Spring 1986

DEEP FOUNDATIONS JOURNAL

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

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Dewatering Practice For Deep Basements In Hawaii

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A brief discussion of the methodologies involved in dewatering practices, particularly as used in Hawaii in coralline formations. A description of early dewatering practices includes a number of case histories; later dewatering practices also are detailed with case histories. There were disasters and some successes. Grouting generally was not successful. Deep wells are technically feasible and generally the economical solution to deep basement construction.

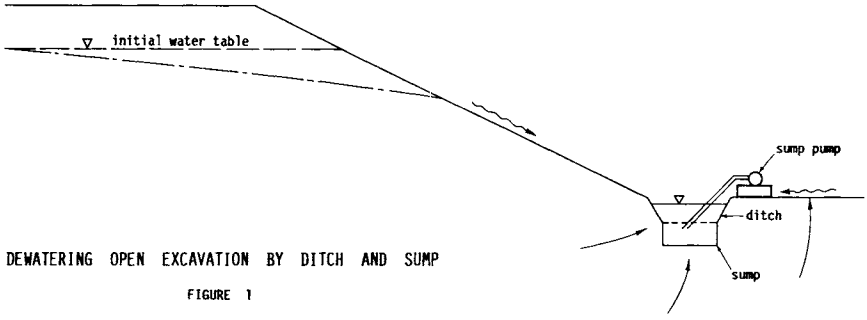
The proximity to sea level and the relatively permeable coralline and alluvial settlements underlying the coastal plains of Oahu have discouraged deep basement excavations. Prior to 1981, basements deeper than two stories, other than a few deep pump station excavations, were not constructed. A general rule of thumb regarding dewatering was " don't go below the water table in coral."

However, increasing land values, building code parking requirements for planned developments, and increasing demand for open space at ground level have made deep basement excavations for parking more attractive.

Methodologies that have been applied in Hawaii include sumps and ditches, cutoffs, and wells.

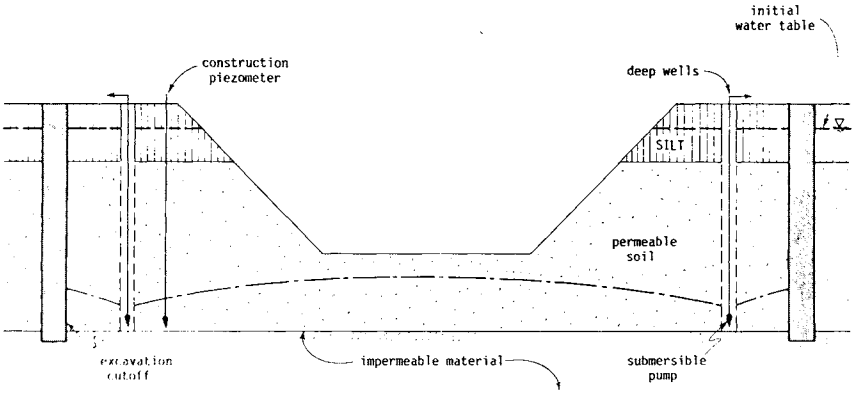
Sumps and ditches (Fig. 1) are generally used to collect water entering an excavation or structure. They generally can lower the water level only a few feet and are used to collect water from within sheet pile enclosures and similar types of excavations.

Cutoffs (Fig. 2) can stop or minimize seepage into an excavation when they are installed down to an impervious stratum. The most widely used cutoff in Hawaii is sheet piling although there are at least two (relatively unsuccessful) case histories of grouting. Difficulties that have been encountered with cutoffs include unsuitable material for sheet piling installation, lack of an underlying impervious stratum, and unsuitable material for grout-



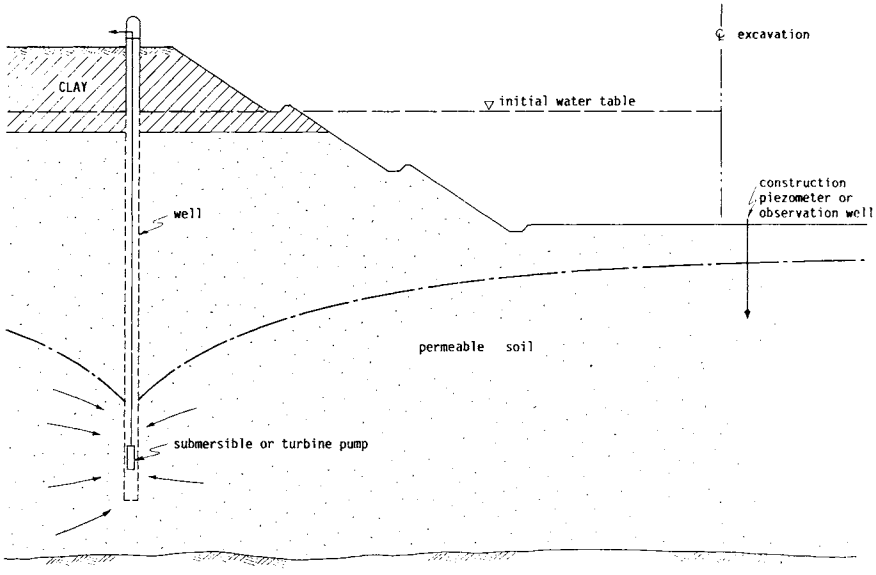
DEWATERING OPEN EXCAVATION BY DITCH AND SUMP

FIGURE 1



CUTOFF AROUND AN EXCAVATION

FIGURE 2



DEEP-WELL SYSTEM FOR DEWATERING AN EXCAVATION

FIGURE 3

ing. Sheet piling cutoffs have often been beneficial when the primary use has been for excavation shoring or bracing, with seepage reduction a secondary use.

Deep well systems (Fig. 3) are used to dewater soils that can be drained by gravity flow. They are usually for large deep excavations when large flows are expected. However, in Hawaii, because of the highly permeable characteristics of the soils, they are also useful and economical for relatively shallow excavations. An advantage is that they can be installed around the periphery of the excavations thus removing dewatering equipment from within the excavation.

Other common dewatering methods have not been widely applied in Hawaii. Well points are probably the most commonly used dewatering method throughout the United States, but have been applied to few projects in Hawaii, primarily due to the high permeability of permeable soils in Hawaii. Slurry wall cutoffs, vertical sand drains, and electro-osmosis have not been used, to our knowledge.

EARLY DEWATERING PRACTICES

Early dewatering practices in Hawaii generally consisted of open pumping with sumps and ditches with a few instances of the use of deep wells. Definitive data on pumping qualitites are not readily available, but the few instances of deep well dewatering appeared to be more successful with less pumping per foot of required dewatering. There were a few bright spots, but there were also a number of notable dewatering failures. Each of these dewatering problem case histories provides an insight into possible dewatering problems in Hawaii.

The first case history was described by J. Patrick Powers in his book on construction dewatering (Powers, 1981). At an unnamed pump station in Honolulu, open sump and ditch pumping at 10,000 gallons per minute (gpm) failed by only a few feet to achieve the necessary drawdown of 30 ft. More pumps were ordered; but as pumping continued, the sand within the voids in the coral were gradually eroded and washed into the excavation to the extent that the pumping requirement increased to 30,000 gpm.

This incident points out one of the major advantages of deep well pumping over open sump and ditch pumping within coralline materials. Deep well pumping does not tend to increase the transmissivity of the coralline materials in the vicinity, as fines are generally not removed with the pumping. Deep well pumping also makes it much easier to dispose of the water because it is not cloudy and is not likely to attract the attention of environmentalists. The clean water also does not silt up and clog storm drains.

The second case history is a proposed high-rise on the corner of Ward and Kapiolani in the Kakaako area of Honolulu. At this site two permeable zones are separated by a thin, relatively impermeable alluvial layer. Construction of a high-rise building with a basement was attempted. Dewatering with the use of open sumps and ditches proved unsuccessful.

Analysis would have indicated that dewatering or depressurizing of the lower permeable zone was necessary to assure stability of the basement excavation in this case. However, this apparently was not realized and, as a result, heaving and boiling of the excavation bottom was encountered. The basement excavation was filled in and a low-rise structure was constructed. In this case, depressurizing of the lower layer was feasible with relatively limited deep well pumping.

The Federal Office Building in downtown Honolulu is a classic example of how not to dewater a site. Approximately $18 million worth of construction claims were filed by Haas and Haynie, the contractor. Most of these claims were directly or indirectly related to the failure to properly dewater the site. The claims were settled by the General Services Administration (GSA) for $9 million; Haas and Haynie went into bankruptcy. The unfortunate thing was that dewatering of the site was a problem in terms of scale, but does not appear to have been an unfeasible technical problem. The conclusion of GSA's outside geotechnical consultant (McClelland Engineers, 1976) was that dewatering of the site was technically feasible and proper dewatering of the site would have prevented most of the problems that occurred.

Examination of the construction problems at the Federal Office Building illustrates how poor dewatering practice can affect all phases of the foundation construction process—for the worse. Inadequate dewatering was extremely detrimental to Haas and Haynie in terms of job progress, time, and cost. Problems in excavation, pile driving, and placement of steel and concrete can be attributed to dewatering problems. Because the soils were not predrained, the attendant difficulties of excavating "under water" were encountered.

As a result of using open sump pumping and allowing water to flow into the excavation, the water was very cloudy and Haas and Haynie was denied permission to directly discharge into the large storm sewers in the vicinity without the use of sedimentation basins.

During pile driving, claiming that dewatering of the site was "impossible," Haas and Haynie drove piles from a higher elevation than necessary. This resulted in a large number of piles being misplaced at the foundation level, sometimes requiring replacement or additional piling. The use of a follower was also made necessary by this action.

Placement of steel and concrete for pads and pile caps was made extremely difficult by the flowing water. A number of pads required repair as concrete was damaged by flowing ground water during construction and were rejected by GSA.

It was pointed out that a significant advantage of predrainage by deep wells is the ease with which excavation work can proceed. Excavated materials are lighter as they are not saturated with water, are easier to handle because of their dry condition, and are more readily reused as structural backfill.

LATER DEWATERING PRACTICE

Later dewatering practice introduced the use of investigations and tests specifically for dewatering and the use of preliminary dewatering system designs. In such cases dewatering was more successful, although there still continued to be examples of poor dewatering practices.

A later case history, is the Ewa Beach pump station. Although Dames & Moore was not directly involved, site visits were made due to our technical interest in the grouting being attempted by the contractor. From previous experiments with grouting in coral there was general conclusion that natural coralline formations were not particularly suitable for grouting. The relative heterogeneity of coralline formations would make grouting very difficult to control and the formation of a uniform and relatively watertight grout curtain would be very difficult to achieve. A second requirement for an effective cutoff is an impermeable base. Such a base is often either not present, is discontinuous, or is too thin, in which case dewatering of the permeable material underlying the base would be required in order to achieve excavation stability.

Based on discussions with construction personnel at the site and visual observations, it appeared that grouting at the Ewa Beach pump station had not been successful. Although grouting at about 3 ft intervals was eventually used, it was clear that an effective cutoff even within the grouted zone had not been achieved. Water entering the excavation from the sides of the excavation was clearly visible.

Also, the grouting did not extend to an impermeable base. Analysis would have indicated that grouting without an impermeable base could, even at maximum efficiency, only cut off 20 percent of the flow. Dewatering eventually was achieved at this site through massive quantities of open pumping, estimated at up to 30,000 to 35,000 gpm. The case history, however, is useful in confirming our expectations that grouting of natural coral formations generally does not create an effective cutoff.

The next case history is an example of a successful deep well dewatering effort. Century Square, in downtown Honolulu, consisted of a 37-story tower, and a four-story underground parking structure. The underground parking was 30,000 sq ft; the bottom two stories were under water. A dewatering investigation was conducted, a deep well dewatering system was designed and implemented, and the results were highly satisfactory (Fujioka and Yao, 1982).

The effects of a successful dewatering system on a construction project can be dramatic. The successful deep well system resulted in much time and cost savings as well as reducing the impact of the construction on the environment.

Specific effects included:

1. The dewatering system was entirely on the perimeter and did not result in obstructions within the excavation. Sumps, hoses, and pumps often found within a dewatered excavation were not necessary.

2. There were no delays to the excavation and construction work once the men and equipment were mobilized. Other forms of dewatering, particularly sump pumping, often result in periodic delays when water must be removed as the construction proceeds.

3. All excavated materials were dry, resulting in much easier digging, scraping, loading, and general handling of the material.

4. The excavation and the vicinity were kept clean because of the dry conditions. Despite heavy vehicular traffic into and out of the excavation, the adjacent streets (a busy downtown area) required minimal cleanup.

5. The pumped water was clean and could be directly disposed of into the storm drain system. The clean brackish water had no impact on the storm drain system or on the Honolulu Harbor waters, which eventually received disposal of the brackish water. A desilting basin was not required (there was no area of the site that could have been used for such a basin).

6. The dry conditions within the excavation resulted in efficient steel placement, forms construction, and concrete placing.

7. The waterproofing system, consisting of bentonite panels placed between the basement walls and the gunnited excavation face, could only have been constructed under dry conditions. The waterproofing system was very inexpensive and quickly installed. Alternative waterproofing systems, installable under less ideal conditions, would have been more expensive and would have required more time to install.

Two other projects, Executive Center and Bishop Square, were constructed in the downtown area shortly after Century Square. These projects utilized deep wells for dewatering of relatively shallow excavations. These projects are examples of the utility of deep wells for relatively shallow excavations due to the high permeability of the coral, which results in relatively flat drawdown curves. A few high-capacity wells and pumps can efficiently dewater even large excavations to shallow depths.

CONCLUSIONS

A review of the case histories presented in this paper results in the following general conclusions:

- The most common dewatering practices in Hawaii have been perimeter deep wells and open sump pumping.
- Open sumps and ditches have been fairly successful on some projects and have resulted in notable dewatering failures on others.
- The use of a properly designed and executed deep well system has generally resulted in lower pumping quantities with substantial construction time and cost savings.
- Despite limited actual experience in grouting, it would appear that the method is not generally applicable to natural coralline materials in Hawaii.
- For the general complexity of subsurface conditions encountered in Hawaii, particularly in coralline and alluvial deposits, deep well dewatering

is, in general, technically feasible. Considering overall construction costs, it is usually a more economically feasible approach to deep basement construction.

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Allowable Compressive Design Stresses For Pressure-Treated Round Timber Foundation Piling

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**Presented before the American Wood-Preservers' Association
Reno, Nevada, May 1985**

Research at the University of Colorado on southern pine and Douglas-fir piling has determined that the existing building code design stress values of 1200 psi and 1250 psi, as determined by ASTM D-2899, are substantiated; but testing procedures should be revised. All full size piling tests should be standardized at 1/d ratios of 2:1, or confined tests performed. Also, it has been determined that theoretical design values for timber piling should not be based on the lumber criteria of D-245 and D-2555, but instead should follow the pattern set by engineers in the utility industry for timber poles, in accordance with ANSI 05.1.

Untreated timber foundation piling has been used for centuries in areas where the wood was below the ground water level not subject to marine borer attack. Pressure treated timber foundation piling was developed some 100 years ago for use in areas where the wood extended above the ground water table.

Most treated timber piling was installed with the knowledge that 1200 psi could be used for design to support the vertical compressive loads. There

AASHTO	American Assoc. of State Highway and Transportation Officals
ANSI	American National Standards Institute
ASTM	American Society for Testing & Materials
BOCA	Building Officials Conference of America
SBC	Southern Building Code
UBC	Uniform Building Code

TABLE 1

Southern Pine

	Length	Weight	Rings/ Inch	Spec. Grav.	Moisture Content	Creosote Content	Diameter Max.	Diameter Min.	Knots[a] KS/D	Knots[a] KL/D	Ultimate Strength
1	36.0"	64 lbs	7	.53	50%	15 pcf	8.5"	8.4"	--	--	2720 psi
2	36.5	74	7	--	--	--	8.1	8.1	.35	.16	2460
3	36.0	94	9	.60	46	13	9.8	9.4	.16	.09	3400
4	35.9	89	5	.49	56	14	9.5	8.4	.53	.15	2240
5	35.8	58	8	.50	61	18	8.9	8.3	.19	.19	2680
6	36.0	89	10	.57	47	12	9.7	9.5	--	--	2910
7	35.4	84	8	.57	43	17	9.2	9.1	.03	.03	2610
8	35.7	92	8	.61	51	11	9.7	9.5	.05	.05	2730
9	36.0	85	8	.63	42	12	9.2	9.0	--	--	2420
10	36.0	78	7	.66	48	10	8.8	8.5	.18	.11	3040
11	36.3	56	9	.52	62	14	7.6	7.3	.40	.15	3040
12	36.1	54	10	.66	40	13	7.4	7.2	.27	.10	3000
13	36.3	54	13	.60	38	13	8.0	7.0	.33	.11	3830
14	36.3	54	7	.52	80	22	7.5	7.1	--	--	2910
15	36.1	53	7	.64	51	10	7.4	7.4	.24	.18	2740
16	36.0	49	10	.59	35	14	7.2	7.0	.30	.19	2840
17	35.1	49	10	.62	49	10	7.5	7.1	.39	.17	2760
18	35.6	53	12	.72	31	18	7.3	7.0	.07	.07	2850
19	35.8	65	9	.54	60	8	8.0	7.8	.33	.13	2290
20	36.0	51	13	.66	33	13	7.3	7.2	.13	.09	3310

Average = 2839 psi
Standard Deviation = 385

a. Knots
KS = Sum of knot diameter/ft.
KL = Largest knot diameter
D = Diameter where knot(s) located

Coefficient of Variation = 366/2839 = 13.6
Variability Factor = 100 - 13.6/2 = 93.2%

have been no failures of the creosoted timber foundation piling material using the new psi design value.

In the late 1960's, the wood treating industry observed the engineering community was not taking into consideration all of the potential economic advantages that timber piling had to offer. Specifications were calling for either Class A or Class B piling that limited the supply to just two sizes when in fact the material grew and was available in a much broader range of sizes.

It was also observed that various soil conditions provided an opportunity for the engineer to specify the size of timber piling in accordance with these conditions. In the case of a friction pile, the butt size is more critical and, therefore, more piles with smaller minimum tips could be utilized.

In the case of a point bearing pile, the tip size is more critical and, therefore, more piles with smaller minimum butts could be utilized. Also, where various projects and soil conditions provide design loads in a range from 10 to 50 tons, there should normally be an opportunity to specify various minimum size requirements for the piling material itself, based upon the most economical minimum size need for the design load required.

With this thinking, ASTM* Committee D-7 revised the ASTM D-25 Standard Specification for Round Timber Piles and incorporated within this specification a means of specifying the size piling needed, as shown in Table 1 or Table 2. In conjunction with this, there was also a need to tell the engineer how to establish the design stresses for the various size timber piles, and ASTM Committee D-7 adopted and published the method for doing this in ASTM Specification D-2899 in 1970.

In recent years these design values have been criticized by several people. The criticism originated from those directly representing competitive products. The criticism was made even though D-2899 design stresses for Southern Pine and Douglas-fir were in line with the 1200 psi empirical values previously used by engineers. In defense of their criticism, they were not aware of the differences in strength considerations of round versus sawn material.

The purpose of this paper is to substantiate the use of the "bottom-line" design stresses of 1200 psi in D-2899, and recommend certain changes in the derivation of the theoretical equations. Research at the University of Colorado has disclosed prior testing errors, and industry surveys have uncovered production procedures that were not privy to the original developers of D-2899.

