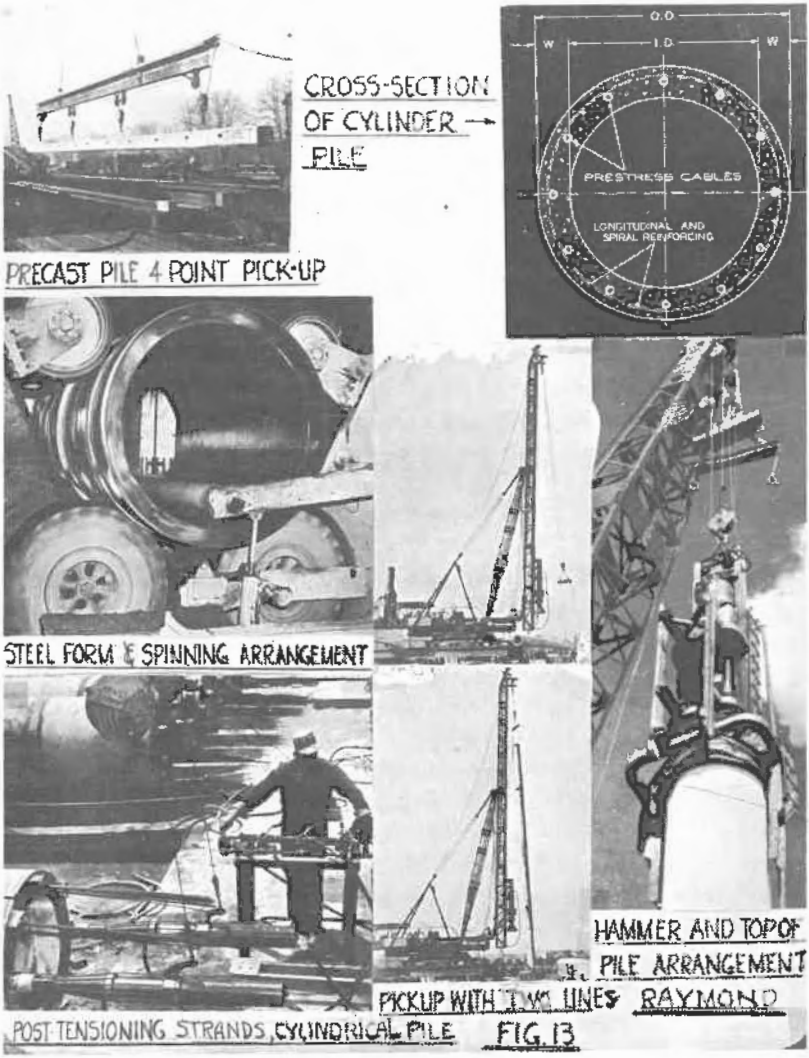
A pre-cast concrete pile may be of any cross-section, from circular to rectangular and tongue and grooved for sheeting; some are cast hollow. Concrete is placed with the pile in a horizontal position. The concrete must develop predetermined strengths before the pile can be removed from the form and handled from the horizontal to the vertical position and driven. Gain in strength is hastened by steam-curling.

Pre-cast concrete piles are used in open water and occasionally for elevated structures inland. Predetermination of pile length, except when driven to level bedrock, is not possible. When cast too long the pile must be cut off and the excess length wasted. When cast too short, the pile must be lengthened. To lessen the resultant delay in driving, splicing compounds have been developed. The writer has had but casual experience with the use of these compounds. The life service of these compounds in sea-water exposure is yet to be proven.

## Pre-Tensioned Concrete Piles

Pre-tensioning strands for the casting of concrete piles is of relatively recent usage in the United States. The abutments of the casting beds, to resist movement under the forces required to stretch the

strands, must be rugged. They are set far enough apart to accommodate casting a number of piles in between. With modern equipment and methods, piles are removed from the form within 2 days after casting. Within a week they seemingly are handled with but little attention paid to pick-up points, and driven (see Fig. 13).




Post-Tensioned Cylinder Piles (Raymond)

This is a post-tensioned pre-cast concrete pile in the form of assem-

bled hollow cylinders. Each cylinder is 16 feet long. The strands for any number of cylinders, end to end can be post-tensioned to fabricate the desired length of pile. Joints between cylinders are sealed with a mastic compound. Present sizes are 36" and 54" O.D. with 4" to 4.5" and 4" to 5" wall thickness, respectively. The number of post-stressed strands varies from 8 to 24.

Each cylinder is cast in a horizontal position in a steel form that is spun on trunnions to distribute and then wring excess water from the concrete. Strong concrete is developed. Longitudinal and spiral reinforcing steel is used. A hole for each set of strands is prepared by housing a steel rod with a rubber tube. Almost immediately after the completion of spinning, the cylindrical form with its fresh concrete is moved to a standing position, then to a curing yard. Within a few hours thereafter the form, steel rods and rubber tubes are removed. Curing of the concrete is hastened by steaming.

While driven for axial bearing, this pile possesses the added factor of stiffness. It therefore is best suited to support over-water and above-ground structures. The pile itself is heavy, the equipment to handle and drive it properly must be powerful and for economy the force or forces it is to resist must be above the usual.

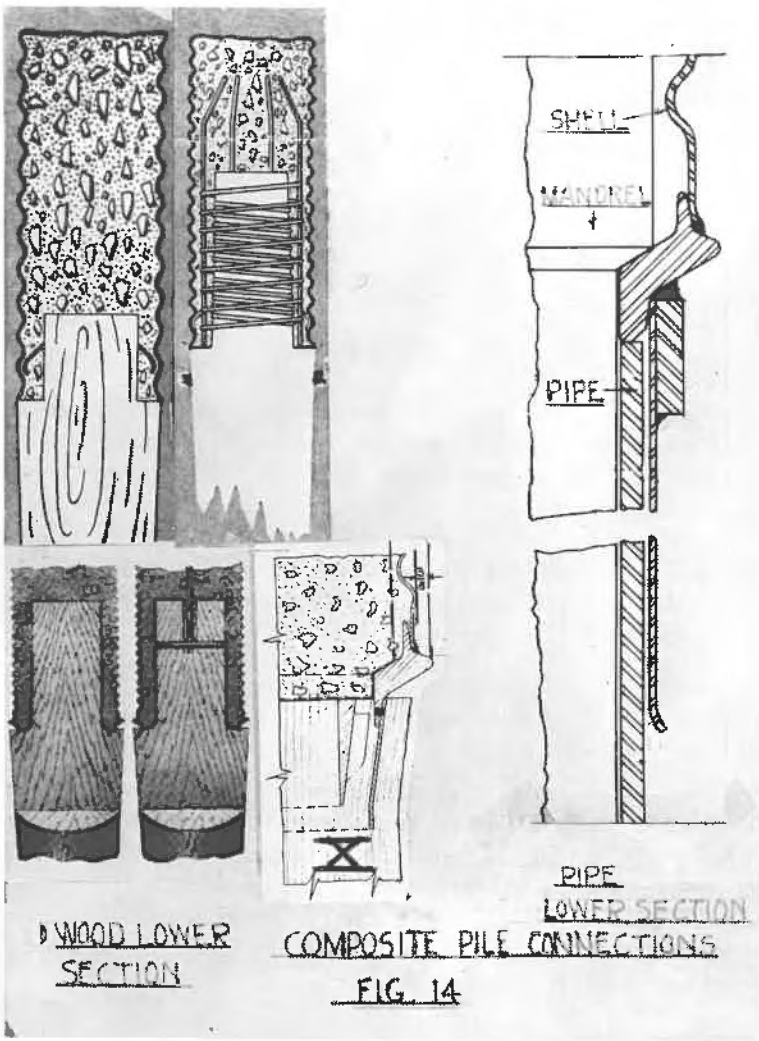
## Composite Piles

A composite pile may be a combination of any two of the previously identified piles, except the treated wood, compacted concrete and cylinder piles. In this country the combinations have reduced to two: the untreated wood-pile lower section or the closed-end-pipe lower section, both in combination with the cast-in-place-concrete, permanent-shell upper section.

A watertight joint between the two sections is imperative, as is a positive connection to prevent vertical separation and horizontal offset of the upper section by the driving of subsequent piles. The connection must be mechanical or welded and effective whether or not the concrete has been placed and had time to gain strength. For wood-pile lower sections the types of connections are numerous and mostly ineffective. Those for the pipe-pile lower section, both sections being of the same material, are easier to make secure.

Of the several wood-to-concrete connections shown, and there are many others, the only positive one is marked X in Fig. 14. The outside steel band protects the wood-pile head from splitting and the inner tapered ring wedges the wood between the two so tightly that the shell, whether or not filled with concrete that has cured, will not heave during

the driving of subsequent piles.




COMPOSITE PILE CONNECTIONS

FIG. 14

The wood composite pile is difficult to install. Its proper installation requires the full time of an experienced inspector. A wood pile section length cannot be accurately predetermined. When judged too long it will be overdriven in an effort to put its top at permanent water level. When judged too short it will be soft driven to avoid the cost of added footage of concrete upper section. One party to the contract bears the

onus of this predetermination. Still, while many structures are resting satisfactorily on this type of pile, it also has given an inordinate amount of trouble.

The pipe composite pile is not too difficult to install, the bottom section being straight to start with.

The wood composite pile is used in lieu of untreated wood where the permanent water level is much below normal cut-off and the pipe composite is used where the length of pile exceeds that of the cast-in-place concrete pile mandrel.

The design load for each type is that allowed for the weaker of the two sections.

## Pile Driving Machines

Other than machines built for special work, land pile machines generally are the self propelled crane type, traveling on caterpillar tracks or on pneumatic tire-equipped wheels. They might well be termed universal. The leads can be changed from plumb to a maximum in or out batter of 5 horizontal to 12 vertical in a matter of minutes. Most machines are equipped with boilers of a capacity to furnish the needed steam, or lacking the boiler, with independent compressors that supply air to drive the hammer at its rated speed (see Fig. 15).

## Pile-Hammers

Pile hammers, other than the resonant hammer, drive by impact, The hammer is driven by steam, by compressed air or by oil. The energy in foot-pounds per blow of manufactured hammers ranges from a high of 114,000 to a low of 3700; the weight of ram from 40,000 to 700 pounds and the blows per minute from 150 to 50. By comparison, piles, other than the cylinder pile, may weigh from 40 tons for pre-cast concrete to less than 0.3 tons for untreated wood.