History of Timber Foundation Piles

Prehistoric man knew that timber piles had to be driven into the ground below the water table. The first use of timber piles is credited to a group of Swiss Lake Cultures who migrated to Central Europe and, for protection, built homes out over lakes. The homes were supported on timber pile foundations that are reported to still be in good condition. Other civilizations have followed the same pattern; for example, Venice, built about 1000 A.D.,

TABLE 2

Douglas-Fir

	Length	Weight	Rings/ Inch	Spec. Grav.	Moisture Content	Creosote Content	Diameter Max.	Diameter Min.	Knots KS/D	Knots KL/D	Ultimate Strength
21	36.0"	69 lbs	14	.45	27%	15pcf	9.7"	9.4"	.34	.10	3000 psi
22	35.6	59	12	.48	35	10	9.4	8.9	.05	.05	4120
23	35.3	73	12	.44	19	28	9.6	9.4	.16	.16	3350
24	35.6	58	12	.43	40	7	9.5	9.2	.53	.14	3160
25	35.9	86	12	.43	28	27	11.2	11.0	.11	.06	2760
26	35.8	83	11	.48	67	13	10.4	10.2	.39	.06	3070
27	35.8	90	13	.42	6	26	10.4	10.1	.39	.10	3380
28	35.8	79	6	.43	26	16	11.2	10.9	.41	.10	2710
29	35.0	64	16	.38	32	33	8.8	8.6	.12	.12	2380
30	35.5	80	12	.44	28	13	9.8	9.5	.26	.10	2420
31	35.5	46	14	.42	50	1	8.8	8.4	.35	.12	2220
32	34.8	58	11	.41	32	17	9.0	8.8	.31	.08	3060
33	35.0	55	7	.38	62	13	8.9	8.7	.70	.23	2480
34	35.3	51	8	.33	52	38	8.4	8.0	.22	.13	2140
35	35.1	58	5	.39	47	16	9.0	8.6	.44	.17	2700
36	35.0	46	7	.42	38	11	8.6	8.3	--	--	2980
37	35.0	63	11	.43	28	19	8.4	8.9	.26	.14	2960
38	35.1	71	10	.41	23	30	9.6	9.5	.67	.16	2720
39	35.3	58	9	.37	42	14	9.1	8.7	.23	.17	2920
40	35.0	50	10	.39	27	16	8.6	8.4	.77	.15	2580

Average = 2856 psi

Standard Deviation = 426

Coefficient of Variation = 426/2856 = 14.9%

Variability Factor = 100 - 14.9/2 = 92.53%

was constructed on timber piles driven into mud flats in the North Adriatic Sea (Haldeman, 1981).

Many important structures throughout the world are supported by timber pile foundations. This includes the following: London Bridge; Campanile Tower and Rialto Bridge in Venice; Pont Nortre-Dame Bridge in Paris; Royal Palace in Amsterdam; Berlin Castle and Opera House; Washington D.C. post office, etc. Even the 100-year-old Brooklyn Bridge is supported by a timber cribbage foundation that is in excellent shape where always kept wet.

Ancient practice limited the use of timber piling to conditions in which there was complete submergence of the pile under the permanent ground water level. There were exceptions, however. Around 600 B.C., during the Greek Civilization, the Temple of Diana in Ephesus was built in what is today Western Turkey. Charred timber piles were used in marshy soils. The Romans also used charred timber piles for bridge construction; however, the major use of timber piles was generally limited to those sites where the piles were completely submerged.

About 1880, the railroads in the United States started using pressure creosoted treated timber piles above the water line and with great success. As reported some 40 years later to the U.S. Department of Commerce (U.S. Department of Commerce, 1931), railroad engineers found it was practically impossible for the preservative to leach out of the underground portions of the piles and, so far as could be determined, piles embedded in concrete caps were permanently preserved. (Credit the Louisville and Nashville R.R. for first using creosoted timber piles.)

Since 1880, many thousands of structures have been constructed in the United States on pressure treated creosoted timber foundation piles. Except where soil settlement dragged-down the piles (and all pile types would fail), there has been no record of treated timber pile failures by decay or overstressing.

One example of a successful timber pile job was the Santa Fe Railroad's grain elevator, constructed in 1905 near Kansas City, where over 23,000 creosoted piles were used for a heavily loaded structure. Other projects are: Union Passenger Terminal Station in Cincinnati; Dauphin Island Parkway and I-10 Interchange, Mobile, Alabama; U. S. Coast Guard, Mess Hall, Alameda, California; Sports Arena, San Diego; Coliseum, Oakland, California; Frozen Foods Warehouse, Georgia Port Authority, Savannah, Georgia; Freemont Interchange Viaduct, Portland, Oregon; State Highway Department Building, Olympia, Washington; Channel 2 TV Tower, Baton Rouge, Louisiana; Fairmont Bridge, Minnesota; Ideal Toy, Newark, New Jersey; Food Center, Philadelphia, Pennsylvania; Sears Parking Garage, Memphis, Tennessee.

Some 500,000 timber foundation piles are used per year throughout the United States. One recent job of considerable significance concerning allowable design stress is the viaduct at Winnemuca, Nevada, constructed in 1978 with 70 ton design loads (Goble, 1981).

Allowable Design Stresses for Timber Piling

The most often used design value for timber piles is 1200 psi in compression parallel to grain. The 1200 psi was probably an empirical value developed by experienced engineers. Older editions of the U. S. Department of Agriculture, Handbook No. 72, *Wood Handbook*, page 483 (USDA, 1948), for piles *end bearing* on solid stratum, states that design stresses should not exceed 75 percent of the values of Table No. 25. For southern pine and Douglas-fir, this recommendation results in compression parallel to grain values of 1087 psi for non-dense and 1275 psi for dense piling. Most southern pine and Douglas-fir piling are taken from pole stock and are dense; i.e., have more than 6 growth rings per inch. Furthermore, the Handbook states:

> "It is therefore not possible to arrive at basic stresses by mathematical calculation alone. Experience and judgement, backed by pertinent test data, must be employed to formulate basic stresses that are applicable to certain specific conditions."

For *friction piles*, the handbook recommended that timber piles are particularly efficient because of their taper. Pile capacity was considered to be a function of the soil strength parameters, and the ultimate, compressive strength of the pile was generally not a factor.

In 1941, the American Association of State Highway and Transportation Officials (AASHTO) adopted 1200 psi as the design stress for Douglas-fir and in 1953, specified 1200 psi for non-dense and 1400 psi for dense southern pine (AASHTO, 1977). The New York City Building Code specified 1200 psi in the early '60s, and the major building codes (i.e., BOCA, UBC, SBC) soon followed with design stresses of 1200 psi for southern pine and 1250 for Douglas-fir based on historical considerations or on ASTM D-2899, "Establishing Design Stresses for Round Timber Piles" (ASTM, 1970), which was first published in 1970. Proof of performance by years of use is the reason the value of 1200 psi is generally used by engineers throughout the United States as the allowable design stress for timber piles (BOCA, 1978; SBC, 1979; and UBC, 1979).

Timber Technology

Modern timber design is based on probability theory as developed at the U. S. Department of Agriculture, Forest Products Laboratory in Madison, Wisconsin. This technology was adopted to timber pile design through ASTM D-2899. Even the current edition of the Wood Handbook (USDA, 1974) refers to ASTM for stress in piles. What is the principle of timber design? Simply that wood is grown in nature, and not man-made. Trees are not uniform and, therefore, their strength characteristics will vary. Hence the need for specifications and standards to assure the design engineer this product of nature will reliably meet or exceed the minimum design values used.

Thousands of tests have been performed on what is called "small-clear

samples." These samples are free from knots and other defects. For compression parallel to grain, the specimens are 1 x 1 x 4 in; for bending they are 1 x 1 x 18 in. Individual bending tests have been continued for nearly ten years on a single sample, resulting in a bending stress duration of load relationship (USDA, 1974). The test results for all small-clear specimens are published in ASTM D-2555, "Establishing Clear Wood Strength Values" (ASTM, 1981).

Results from the small-clear tests are correlated to full-scale timber design by using a series of reduction factors multiplied by one-another, and arriving at what is called a "near-minimum" factor of safety of 1.25 for compression parallel to grain (ASTM, 1981). Why a near-minimum value? Because, it is very unlikely that all the reduction factors would occur on the same member at the same time. The real safety factor is probably about 2.00 to 2.50, and nearly all values are in the range of 1 to 4 (Wood, 1969). The reduction factors for lumber in D-245 are (ASTM, 1981):

	Approximate Values for Lumber
1. *Variability* (The 5% exclusion limit)	0.70
2. *Strength Ratio* (Ratio of cross-sectional area with knots deducted to cross-section of structural lumber, depending on grade specified)	0.65
3. *Special Factors*	
Dense Lumber (6 growth rings per inch and 1/3 summerwood, etc.)	1.17
Adjustment Factor (Includes a reduction factor of 1/1.6 for load duration and 1/1.25 for formal factor of safety)	1/2.0

The allowable design stress for dense southern pine lumber parallel to grain, depending on the specified grade, would be approximately:

Design Stress

$$\text{compression parallel to grain} = \frac{(0.65)\,(0.70)\,(1.17)}{(2.0)} \quad (3627 \text{ psi})$$

$$= (0.27)\,(3627) \text{ psi} = 965 \text{ psi}$$

where: 3627 psi = crushing strength from ASTM D-2555, weighted average (ASTM, 1983)

In the late 1960's and early 1970's, a series of compression tests parallel to grain were performed on treated and untreated southern pine, Douglas-fir, and oak pile material. Small clear samples were often correlated to companion round piles, 3 ft long and about 6 in. to about 14 in. diameter. The test results from 150 southern pine tips tested at Mississippi State, 78 Douglas-fir tips tested at Oregon State, and 15 each pine, fir and oak, butts and tips, tested at the Forest Products Lab were then made available to ASTM Committee D7.07 (Peterson, 1970; Thompson, 1970; Wilkinson, 1968).

The committee evaluated the data and presented a normalized equation from which the small clear values in D-2555 could be used as a basis for developing design stresses for round timber piles. The committee apparently followed the D-245 procedure for lumber:

	Approximate Values for Lumber
1. *Variability* (The 5% exclusion limit for southern pine)	.0.72
2. *Strength Ratio* (Reductions for imperfections at the pile tip)	.0.81
3. *Special Factors*	.0.00
a) *Dense Piling* (The committee forgot to consider that nearly all tests were performed on dense materials)	
b) *Treatment*	
(1) Boulton process	.0.90
(2) Steam conditioning	.0.85
4. *Adjustment Factor*	
a) *Load Sharing* (The normal load duration factor reduced by 5%)	1/1.52
b) *No Load Sharing* (Includes a load duration factor of 1/1.52 and a 1/1.25 factor of safety)	1/1.9

The D-2899 allowable design stress for treated southern pine piling, parallel to grain, is approximately:

Design Stress

$$\text{compression parallel} = \frac{(0.72)\,(0.81)\,(0.85)}{(1.9)} \quad (3627 \text{ psi})$$

$$\text{to grain} = (0.25)\,(3627 \text{ psi}) = 908 \text{ psi}$$

$$= \text{load sharing} = 1183 \text{ psi}$$

Unfortunately, many of the pile tip tests were performed on samples that were too slender and failed prematurely (Graham, 1984). According to ASTM D-298 "Static Tests of Timbers in Structural Sizes" (ASTM, 1983), all tests should have been performed with 1/r values between 10 and 17, however, most of the southern pine pile tips were tested at 1/r ratios exceeding 17, and the results caused concerned with geotechnical engineers (Armstrong, 1979).

Research at University of Colorado

Testing is being performed on 2 and 3 ft long timber piling tips at the University of Colorado (Goble, 1983; Phillips, 1983; Stark, 1982; Urrego, 1981). Tests to date include 20 matched treated and untreated unconfined southern pine and 20 matched treated and untreated Douglas-fir. Consideration was given to the 1/r requirement to prevent buckling. ASTM D-198 sets the maximum ratio at 17 (ASTM, 1984). The first 20 southern pine treated and untreated samples were run with lengths cut at 3 ft, some at 1/r values greater than 17. In accordance with ASTM D-198 "Static Tests of Timbers in Structural Sizes," all subsequent samples with diameters less than 8.5 in. were cut to 2 ft lengths (i.e., 1/r must be less than 17). Average test results on unconfined samples were as follows:

	Diameter	*Crushing Strength*
Douglas-fir		
treated	9.3 in.	2860 psi
untreated		2760
Southern pine		
treated	8.0	2840
untreated		3240

Reduction factor results on 20 pieces each of Douglas-fir and southern pine samples were:

	Defects	*Treatment*
Douglas-fir	0.80	1.00
Southern pine	0.90	0.83

Similar to the Oregon State test results, Douglas-fir piling strength was not reduced by Boultonizing.

Long-term tests were also run at Colorado. The full-size piling sample results correlate with existing long-term bending tests. Test results on small clears were inconclusive due to an equipment malfunction.

Confined Tests

Engineers often ask what is the effect of soil confinements on timber piling strength? Triaxial-type tests were run by the Forest Products Lab in 1964 to determine if round timbers are stronger when confined by a soil overburden (Wilkenson, 1964). The tests used oil as the confining material. The oil penetrated into the wood and actually reduced the strength values below those of unconfined samples. Additional confined tests on untreated round timber piles have recently been performed at the University of Colorado. The Colorado confined tests were performed in dry sand compacted to approximately 40 ft of equivalent overburden pressure, i.e., 30 psi. The tests resulted in higher strength values of confined as compared to unconfined matched pairs, averaging some 20 percent:

	Diameter	*Crushing Strength*
Southern pine	±9 in.	
confined		3770 psi
unconfined		3110

Most of the increase occurred in the lower strength samples.

These test results showed that the lower unconfined strength samples are some 40 percent stronger when confined and the higher strength samples were increased only by 10 percent when confined.

LOAD SHARING

Pile foundations are often designed as a cluster of piles grouped beneath a pile cap, supporting one heavy column load. Load sharing is often assumed in design of pile foundations.

Dr. Roy Olsen, of the University of Texas, discussed load sharing for timber piles (Olsen, 1984):

> "Design at the lower 5 percent level of a probability curve should not be included. For a one-pile group the probability of failure should be less. For a multi-pile group the probability of overstressing any one pile should be raised. If we call Pf the probability of failure obtained from the probability density function in terms of the applied stress, the probability of failure of a one-pile group is Pf, but the probability that all piles in a group of n piles will be overstressed is Pf." If Pf = 0.05 and n = 4, then the probability of all piles being overstressed is only one in 160,000. - - - - the calculation indicates that it is uneconomical to set limiting stresses on piles in groups as if the group would fail if any one pile failed."

Edward Diekmann, G.F.D.S. Engineers, San Francisco (ASTM, 1972), recommended a load sharing value of 1.15 be used for timber piles in ASTM D-2899. The 1.15 factor is normally used in timber design for load sharing.

Utility Pole Bending Strength

Unlike lumber which is sawn into various dimensions, wood utility poles are obtained from trees carefully selected for density (6 rings per inch, etc), straightness, minimum defects, etc., and are used as an independent structural member. The design criteria for dimension lumber is not considered practical for use in evaluating the reactions of a wood pole that is supporting utility lines.

Starting about 1925, the utility industry helped to develop American National Standard Institute 05.1 "Specifications and Dimensions for Wood Poles" (ANSI, 1979). The current ANSI edition on wood poles was based on research evaluated at the U. S. Government Forest Products Laboratory (Wood and Markwordt, 1965). Steam conditioned southern pine and Boulton dried Douglas-fir were evaluated at 8000 psi bending strength using the following parameters:

	Values for Utility Poles
1. *Variability* (The average value of the standard deviation, i.e., 100 − 14/2 = 93)	0.93
2. *Form Factor.* (Comparing the strength of small, clear specimens with untreated poles)	1.08
3. *Moisture Content* (Based on expectation that service moisture contents 4 ft above the ground line would not exceed 20%)	1.16
4. *Load Duration* (Duration of load is omitted because service loadings from wind gusts on ice-covered conductors are of short duration compared to the 15-minute pole tests)	0.00
5. *Treatment*	
(1) Boultonized	0.90
(2) Steamed at 245°F	0.85

The resulting bending strength values of 8000 psi for southern pine and Douglas-fir, given in ANSI 05.1, are for groundline stresses approximately

6 ft from the butt, when a horizontal load is applied 2 ft from top of the pole. For strengths above the groundline, a reduction factor is used:

$$F_2 = F_1 (1 - 0.5\ H/L)$$

where: the maximum value of H can be L/2 and

F_1 = fiber stress value
F_2 = fiber stress at distance H
H = distance above groundline
L = length of pole from groundline to top

The maximum reduction should not exceed 25 percent of the species fiber stress value at the groundline.

From the above, minimum pole circumferences specified at 6 ft from the butt have been calculated for each species in a given class to develop approximately equal groundline stresses. Poles are further proportioned into 15 classes, depending on the magnitude of horizontal loads. For example, a 40 ft Class 3 pole will support a horizontal line pull of 3000 lb at 2 ft from the top of the pole with a minimum circumference of 36 in., 6 ft from the butt, and 23 in. at the top.

Production of Timber Piling

About 500,000 timber piles are produced each year from the same group of southern pine and Douglas-fir trees that provide many of the 3,000,000 utility poles sold annually. The trees are carefully selected, dense (i.e., minimum 6 rings per inch), straight, have few knots, etc. in accordance with ANSI 05.1 (ANSI, 1979), and ASTM D-25 "Standard Specification for Round Timber Piles" (ASTM, 1979).

The standing volume of subspecies for southern pine and Douglas-fir, as given in ASTM D-2555 (ASTM, 1981), should not be used to determine the proportion of trees used for foundation piling. The timber utility pole and piling specifications are too stringent. Instead of using D-2555, plant surveys indicate the foundation piling proportion for southern pine subspecies is 50 percent Loblolly/Shortleaf and 25 percent each for Slash and Longleaf, and that nearly all the Douglas-fir foundation piles are produced from Coastal subspecies.

Most timber foundation piling is pressure treated with creosote in accordance with the American Wood-Preservers' Association Standard C3 (AWPA, 1981):

	Retention	*Penetration*
Southern Pine	12 pcf @ 0 to 2 in.	3 in. or 90% sapwood
Douglas-fir	17 pcf @ 0 to 1 in.	.75 in. and 85% sapwood

Prior to treatment, southern pine piling shall not be steamed at 240°-245°F for more than one (1) hour per inch of average midpoint diameter.

Douglas-fir is usually Boultonized, boiled in creosote under a vacuum at a maximum of 220°, in accordance with AWPA requirements. (For southern pine, the 15 hours maximum steaming at 245°F includes all steaming time at which the temperature exceeds 200°F.)

Results and Discussions

Material strength factors for lumber and timber, timber piling, and wood utility poles were reviewed in the preceding sections. The allowable design stresses for lumber and timber were shown to be based on consideration of lumber's variability (5% exclusion limit), imperfections, density, load duration, moisture content, safety factor, etc. Bending stength at the groundline was the prime consideration for utility poles strength factors, and axial compression at the pile tip apparently controlled timber piling design.

Ever since publication in 1970, ASTM Standard D-2899 on timber piling stresses has been under attack. The main criticism of D-2899 has been the absence of a "formal factor of safety" and improper reduction factors for subspecies, treatment, knots, and load duration. Also, some of the piling tests failed at very low values.

The problem of low piling tip strength results was resolved when it was discovered the prior crushing tests were performed on samples that were too slender and failed prematurely (Graham, 1984). Also, the critics had based their entire stress evaluation on lumber design criteria, not knowing that piling was produced from utility pole stock, which incorporates an entirely different set of design parameters, starting with subspecies selection and extending throughout the entire stress reduction factor analysis.

Engineers can now use the University of Colorado full-scale test results to determine the allowable design stresses for treated round timber piling. Average test values of 2840 psi for southern pine and 2860 psi for Douglas-fir incorporate all the reduction factors normally used for design, except for variability and safety, including load duration. Factors for knots, size, density, moisture content, treatment, etc. are included.

Test results indicate the variability for piling is the same as utility poles, i.e., 0.93; however, 0.90 should be used because of the limited number of tests performed to date.

The safety factors should be 1/2.0 based in part on timber technology and part on geotechnical considerations. Timber technologists use an adjustment factor of 1/2.0 in D-245, which includes 1/1.6 for load duration and 1/1.25 as the "formal factor of safety." Geotechnical engineers use a safety factor of 1/2.0 for static design and load test analysis. The use of a safety factor of 1/2.0 for non-confined timber piles is justified.

To insure uniform strength characteristics, a procedure should be developed for the quality control of timber foundation piling. Owners, engineers and contractors should be able to order 2800 psi or other strength level timber piling, depending upon design, and be assured they receive what they ordered.