## Bodine Resonant Driver

This machine is high speed, powerful and complicated. As presently developed, it is driven by two 500 H.P. gasoline engines. The working force is generated by two heavy weights rotating in an orbital path in a strong enclosure. The two weights, or rollers, are in phase with each

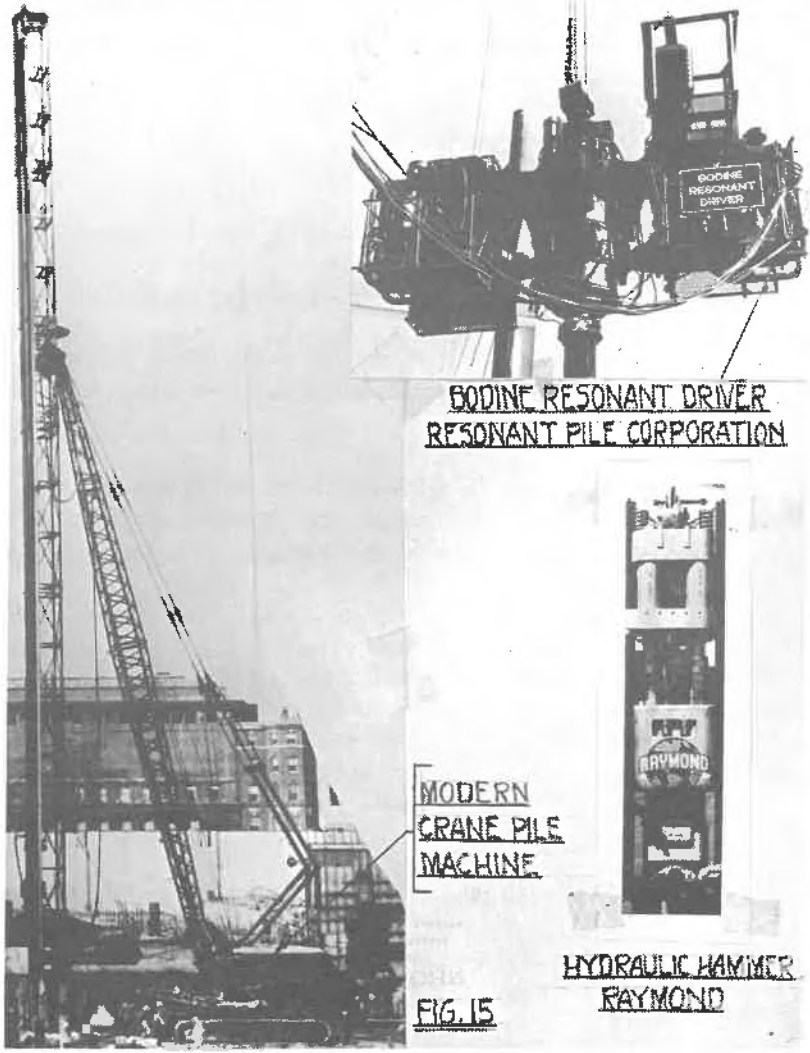



FIG. 15

other, but are rotating in opposite directions. The horizontal force, or impulses, cancel out. The vertical impulses are additive and create the longitudinal vibratory waves which alternately lengthen and shorten the pile or pipe. The impulses range between approximately 60 and 150 cycles per second.

The speed at which a pile or pipe is resonated is determined by its length. Structural members up to 360 feet in length have been driven with this hammer. The pile being driven must be rigidly attached to the

resonant driver to form part of the resonating system. Powerful hydraulic clamps designed for most types of piles accomplish this requirement.

Final driving characteristics of a pile are obtained from gages on the operator's console which indicate the horsepower input and the rate per second at which the pile is penetrating the ground.

The resonant driver will:

1. Drive piles fast.
2. Drive piles through frozen ground, (50 feet has been penetrated by an H-Pile in a recent test in 25 minutes).
3. Drive plumb piles that have been set firmly in the ground without fixed leads.
4. Drive open-end pipes with the soil inside raising instead of lowering (plugging) as with impact hammers, producing a truly non-displacement pile.
5. Pull a pile as readily as drive one.
6. Create no objectionable ground vibrations during driving.
7. Drive piles but not steel sheeting with the only noise being that of the muffled gasoline engines driving the unit.

## Raymond Hydraulic Hammer

This hammer is operated hydraulically under oil pressures of 5000 to 5500 psi. Its operating mechanism is similar to that of the differential steam hammer. Its speed of operation is from 120 to 130 blows per minute. It drives by impact.

## Pile Hammers, Anvil Base or Followers, Extractor, Cushions

Fig. 15A should be referred to for the following item explanations.

A A Number 1 Vulcan, single acting pile hammer; steam to raise the drive weight, gravity to pull it down; 56 to 60 blows per minute. For some time it was fancied by many engineers and contractors that a pile driven anywhere to 4 blows to the last inch with this hammer would safely support 30 tons. Irrational ideas soon end as did this one. Vulcan also manufactures double acting hammers (steam up, steam and gravity down) and hammers that operate under water.

B A McKiernan-Terry enclosed, double acting hammer; 110 blows per minute. This hammer will operate under water. To do so, compressed air is fed into the base to exclude water from the ram chamber and from the top surface of the anvil base. A positive

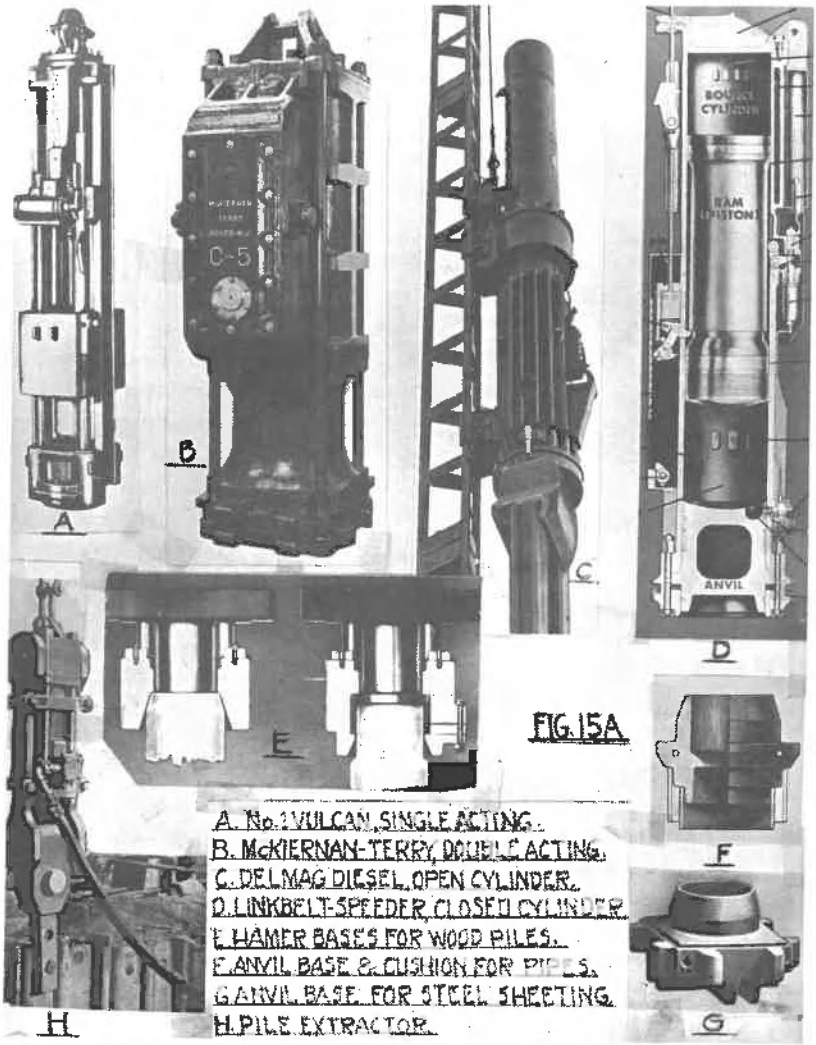


FIG. 15A

means for determining final penetration must be provided and a hard-wall exhaust hose used to prevent collapse from water pressure. They also manufacture single acting and diesel hammers.

C A Delmag Diesel. Since the top cylinder head is open it is single acting. The height of fall is determined by observing the distance the plunger projects above the cylinder at the top of the stroke. The hammer operates on oil at roughly 50 strokes per minute when determining the final driving resistance. The pile is moving slowly at

this point and the plunger gains the advantage of the rebound.

D A Link-belt Speeder Diesel, has a closed top cylinder, thus creating what is termed a bounce chamber; 100 to 110 blows per minute. It is partially double – acting to the extent of increasing its speed . Its operation is entirely blind, but from gage recordings of bounce-chamber pressure its energy per blow is established empirically. The indented anvil base shown on this hammer is properly shaped for driving wood piles. Diesel hammers are being used more often with time in the United States.

E Shows two types of hammer bases for protecting the heads of wood piles during driving. Just because this picture has been shown in sales catalogues for years, which is the contractor's argument, it does not make this arrangement proper.

Left: : The hammer plunger will form a cavity in the top of the pile during the driving operation. To correct, insert between the pile and plunger a 1" thick steel plate of the top diameter of the truncated cone in the hammer base. This is the proper arrangement for driving a wood pile with this base.

Right: A steel plate is shown to protect this pile head. For the largest hammer that should be used to drive a wood pile the tenon shown would be approximately 11" in diameter. Even with the steel plate an unbanded pile head will split and broom during driving. Wood piles, for driving, are headed by dock builders. For this purpose chain saws are now used instead of an axe. The flat top should be not less than 11" in diameter. Still the writer has seen not one but many pile heads dressed with chain saw to 4" square. When the hammer is raised after driving the pile head resembles a weeping-willow tree. Proper procedure is to scribe the head of the wood pile to the flat under-surface diameter of the anvil and chamfer the edge with an axe to allow the pile to enter the anvil reasonably snug. In this work the best is not good enough.

*General*

Between the hammer and pile is inserted an anvil base or follower. Its function is to center the hammer on the pile during driving. Its upper face is prepared to receive the cushion needed to prevent a direct blow of the ram on the pile. Its lower face is indented to fit the cross section of the pile to be driven or to enclose its diagonals, thus permitting the pile to rotate during driving. Square followers, especially tight fitting ones for precast piles, often result in damaged heads. Indented followers to fit H sections might account for some

of the damage done to these piles during driving.