CONCLUSIONS

The following conclusions are based on more than three years' research at the University of Colorado on the crushing strength of southern pine and Douglas-fir timber piling.

The length to diameter ratio (l/d) for unconfined compression tests parallel to grain should be standardized for timber piling at about 2:1. A series of laboratory tests are required to finalize the actual l/d ratio that minimizes the effects of premature buckling type failures in round timbers, and simulates the effect of soil confinements at the pile tip.

The allowable design stresses for round, treated southern pine and Douglas-fir foundation piling should be based on the strength of material criteria already developed for wood utility poles, especially since most timber piling is taken from pole stock:

Subspecies for southern pile should be proportioned at 50 percent Loblolly/Shortleaf and 25 percent each for Slash and Longleaf; Douglas-fir should be Coastal.

Basic strength parameter should be 2800 psi axial crushing strength on a treated, 8 in. diameter pile tip, which includes all the usual reduction factors except variability and safety. This strength value, based on the 3 ft long Colorado pile tip tests, should be updated when 1/d test ratios are standardized for timber piling.

Reduction factors of 0.90 for variability (rounded average standard deviation) and 1/2.00 safety factor (including 1/1.60 load duration and 1/1.25 for formal factor of safety) results in an allowable design stress at 1250 psi for the southern pines and coastal Douglas-fir.

This author believes the allowable design stresses for the southern pines and coastal Douglas-fir should be established at only one value, 1250 psi, and all other factors should be dropped, except for consideration of driving stresses. The engineer should be aware that:

(1) pile tip strength is increased by soil confinement an average of 20 percent for piles 40 ft long or longer in granular soil;

(2) pile strength increases from tip to butt at a rate of 10 percent per every 10 ft of length up to a maximum of 25 percent;

(3) load sharing usually permits about a 15 percent increase in allowable design stresses in lumber used as floor joist, etc., and is probably applicable to piling used in a group connected with a concrete pile cap,

(4) timber piling is usually supplied oversize by at least a volume of 15 percent.

Knowledge of the existence of these additional strength factors should assure the engineer that the recommended design stresses are quite safe.

Research is needed to establish more information on allowable driving stresses for timber piles, especially when the driving stresses can now be estimated in the office using wave equation analysis and measured in the field using a dynamic testing device. At the present time, engineers are

estimating the theoretical allowable driving stresses of timber piling at 3000 psi. A more accurate driving stress determination is required for timber piling to stay competitive.

Finally, the timber piling industry needs to investigate the possibility of establishing a non-destructive testing program, such as ultrasonic tests, for timber piling. If such a program exists, timber piling could be pre-tested at the plant and certified to meet specific design stress requirements, probably for 50 ton and higher design loads.

The National Timber Piling Council, in the short time it has been established, has within its present goals, programs underway to develop this needed information. The program, when completed, will give the designer, the specifying engineer and the contractor the knowledge and quality assurance of product to enable our industry to be a more important consideration in future foundation design.

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Crane Boom Loadings From Pile Driving

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Loads and stresses from pile driving are increasing from heavier piles and more powerful hammers. Stresses can be dangerous to equipment stability. Equipment developer MacKinnon tells how to compute stresses for all positions of the crane boom and angles of pile batter. From a paper presented at the DFI-University of Hawaii Seminar, April 1985.

Piles are being driven in longer sections and ever increasing weights. The reasons for these increases are purely economic.

The driven pile length has increased for several reasons: It is more productive to drive the pile in a single length rather than in multiple sections; pile capacities have generally increased requiring them to go deeper to attain bearing; there seems to be a higher percentage of batter piles, which should, from a production standpoint, be driven in one piece.

The total pile weights are increasing because of the increased length, larger sizes to provide higher capacities, and the increasing use of precast concrete piles. Heavier hammers add to the crane load.

The contractor has had to change from relatively short swinging leads to long SEMIFIXED or fixed leads with hydraulic bottom braces. The loading imposed on the crane boom is considerably different.

Swinging leads generally impose only vertical loads. Since the lead is cable supported, the shock loads from driving are dampened. Failures with swinging leads generally are related to stability of the crane, not structural capacity of the boom.

SEMIFIXED (see sketches) and fixed leads impose vertical loads, side loads, and standing moments on the boom.

The SEMIFIXED and fixed leads differ in how they introduce their vertical load component into the boom. The SEMIFIXED system transfers its vertical load via cables over the boom tip sheaves; the fixed system transfers its vertical load via a solid connection to the boom tip shaft. The SEMI-

FIXED system will dampen the shock loads while the fixed leads introduce these shocks into the boom.

The side load and standing moment are introduced in the same manner in both SEMIFIXED and fixed design. The differences are only in the geometry of the respective structure.

The contractor and his staff should be aware of how to compute the various load components for fore, aft and side batter. It should be remembered that any failure in the boom will be almost instantaneous since it is under compression.

Analyzing fore-aft batter (see Bulletin No. 283) evolves into determining the equivalent vertical tabular load W_E that when placed on the crane produces the same moment as the pile driving equipment. The tabular load W_E is then compared to the crane capacity W_L at the design radius, such that $W_L \geqq W_E$. In event that the crane is going to crawl with the lead, the capacity of W_L should be reduced to 65 percent of the tipping capacity rather than the 75 percent factor used in W_L from the crane tables. The same effect can be obtained by increasing W_E such that $W_L \geqq 1.15\, W_E$.

The value of W_E in the plumb position (Bulletin 283–2) with hammer, drive cap and pile, is determined first since if the crane cannot handle the load plumb, it will never get to the batter position. Second, the value of W_E in fore batter (Bulletin 283–3) is determined with the hammer and drive cap at the bottom of the leads as would occur immediately after driving the pile and the hammer is lifted off the pile. Lastly, determine the value of W_E for aft batter (Bulletin 283–4) with hammer and drive cap at top of leads with full pile not touching the ground. In all of the above cases, $W_L \geqq W_E$) if not either the operating radius or the load must be reduced. All of the above equations take their moments about the center of crane rotation and are applicable for both SEMIFIXED and fixed systems.

The SEMIFIXED system can increase the effective capacity of the crane because of its ability to place the lead on the ground. The lead system then becomes similar to a stiff leg derrick. The vertical component is transferred to the ground via the lead and the crane only accepts the lateral component and acts as the hoisting mechanism for the hammer and pile. To evaluate this configuration, the moments are taken about the base of the lead to solve for the unbalanced moment M_C from the crane, this moment is then divided by the radius R to determine W_E, $W_E = \frac{M}{R}C$. This procedure may be utilized for plumb and aft batter but should only be utilized with caution for the fore batter configuration.

The fore batter can create a negative value for W_E at which time the boom will rotate backward over the crane—the only thing that prevents this is the line between the lead and the boom.

Analyzing side batter (see Bulletin 183) evolves down to determining the side batter load on the boom S_B and the standing moment M_A induced in the boom as well as W_E for the vertical position. The value of S_B is compared against the allowable "horizontal side load of 2 percent of the rated lifted

load applied to and at the load radius," per SAE J987, Crane Structures–Method of Test. There is no published allowable value for the induced moment M_A. The value for W_E is compared against W_L from the crane tables.

There are four conditions to be determined. First, S_B is determined with the lead side battered (Bulletin 183–5) and the hammer and drive cap at the base of the leads, this will normally produce the largest value of S_B. Second, S_B is evaluated with the lead side battered (Bulletin 183–6) and the hammer and drive cap at top of lead with full pile, this value while less than from condition 1, will be combined with W_E vertical and M_A from condition 4 and may prove to be the worst loaded case under combined stress analysis. Third, S_B is evaluated (Bulletin 183–7) with the same configuration as in condition 2 but the pile just touching the ground, this will produce the maximum torque to the crane's swing system.

It is advised that the house lock be engaged before touching the ground, thereby eliminating any possibility that the crane would be revolved around. Lastly, the moment induced in the boom M_A by the offset loads is determined (Bulletin 183–8).

The evaluation of whether the boom can sustain the induced loads must be made by the crane manufacturer. The information required is as described in Bulletin 185 along with a listing of some of the manufacturers.

It is hoped that this paper will assist the reader to determine the loads his operations will induce in the crane and, therefore, forewarn him of any impending disasters.

ACKNOWLEDGEMENT

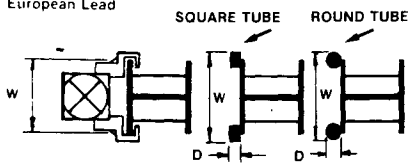
The author acknowledges the assistance of Fritz Koltermann, V.P. and Chief Engineer of The Foundation Equipment Corporation, for his help in the preparation of this paper. The comments and assistance of the crane manufacturers, American Hoist and Derrick, Koehring, Link-Belt, Manitowoc, Northwest and Harnischfeger are appreciated.

Editor's Note: Sketches of the leads were made available earlier by author Mac Kinnon and Foundation Equipment Corp. for use in the DFI Glossary of Foundation Terms.

LEAD TYPES

H-BEAM

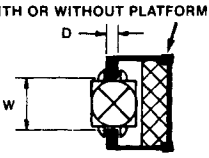
Also Called Spud Lead, Wide Flange, Monkey Stick, European Lead



GUIDE PROFILES & APPROX. WEIGHTS:
- 10" W — 50 lb./ft.
- 12" W — 60-80 lb./ft.
- 14" W — 80-120 lb./ft.
- 21" W — 145 lb./ft.
- 3" D x 15¾" W — 80 lb./ft.
- 2¾" Dia. x 15¾" W — 80-130 lb./ft.

BOX LEAD

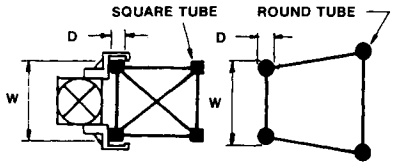
Also Called "U" Lead, Steam Lead



GUIDE PROFILES & APPROX. WEIGHTS:
- 6", 7" or 8" D x 20" W — 65-90 lb./ft.
- 8" or 9" D x 26" W — 80-160 lb./ft.
- 8", 9" or 10" D x 32" W — 130-210 lb./ft.
- 10" or 11" D x 37" W — 200-280 lb./ft.
- 11" D x 56" W — 210-300 lb./ft.

TRUSS LEAD

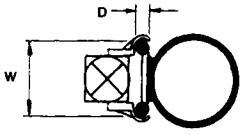
Also Called Monkey Stick, Spud Lead, European Lead



GUIDE PROFILES & APPROX. WEIGHTS:
- 3" D x 15¾" W — 60-100 lb./ft.
- 3" D x 21½" W — 60 lb./ft.
- 3" D x 28½" W — 100 lb./ft.
- 5" D x 28½" W — 150-190 lb./ft.
- 2" Dia. x 11" W — 30 lb./ft.
- 2¾" Dia. x 15¾" W — 70-125 lb./ft.

PIPE LEAD

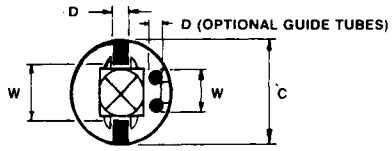
Also Called Monkey Stick, Spud Lead, European Lead



GUIDE PROFILES & APPROX. WEIGHTS:
- 2" Dia. x 11" W — 30-60 lb./ft.
- 2¾" Dia. x 15¾" W — 70-125 lb./ft.

OFFSHORE LEAD

Also Called Chuck Lead, Can, Rope Suspended Lead



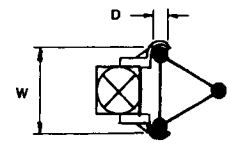
GUIDE PROFILES & APPROX. WEIGHTS:
Diesel Hammers —
- 6" D x 20" W x 24" C — 4,000 lb.
- 8" D x 26" W x 36" C — 6,500 lb.
- 10" D x 32" W x 48" C — 10,000 lb.
- 2" Dia. x 11" W x 24" C — 4,000 lb.
- 2¾" Dia. x 15¾" W x 36" C — 6,500 lb.

Air/Steam Hammers —
- 10" D x 54" W x 48" C — 16,850 lb.
- 14" D x 80" W x 72" C — 33,700 lb.
- 19" D x 88" W x 72" C — 68,000 lb.
- 22" D x 144" W x 120" C — 137,000 lb.

Weights can vary due to hammer size and thickness of drive plate.

TRIANGULAR LEAD

Also Called Monkey Stick, Spud Lead, European Lead



GUIDE PROFILES & APPROX. WEIGHTS:
- 2" Dia. x 11" W — 20 lb./ft.
- 2¾" Dia. x 15¾" W — 40 lb./ft.

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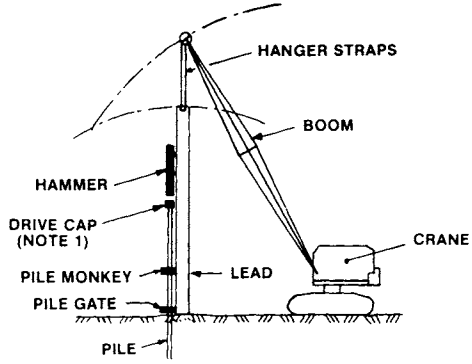
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SWINGING LEAD SYSTEMS

SKETCHES INDICATE HAMMER MOUNTED IN FRONT OF LEAD.
ALL CONFIGURATIONS ARE POSSIBLE IN BOX LEAD.

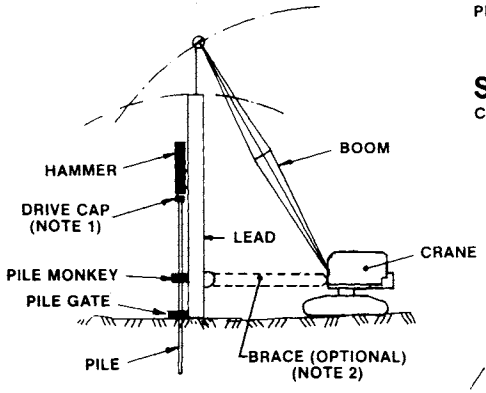
UNDERHUNG LEAD

CABLES REQUIRED:
1 — HAMMER
1 — PILE (OPTIONAL)
1 — PILE MONKEY (OPTIONAL)



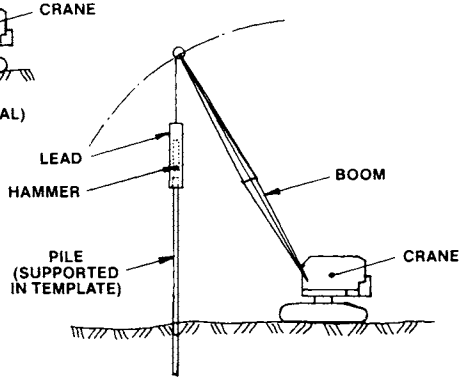
SWINGING LEAD (NOTE 3)

CABLES REQUIRED:
1 — HAMMER
1 — LEAD
1 — PILE (OPTIONAL)
1 — PILE MONKEY (OPTIONAL)



OFFSHORE LEAD

CABLES REQUIRED:
1 — HAMMER



NOTE 1: ALSO CALLED ANVIL BLOCK, BONNET, CAP, DRIVING HEAD, FOLLOW CAP, HELMET, HOOD, RIDER CAP
NOTE 2: ALSO CALLED A-FRAME, APRON, BOTTOM BRACE, BOTTOM STRUT, KICKER, PARALLELOGRAM, PLATFORM, SPIDER, SPOTTER, SPREADER, SPREADER BARS
NOTE 3: ALSO CALLED HANGING LEAD

FEC THE FOUNDATION EQUIPMENT CORP.

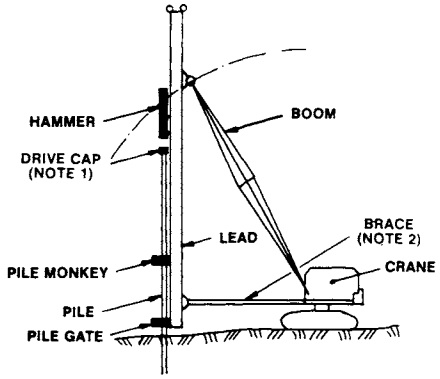
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FIXED LEAD SYSTEMS

SKETCHES INDICATE HAMMER MOUNTED IN FRONT OF LEAD.
ALL CONFIGURATIONS ARE POSSIBLE IN BOX LEAD.

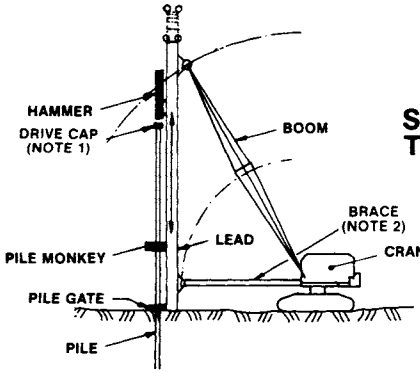
FIXED OR EXTENDED WITH BRACE

CABLES REQUIRED:
1 — HAMMER
1 — PILE (OPTIONAL)
1 — PILE MONKEY (OPTIONAL)
1 — BRACE (OPTIONAL)



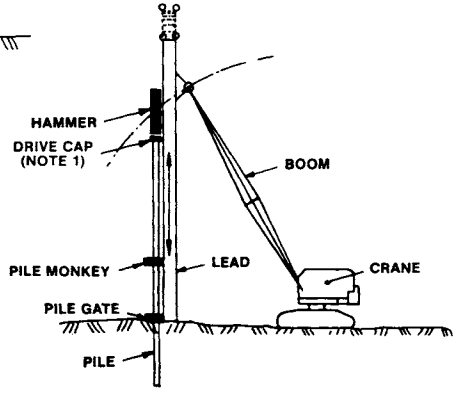
SEMIFIXED OR VERTICAL TRAVEL WITH BRACE

CABLES REQUIRED:
1 — HAMMER
1 — LEAD
1 — PILE (OPTIONAL)
1 — PILE MONKEY (OPTIONAL)
1 — BRACE (OPTIONAL)



SEMIFIXED OR VERTICAL TRAVEL WITHOUT BRACE

CABLES REQUIRED:
1 — HAMMER
1 — LEAD
1 — PILE (OPTIONAL)
1 — PILE MONKEY (OPTIONAL)



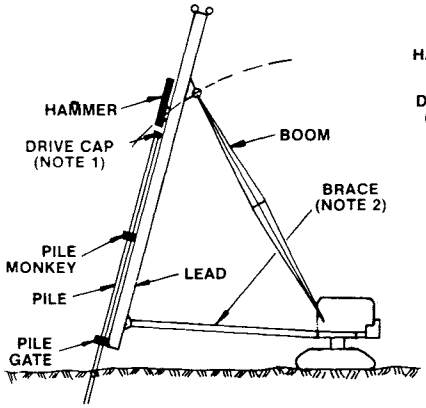
NOTE 1: ALSO CALLED ANVIL BLOCK, BONNET, CAP, DRIVING HEAD, FOLLOW CAP, HELMET, HOOD, RIDER CAP
NOTE 2: ALSO CALLED A-FRAME, APRON, BOTTOM BRACE, BOTTOM STRUT, KICKER, PARALLELOGRAM, PLATFORM, SPIDER, SPOTTER, SPREADER, SPREADER BARS

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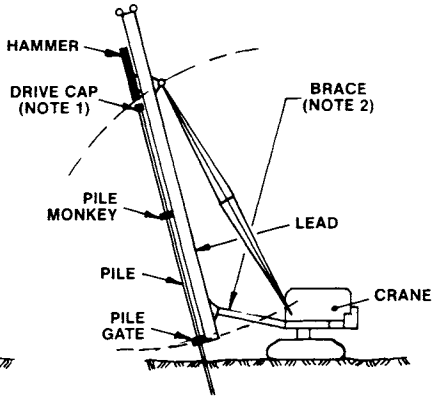
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BATTER CONFIGURATIONS

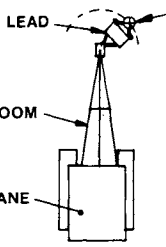
SKETCHES INDICATE HAMMER MOUNTED IN FRONT OF LEAD.
ALL CONFIGURATIONS ARE POSSIBLE IN BOX LEAD.



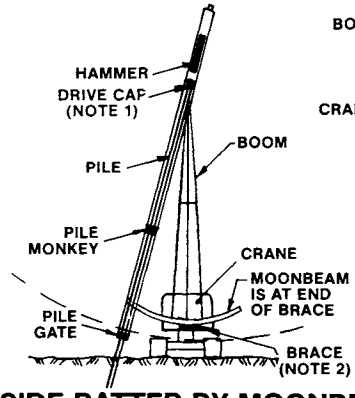
FORE (POSITIVE) BATTER



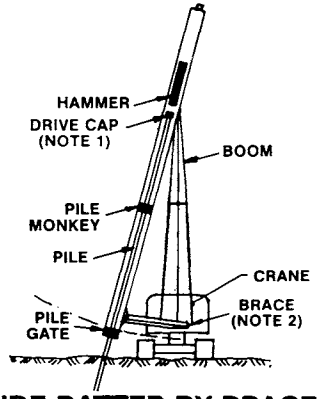
AFT (NEGATIVE) BATTER



CARDONIC (SWIVEL LEAD)



SIDE BATTER BY MOONBEAM



SIDE BATTER BY BRACE

NOTE 1: ALSO CALLED ANVIL BLOCK, BONNET, CAP, DRIVING HEAD, FOLLOW CAP, HELMET, HOOD, RIDER CAP

NOTE 2: ALSO CALLED A-FRAME, APRON, BOTTOM BRACE, BOTTOM STRUT, KICKER, PARALLELOGRAM, PLATFORM, SPIDER, SPOTTER, SPREADER, SPREADER BARS

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INSTRUCTIONS FOR GEOMETRIC SIDE BATTER CHART

The chart gives the ranges of possible side batter which can be achieved under the following assumptions:

1. The boom foot is 4 ft. from the centerline of rotation.
2. The boom foot is 6 ft. above ground.
3. The boom tip connector is 2 ft. above the boom point point sheaves.
4. The brace is held horizontal.

The following shall explain how the chart has developed such that one can make his own chart if the conditions vary considerable from the above assumptions.

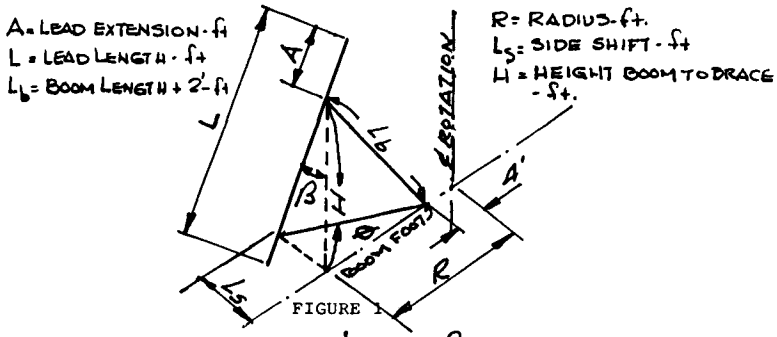


FIGURE 1

The side batter is the ratio $\frac{L_s}{H} = \tan\beta$ or

expressed as a whole number $S_b = \frac{H}{L_s} = \frac{1}{\tan\beta}$

For instance a side batter ratio of 1 in 4 is expressed as:

Side batter ratio S_r = 1:4 or

The angle of tan = 0.25 is 14.03 degrees

the side batter $S_b = \frac{1}{0.25} = 4$

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In fig. 1 the side batter ratio would be:

$$Sr = \frac{L_s}{H}$$

and the side batter:

$$S_b = \frac{H}{L_s}$$

H can be found to be:

$$H = \sqrt{(L_b)^2 - (R-4)^2} \quad (1)$$

and is:

$$L_s = (R-4)\tan\phi \quad (2)$$

The side batter S_b can then be expressed as:

$$S_b = \frac{\sqrt{(L_b)^2 - (R-4)^2}}{(R-4)\tan\phi} \quad (3)$$

Since the side batter ratio usually is given, formula (3) can be solved for the boom point radius R:

$$R = \frac{(L_b)}{\sqrt{S_b^2 \tan^2\phi + 1}} + 4$$

For instance:

$S_r = 1:4$

$\therefore S_b = 4$

$L_b = 50+2$

$\phi = 26°$

$$R = \frac{52}{\sqrt{4^2(\tan^2 26°)+1}} + 4$$

$$R = 27.72'$$

or for any practical purpose R=28 ft.
Drawing a vertical line at R=28 ft. in the chart will show a possible side batter 1:4 at point (1) where this line intersects the 50 ft. boom line. Point (2) where this line intersects the boom point radius will give the boom point height above the boom foot.

The chart further shows that 14 ft. double throw brace fully extended will just reach this radius.

It is of the utmost importance to understand that the chart only shows what geometrically is possible, it does not give any indication whether the brace or the crane boom can handle the introduced side loading.

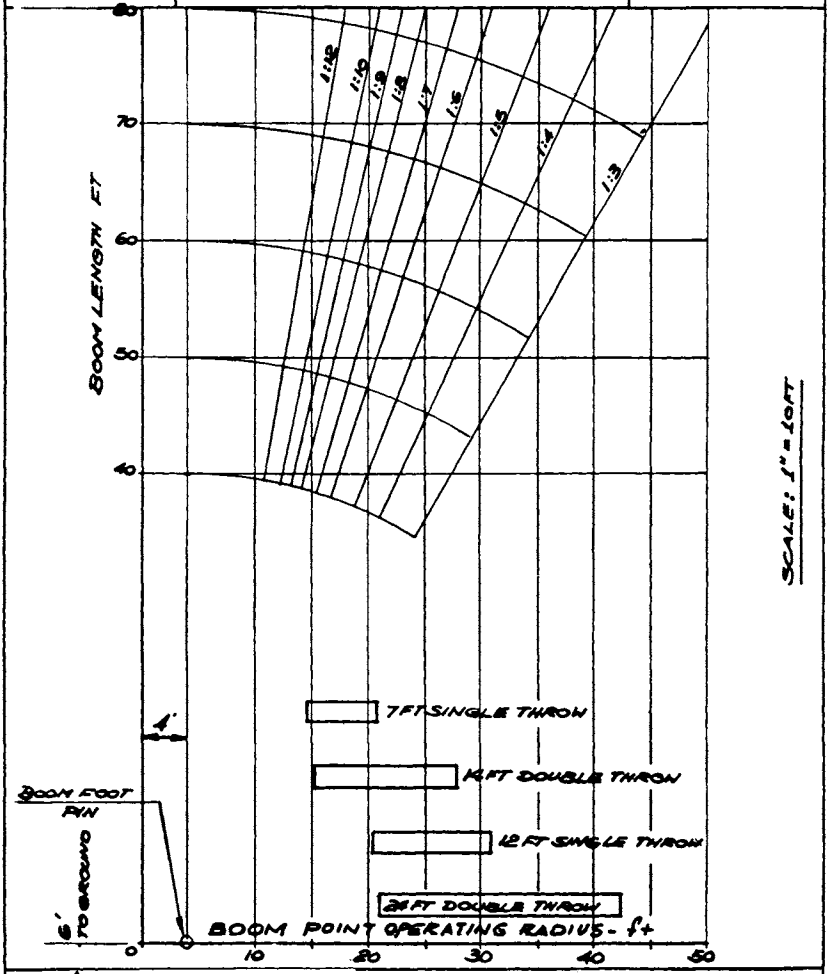
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TECHNICAL BULLETIN

SIDE BATTER

BULLETIN NO.
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SIDE LOADING OF CRANE BOOM

To find the loadings induced in the crane boom, four conditions must be checked.

CONDITION 1. Figure 5 pg. 5 of 8 — The lead side battered with hammer and drive cap at the bottom of the leads as would occur when the pile had been driven and the hammer and drive cap are lifted off the pile; this normally produces the largest (S_B) side batter load.

CONDITION 2. Figure 6 pg. 6 of 8 — The lead side battered with hammer and drive cap at top of leads, with pile, off the ground.

CONDITION 3. Figure 7 pg. 7 of 8 — The lead side battered with hammer and drive cap at top of leads, with pile touching ground. This will produce the maximum load on the cranes swing system until the pile enters the ground and accepts lateral load.

CONDITION 4. Figure 8 pg. 8 of 8 — The lead side battered with hammer and drive cap at top of leads, with pile off the ground. The offset of the equipment to the boom will create a standing moment in the boom for conservative evaluation assume 100% of this moment enters boom without assistance from brace or ground.

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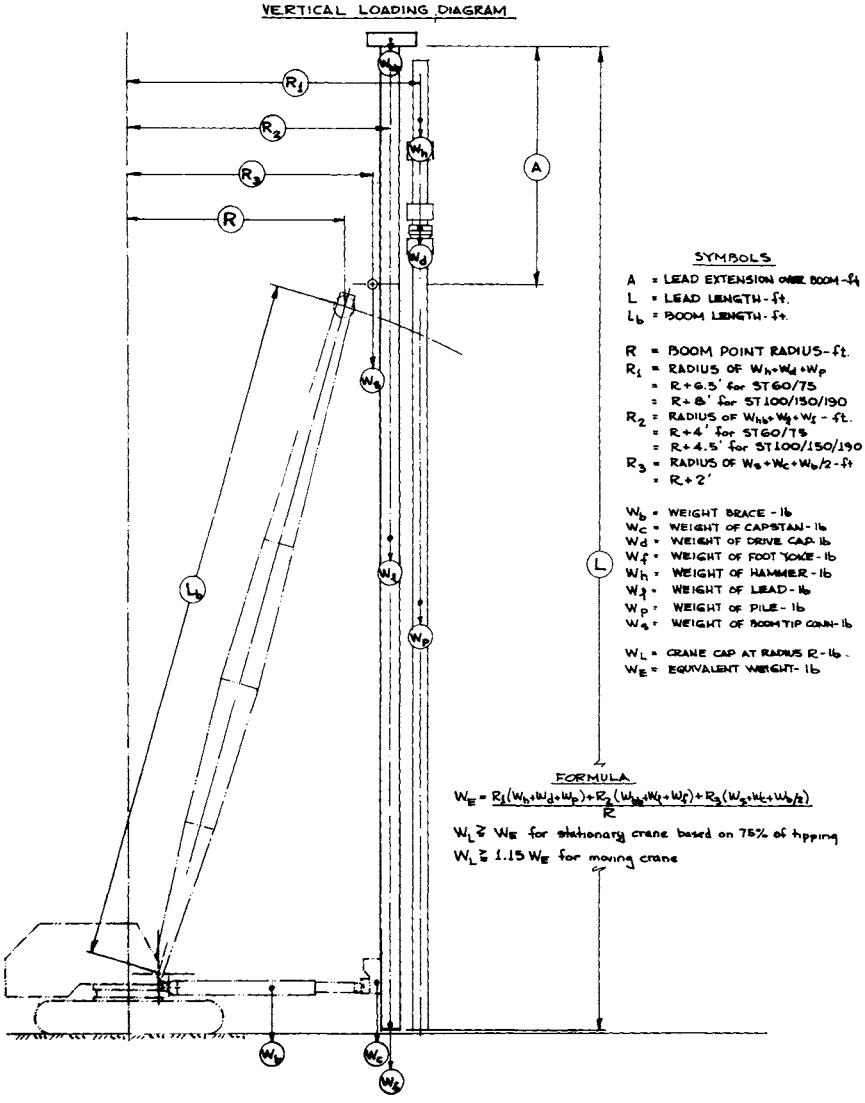

VERTICAL LOADING DIAGRAM
SYMBOLS
A = LEAD EXTENSION OVER BOOM - ft
L = LEAD LENGTH - ft.
Lb = BOOM LENGTH - ft.
R = BOOM POINT RADIUS - ft.
R1 = RADIUS OF Wh + Wd + Wp
= R + 6.5' for ST 60/75
= R + 8' for ST 100/150/190
R2 = RADIUS OF Whb + Wg + Ws - ft.
= R + 4' for ST 60/75
= R + 4.5' for ST 100/150/190
R3 = RADIUS OF Wa + Wc + Wb/2 - ft
= R + 2'
Wb = WEIGHT BRACE - lb
Wc = WEIGHT OF CAPSTAN - lb
Wd = WEIGHT OF DRIVE CAP - lb
Wf = WEIGHT OF FOOT YOKE - lb
Wh = WEIGHT OF HAMMER - lb
Wl = WEIGHT OF LEAD - lb
Wp = WEIGHT OF PILE - lb
Ws = WEIGHT OF BOOM TIP CONN - lb
WL = CRANE CAP AT RADIUS R - lb.
WE = EQUIVALENT WEIGHT - lb
FORMULA
WL ≥ WE for stationary crane based on 75% of tipping
WL ≥ 1.15 WE for moving crane

FIGURE 1

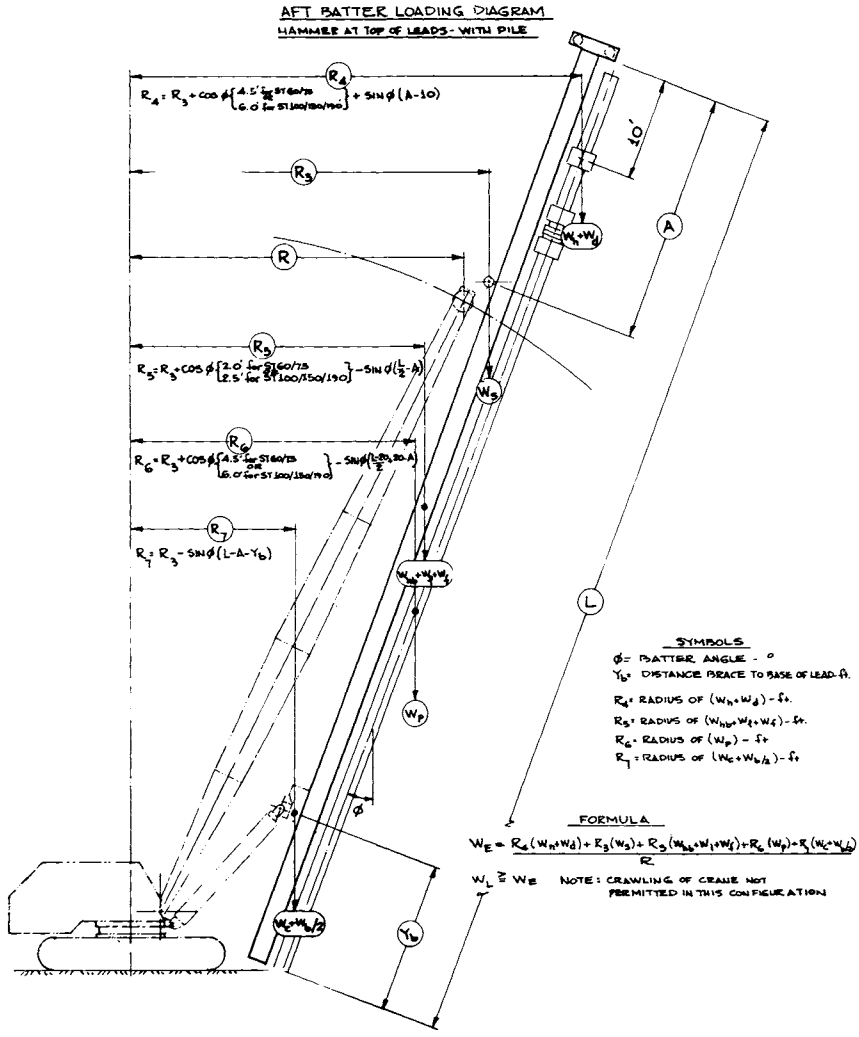

AFT BATTER LOADING DIAGRAM
HAMMER AT TOP OF LEADS - WITH PILE
SYMBOLS
FORMULA
NOTE: CRAWLING OF CRANE NOT PERMITTED IN THIS CONFIGURATION

FIGURE 2

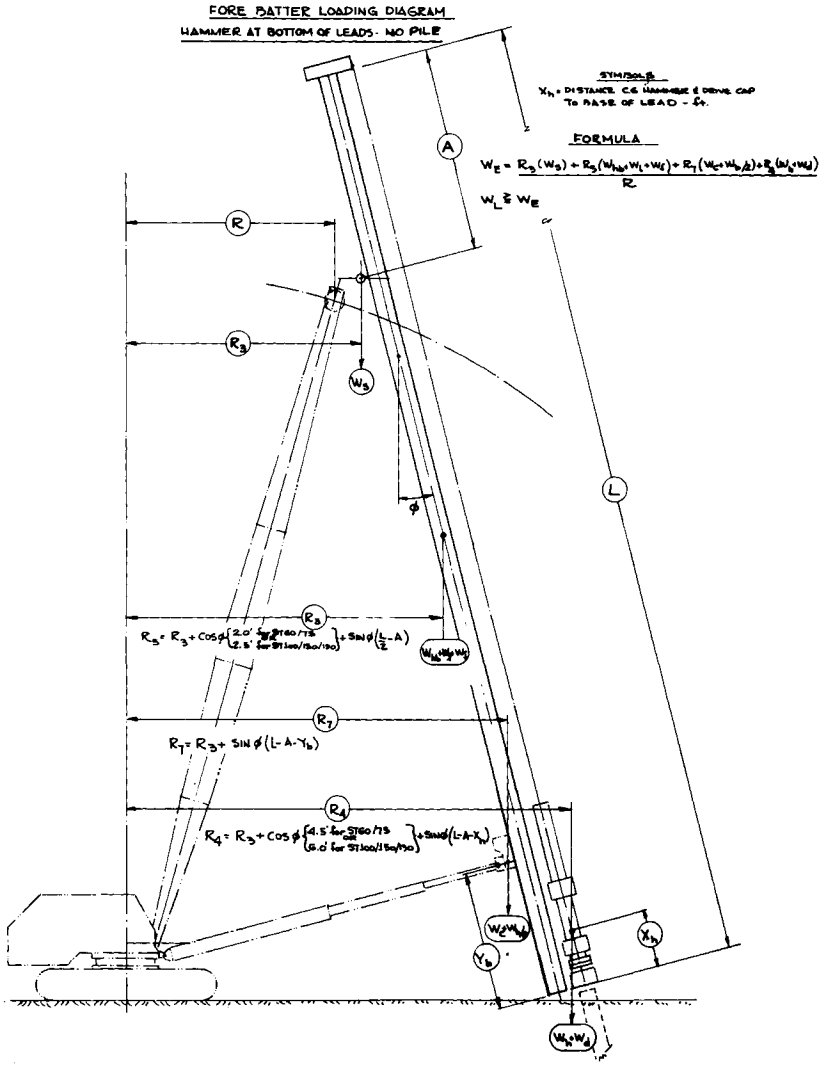

FORE BATTER LOADING DIAGRAM
HAMMER AT BOTTOM OF LEADS - NO PILE
SYMBOLS
X_h = DISTANCE C.G. HAMMER & DRIVE CAP TO BASE OF LEAD - Ft.
FORMULA
$W_E = \frac{R_2(W_3) + R_3(W_{H\&C} W_L + W_E) + R_7(W_C + W_D \frac{L}{2}) + R_4(W_2 W_H)}{R}$
$W_L \cong W_E$
A
R
R_2
W_3
ϕ
L
R_3
$R_3 = R_2 + \cos\phi\left[\begin{matrix}2.0' \text{ for ST60/75} \\ 2.5' \text{ for ST100/120/150}\end{matrix}\right] + \sin\phi\left(\frac{L}{2} - A\right)$
$W_H W_C W_C$
R_7
$R_7 = R_3 + \sin\phi(L - A - Y_b)$
R_4
$R_4 = R_3 + \cos\phi\left[\begin{matrix}4.5' \text{ for ST60/75} \\ 6.0' \text{ for ST100/120/150}\end{matrix}\right] + \sin\phi(L - A - X_h)$
$W_2 W_H$
X_h
Y_b
$W_H + W_A$