F A pipe-pile anvil base or follower. Two pipe sizes are shown, a smaller pipe would fit inside. The cushion in its top apparently is a lathe-turned, hardwood block. Again, as shown, the arrangement is wrong. Correct procedure would be to shorten the block 1" and insert a 1" thick, snug fitting steel plate. Without a top steel plate one is reverting to the situation criticized in "E Left", above.

G Steel sheet pile anvil base or follower. The ears on 4 sides indicate that it may be turned 90 degrees in the leads for driving alongside or astride the line of sheeting. Its top shield, now shown as a cavity needs a cushion block and steel plate.

## Cushions

Normally cushions are set between the ram and anvil base or follower to prevent a direct metal to metal blow. They may be elaborately made of laminated materials, discs of asbestos or of soft metal or lathe-turned hardwood blocks, or boards or rings of old cable or rope or of any scrap material. Between the top of a pre-cast concrete pile and the follower is placed a cushion of 1" hardwood boards 3 to 4 inches thick and free of knots. Holes through this cushion for projecting steel dowels must be accurately spaced and generous in diameter. It is not possible to even judge the actual life of any type of cushion. At the moment the efficiency of one type of cushion as compared to another is a guess.

Assume the efficiency of a lathe-turned, parallel-to-the-grain, snug-fitting, seasoned, hardwood block not more than 6 inches long; confined within a cast steel shield, between top and bottom thick steel plates, as approximately 1.0. Whenever the cushion arrangement to be used appears less efficient, increase the percentage of overdriving; if more efficient let up on the percentage of overdriving. Permit no renewal of the cushion during the last 3 feet of driving of any pile. If renewal occasionally is unavoidable then overdrive that occasional pile much more.

A new cushion, which allows the minimum height of ram fall, begins to shorten under the blows of the hammer. The cushion is replaced when it has been beaten to a pulp and often afire when the height of ram fall is a maximum, but before the ram strikes and drives the base off the rods that hold the hammer together.

Energies of pile hammers seldom coincide with design pile capacities in pounds multiplied by 0.14. Whenever the energy of an available

hammer is less than the computed energy, but always by less than 5%, increase the percentage of overdriving; if of greater energy decrease the percentage of overdriving.

This analysis seems valid where the point of the pile is finding its resistance in the specified load-supporting material. A final resistance exceeding that required by the formula by more than 100% would indicate a new judgment or rejection of the proposed equipment.

No analysis or formula is valid when the pile develops the required resistance where its point is above the specified highest bearing material. Unaccompanied by common sense and practical experience no pile driving formula is of any use.

## Pile Extractors (Fig. 15 A-H)

Very occasionally an axially loaded pile is pulled; steel sheeting very often. When placed in the line of a strong purchase, this tool is of material help when pulling a pile. On the end of a single line or in the bight of a double line the extractor is not of much help. The sheeting shown in Fig. 15A is not deeply embedded or the handling holes would rip out during the pulling operation, which did not happen.

## Leads

To drive piles properly requires that the pile and hammer be centered with respect to each other and the two held to the plan alignment during the entire driving of the pile. No other method of driving piles known to the writer provides these two requirements with reasonable speed of operation. Only at pile locations where the machine cannot be positioned for driving with fixed leads would the writer permit the use of a swinging hammer or a hammer riding in the ways of swinging leads to drive piles.

## Jetting

Jetting, the loosening and removal of soil with a stream of water under pressure, is used to reduce the amount of driving or to get the pile past a dense or heavy material. A hole may be made in which to insert pile or the jet may be operated during driving. At times both methods fail to work. Whether true or not, the writer has concluded that jetting will be ineffective at locations where the jet water and loosened soil fail

to return to the surface. In some instances success with jetting can be known only after a try is made.

A water jet, properly made, is of extra-heavy, flush-coupled pipe with a long-radius bend at the top to prevent a kinked hose and a machined nozzle at the lower end, not the pipe beaten in.

## Spudding

Where the soil crust or plant-waste material is dense, a hole is first made by driving and pulling a steel spud in which to start the pile. Sometimes this procedure will break up a layer of frozen ground. Where the crust is not too thick, spudding will prevent damage when driving wood piles and make way for other types of piles to be driven more to plan alignment and straighter.

## Pre-Excavating

To prevent heave and to lessen the skin-friction during and after driving, a hole is preaugered for each pile. A rotary bit usually 2 inches larger in diameter than the pile is used with a re-circulating liquid to auger a hole practically to the bottom of the clay or to the bottom of a top dense material. To remove the cuttings from the hole, the bit should have 2 small nozzles directed upward, otherwise large volume pieces settle to the bottom of the hole where they are not wanted.

## Dry and Wet Augering

Dry augering a hole for each pile by machine through a dense top material, will lessen the difficulty of driving piles at that location. These holes should be excavated just ahead of the driving and not sooner.

Wet augering can be used to make a hole for each pile to be driven through granular and plastic soils. A prepared mud to stabilize the wall and bottom of the hole should be used for the circulating liquid. To minimize the volume of settled solids the auger should be equipped with a concentric cylinder into which the coarse particles settle at the change in rate of upward flow at its top.

Fill or soil containing particles larger than 3 to 4 inches in diameter may hinder or prevent augering by either method.

Visualize the many shapes, sizes, lengths, weights, materials of the driven part, pile surfaces and the equipment just described. Consider the many types of soils through and into which piles are driven. Then try to derive a pile-driving formula that will include only the major factors. A formidable assignment in a rough operation, but the profession still keeps trying. Developing a universal pile driving formula is as likely as the development of an all purpose pile.

Why not consider the following procedure?

1. Study the soil conditions disclosed by reliable borings properly logged.

2. Examine in detail and test, where necessary, representative soil samples.

3. Determine the stratum on or in which the pile point must rest.

4. Specify the type of pile, the minimum energy per blow of the hammer when operated at full speed, the acceptable cushion and anvil arrangement without being too fussy, and the minimum number of blows for the last 3 or 6 inches. (The number of blows for the last 1 inch are easily misread).

5. The present Boston Building Code requires the hammer energy in foot pounds per blow to be numerically not less than 0.14 of the design capacity of the pile in pounds. The constant 2 in the numerator of the Engineering News formula is changed to 1.7. The factor 0.1 is multiplied by the square root of the ratio of the weight of the driven part to the weight of the hammer-ram; in no event shall this ratio as entered into the formula be less than 1.0, nor shall it exceed 3.5. Adoption of any formula is not the whole solution. Its answer is a point from which to start.

6. Plot the driven pile lengths alongside the applicable boring or borings. Willing compliance with this simple procedure seems repulsive to many construction engineers; they are disinterested. To them and the contractors' organization satisfying the requirements of some formula is enough. And too many engineers are overly willing to abandon to others the construction of their design; a serious lack of continuity in the completion of any construction operation.

7. Specify spudding, jetting, pre-augering wet or dry, or other special procedure as the study may determine.

8. Require static load tests for piles driven in unusual soil conditions, with new types of equipment, for unusually high design loads and not otherwise. Require redriving under some soil conditions to determine any increase or decrease in final set of the pile with time.

9. Stay away from piles, except the open end pipe, where at all

possible, at locations where obstructions including boulders and rip-rap are known to exist or where the pile point must reach a specified elevation.

10. Soil test borings are made, the samples obtained and the results logged for the benefit of the design, the soils and construction engineers and for the contractors' organization; they are not made to benefit one party alone.

## Payment for Piles

Payment for driven piles may be constructed for one the following bases:

1. The length measured from plant cut-off to actual point elevation, per linear foot. This basis of payment is equitable provided that the moving on and off charges are a minor part of the total cost and the estimated is close to the actual footage. Otherwise, should the actual footage be greater than the estimated, the client suffers financially. The contractor suffers should the actual footage be materially less than the estimated.

2. Per linear foot for the length of a one piece pile set under the hammer. The owner suffers unless strict limitations for ordered lengths are placed on the contractor.

3. Per linear foot for the length of a one piece pile set under the hammer and pay for length cut off at a lesser price per foot. Payment methods 2 and 3 are equitable for wood and treated piles which are produced by nature. There should be no payment for lengths cut off of man-made piles. For the latter, a length contingency should be included in the estimate of cost for bidding.

4. A lump sum price for moving equipment and organization to and from the construction site plus a unit price per foot of driven pile measured in accordance with the provisions of basis 1. This method of payment is equitable in all cases for man-made piles. For timber piles the pay length should be determined as is outlines in cases 2 or 3.

5. A principal sum, consisting of moving on-and-off charges and the price for a given number of piles of a stated base length, the principal sum being adjusted for the difference between the stated and the actual lengths of individual piles or for the aggregate difference between the stated and actual lengths. Since the quoted price per foot for omitted length is seldom more than half that for added length, it is most equitable when the adjustment is made on the change in aggregate footage, not on change in individual pile lengths.

6. A lump sum for the number of piles shown regardless of length

with no adjustment in number or price. Many owners prefer this form of contract to one involving adjustments. A lump sum price is obtainable very rarely and then only where the site has been thoroughly explored by reliable soil test borings, where the soil conditions are not erratic, where the piles are to be driven to a dense granular stratum, to hardpan or to bedrock and where the pile contractor can add to his usual price a contingency to offset the assumed risk.

7. A cost plus percentage form of contract; it is negotiated. It is resorted to in times of uncertainty or when volume and coordination are indeterminate or where time, not cost, is the controlling factor. Other circumstances often force its adoption. The owner pays all cost directly connected with the work in the field. The costs generally include: transportation, rental of power driven equipment at agreed or published rates, materials, hand tools and consumable supplies, labor, supervision, insurance and so forth plus a percentage of the whole for general overhead and profit. When actual costs are high, the contractor is exposed to the serious charge of neglect or abetment in order to increase the amount of his fee. Often the criticism is justified.

8. A cost plus fixed fee form of contract eliminates the criticism cited under the cost plus percentage contract. After establishing the estimated cost and fee, any reduction in actual cost increases the percentage of profit, an inducement. The fee is not increased should the costs be greater.

9. The cost plus fixed fee, with an outside price form of contract is resorted to at times. It is not equitable unless the contractor participates at least equally with the owner in any savings. The contractor's fee is lessened or it may disappear should the actual cost overrun. Many forms of this type of contract are practiced.

*Accessory Items*

1) Spudding, 2) Jetting, 3) Wet-augering, 4) Pre-excavating or dry augering.

5) Re-driving

Bid price for each of these items may be quoted per unit, or be included in the quoted price per foot. From his study the engineer should determine which method or any other method of payment is most equitable to both the contractor and the owner. Very often adopting one of these procedures materially increases the rate or progress when driving piles.

*Obstructions*

Normally, the cost of removing man-placed obstructions is the burden of the contractor.

The footage of a pile encountering a nature-placed obstruction is paid for as if it were complete as well as the footage of a replacement pile or piles driven to one side.

## Caissons

*(Cylindrical excavations with or without an enlarged base made on land and filled with concrete to plan cutoff elevation.)*

Construction of this type of foundation unit when completed under compressed air is recognized everywhere as a caisson. Otherwise the term "caisson" when used to identify a cylindrical foundation unit on land may be misleading. It is known differently in different parts of the country. Since this subject is "Pile and Caisson Foundations" the term will be used as it is recognized locally.

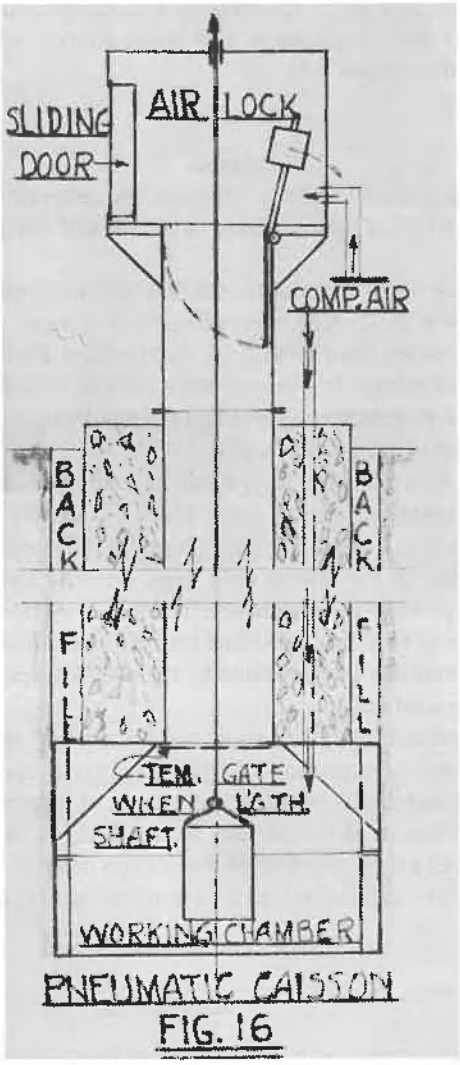
Open caisson work most everywhere necessitates groundwater control. In an excavation, ground water may be controlled 1) by surface pumping to the depth where soil grains begin to move with the water, 2) by pumping through well-points or in deep, properly screened sumps or wells, 3) by air pressure in an inclosed chamber, 4) by freezing and 5) by electro-osmosis in very fine individual grained soils. In a flooded operation ground water can be controlled by the over-balance in liquid pressure with a prepared mud.

Accurate subsurface information and satisfactory soil samples are essential to a predetermination of the feasibility of open caisson construction. A successfully installed test caisson at a given site does not mean that all caissons at the site can be installed. A changeover, after starting work, to a different type of foundation is not uncommon. The installation of caisson foundations is hazardous construction.

### *Pneumatic Caisson*

A working chamber with a cutting edge forms the bottom on the end of a vertical hollow concrete shaft. At the top of the shaft is an equalizing chamber, the air lock, which by manipulation of the two doors provides means of ingress and egress for men and material between atmospheric and working pressures. Ground water is kept at the bottom of the working chamber by increasing the air pressure as the excavation is deepened (see Fig. 16).

State laws and working conditions set by the unions, while justified to a great extent, have pretty well eliminated the use of pneumatic caisson construction inland. Other methods have replaced it.




PNEUMATIC CAISSON
FIG. 16

*Drilled-In-Caisson*

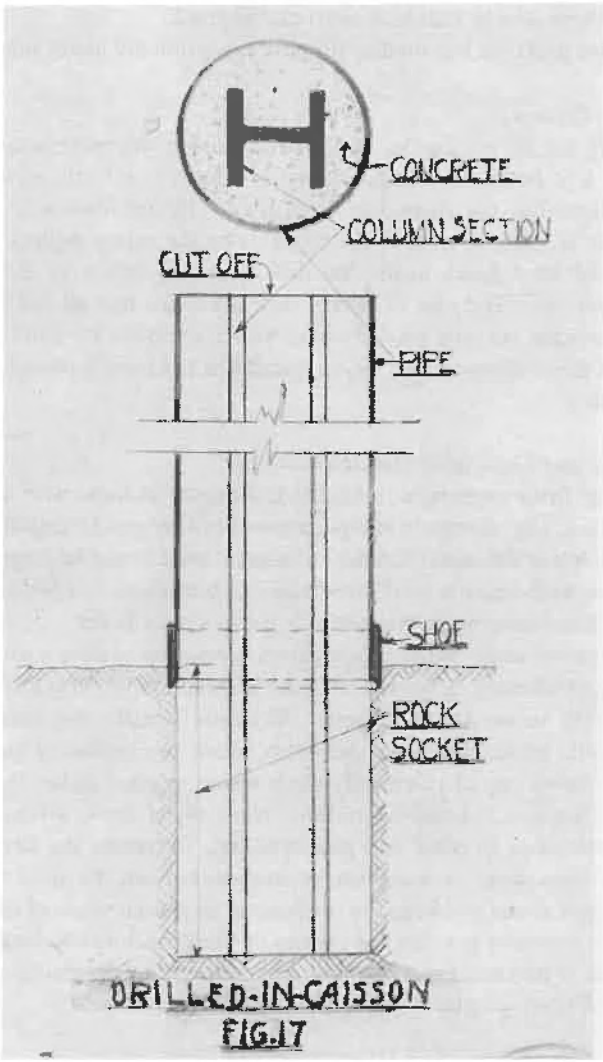
A drilled-in-caisson unit consists of (Fig. 17):

1. An open end pipe fitted on the bottom end with a thick, wide steel band driven to and sealed on bedrock. The pipe is cleaned out by the well-drilling method.

2. A socket the size of the inside diameter of the pipe excavated in the bedrock by the well-drilling method.

3. A steel column section through the full length of the unit, centrally positioned in the socket and pipe.

4. The space between the column section, socket and pipe wall is filled with concrete.




DRILLED-IN-CAISSON
FIG.17

The cross-sectional area of the pipe is allowed one stress, that of the column section another and that of the concrete still another. At the lower end the total load is transferred at an allowed unit per square foot of bearing on the bedrock and an allowed unit per square inch of socket

surface.

Except for cleaning out the driven pipe and excavating the rock socket, all work is done in the open. Where the pipe cannot be sealed on bedrock, it is filled high enough with water or a prepared mud to overcome the inward movement of water and soil at the leak. A socket cannot be excavated in friable or shattered bedrock.

These units are installed to support exceptionally heavy loads.

*Powell (Caisson)*

Only for the reason that this type of caisson was recently advocated for use is it being mentioned. Not for many years, to the writer's best knowledge, has the procedure been used. Its installation follows the principle of sinking wells in the oil fields by the rotary drilling method: a flow of mud down inside the drill stem and return in the annular space outside. The rate of return flow is so slow that all but the finest soils particles become settled solids to be removed by hand. In part this is a flooded operation. Economically, it has been replaced by other methods.

*Williams and Calweld (Rotary Excavators)*

Both firms manufacture bucket and auger machines with all attachments (Fig. 18). Competition has resulted in very good equipment. The operation is in the open. Control of ground water must be from isolated sumps or wells because a full sized auger or bucket cannot be operated in a cylindrical excavation containing a pump and/or hoses.

A loaded auger when raised above the ground surface is spun for unloading (Williams), a bucket must be removed from atop a cylindrical excavation to unload (Caldwell). As depth is made, the excavation is lined with telescoping steel cylinders which are reclaimed as the concrete is being placed. Generally these are set in place rather than driven as in a hand-excavating operation. Normally a crane attends one of these machines to place and pull cylinders, to handle the hand loaded bucket used when cleaning the excavation bottom, to move tools and equipment ahead and keep the machine at its special work of excavating.

It is common practice for owners of these machines to test-excavate holes at a particular site at their own expense to determine the feasibility of open caisson construction.

*Benoto (Machine and Method)*

This is a European developed machine. The writer has had but limited experience with the method. Essentially its operation is as follows (Fig. 19):