FIGURE 3

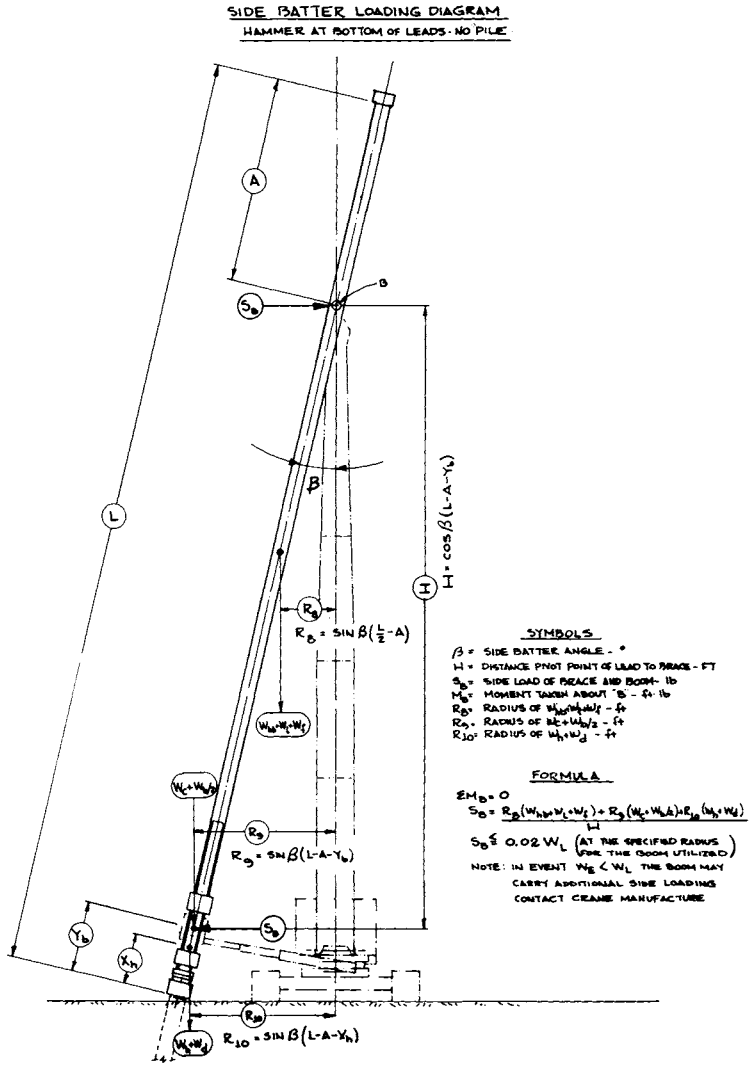

SIDE BATTER LOADING DIAGRAM
HAMMER AT BOTTOM OF LEADS - NO PILE
A
B
S_B
β
L
H
$H \cdot \cos\beta(L-A-Y_h)$
R_B
$R_B = \sin\beta(\frac{L}{2}-A)$
$W_H+W_L+W_T$
W_L+W_{LB}
R_3
$R_3 = \sin\beta(L-A-Y_h)$
Y_h
X_h
S_B
R_{10}
W_H+W_d
$R_{10} = \sin\beta(L-A-Y_h)$
SYMBOLS
β = SIDE BATTER ANGLE - °
H = DISTANCE PIVOT POINT OF LEAD TO BRACE - FT
S_B = SIDE LOAD OF BRACE AND BOOM - lb
M_B = MOMENT TAKEN ABOUT 'B' - ft-lb
R_B = RADIUS OF $W_{H+W_L+W_T}$ - ft
R_3 = RADIUS OF $W_L+W_{LB}/2$ - ft
R_{10} = RADIUS OF W_H+W_d - ft
FORMULA
$\Sigma M_B = 0$
$S_B = \frac{R_B(W_{H+W_L+W_T}) + R_3(W_L+W_{LB}) + R_{10}(W_H+W_d)}{H}$
$S_B \leq 0.02 W_L$ (AT THE SPECIFIED RADIUS FOR THE BOOM UTILIZED)
NOTE: IN EVENT $W_B < W_L$ THE BOOM MAY CARRY ADDITIONAL SIDE LOADING CONTACT CRANE MANUFACTURE

FIGURE 5

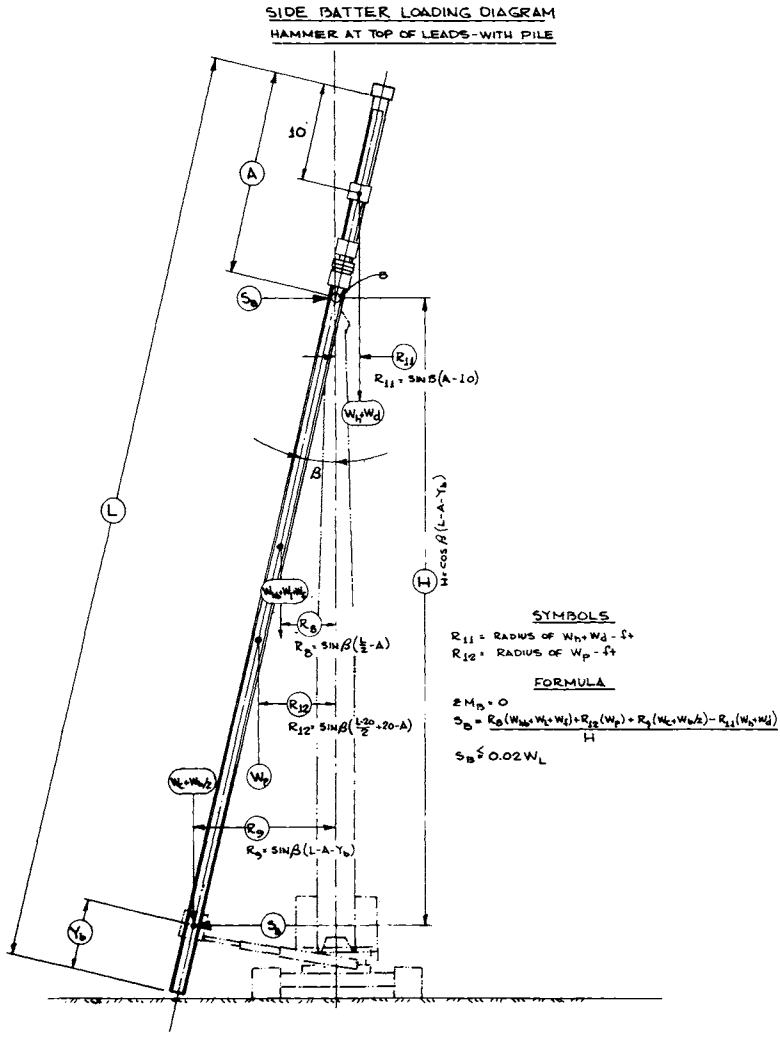

SIDE BATTER LOADING DIAGRAM
HAMMER AT TOP OF LEADS - WITH PILE
10'
A
L
H
B
S_B
R_{11}
$R_{11} = \sin\beta(A-10)$
W_h+W_d
$H = \cos\beta(L-A-Y_b)$
$W_h+W_d+W_L$
R_B
$R_B = \sin\beta(\frac{L}{2}-A)$
R_{12}
$R_{12} = \sin\beta(\frac{L-20}{2}+20-A)$
$W_L+W_b/2$
W_p
R_p
$R_p = \sin\beta(L-A-Y_b)$
Y_b
S_B
SYMBOLS
R_{11} = RADIUS OF W_h+W_d - ft
R_{12} = RADIUS OF W_p - ft
FORMULA
$\Sigma M_b = 0$
$S_B = \frac{R_B(W_h+W_d+W_L)+R_{12}(W_p)+R_p(W_L+W_b/2)-R_{11}(W_h+W_d)}{H}$
$S_B \leq 0.02 W_L$

FIGURE 6

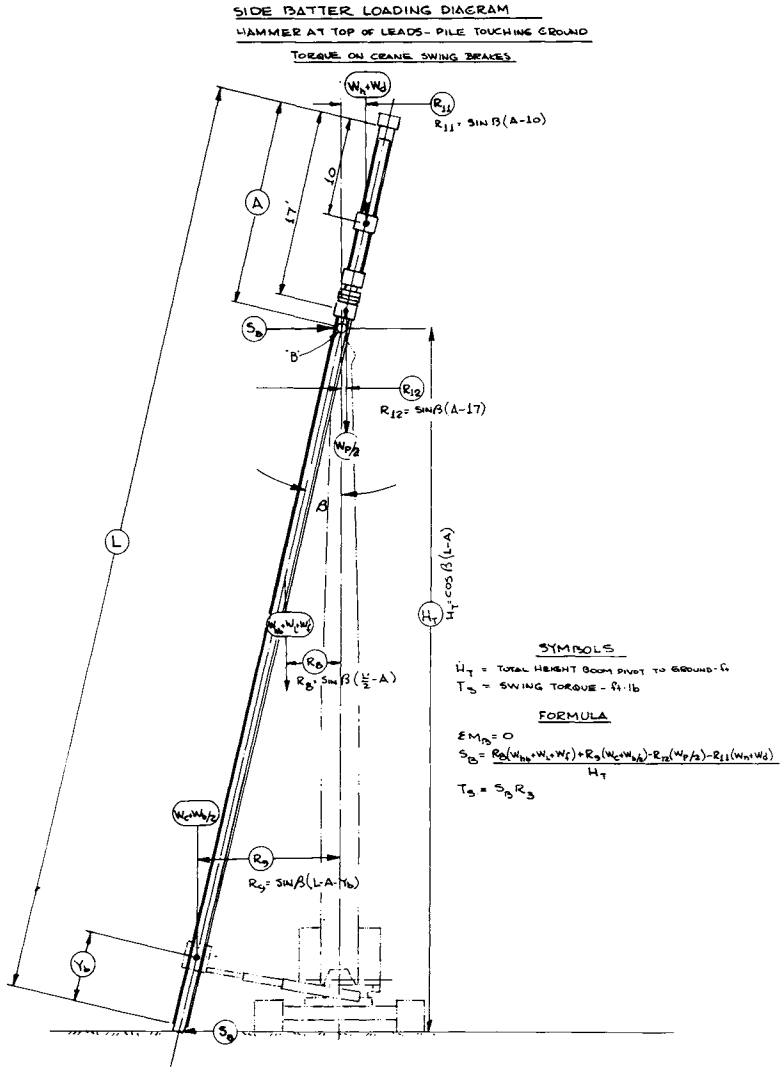
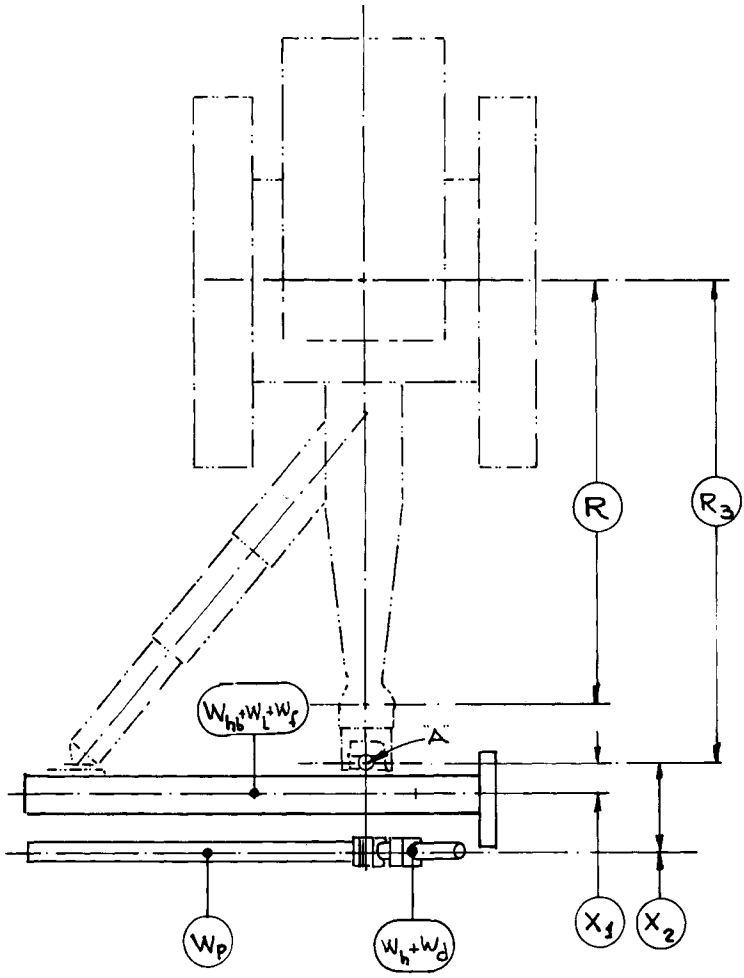

SIDE BATTER LOADING DIAGRAM
HAMMER AT TOP OF LEADS - PILE TOUCHING GROUND
TORQUE ON CRANE SWING BRAKES
SYMBOLS
FORMULA

FIGURE 7

SIDE BATTER LOADING DIAGRAM

1-HAMMER AT TOP OF LEADS - WITH PILE

ROTATIONAL MOMENT



SYMBOLS

X_1 = DISTANCE POINT "A" TO ℄ LEAD (2' FOR ST60/75 - 2.5' FOR ST100/150/190)

X_2 = DISTANCE POINT "A" TO ℄ PILE (4.5' FOR ST60/75 - 6.0' FOR ST100/150/190)

M_A = MOMENT ABOUT BOOM FT-lb

FORMULA

$$M_A = X_1 \sin\beta(W_{hb}+W_L+W_f) + X_2 \sin\beta(W_p+W_h+W_d)$$

FIGURE 8

TECHNICAL BULLETIN

FORE-AFT BATTER

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The loads induced on the crane boom from fore and aft batter are calculated by determining the equivalent tabular load at the specified boom radius.

The load for the fore and aft batter must be checked for the following conditions:

Condition 1. Figure 1 pg. 2 of 4 — The lead plumb with hammer, drive cap, and full pile weight. The logic of this is that the batter will never be achieved if the crane won't handle the weight plumb. This is not required if a handling crane is employed.

Condition 2. Figure 3 pg. 3 of 4 — The lead fore battered with the hammer and drive cap concentrated at the bottom of the leads as would occur when the hammer is raised off the piling it has just driven.

Condition 3. Figure 2 pg. 4 of 4 — The lead aft battered with the hammer and drive cap at the top of leads and the full pile in the leads.

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TECHNICAL BULLETIN

FORE AFT BATTER

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DATE August 1983

VERTICAL LOADING DIAGRAM

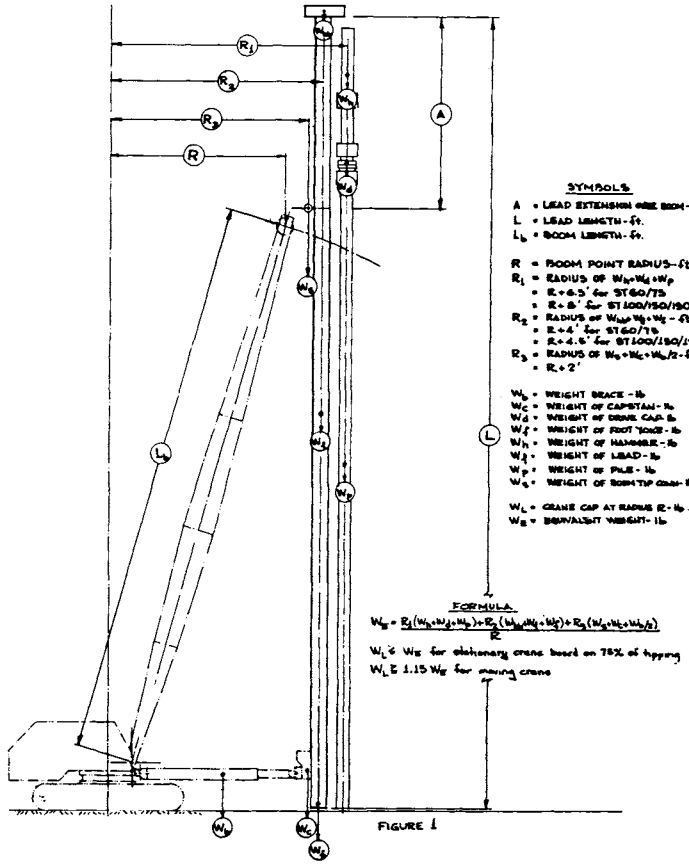


FIGURE 1

THE FOUNDATION EQUIPMENT CORP.

364 FLORENCE AVENUE
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DOVER, OHIO 44622
Telephone: 800-321-5539
In Ohio: 216-364-7521

TECHNICAL BULLETIN

FORE AFT BATTER

BULLETIN NO. 283 page 3 of 4

DATE August 1983

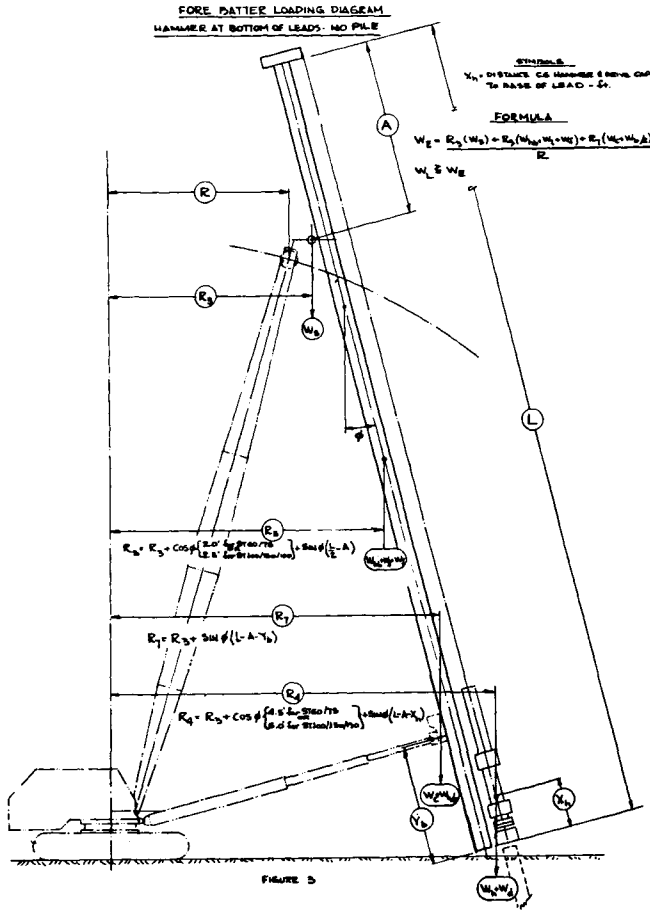


FIGURE 3

THE FOUNDATION EQUIPMENT CORP.
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BULLETIN NO. 283 page 4 of 4

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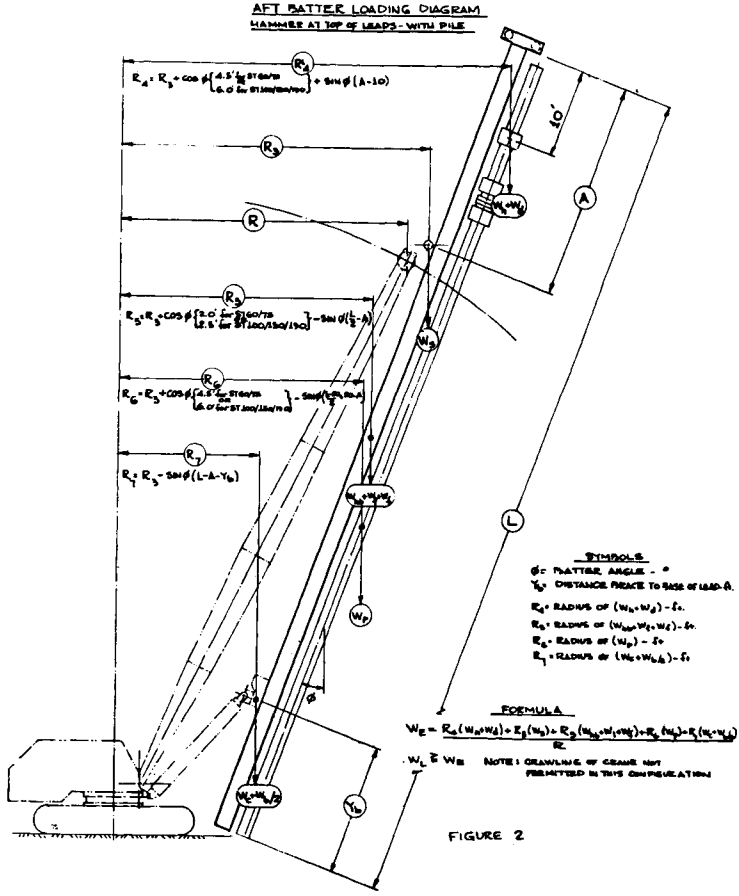


FIGURE 2

THE FOUNDATION EQUIPMENT CORP.
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TECHNICAL BULLETIN

BOOM LOADING

BULLETIN NO. 185 Page 1 of 2

DATE April 1985

The following data should be reported to the crane manufacturer when attempting to evaluate the capacity of the crane boom under side loading or compound loading.