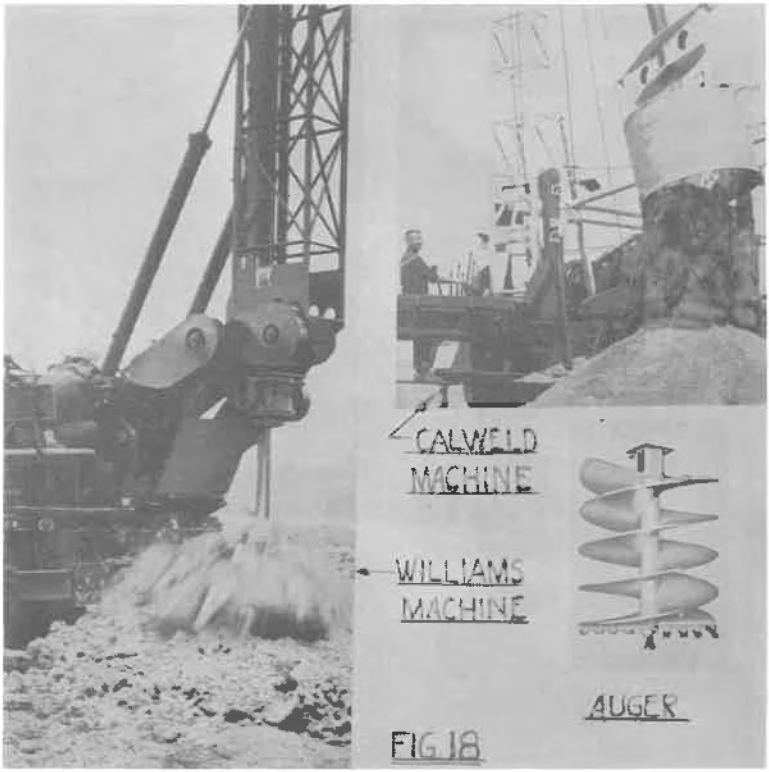


FIG 18

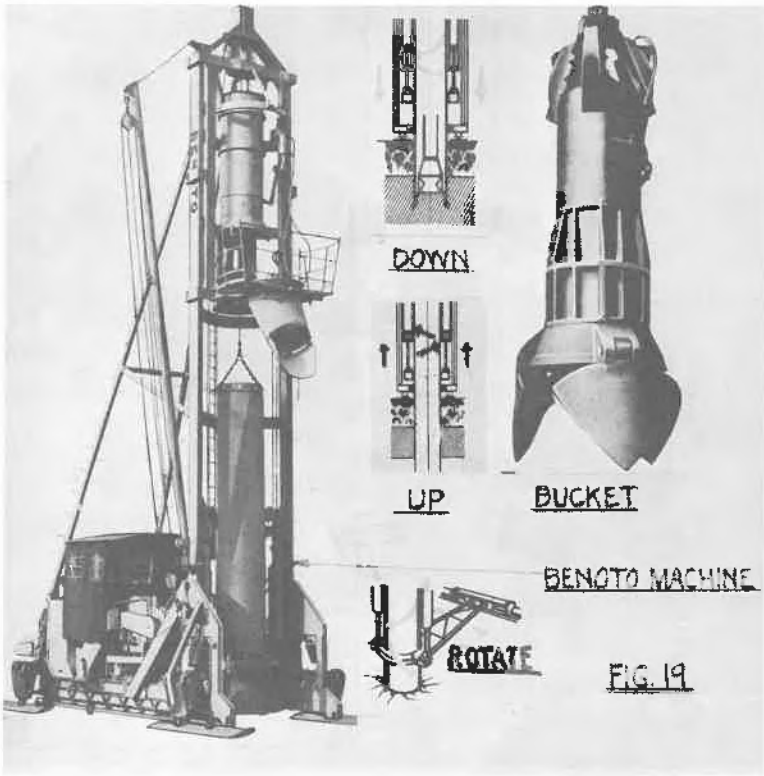
1. All cylinders to line the same shaft excavation are of equal diameter. They are joined together in a manner to resist compression, tension and reciprocating circumferential motion. The bottom cylinder is fitted with a cutting edge.

2. To resist uplift of the machine when the cylinder is being pushed down, the weight of the front end is increased by the aid of earth anchors.

3. The cylinder is entered into the ground and removed, hydraulically as the excavation is deepened and the concrete is placed, respectively. During each vertical movement the string of cylinders is partially rotated hydraulically, with a compressed air tightened band gripping the top cylinder.

4. Excavating inside the cylinder is accomplished by means of strongly built clam-shell or orange-peel buckets with rounded doors that are operated by the main hoist on the machine.

This process can be operated in an open or flooded excavation. Very dense materials can be loosened or broken up and removed by use




of these buckets. The process is usable but much slower than the auger or bucket machines in soils that are suitable for use of the latter. It is usable in soil unsuitable for the auger or bucket-machine.

*Belled Caisson*

To successfully install this type footing (Fig. 20) requires a material that will stand alone during the time required to excavate and fill the bell excavation with concrete. The bottom of the belling material must be at the top of the bearing stratum. Minimum thickness of the belling material must be equal to the design height of bell, plus 8 to 12 inches to allow for driving the bottom cylinder enough to seal out any top water. This work is performed in the open. Proper ground water control is an important factor. The writer knows of no such work having been done in a flooded excavation except where a bell roof developed

weakness and a diver was used to complete the bell excavation in a water-filled hole.

Mistaken judgment in the feasibility of this type of foundation leads to trouble for all parties concerned. Where soil conditions are suitable, it is the most economical type of foundation. The heavier the load, the greater its economy.

All such work, except in restricted areas and limited head room, is presently being done by machine. Much of it, because of the time element, would in the writer's opinion be impossible for a hand operation.

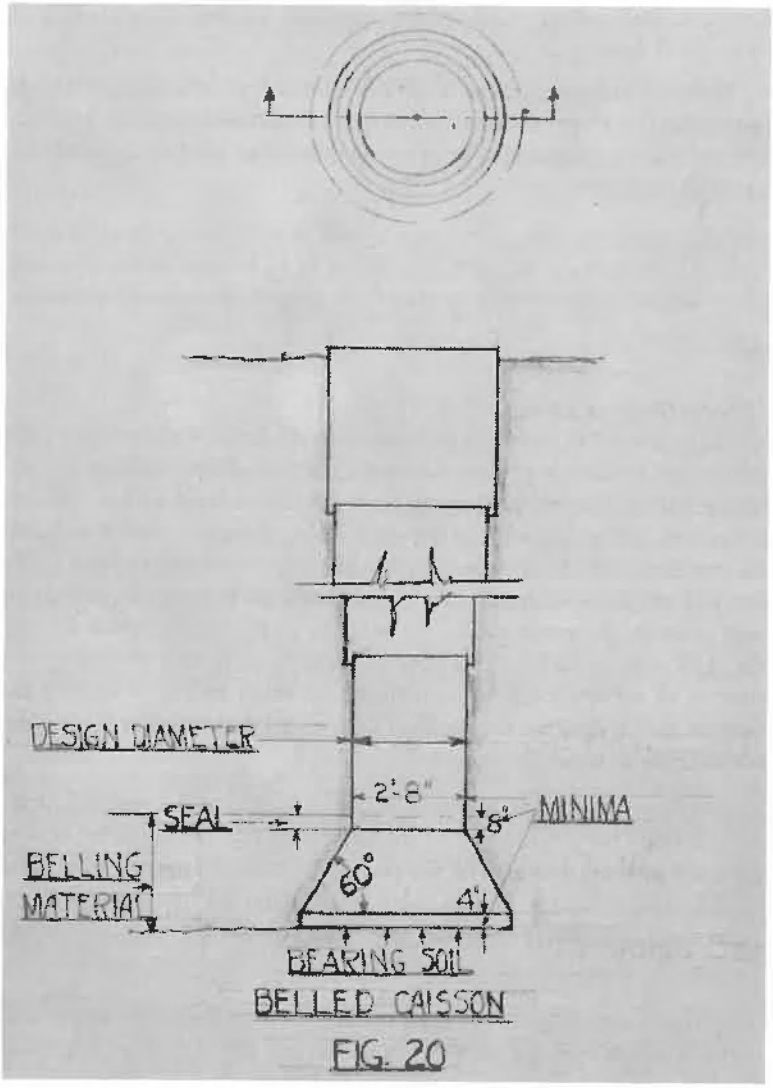
*Calweld (Reverse Flow)*

The Calweld Company has done much to develop and promote the process of reverse mud flow for installing cylindrical footings by wet rotary drilling. Except for the top few feet, which is lined with a cylinder to prevent caving or to seal out top open water, the excavation is unlined. Its exposed surfaces are stabilized by keeping the excavation and cylinder full of prepared mud. All loosened soil particles small enough to pass through the pump set at the working surface are removed through the drill pipe or kelly. This process reduces, or it may eliminate, the volume of settled solids to be removed by other means. Cleaning the bottom and leveling the rock surface with a well drill or roller bit may be bothersome at some locations.

As the excavation is deepened, sections of drill pipe are added. Each section must be not longer than the height the pump will lift water that is loaded with soil grains. With the pump set below the water swivel, that height, approximately 18 feet, is measured from the mud surface in the excavation to the top of the side opening on the swivel. Mud is fed in at the top of the cylinder.

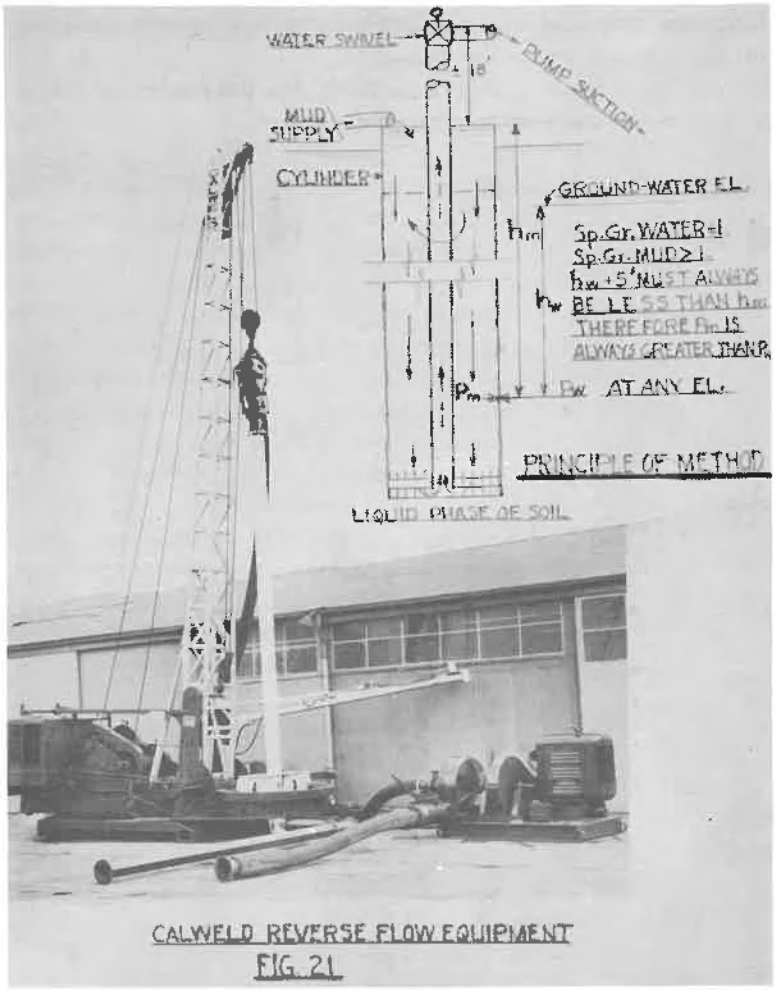
After completing the excavation the mud is displaced by tremie concrete. The method is positive, it is automatic and it is less hazardous than compressed air or the dewatering of large areas.

The mechanics of this method are shown graphically in the upper-right quardrant of Fig. 21. Since the writer knows of one collapse when the differential in hydrostatic head was 3 feet when using dirty water as the circulating liquid he assumes that a minimum differential of 5 feet is always safe with mud.




Basis for Payment

Except for the drilled-in-caisson type of construction this type of work is estimated and sold by volume. A form of contract that is suitable for the installation of a pile foundation is applicable to cylindrical caisson construction except that the pay unit is cubic rather than linear. Pay volume should be computed on the basis of neat design sizes measured from cut-off to actual bottom. To avoid argument this manner




CALWELD REVERSE FLOW EQUIPMENT
FIG. 21

of determination should be basic; let the contractor estimate the swell in volume due to increase in cylinder sizes and excavation above cut-off if there is any.

Ordinarily the removal of man-placed obstructions usually found near the surface is the burden of the contractor. Obstructions found below disturbed soil that interfere with usual progress are removed by volume or at a stated price per hour for use of machine and crew. An

obstruction projecting into the cylindrical hole is more troublesome than one wholely within the excavation.

Unless specifically provided for otherwise, the proper control of all water is the contractor's responsibility.

# HISTORY OF THE DEVELOPMENT OF BRIDGES

by
William A. Henderson
Member and Past President*

*(Presented at a Meeting of the Structural Section, November 7, 1966)*

Engineers have spent much time peering back into the pages of the past and trying to determine just what the first bridges looked like. Such speculation can of course never lead to any tangible conclusion. But it is self-evident that from the earliest times the choice of structure type has been limited basically by the types of construction material available. The development of new bridge forms has proceeded hand in hand with the development of better and stronger materials.

Since the first recorded structures were built of available natural materials that were weak in tension, it is not surprising that they were arch forms. The first arches date from as far back as 1500 B.C., were of brick and were found in Babylonia, Assyria, and Egypt. The ancient Hindus had a saying to the effect that "an arch never sleeps", implying thereby that the constant thrust of the arch produces settlements that could become troublesome. Indeed the Hindus scrupulously avoided the use of arches. The Egyptians, for the same reason, used them but little.

Fig. 1. Pont du Gard at Nismes, France

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It remained for the Romans to develop the use of arches, which they used in the construction of aqueducts, viaducts, bridges and buildings. For these structures they made extensive use of stone, partly because it was in plentiful supply but also because of the development of better cutting tools than had previously been known. Many of the Roman bridges were naturally over rivers. Where the piers for these bridges were founded on rock or ledge the structures endured amazingly well. Where the piers were on compressible or otherwise unsuitable material, the Romans invariably overcame the difficulty by widening the piers to decrease the unit pressure under the foundation. Wide piers had one additional advantage and, paradoxically, a serious shortcoming: if by chance one span in a series were to collapse, the wide pier could resist the unbalanced thrust from the adjacent span without undue strain. On the other hand, many arch structures were destroyed by undermining because of the high stream velocity from over-constriction of the waterway.

No discussion of arch structures would be complete without mention of the aqueducts in the design and construction of which the Romans excelled. These were composed of up to three tiers of arches superimposed one on the other, with total heights of as much as 160 feet. Remains of the Martian aqueduct, built over 2100 years ago, may still be seen. The original structure contained almost 7000 individual arches in its 39 miles of length. It can be seen that the Romans were well advanced for their day in the science of engineering.

After the downfall of the Western Empire there were few bridges built until the twelfth century. At that time travel and trade were increasing and the rapid development of cities made better stream-crossing facilities necessary. In Spain many bridges were built by the Moors, who in a great many instances made military considerations the prime factor. This is shown by their use of fortified approaches and by the occasional placement of angle points in the decks for better defense against attack.

The basic type in all this construction was still the arch, which has until now remained in high favor wherever particularly graceful or attractive form is required.

In the twelfth century the Benedictine monks formed an association called "Brothers of the Bridge" which devoted itself not only to the building of stream crossings but also to the construction of adjacent houses for the benefit of the sundry travelers. It might be said facetiously that this marked the beginning of the modern motel system. The bridges, mostly arches, were very narrow, varying in width from six feet to seldom more than twenty, and the largest span was approxi-

mately 110 feet, although the greatest over-all length is said to have been 2200 feet. Since the monks themselves raised the funds for building the structures, their coffers were seldom overflowing and the materials used in the structures were of rather poor quality. It is therefore remarkable that a few arches of a stone bridge built in 1177 across the Rhine at Avignon remain to this day.

Bridge of St. Benezet Across the Rhone at Avignon (1177 - 1187)

Actually of all the bridges built in the twelfth century and for the ensuing three hundred years, practically all were cheaply constructed, rested on unduly thick piers, and consisted of arch spans with earth-filled spandrels. It is surprising that the bulky houses supported on some of the narrow bridges remained as stable as they did.

A notable structure, begun in 1176 and completed in 1209, was the old London Bridge over the Thames. It consisted of a drawbridge and nineteen pointed arch spans, with houses resting thereon. The piers were so wide that they reduced the overall available waterway width from 900 feet to 194 and it is recorded that the restriction caused a drop of about five feet in the stream at the bridge. After eighty years it was in such bad shape that auxiliary supports had to be built for the houses. In 1758 the houses were dismantled and new arches built, and early in the nineteenth century the whole structure was replaced at a cost of over 40,000 pounds, a fabulous sum for those days.

Bridge at Kreuznach, Germany. Note the houses and the constriction of the waterway by the wide piers.

It is interesting on occasion to forget for a moment the technical side of our story and to take a quick look at the human phase. We find that in 1846 a certain Jonathan Edwards was commissioned to design an arch bridge spanning 140 feet over the Taaf River in Wales. But Edwards accepted a contract under which he agreed to rebuild the structure at his own expense if it failed to stand for seven years. Unfortunately for him, an unduly severe freshet occurred after two and a half years and Edwards rebuilt the bridge in accordance with the terms of his contract. The second bridge failed during construction, but noting that the crown of the arch had moved upward to cause the collapse, Edwards made the indicated revisions in design and finally produced a lasting bridge. When, or even whether, Edwards ever received payment for his work is not recorded.

Another incident, if we may go back still farther in history, concerns a new bridge in Toledo, Spain, in 1390. When the arch spans were nearly completed on the erection falsework, the designer realized for some reason that the bridge would collapse when the centering was removed. Terrified, he rushed home and told all to his spouse. She rose nobly to the occasion and that night set fire to the falsework and ruined the arch. Her husband's second construction attempt was suc-

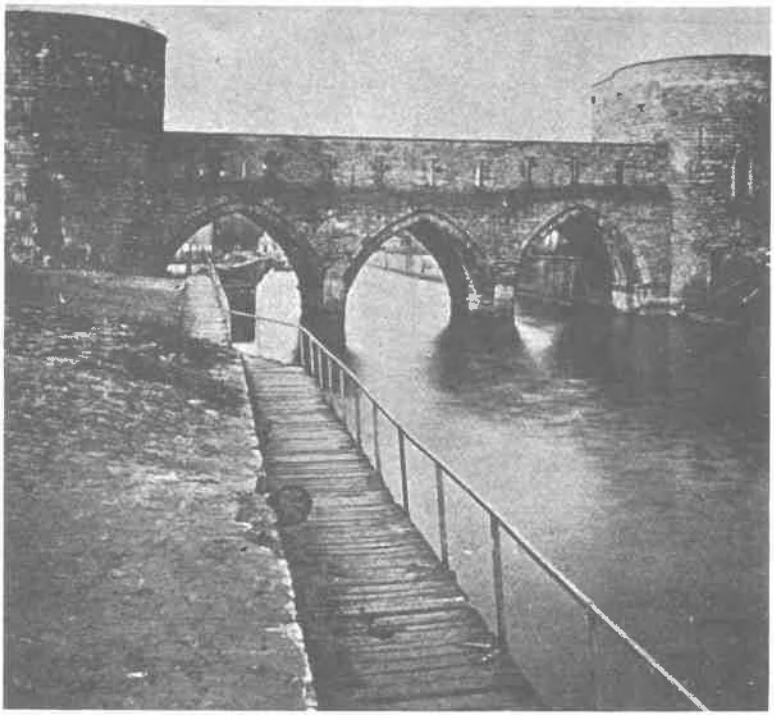

Fig. 4 Bridge at Tournai, Belgium. Note chapels at abutments.

cessful. But it is a known fact that on very rare occasions a wife cannot keep a secret, and the Archbishop under whose sponsorship the bridge had been built, heard the whole story. However, being a wise man, rather than reprimand the couple he congratulated the designer on having such a reliable spouse.

One of the earliest materials to be used was, of course, wood. Lumber was plentiful. One or two men with a saw and a pair of horses or oxen could snake out several logs a day, and the small local sawmill was very much a going industry. Wood was, in addition, easily worked and has been, until recently, comparatively cheap. It does have several disadvantages in that if untreated it rots readily when exposed to the weather, it is not particularly good in tension, and efficient end connections of tension members are not easily made. The development of treenails, or wooden pegs driven into pre-drilled holes in the members to be connected, went far toward the practical development of the wooden truss, and the covering of the trusses with shingled roof and sheathed sides practically eliminated the problem of rot. Indeed the wooden covered bridge became an inherent part of the early American countryside. Some were

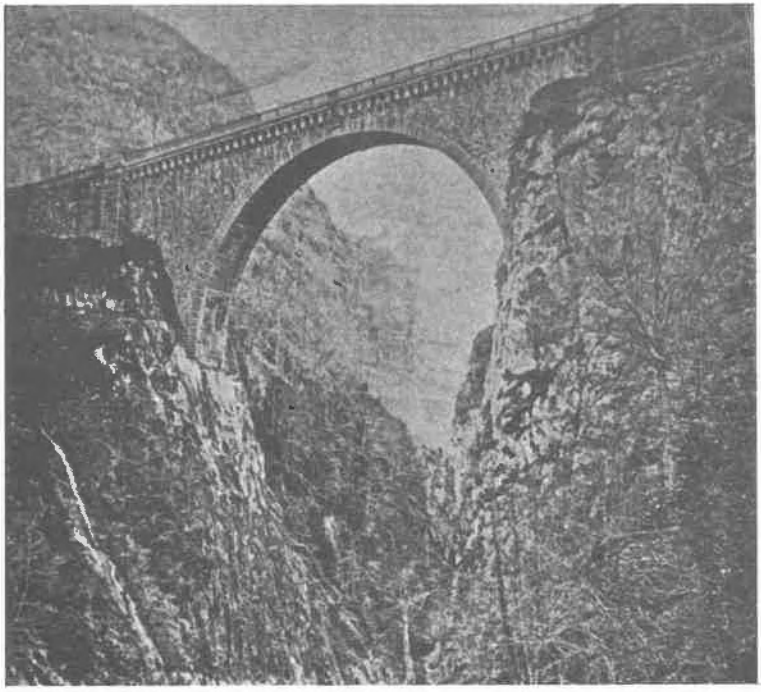

Fig. 5 Bridge of St. Sauveur in the French Alps

basically trusses, some were arches, and many were a combination of the two. While the exact distribution of load between arch and truss was never known, arch-truss combinations, known as Burr trusses, afforded a particularly stiff structure and lasted for many decades. Their obsolescence is the result of present day traffic requirements rather than their own decay, although during the last few years, partly due to the urging of the Society for the Preservation of Covered Bridges, the Commonwealth of Massachusetts has built such wooden covered bridges in Charlemont and Sheffield. But with increased standard for width and headroom, and with the virtual elimination of square crossings because of their inherently poor approach alignment, the modern bulky covered bridge is a far cry from the cozy little structure of nostalgic memory.