1. Crane model and serial number
2. Boom length and boom number
3. Operating radius R
4. Loading conditions and values for the combined loads of W_E, S_B, M_A

a. Hammer and drive cap top of leads with pile

SIDE BATTER ONLY	COMPOUND BATTER
W_E = plumb with pile-lb	W_E = batter if greater than plumb - lb
S_B = with pile - lb	S_B = with pile - lb
M_A = with pile - ft-lb	M_A = with pile - ft-lb

b. Hammer and drive cap bottom of leads without pile

SIDE BATTER ONLY	COMPOUND BATTER
W_E = plumb without pile - lb	W_E = batter if greater than plumb - lb
S_B = without pile - lb	S_B = without pile - lb
M_A = without pile - ft-lb	M_A = without pile - ft lb

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Case History On Pipe Pile Installation

Jerry F. Parola, P.E., Vice President
Case International Corporation, Roselle, IL

**Presented at an Associated Pile & Fitting Corp. GEO-Pile Conference
San Francisco, CA**

In current practice, pile foundation design is based on a general understanding of pile-soil interaction during installation and superstructure-pile-soil interaction after construction. The pile design is a reflection of experience, precedent and judgment for normal piling practice of working load capacities up to approximately 100 tons with corresponding ultimate capacities up to 200 tons. For higher capacity piles, pile design is even less predictable. In particular, the feasibility of pile installation is an important design consideration that must be given the proper attention. An installation capability that is not given the required emphasis can cause unnecessary problems.

The pile must be installed to provide the required foundation behavior. Static driving (jacking) is impractical; therefore, dynamic driving is utilized. The pile-hammer-soil system controls the feasibility of any installation procedure. This is more important for higher capacity piles.

Considerations of pile installation in general can be characterized by the pile hammer as the driving energy source, pile as the transmitting element and soil conditions as the resisting force to penetration. The pile hammer is an energy source used to generate a force pulse in the pile, as shown in Fig. 1. This generated force pulse must overcome soil resistance to achieve penetration and capacity. The shape of the generated force pulse or energy must also be proper to achieve penetration. This energy source is shaped by the hammer properties of ram weight, stroke, cushion and energy losses.

The pile must transmit an incident force pulse which is capable of overcoming the soil resistance. The pile ability to transmit the force pulse is a

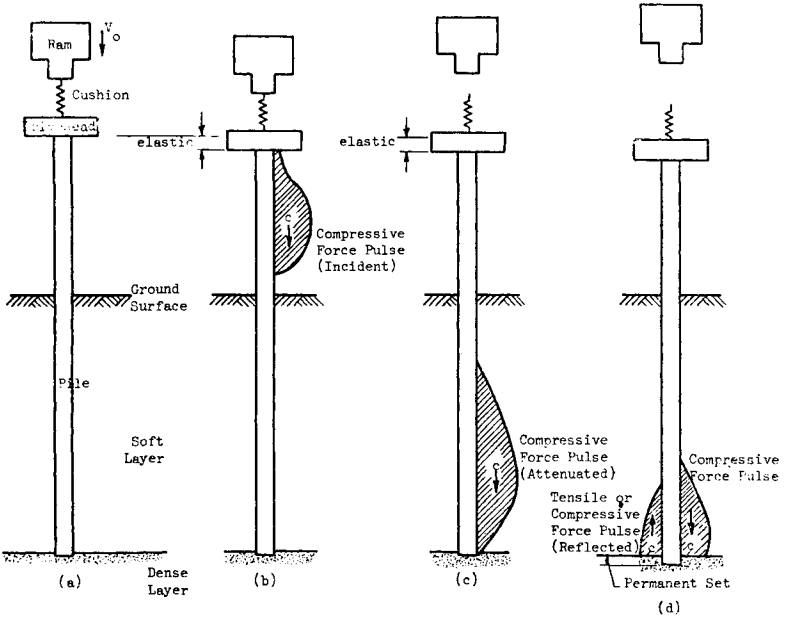


Figure 1 HAMMER-PILE-SOIL SYSTEM

function of pile impedance or area. Factors that affect the ability to transmit are structural damage, change in section or boundary conditions such as length. The soil characteristics which provide resistance to overcome include soil distribution, soil properties and freeze or relaxation. Freeze or relaxation can be defined as the difference in pile-soil behavior during and after pile driving.

To highlight the importance and feasibility of pile installation procedure, a unique case history is presented for the pile foundation of a high chimney stack at a power plant. The pile foundation required a 200 ft long pile, which was load tested to 700 ton.

Scope of Work

The pile foundation is for a 650 ft high concrete chimney stack at an existing coal power plant in Michigan. The effect of effluent discharge of the existing 170 ft high stack because of air-currents dictated the requirement of a new and higher stack.

The pile foundation for the chimney consists of 24 in. diameter concrete filled pipe piles driven to sound Marshall sandstone about 200 ft below grade. The 120 piles are driven in a multiple circular pattern with a working load of 300 tons and corresponding ultimate capacity of 600 tons. The pipe piles with 1/2 in. wall thickness, ASTM 252 Grade 2, were furnished by the owner in 50 ft lengths.

The site is a level, filled and graded industrial area. The upper 85 ft of soil profile consists of loose to medium dense silty fine sands or sandy silt with organic and lenses of organic and silt present. The standard penetration tests, N values, range from 3 to 32 blows per foot for the upper soil profile. A stiff brown and gray clay with traces of sand and silt and partings of fine sand is present below the upper sand and silt zone and extends to bedrock at about 200 ft below grade.

The stiff clay layer has a range of N-values of 10 to 21 blows per foot, with unconfined compressive strengths of 1.2 to 2.4 tsf, and water contents of 22 to 30 percent. The sound bedrock is Marshall sandstone at 200 ft below grade with a few feet of possible upper weathered zone. The water table is about 5 ft below grade. This soil profile is shown in Fig. 2.

The specifications to install the proposed pipe piles for the given soil profile are highlighted herein. All pipe piles shall be driven open-end with a minimum hammer energy of 35,000 ft-lbs with a final driving criterion of 15 blows for the last 3 in. This pile casing shall be seated on sandstone or dense material directly overlying the sandstone. After the pile casing has been driven to the specified resistance, the casing shall be cleaned of all soil and debris and redriven to a final resistance of 27 hammer blows per final 3 in. of penetration with a hammer energy of 26,000 ft-lbs. If another type hammer is used, the final driving resistance shall be determined in accordance with the Modified Engineering News Record Formula. After final seating, the pipe will be cleaned and filled with 5200 psi concrete or 5700 psi tremie plug concrete.

An open-prejet is allowed only to a maximum depth of 30 ft and jetting during driving will be restricted to removing soil plugs from within the pile only. Jetting shall not extend beyond the pile tip elevation during driving. Nearby silo structures on mat foundations restrict the use of jetting.

All splices shall be butt welded with 60 deg. bevel and 10 gage back-up plates. No pile tip reinforcement was specified. The location tolerances were 6 in. and 2 percent plumb.

Pile Installation Considerations

To achieve the required pile penetration to rock and pile performance for the chimney structure, pile installation equipment and procedure must be given proper attention. The construction considerations include the selection of pile hammer and effects on soil resistance during driving for the open-end pipe piles. THe upper soil profile of silts and fine sands can produce densification or compaction during driving and time effects on driving resistance such as freeze because of required splicing time on the long piles.

First, consider the selection of the pile hammer for the given pipe pile and soil conditions. The upper hammer energy can be approximated by the relationship of 2000 ft-lbs per sq. in. for steel piles. For the 24 in. diameter x 1/2 in. wall pipe with area steel equal to 37 sq. in., energy should be about 74000 ft-lbs, which is considerably greater than the required minimum hammer energy.

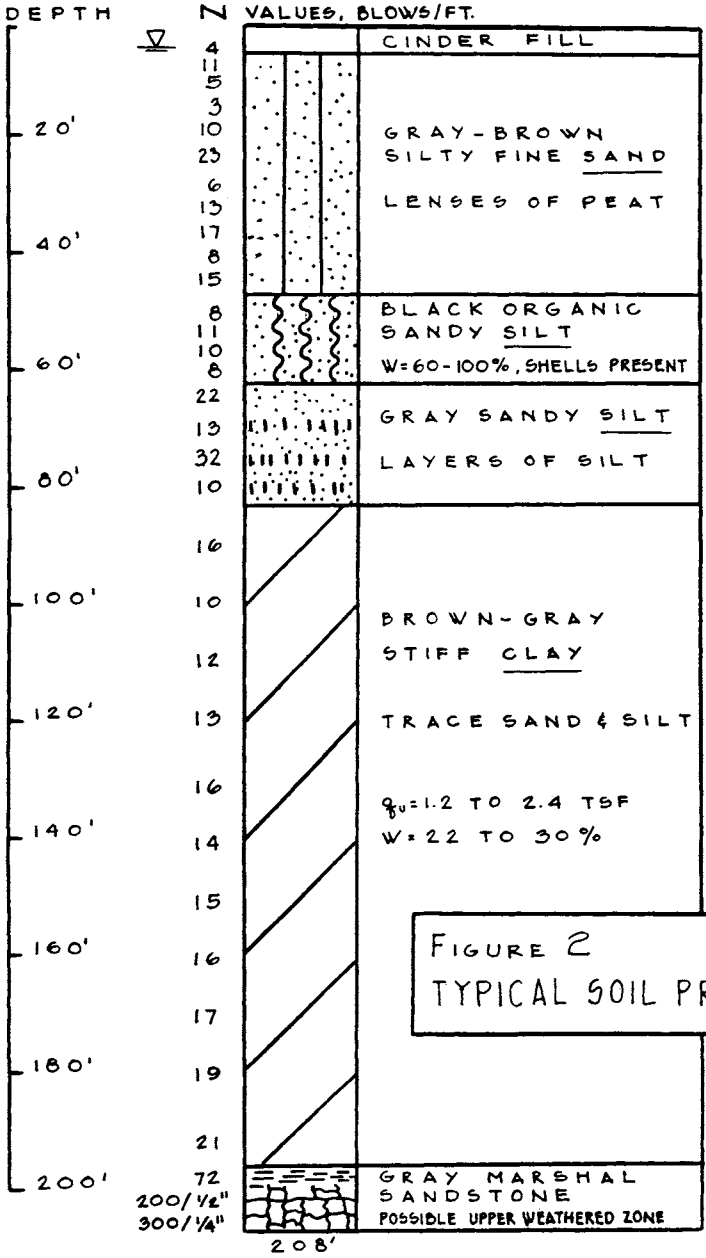


FIGURE 2
TYPICAL SOIL PROFILE

To investigate the driving capability of different hammers, a theoretical driving log is produced with the use of the wave equation analysis and static soil analysis. The wave equation is represented by a lump-mass and spring model as illustrated in Figure 3; considerable effort has been expended in this area with several references readily available. The wave equation theory simulates the actual pile hammer behavior.

With consideration of hammer energy in the range of 60,000 to 80,000 ft-lbs, the Vulcan 020 single-acting hammer with rated energy of 60,000 ft-lbs and the IHI-J44 single-acting diesel with a rated energy of 79,600 ft-lbs will be investigated. The wave equation analysis for the ultimate capacity and maximum stress versus driving resistance is shown for the two hammers in Fig. 4. The predicated ultimate dynamic capacity for the given pile is about 400 tons.

Unless the specified pile can be seated upon sound bedrock to act similar to a structural column, the pipe cannot be driven to the required 600 ton ultimate capacity. The recommended dynamic design stresses used for steel piles is 12 ksi; therefore, the supplied pipe pile has a designed load of 222.5 tons or ultimate driven capacity of 445 tons. This ultimate load is consistent with the wave equation results, and any difference attributed to the length effect.

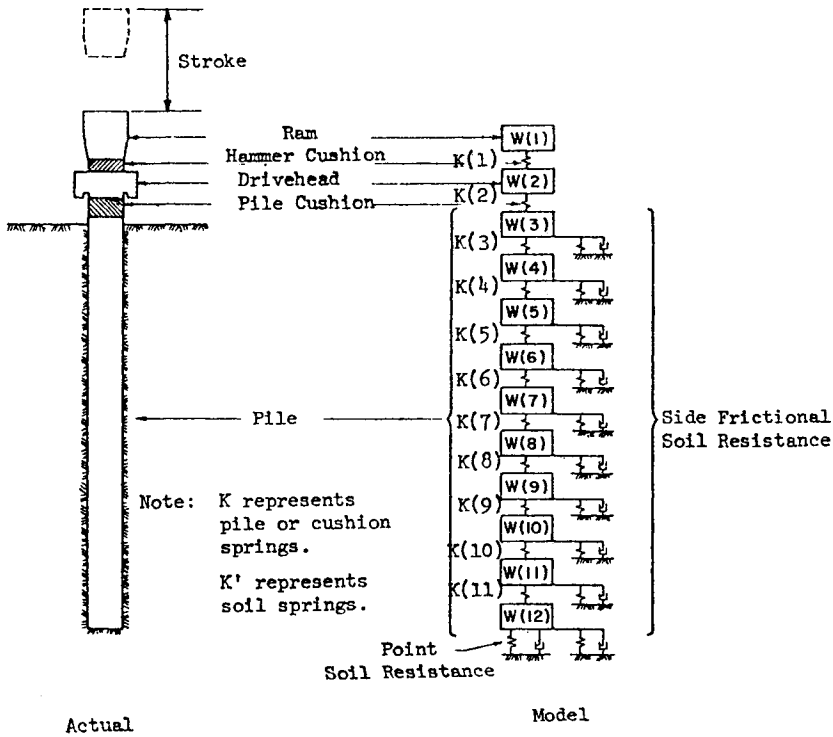


Figure 3 SMITH'S MATHEMATICAL MODEL

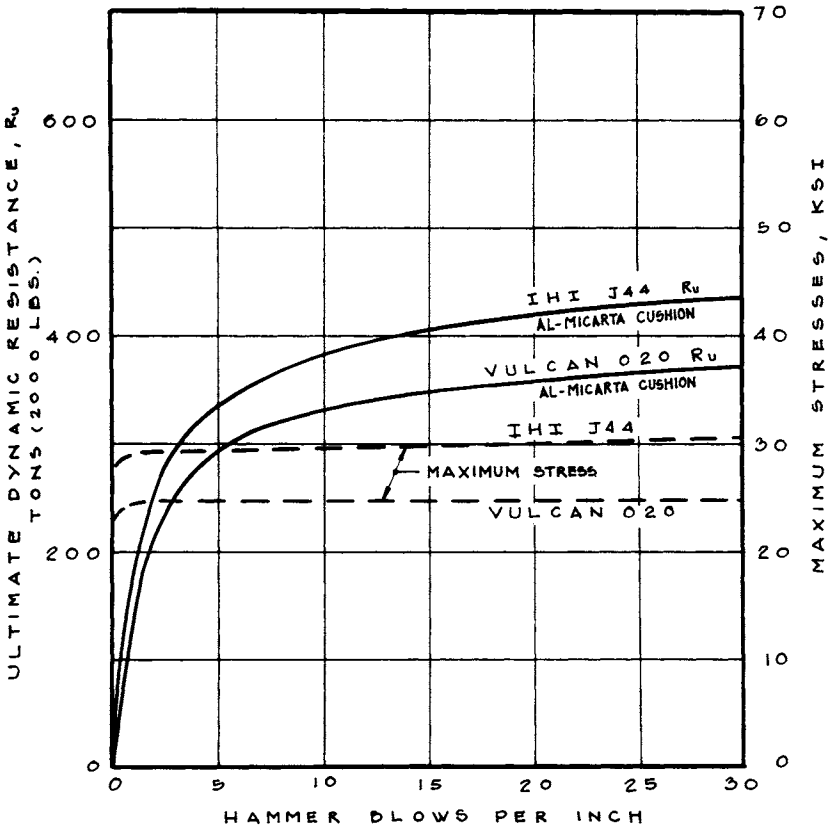


FIG. 4 WAVE EQUATION ANALYSIS

The static soil analysis coupled with the wave equation results can produce theoretical driving logs for the two hammers being considered with consideration of the tip resistance being aided by jetting, as shown in Fig. 5. It is apparent that the given pipe pile is not capable of overcoming the frictional soil resistance to achieve the desired penetration and capacity. The given pipe pile is obviously not capable of transmitting the incident force pulse required to overcome the soil resistance.

The driveability of long steel piles is limited by axial stiffness rather than yield stress of the material. The static soil analysis must account for the possible effects of compaction and the increase in friction strength with time as the result of splicing for the upper granular soils.

It is apparent from the possible restrictions to achieve the required penetration to rock, the pile driving will require the proper aid in order to overcome the soil resistance. Consideration of removing the soil resistance at the pile tip only during driving by jetting or other means is not sufficient to achieve penetration to rock. Also, jetting is restrictive because of adjacent silo structures founded on mat foundations. The pile improvement for drive-

ability could be considered in some cases; however, the pile material provided at the site cannot be readily improved for this project. Therefore, efforts should be considered to minimize the upper soil frictional resistance without detrimental effects so that the pile can be readily installed to rock.

The construction techniques of predrilling under a slurry head of the upper 100 ft and driving the pile to rock were a possible solution. With consideration of the IHI-J44 pile hammer because of better driving capability, the theoretical driving logs as shown in Fig. 6 can be constructed with the use of the wave equation analysis and the soil resistance. The theoretical drive log curve indicates that the pipe pile can achieve the required penetration.

The driving effect of slurry is bounded by the full effect of slurry, i.e., no upper 100 ft of soil friction and the partial effect of reducing the upper 100 ft of frictional resistance by one-half. The construction procedure of predrilling and slurry was used for this project. The pile driving for the original plant encountered long hammer times, freeze effect and use of and effect of jetting support the proposed construction procedure.

Pile Installation Procedure

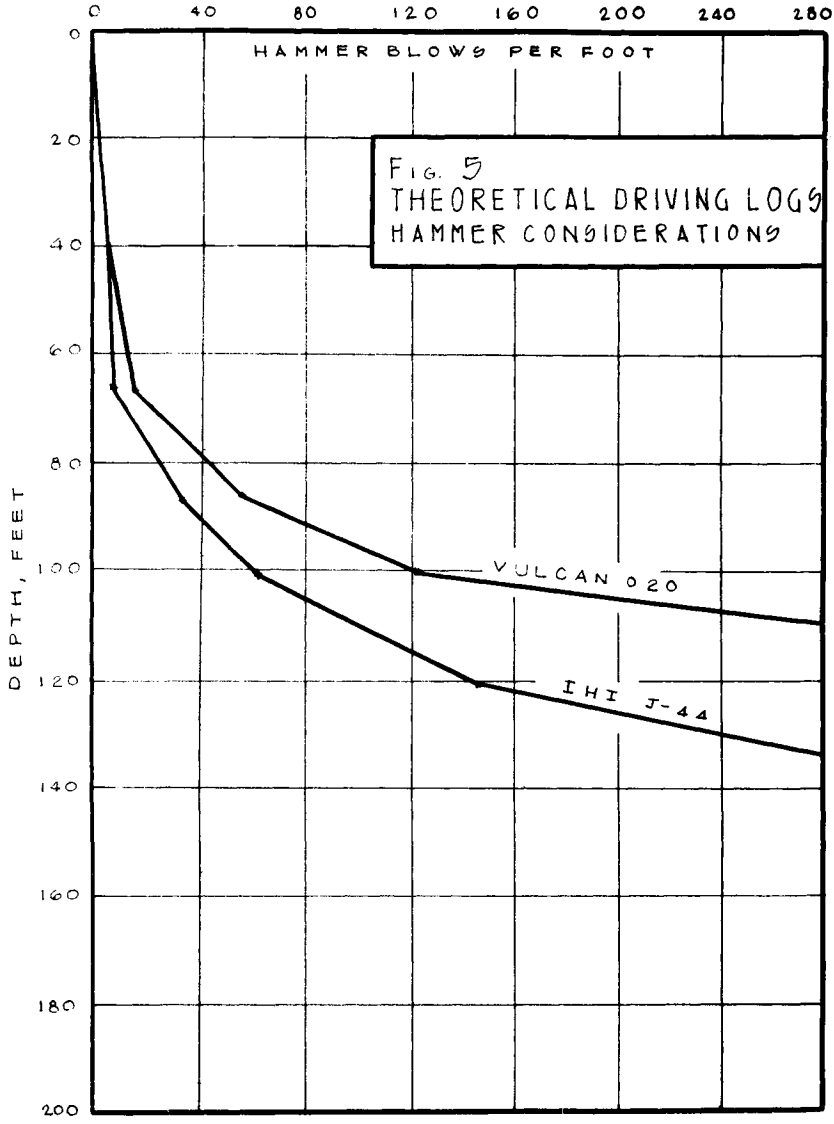
The construction procedure to successfully achieve the required penetration and capacity included augering a 24 in. diameter hole under a head of slurry to 100 ft depth. After completion of slurry hole, set a welded 100 ft pipe section and seat pile to nearly 100 ft with pile hammer. Weld on additional 50 ft section and drive to grade, then weld on final 50 ft section and drive to sandstone with a final resistance of 10 blows per inch for three consecutive inches with the IHI-J44 pile hammer. After driving is completed, mechanically clean with auger and kelly bar to pile tip or refusal and maintain water level in pipe pile. It should be noted that the pile tip was outside reinforced to ensure proper seating without damage, avoid section change for inside augering and reduce the skin friction.

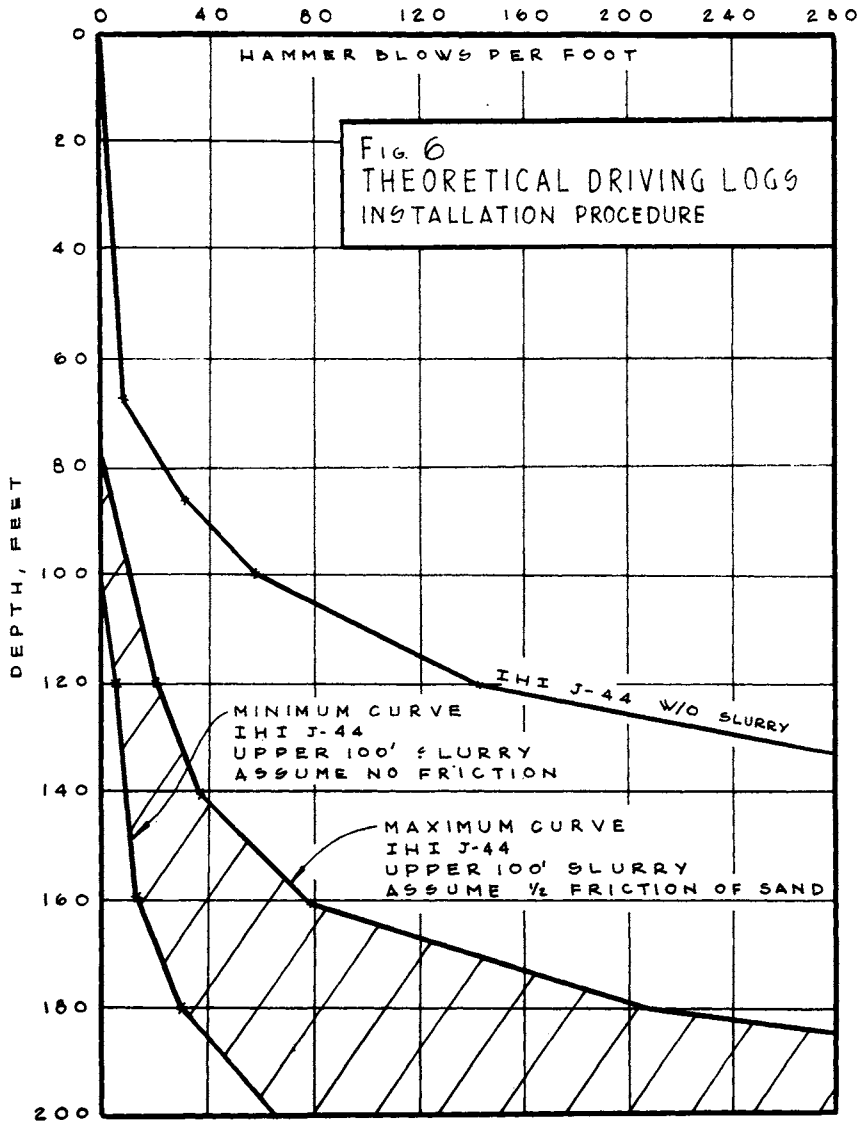
After mechanically cleaning the pipe, redrive the pipe pile with the IHI-J44 hammer to a final driving resistance of 15 blows per last inch. Clean pipe with air-lift or blow pipe and maintain a positive water head at all times. A 20 ft concrete plug was placed with a bottom dump bucket. Complete concrete operation to cut-off after cleaning above plug with blow pipe and pump out water.

The proposed installation procedure was successful and successfully predicted by the theoretical driving log. A comparison of the predicated and actual driving logs are shown in Fig. 7. The actual driving behavior corresponds favorably with the predicted results.

Load Test Results

The proof of the pile installation procedure is further verified by the performance of a pile load test. A pile was selected and load tested to a maximum of 700 tons.


0
40
80
120
160
200
240
280
HAMMER BLOWS PER FOOT
FIG. 5
THEORETICAL DRIVING LOGS
HAMMER CONSIDERATIONS
DEPTH, FEET
0
20
40
60
80
100
120
140
160
180
200
VULCAN 020
IHI J-44


0
40
80
120
160
200
240
280
HAMMER BLOWS PER FOOT
FIG. 6
THEORETICAL DRIVING LOGS
INSTALLATION PROCEDURE
DEPTH, FEET
0
20
40
60
80
100
120
140
160
180
200
IHI J-44 W/O SLURRY
MINIMUM CURVE
IHI J-44
UPPER 100' SLURRY
ASSUME NO FRICTION
MAXIMUM CURVE
IHI J-44
UPPER 100' SLURRY
ASSUME ½ FRICTION OF SAND

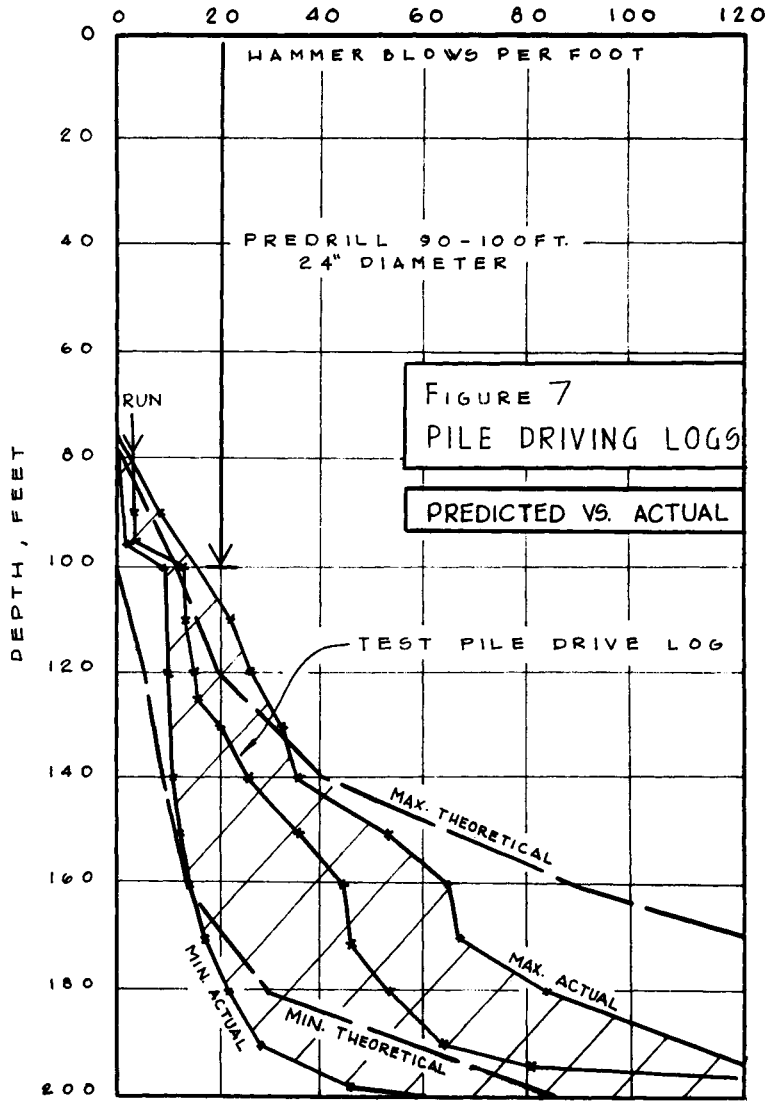
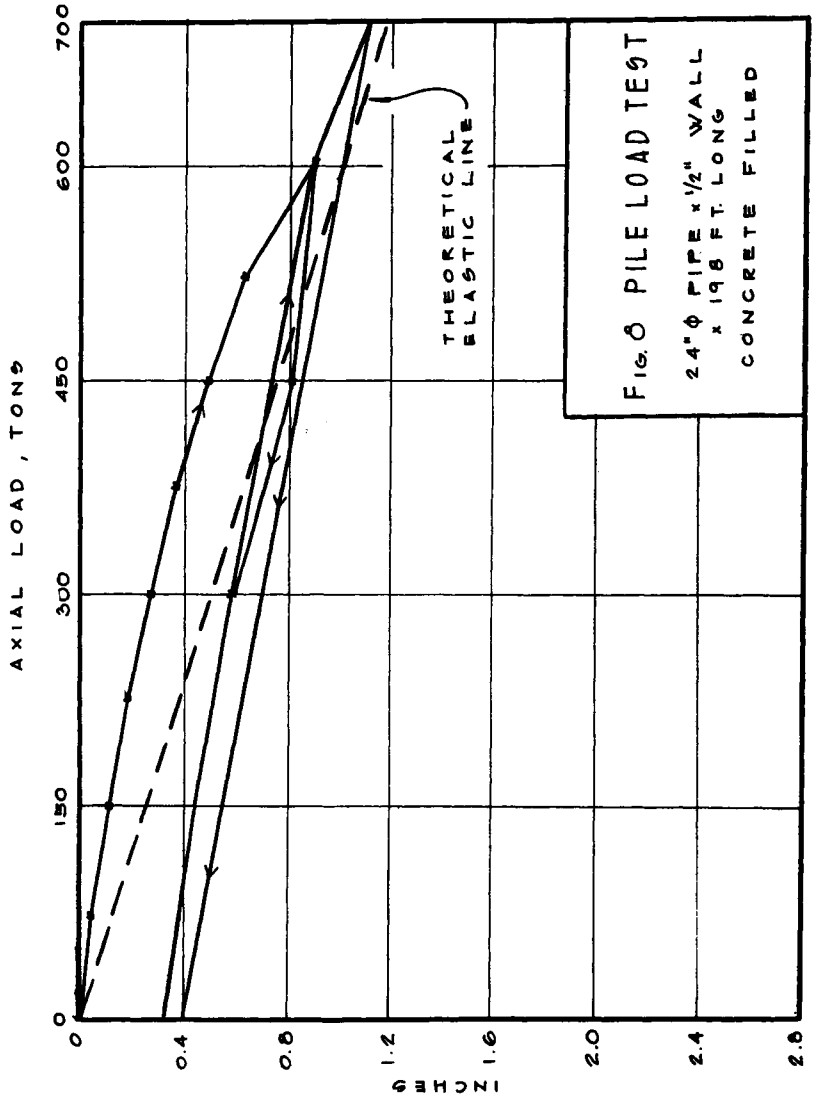


FIGURE 7
PILE DRIVING LOGS
PREDICTED VS. ACTUAL

AVERAGE AXIAL SETTLEMENT @ PILE HEAD



The pile load was applied by a calibrated hydraulic jack with the capacity of 700 tons acting against a reaction support of anchor piles and reaction frame. Adjacent production piles were used as reaction anchor piles. The settlement measurements include two diametrically spaced dial indicators with sensitivity of 0.001 in., level readings and draft readings at the pile head. The measurement reference frame was independent of the test pile or reaction piles.

The pile test load was applied in increments of 25 percent of the 300 ton working load (75 ton increments) to a maximum of 700 tons. Each load was maintained for one hour increment except the 600 ton load was maintained for 16 hours. After completion of the 600 ton load, the pile was unloaded in 150 ton increments. The pile was reloaded within one hour to 700 tons and maintained one hour before final unloading.

The test results of the pile load are shown in Fig. 8. These test results clearly indicate the pile is capable of supporting the test load of 700 tons, thereby supporting the proposed installation procedure. The theoretical elastic line of a free-standing column is plotted with the test results to serve as an index for interpreting the load test. It should be noted that several building codes permit up to 0.01 in. of net settlement per ton of ultimate load; therefore, the net settlement of 7.00 in. would be considered for the test load of 700 tons. It is obvious that some present building codes may not be applicable and must be used with care for interpretation of high capacity piles.

Summary

The feasibility of pile installation is an important design consideration that must be given knowledgeable attention. The case history presented herein typified the importance of the proper installation procedure to achieve the desired pile performance. The pile load test results as well as the ultimate proof test of the completed chimney structure successfully verify the pile performance of this case history. It should be cautioned that considerations of installation procedure are more necessary for higher capacity piles.

Understanding Soils Reports

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Presented at a Structural Engineers Association of Northern California Continuing Education Seminar on Foundations/Excavations, March 1984

Preparation, understanding and use of soils reports must be a team effort to be fully effective. The soils engineer must know what is to be built and exactly where. The structural engineer and owner must be fully informed on the foundation problems of the site. There must be frequent team review of information obtained from subsurface investigation and of design as details develop. The soils engineer should be available on a continuing team basis as design proceeds and should be directly involved with inspection and interpretation of site conditions as construction continues.

Communications — The Soils or Geotechnical Report is the primary means of communication among the soils engineer, the Owner and other members of the design and construction team. The report must be complete and clearly written if it is to provide effective communications.

Guidance for Structural Engineer — The structural engineer uses the recommendations in the soils report to design foundations, walls, pavement and the like.

Information for Contractor — Despite all the disclaimer clauses that may be written in the report or the project specifications, the soils report will give the contractor his primary information regarding subsurface conditions. The report will play an important role in any claims for changed conditions or construction difficulties.

Working with a Soils Engineer

Selection of Soils Engineer — A soils engineer should be selected for his experience and his qualifications. Previous experience on adjacent or nearby parcels can be an advantage. The soils engineer should be one who communicates well with the structural engineer. Beware the trend of selection based

on the low bid—as the low bidder has probably reduced field drilling and other services that will be needed for a prudent and economical design.

Scope of Work — The soils engineer's scope of work should be determined at a meeting with the structural engineer. The soils engineer needs to know the type of building being designed, the depth of possible excavation and the design recommendations required by the structural engineer. The number of borings should cover the building site and relate to previous nearby borings. Soil borings must be deep enough to sample all significant bearing materials and sampling must be frequent so enough tests can be performed in the laboratory to determine strength and deformation properties of the soils.

Progress Meeting — After the soils engineer has completed the field borings and laboratory testing and is well along with his analysis and recommendations, a meeting should be held with the structural engineer. The structural engineer will update the soils engineer on the design progress of the project while the soils engineer presents a preview of his recommendations. Potential settlement of various foundation solutions is discussed. Any other potential problems are discussed. At this meeting the foundation system is usually selected by the structural engineer.

Review of Draft Report — The structural engineer should always review a draft of the soils report before it is issued. He has to live with all of its recommendations; if he does not like what the report says, he should discuss the draft with the soils engineer before it is finalized. Reviewing a draft adds a few days to the process, but it is usually well worth the time.

If a soils report was done previously for an earlier project, or for other reasons, the structural engineer must insist that the soils engineer be engaged for further consultation. A previous soils report can seldom properly present recommendations for a new building concept; sometimes even more borings or laboratory tests may be required.

Use the Soils Engineer's Recommendations — It is the responsibility of the structural engineer to take the recommendations of the soils engineer and transform them into plans and specifications. If things are ignored or omitted, the structural engineer may assume liability for not following the soils engineer's recommendations. Send foundation plans and specifications to the soils engineer for his review after design and before construction. Make sure his recommendations are interpreted correctly. Welcome his comments and constructive criticism.

Follow Through During Construction — It must be recognized that the soils engineer extrapolates information from a few borings to cover recommendations for the entire site. He must be engaged during construction to inspect footing excavations, pile installation, backfill operations, etc. Soils will vary and special precautions will be necessary. The soils engineer who prepares the design recommendations should provide Special Inspections for work covered by his recommendations.

CONTENTS OF SOILS REPORT

Factual Information — must include:

- Description of Site
- Scope of Work
- Boring Logs
- Soil Profile
- Test Results

Foundation Design Recommendations

Recommendations are probably the most important item in the soils report. If spread footings or a mat is anticipated, allowable bearing values for various load combinations will be given. If piles or caissons are anticipated, driving or drilling criteria should be given as well as all pertinent information, including indicator pile programs, load tests, different types of piles, and the like. These recommendations must be correlated with anticipated settlement discussed in the next section.

The constructibility of the foundation system must be addressed. Can it be built practically? Can the material be excavated? Can piles be installed? Can caissons be drilled?

Are the design recommendations suitable? Does the soils engineer understand the loads which are critical for foundation design? For example, seismic overturning loads may be governing foundation design when combined with gravity loads based on a one-third increase in bearing loads. If asked, the soils engineer may be able to justify a 50 percent or greater increase in values for earthquake due to the very short duration of that loading. The soils engineer can provide the structural engineer better values for design if he understands the situation and any potential problems.

Estimation of Settlement

When comparing alternate foundation systems, the most important factor is the potential settlement. The selection between spread footings, mats, short piles, long piles, etc., basically depends on how much settlement can be tolerated.

Differential settlement is really of greater concern to the structural engineer. If the whole building settles uniformly 2 in., the structural enginner has no problem provided he coordinates drainage, access and utility connections with the architect. However, if there is 1 in. of differential settlement between adjacent footings, the structural engineer usually has a major problem!

If building settlements are going to be large, the effect of those settlements on adjacent buildings must be considered. Damage to adjacent property is possible and must be considered. When adding on horizontally to an existing building, the original foundation solution may not be appropriate. For example, consider the original building on spread footings which settled 1 in. An addition with spread footings would have to be built 1 in. high in

hopes of eventual settlement or it would end up 1 in. low. A better solution is probably a mat, strip footings, or piers or piles.

The structural engineer and soils engineer must understand where each is coming from on the problem of settlement. The structural engineer must understand that all buildings settle. He must understand that the soils engineer considers only the soil when calculating settlement, and variations in settlement assume zero stiffness of the structure above, which is far from reality. Soils engineers have the tools to reasonably predict average settlement, but they guess a bit at differential settlement as it is influenced by subtle or gross changes in soil conditions under various columns. Believe the estimates as you will and question the soils engineer on how confident he is in the estimate, his assumptions, etc. It is essential that the right answer be reached to control settlement. Big dollars can be wasted on excess foundations to minimize settlements, but bigger dollars can be spent to correct excessive settlement. No one wants a Tower of Pisa in their resume. Be careful and prudent!

Lateral Earth Pressures on Walls

The soils engineer will provide lateral earth pressure estimates for design of basement walls or cantilever retaining walls. Values will be at rest or active pressures plus possible surcharges and water pressures or some variation thereof. It used to be 30 pcf worked fine for most everthing; but. thanks to a few problems and lots of litigation, higher design pressures are common. In most cases, the higher loads can be easily handled. They result in somewhat heavier walls that do not crack as much. However, if loads become unmanageable, question the soils engineer on solutions. There may be a large factor of safety included, which only compounds with the factor of safety provided in the structural design.