Wood structures are still used to some extent in the western part of this country and in such places as Alaska where the native timber is readily available. Preservation of the lumber is now obtained through creosoting or similar treatment, and the tension members, as in the past, are usually of metal.

The development of the manufacture of metals was of course a great

boon to the evolution of bridges. Cast iron was first manufactured in the fifteenth century, though not in great quantity. Being crystalline in texture it is weak in tension and in resistance to shock and impact. The first bridge of cast iron was built in 1776 over the Severn River in England, consisted of an arch of 100 feet and was, at least until recently, still in use. Many other structures were built of the material, but the percentage of failures, especially of railroad bridges, was unduly high. The major use of cast iron later was for compression members or as bearing blocks at the joints of trusses.

Wrought iron, being superior in tension and in resistance to shock, superseded cast iron and was the most common metal in bridges from about 1850 to 1890, although the puddling process for making it had been introduced in 1780. It was rolled into many shapes and there are still hundreds of bridges of wrought iron in existence, especially on roads in the country where traffic is not excessively heavy. In the last decade of the nineteenth century the use of wrought iron was largely discontinued when steel was produced in quantity.

Although many truss bridges had been built before 1847, little had been understood about the theory of their design, and the proportioning of the members had been done through a combination of experience and a sense of the general fitness of things. But in that year Squire Whipple, an instrument maker in Utica, New York, and somewhat of a philosopher, produced two treatises outlining the theory of stresses in trusses and giving plans and details for wooden and iron bridges. Four years later, although it does not appear that he had ever seen Squire Whipple's essays, one Hermann Haupt in Germany published his work on the same subject, but in a much less complete form. The way was now open for rational and more precise methods of analysis of trusses and designers availed themselves of the opportunity to create larger and bolder structures.

After the establishment of a rational method of analysis for truss design, it was only natural that there should appear many new types of trusses. Some were very practical and were in use for many years, while others, being highly theoretical, were soon forgotten. The parabolic truss, in which the stress in the straight chord is constant under uniform loading, became one of the more common types. So also did the truss commonly known as the lenticular or fish-belly, which has parabolic chords both top and bottom. This truss was built extensively in New England during the last quarter of the nineteenth century and several excellent examples still are in use on back roads where loads are not too heavy.

On the other hand, it was only three years after the publication of

Squire Whipple's treatise that the Bollman truss appeared. From the bottom of each vertical were two diagonals, of which one extended to each abutment. The large number of long diagonals and the difficulty in making the connections rendered the type unpopular, and deservedly so. Only one example of the type still remains. Its main advantage, i.e., that the stress in the chord was constant, could more readily and simply be realized with a parabolic truss. The Pauli truss, with two parabolic chords, was arranged so that the maximum stress in each chord was constant throughout its length. The slight saving in material was more than offset by the additional cost of fabrication and erection. The Schwedler truss was proportioned in such a way that the curved chord (the other chord being straight) would carry just as much of the shear as to eliminate any reversal of the stress in the diagonals and therefore obviated the need for counters.

It can be seen that contrary to what might have been expected, the first types of truss devised after Squire Whipple's treatise was published were rather complex and unwieldy, and that simplicity was obtained only after further development had been made. During this period there was a great difference of opinion among bridge engineers as to the relative merits of pinned and riveted connections. European engineers favored riveted joints because they provided a much more rigid structure, while American engineers preferred the pin connections because secondary stresses, in theory at least, were eliminated and the pins permitted much easier and faster erection of the structure. Indeed a pin-connected bridge about 160 feet long has been erected in this country in what is probably the record time of eight and one half hours. Because of the development of high strength steels in recent years, trusses are less common than formerly, but when they are used riveted joints are almost universally chosen.

It is generally supposed that the suspension bridge is strictly a modern invention. It is surprising to find, then, that the first record of a structure of such type had to do with one in China in 1667. The first suspension bridge built in this country had a span of only 70 feet, having been built in 1796. Within the next fifteen years or so, some forty suspension structures were in existence, with a maximum span of 306 feet. Taking into consideration the limited knowledge of structural theory available at the time, it must be concluded that these structures were daring indeed.

One suspension bridge built by Theodore Burr at Schenectady in 1808 was unique since the curved suspension members were fabricated of wood. There were three of these members, each made up of eight 4 in. by 14 in. white pine planks bolted or spiked at the joints. Some

twenty years later it was found necessary to build auxiliary pier supports under the center of the spans in order to arrest a rather serious sagging, but the structure remained serviceable until 1873, when it was completely replaced.

Only two years after the building of the Schenectady structure a chain suspension bridge with a span of 244 feet was erected across the Merrimack River three miles above Newburyport in Massachusetts. There were two separate roadways and therefore three chains. The individual links were made of forged iron two feet long and one inch in diameter. Each chain, as it went over the supporting towers, was spliced into three smaller chains. From time to time various links were replaced with others of odd sizes. One of the chains broke in 1827 and the structure was rebuilt and lasted until 1907. It was said, however, that the wrought iron in the original chains, unpainted for seventy years, was still in excellent condition.

In 1847 a pedestrian suspension bridge over 1000 feet long was built across the Ohio River at Wheeling, West Virginia. Some fourteen years later a violent windstorm caused it to turn turtle, and as a result there was instituted a study of aerodynamics, stiffening trusses and wind bracing. Further and more advanced study recently had to be made in this area after the failure of the Tacoma Narrows bridge.

Shortly after 1850 there was found to be a need for two unusually large structures. The first, at Niagara, was to carry two decks, with railway traffic on the lower level. The second, a bridge at Montreal, was to be more or less of the same size and for the same purpose. Engineering opinion was greatly divided as to the superiority of one proposed type over the other. John A. Roebling designed and built a suspension bridge for the Niagara site while Robert Stephenson, whose name was equally well known, proposed a tubular bridge at Montreal, with trains going through the tube. The two structures were radically different in concept, but their general histories were remarkably alike. Each lasted about forty-three years, each was succeeded by a heavier bridge to take care of increased traffic demands, and neither was replaced in kind.

One of the most notable of all suspension types is the Brooklyn Bridge, completed in 1883. The original structure was found to be slightly lacking in stiffness and auxiliary stays extending out from the towers were added at a later date. The Brooklyn Bridge marked the first use of galvanized drawn steel wire. During the next two decades or so, several other suspension spans were erected in or near New York. The earliest of these were supported on masonry towers that were strictly practical but of poor appearance. For the design of the Manhattan Bridge an eminent firm of architects was engaged, and for the

first time slender and attractive steel suspension towers were evolved.

One of the most daring engineering feats of this century was the building of the George Washington Bridge since the length of its main span more than doubled that of any predecessor. Up until that time most of the suspension bridges had been built in the vicinity of New York, and for a rather good reason. The cost of such structures is tremendous and their use is justified only in heavily-populated centers where heavy traffic is almost continuous. Recently large suspension bridges have been built on the west coast, notably around San Francisco, but the record for span length rests with the Verrazano-Narrows bridge at the mouth of New York Harbor.

Fig. 6 Bridge over Bixby Creek in California. A slender and attractive concrete arch.

The intended life of a bridge should be measured in decades rather than in years, and since its life expectancy is so great, it should be pleasing to the eye as well as utilitarian. As we have seen, the first bridges were arches, a form conducive to attractive appearance. The early artisans took great pride in their work and generally spared no pains to produce a handsome structure. For that reason early structures that are still extant have a value to our civilization far more important than merely their roots in history. They are a part of and fit into the landscape itself.

With the coming of the steel age and the greater understanding of structural theory, ugly bridges, particularly truss and girder types, proliferated throughout the country. The rapidly spreading railroads were

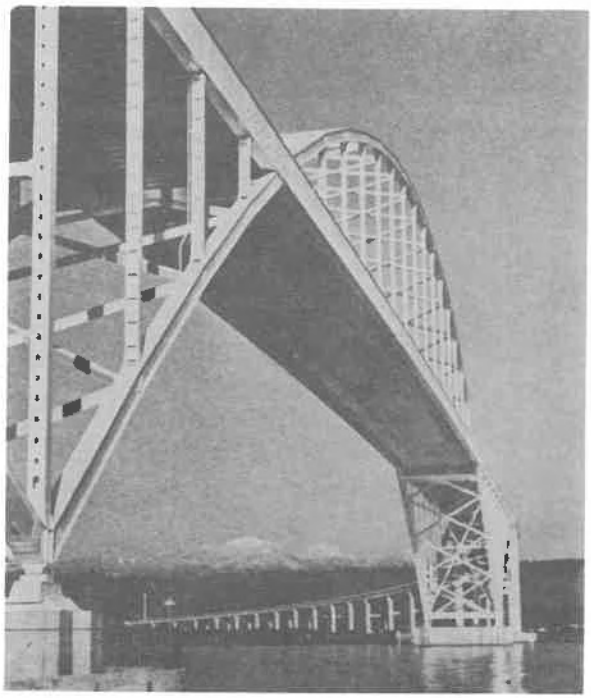

Fig. 7 Modern Orthotropic Steel Bridge in British Columbia. Courtesy of United States Steel Corporation.

prime offenders in this respect since, being profit-making organizations, they insisted on building the least expensive types and the cheapest was seldom the most pleasing. Although many well-proportioned bridges have been recently built, it is an unfortunate fact that a bridge design, except the larger or monumental ones, is still selected far more for reasons of economy than for appearance. Many bridges constructed recently to standardized architectural concepts would appear to have been designed by the mile and cut off in the required lengths. Not only have we perpetuated unattractive bridges, but we have alienated many a young engineer because of the extreme monotony of designing continually the same structure type. It sometimes appear that in this country we alternate between extremes rather than follow the middle of the road, however the pendulum now would appear to be on the downswing. There seems to be dawning an awareness that we should re-assess the value of aesthetics in our structures and emphasize appearance at the risk of slight loss of economy.

And what of the future of bridges and bridge engineering? What

new types will be evolved? What will be the general trends? Obviously the engineer would be most rash even to guess. But some things are known. Successive generations in history have reviewed all the events of their immediate past and have concluded that so rapid has been the advance of knowledge and the arts during their lifetime, surely the end must have been reached. Surely man had discovered everything there was to be learned. And yet the following generation has never failed to advance knowledge even at a far greater pace than has previously been thought possible. The development of bridges and bridge theory is an example. Whenever the engineer has had to stretch out into hitherto unexplored fields, he has been able, through theory and research, to develop new concepts and new standards to meet his specific design requirements. Recent extensions and refinements of aerodynamic theory are a case in point, or when foundations had to be made in deep waters, the pneumatic caisson was evolved.

When the Brooklyn Bridge, with its main span of nearly 1600 feet, was completed over eighty years ago, it was rightly regarded as an engineering marvel. Yet such has been the ensuing increase in structural theory and knowledge that it is now dwarfed by the suspension bridge across the Narrows at the entrance to New York Harbor. There is now contemplated another such bridge in Japan with a main span of a little less than a mile.

With the continuing development of lighter and stronger materials and of research methods, this writer would hazard the guess that future development of bridge types will be limited only by the far horizons of the electronic computer and of that even more remarkable machine, the human mind.

March 1, 1967 – The annual meeting of the Sanitary Section was held in the society rooms and was called to order at 7:00 P.M. by Chairman Robert L. Meserve. The reading of the minutes of the previous meeting were waived. The report of the nominating committee was presented and the following officers were elected for the 1967-68 year:

| | |
|---|---|
| Chairman ..... | Walter M. Newman |
| Vice-Chairman ..... | Charles A. Parthum |
| Clerk ..... | David A. Duncan |
| Executive Committee .. | Allison C. Hayes |
| | Leland F. Carter |
| | Cornelius O'Leary |

Chairman Robert L. Meserve then introduced Mr. David Standley, Associate Air Pollution Control Engineer of the Massachusetts Department of Public Health, who presented a very interesting illustrated lecture entitled "The Status of Air Pollution in Boston".

Mr. Standley described in detail the Metropolitan Area Pollution Control District which involves about 400 square miles and includes about two million people in the Boston area. He reviewed the present laws and regulations pertaining to air pollution and described the detailed survey that was made over the past year and a half by the Massachusetts Public Health Department. Colored slides were shown of the Thanksgiving Week-1966 inversion effect on the Boston area. These slides were taken from an airplane and showed clearly the distinct line of air pollution in the Boston area. Mr. Standley also described the problems connected with air pollution in Massachusetts and attributed much of the air pollution to the burning of fossil fuels. A lively and lengthy question and answer followed the presentation. About 35 members attended the meeting. Adjournment was at 8:45 P.M.

Charles A. Parthum
for David A. Duncan, Clerk

A regular meeting of the Structural Section was held on the evening of April 12, 1967 in Room 1-270 at M.I.T. and was called to order by the Chairman, Charles Ladd, at 7:05 P.M. The Chairman introduced the speaker, Dr. William A. Litle, Associate Professor of Civil Engineering at M.I.T., who spoke on "The Use of Structural Models."

Professor Litle first discussed the role of experiments in structural design, the usefulness of small scale models, and the reliability of these models. The term "small" refers to "table top" size, or 1/8 to 1/28 scale. He then

presented results from three types of models; a space frame, a reinforced concrete slab and buckling of shells.

After a brief question and answer period, the audience was given a tour of the Civil Engineering Structural Models Laboratory at M.I.T. where three projects were described in detail by graduate students. The meeting adjourned about 9:15. Attendance was 41.

Respectively submitted,
C. C. Ladd for
Albert B. Rich, Clerk

April 20, 1967 – A regular meeting of the Boston Society of Civil Engineers was held this evening at the United Community Services Building, 14 Somerset Street, Boston, Mass., and was called to order by President Harry L. Kinsel, at 7:00 P.M.

President Kinsel stated that the minutes of the previous meeting held March 14, 1967 would be published in a forthcoming issue of the Journal, and that the reading of those minutes would be waived unless there was objection.

President Kinsel announced the death of the following members:-

John C. Damon, elected a member February 15, 1938, who died March 8, 1967.

Stuart Huckins, elected a member March 17, 1937, who died March 11, 1967.

Robert M. Becker, elected a member February 16, 1938, who died March 23, 1967.

Clarence R. Bliss, elected a member February 16, 1927, who died March 30, 1967.

The secretary announced the names of applicants for membership and that the following had been elected to membership March 13, 1967:

Grade of Member - Joseph J. Allegro*, John R. Christian, Robert E. Homer

Grade of Junior - John E. Kavanagh, 3rd†,, Dean K. White†

President Kinsel then introduced the speakers of the evening, Prof. Myle J. Holley, Jr., Prof. Allin Cornell of M.I.T., and Mr. John B. Creedon, Liberty Mutual Insurance Company, who gave an interesting talk on "Some Aspects of Structural Safety". The talk was illustrated with slides. A question period followed the talk. Twenty seven members and guests attended the meeting. The Meeting adjourned at 9:00 P.M.

*Trans. from Junior
†Trans. from Student

Respectfully submitted,
Charles O. Baird, Jr., Secretary

On Wednesday, April 26, 1967, a meeting of the B.S.C.E. Transportation Section was held at 47 Winter Street, Boston, Massachusetts. Robert Kiley opened the meeting at 7:05 p.m. as a substitute for Chairman, Casimir J. Kray. Minutes of the previous meeting were read and accepted. Mr. Kiley then read the slate of officers proposed for election for the 1967-68 year as follows:

| | |
|---|---|
| Chairman ..... | Charles Flavin |
| Vice Chairman ..... | James Orpin |
| Clerk ..... | Maurice Freedman |
| Executive Committee Members .. | Charles Shaker |
| | Prof. Alexander J. Bone |
| | Louis A. Forti, Jr. |

The entire slate was then voted upon and accepted.

The meeting was then turned over to Charles Flavin who introduced the newly elected officers and committee members.

Mr. Flavin then introduced the speaker for the evening, Mr. John A. Swanson, Regional Engineer, Bureau of Public Roads, whose talk was entitled "Project Design for Safety." Mr. Swanson gave us the benefit of his vast experience relative to those elements of design which could make a significant contribution to highway safety. Such things as improved guard rails, flatter slopes, break-away sign supports, and adequate lateral clearance adjacent to highways were discussed. Mr. Swanson concluded his remarks with the statement that highway safety could not be achieved unless high regard was given to the four "E's":

Education
Enforcement
Engineering
Equipment

The meeting was concluded at approximately 8:20 p.m.

Respectfully submitted,
Maurice Freedman, Clerk

## NEW MEMBERS

Clifford W. Bowers, 22 Pomeroy St., N. Reading, Mass. 01864
John T. Christian, 51 Kingsbury St., Wellesley, Mass. 02181
Robert F. Daylor, 466 Middle St., Braintree, Mass. 02184
Hans Gerber, 785 Salisbury St., Holden, Mass. 01520
Dennis J. Leary, Woodward, Clyde, Sheard Assoc. 1425 Broad St.
Clifron, New Jersey 07013
Robert E. Moffat, 9 Wedgemere Ave., Winchester, Mass. 02189
Thomas J. Quinn, Jr., 26 Kensington St., Newtonville, Mass. 02160
Herrick H. Spicer, 27 Highland Ave., Dedham, Mass. 02026
Arthur J. Towne, 15 Oakland St., Salem, Mass.. 01970
Mircea S. Vasiliscu, 112 Hammond Road, Belmont, Mass. 02178

## NEW JUNIORS

Edward B. Fitzpatrick, 74 Prentiss St., Watertown, Mass. 02172
William L. Harris, 6 Main St., Durham, New Hampshire 03824
John C. Kavanagh, 3rd., 97 Lowell Ave., Newtonville, Mass. 02160
Donald B. Nichols, 47 Freeman St., Quincy, Mass. 02169
Dean K. White, 16 Hackfeld Road, Worcester, Mass. 01609

## Deaths

Robert M. Becker, March 23, 1967
Clarence R. Bliss, March 30, 1967
John C. Damon, March 8, 1967
Stuart Huckins, March 11, 1967

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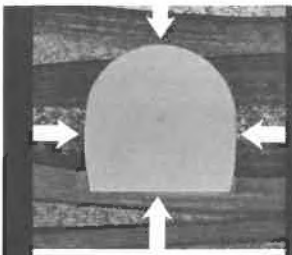

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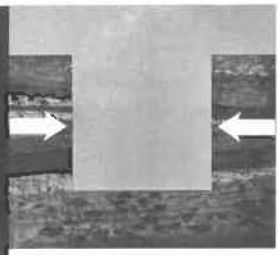

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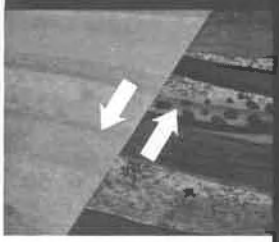

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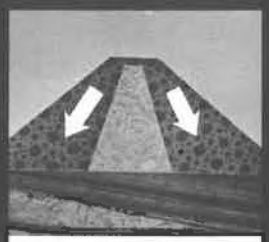

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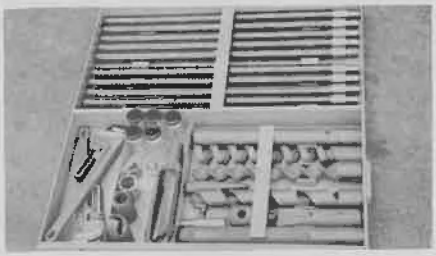

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