Passive pressures are sometimes considered to control sliding of cantilever walls. Recognize that soils engineers traditionally place a factor of safety on passive pressures but that fairly large movements may be required to mobilize these pressures.

Ground Water Considerations

Ground water levels and their estimated variation must be included in the soils report. A conservative design level should be given and followed in design.

If the building extends into the permanent water table, the basement slab must be designed for hydrostatic uplift and the building must be checked for buoyancy. Alternatively, a pumping system can be designed to lower the water table permanently beneath the building. This is best achieved when cut-off walls can be constructed down to an impervious strata. Flow rates and backups for power failures must be considered.

Permanent or temporary pumping to lower the ground water must be considered carefully. Lowering the ground water can cause settlement in adjacent properties or streets due to various causes. Be cautious!

Dewatering may be necessary to preserve soil conditions. Some local soils when excavated and not dewatered turn from firm soil to muddy mush. Other sites may require dewatering of sand layers beneath such strata as bay mud to prevent bottom heave of the excavation.

Seismic Considerations

Liquefaction potential or other geologic hazards may exist. They should be included in the scope of soils work and adequately evaluated. With current knowledge in these areas, there is little excuse if potential hazards are ignored. Question the soils engineer if sandy soils exist below the water table. Recommendations for mitigation may be needed.

Site seismicity and response spectra may require evaluation. For major structures, this information is frequently provided by the soils engineer for design consideration.

Passive pressures for foundation can be important. It has become common practice in recent years to include in soils reports values for passive pressure on foundation and pile bending values to "take the earthquake forces out of the structure." These values have been requested to complete analysis procedures and are required by some regulatory bodies.

In many projects, great efforts are made to prevent this "sliding of the building." There is fallacy in this procedure. It is not necessary to take forces out of the building. What really happens is the ground moves and the building tries to follow. If there is an imperfect soil-building connection, energy is effectively dissipated. This is a form of base isolation. If the force can't get out, it can't get in.

One reason our buildings perform so well in actual earthquakes in this lack of firm connection to the ground. A recommendation is to forget the passive pressure game, provide strong foundation ties to keep the building together at its foundation.

Do not use batter piles or similar means to ensure that the full earthquake forces get into the building. Furthermore, if piles or caissons are used, provide ductile detailing in the region below the pile cap. This is essential at major discontinuities in soil stiffness where bending of the piles will concentrate when the soil moves and distorts during the earthquake and tries to do the same to the piles. Codes are very deficient in this area.

Paving Recommendations — Paving is really a structure of its own; most structural engineers do not understand all the implications. The soils engineer should recommend subgrade preparation and base coarse material. In most cases, the soils engineer should provide pavement design and specification requirements; he is probably the most knowledgeable person on the design team in this area.

Backfill Recommendations — The soils report should contain backfill recommendations and discuss suitability of materials. Can on-site materials be used for backfill? Which ones? If imported materials are required, specifications should be given. Compaction requirements should also be included.

Engineered Fill – On many projects, particularly low-rise or industrial projects, the foundation solution is to excavate to a certain depth and install engineered fill. This may control expansive soils or other potential problems. The soils engineer should give full procedures and recommendations, including material suitability and compaction requirements.

Soil Parameters for Shoring – Shoring design, when required by project layout, is usually the responsibility of the contractor since it involves construction techniques. However, soil design parameters are required for shoring design. It is common to utilize the same soils engineer for shoring design recommendations as for the basic building since that engineer is already familiar with the soil properties, etc.

Consultation by the soils engineer can include: control of ground water and its effects, dewatering recommendations, design pressures for bulkheads including surcharges of adjacent structures and street traffic, passive pressure and earth berm recommendations, tieback friction resistance, underpinning bearing values, etc.

CONCLUSION

The soils report is a communication tool whereby the soils engineer explains his recommendations and concerns for the project. The soils report will only be as good as the soils engineer and structural engineer make it. Talk out the problems! Question why while talk and discussion are possible. The structural engineer must thoroughly review all items and read the draft report carefully. Get it right! As members of the design team, all need to work together closely to obtain economical and sound designs. The structural engineer can only understand and fully utilize the soils report if he has contributed and reviewed it throughout the process.

Long Term Behavior of Piles

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Longevity of structural units, as compared to the planned useful life of a facility, always is a design consideration. Typical concerns include slow decay due to environmental factors, and changes in constitutive relationships governing the performance of the material involved. With respect to foundations, the usual items of interest are corrosion of steel and concrete, and time dependent behavior of the soil/structure system. For piles, as with other foundations, deterioration is precluded, mitigated or otherwise accounted for by assessing properly the ambient situation (sulphates, pH, stray currents, resistivity) and providing safeguards, such as resistant construction products, oversizing, coatings, and/or cathodic protection. On the other hand, long term performance of the pile foundations, as affected by time dependent characteristics of the supporting soil or rock, can be predicted and controlled only with a synergetic coupling of analyses and field tests. The history of a project for which this approach was followed forms the basis for this paper.

In reality, there are only two concerns pertinent to design of pile foundations: tolerable settlement and adequate safety with respect to failure. These can be and are considered in a variety of ways including: Building Code rules or guidelines, theories based on dynamic driving phenomenon, back calculation from the results of load tests, and computations based on static bearing capacity theories. It is the writers' opinion that a combination of static theory and load testing is the most reliable method, especially with respect to allowing prediction of both short and long term behavior. Thus, they advocate the following approach to design of pile foundations:

- define stratigraphy with an appropriate number of borings.
- determine soil properties, especially unit weight, undrained strength (if applicable), angle of shearing resistance (if applicable), modulus of deformation, and pore water pressure/stress relationships.
- decide on pile types, sizes and lengths to be considered.
- for these, compute ultimate capacity using:

Equation 1

$$Q_{ultimate} = Q_{base} + Q_{shaft}$$

Equation 2

$$Q_{ultimate} = (qN_q \text{ or } S_uN_c)\, A_{base} + \int_0^L (a \text{ or } K_s \bar{\sigma}_v \tan \delta)\, A_s dl$$

It is important to note the or. This indicates that S_uN_c and a are used to study undrained behavior while qN_q and $K_s\bar{\sigma}_v \tan \delta$ apply to drained conditions. One or the other apply, not both.

where:

$Q_{ultimate}$	=	the total load required to cause bearing failure
Q_{base}	=	that part of the resistance to failure developed at the base of the pile
Q_{shaft}	=	that part of the resistance to failure developed along the shaft
q	=	the mean normal effective stress in the soil at a depth equal to its embedded length, i.e. $(\bar{\sigma}_v + 2\bar{\sigma}_h)/3$
N_q & N_c	=	bearing capacity factors for deep foundations
S_u	=	the undrained strength of the soil
A_{base}	=	the area at the base of the pile
a	=	the soil adhesion, assumed equal to S_u for soft to medium firm soils and about half S_u for stiff soils
K_s	=	a pile/soil interface earth pressure factor, assumed equal to 1 for driven piles and 1/2 for bored piles
δ	=	the coefficient of friction at the pile/soil interface, assumed equal to ϕ, the angle of shearing resistance
A_s	=	the surface area of the shaft of the pile per unit length, dl
L	=	the embedded length of the pile
$\bar{\sigma}_v$ & $\bar{\sigma}_h$	=	the effective vertical and horizontal stresses in the soil

- select a pile type and size and a range of lengths to be considered based on economic analyses, i.e. cost per ton capacity comparison.
- pick an installation system and criterion to suit the pile and the stratigraphy. Use wave equation or similar dynamic analyses to optimize this selection and to estimate driving stresses.
- if appropriate, install piezometers to measure pore water pressures that develop during pile driving.
- drive several test piles, each to different tip elevations so that the full range of embedded length used in analyses is represented.

- load test the piles to failure in compression and then in tension. Based on the results of these tests, select the length of production piles.
- assign factors of safety for compression and uplift and determine the design loads per pile.
- install the production piles using the verified lengths and the criterion used for installation of the test piles.

Variations on this general theme are, of course, utilized. For example, results from the first test can be used to determine the allocation of available resistance between base and shaft and to confirm data/assumptions on "a" or $K_s \tan \delta$ and ϕ. In some cases, this may lead to redirection with respect to selection of embedded length for the other test piles and the production piles.

Pile Performance – Short and Long Term

In essence, the difference between short and long term behavior of some piles is change with time in their ability to sustain load. Put simply, the capacity of some piles increases with time, while that of others decreases. This phenomenon has been studied extensively and from many perspectives. In fact, it has given rise to new (at one time) words in the geotechnical engineer's vocabulary, e.g. set, relaxation, and freeze. Researchers opinions have been far from unanimous with regard to causative agents. Some blame soil or rock creep. Others believe that the primary influencing factor is pore pressure change. A few think that pile driving hammers get "tired". All agree that dragdown forces, if present, are contributing factors. It is the writers' opinion that for piles supported in soil, the difference between short- and long-term performance is due primarily to a transition from initial undrained to eventual drained behavior of the supporting soil. Thus, a pile driven in clay may lose capacity with time as the strength of the supporting soil changes from undrained to drained. The same can be true in dense sands in which negative excess pore water pressures, and thus higher effective stresses, can develop during driving. As these dissipate, the pile loses capacity. In contrast, in loose to medium sand, positive excess pore water pressures developed during driving render the initial soil strength lower than the eventual strength; i.e., the strength after pore pressures have dissipated. Thus, the pile gains capacity with time. The following case history illustrates this situation.

CASE HISTORY – NEWPORT NEWS, VIRGINIA

Two coal terminals are being constructed at sites located as shown in Fig. 1 adjacent to the James River waterfront in the southern section of Newport News, Virginia. Each complex includes coal piles, reclaimers, car dumpers, transfer towers, and a variety of appurtenant facilities.

Subsurface Conditions

In general, subsurface conditions at the sites can be categorized into nine separate strata. General descriptions of the strata are as follows:

Stratum 1 — Loose (near surface) to medium dense, granular fill of variable composition, usually fairly clean silty sand, often yellow or orange brown in color. This material is found from the surface to depths of up to 25 ft over much of the onshore part of the site. With the exception of occasional fragments of wood or some limonite staining, it is not easily distinguishable from natural granular soils often found underlying the fill.

Stratum 2 — Soft, black organic silts or clays, occasionally sandy. These deposits are found in distinct layers or pockets over much of the reclaimed land portion of the site; i.e., between the existing shoreline and the existing Harbor Road. They appear to be present throughout the river portion of this site.

Stratum 3 — Medium dense, brown to orange brown, coarse to fine silty sand. This formation, probably alluvial in nature, is found beneath most of the onshore part of the site; but is not encountered offshore. Typically, Stratum 3 is found underlying Stratum 1 or Stratum 2.

Stratum 4 — Usually loose or soft, black or gray silty sand or sandy silt, frequently found in conjunction with Stratum 2. In origin, this deposit may be river silt from existing or ancient buried channels. Occasionally it is found as a fill, especially in areas near the present shoreline or behind the existing bulkhead.

Stratum 5 — Soft, gray or green, inorganic clays and silts, probably of marine origin, sometimes interbedded with fine sands, frequently containing gas pockets, encountered near shore and offshore.

Stratum 6 — Medium dense to dense, gray-green, silty or clayey fine sand with abundant shell fragments, known regionally as the Yorktown Formation, found throughout the site, onshore and offshore, to great depths, on the order of more than 200 ft.

Stratum 7 — Fill of very variable composition, density and/or consistency, erratic in constituents, but often sand, gravel, coal and cinders. Found primarily near shore, especially behind the existing bulkhead. Often similar to Stratum 4.

Stratum 8 — Very stiff green clay, found underlying the Yorktown Formation. Probably part of the Calvert Formation.

Stratum 9 — Loose or soft dredge spoil or hydraulic fill composed of materials similar to those of Stratums 2 and 4, usually randomly mixed.

A stratigraphic profile at the site is shown on Fig. 2.

Foundations

Analyses indicated that settlement of many of the more critical or sensitive structures could be held within tolerable limits only by supporting them on piles. Based on anticipated loads, structural configurations, and economic analyses, 12 in. diameter, concrete-filled, steel pipe piles, driven closed

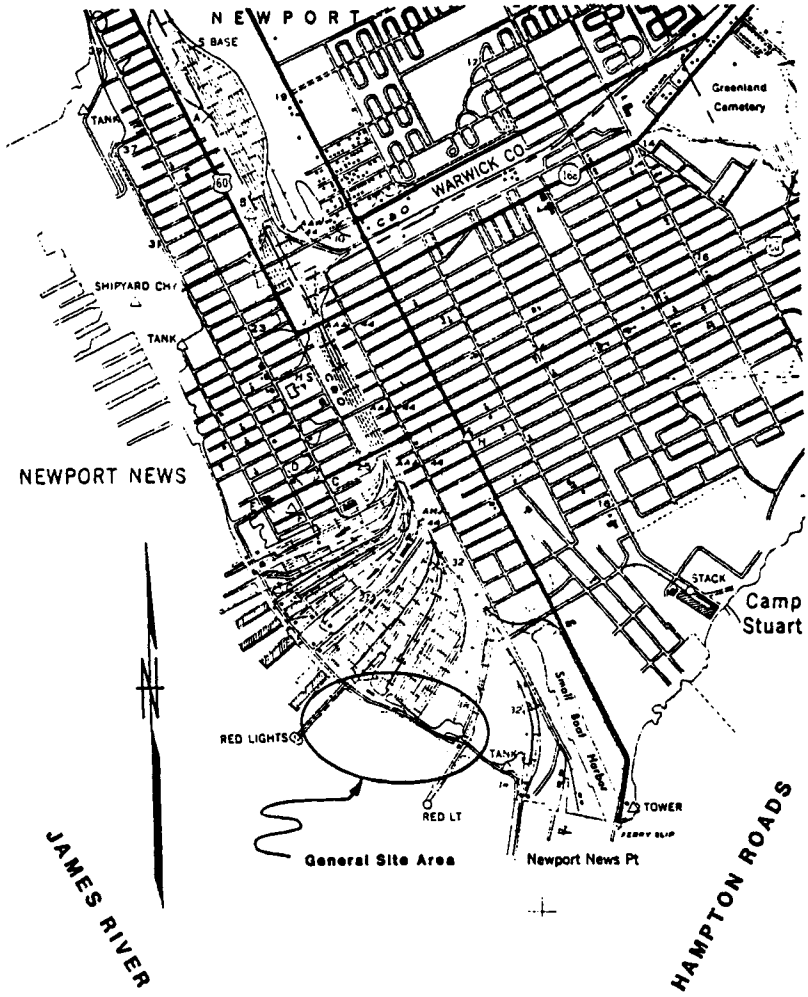


FIGURE 1 - PROJECT LOCATION

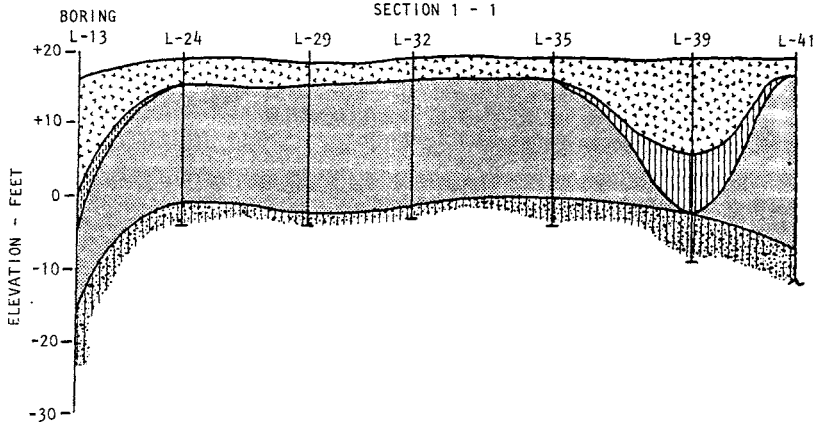
ended, were chosen at both sites. For optimal uses, these were to be designed to sustain compressive loads of 120 tons at one site and 75 tons at the other, and uplift loads of 50 tons at both sites. Factors of safety of 2 in compression and 3 in uplift were used. As soils above the Yorktown Formation are loose and somewhat variable across the sites and dragdown forces sufficient to negate any positive shaft friction above the Yorktown are possible, the lengths of embedment required were computed based on base and shaft resistance available in the Yorktown only.

Two Sites — Two Designers — Two Designs

As the two terminals are separate projects with different owners, designers, geotechnical consultants, and contractors, different procedures for design and installation of piling evolved. Boring data at both sites indicated that the results of the Standard Penetration Tests (SPT) in the Yorktown were quite low, averaging around 20 and sometimes near 0, even at great depths. At one site (say Site A for purposes of designation), the geotechnical consultant interpreted this to indicate very loose conditions and thus he anticipated significant dragdown loads and recommended casing the piles above and driving them to fairly great penetrations into the Yorktown. At the other site (say Site B for purposes of designation), the writers, based on laboratory test data, visual observation of Yorktown samples, and experience with similar weakly cemented fine silty sands, formed the opinion that the low SPTs were caused by high positive pore water pressures developed during driving of the sampling spoon. Thus, they anticipated dragdown forces only on the order of 30 tons and did not foresee a need for casing. Further, based on static bearing capacity calculations, they estimated that the required design loads of 75 tons compression and 50 tons uplift could be supported with suitable factors of safety (2 in compression and 3 in uplift) by driving the piles to predetermined tip elevations corresponding to embedment in the Yorktown of about 40 ft. As tip elevation was established as the control, no other driving criterion was suggested. Based on anticipated driving stresses, estimated by wave equation techniques, a Vulcan 010 hammer with a rated energy of 32,500 ft-lbs was recommended.

Installation of Test Piles — Site A

At Site A several test piles were installed to establish driving criterion suitable to install piles capable of supporting the design load of 120 tons compression with a factor of safety of at least two. A Vulcan 010 hammer was used and various dynamic formulae were tried, but in all cases the piles ran very long and never "fetched up." Thus, the designers and geotechnical consultant decided to try a tip elevation criterion. Additional test piles were driven to penetrations of about 50 ft into the Yorktown Formation. Final resistance was very low, as low as 5 or 6 blows per foot.



LEGEND:

- GRANULAR FILL OF VARIABLE COMPOSITION, USUALLY FAIRLY CLEAN SILTY SAND, OFTEN YELLOW
- BLACK OR GRAY ORGANIC SILTS OR CLAYS, USUALLY SOFT
- BROWN TO ORANGE BROWN COARSE TO FINE SILTY SAND, SOMETIMES DIFFICULT TO DIFFERENTIATE FROM CLEAN GRANULAR FILL
- BLACK OR GRAY SILTY SAND OR SANDY SILT, OFTEN FOUND IN ASSOCIATION WITH ORGANIC SILT, MAY BE RIVER SILT FROM EXISTING OR PREVIOUS CHANNELS
- MEDIUM DENSE TO DENSE GRAY GREEN SILTY OR CLAYEY FINE SAND WITH SHELL FRAGMENTS (YORKTOWN FORMATION)
- BOTTOM OF BORING

FIGURE 2 - TYPICAL SUBSURFACE SECTION

The piles at Site A were expected to sustain 240 tons in compression; i.e., twice design load. Those tested three days after installation failed at about 180 tons. Tests run two weeks after driving indicated ultimate capacities of 225 tons. Two months after they were driven, piles were loaded to 280 tons without showing signs of bearing capacity failure.

To the best of the writers' knowledge, production piles at Site A were installed using a tip elevation criterion corresponding to 45 ft of embedment into the Yorktown and are behaving satisfactorily.

Installation of Test Piles – Site B

Three test piles were driven with a Vulcan 010 hammer to tip elevations corresponding to an effective embedment in the Yorktown Formation of from 40 to 59 ft. Final driving resistance ranged from 4 to 8 blows per foot.

Load Testing

The overall intent of the test pile program was to verify analytically established relationships between penetration into the Yorktown and pile capacity, and to utilize this verification in selecting pile tip elevations for the production piles.

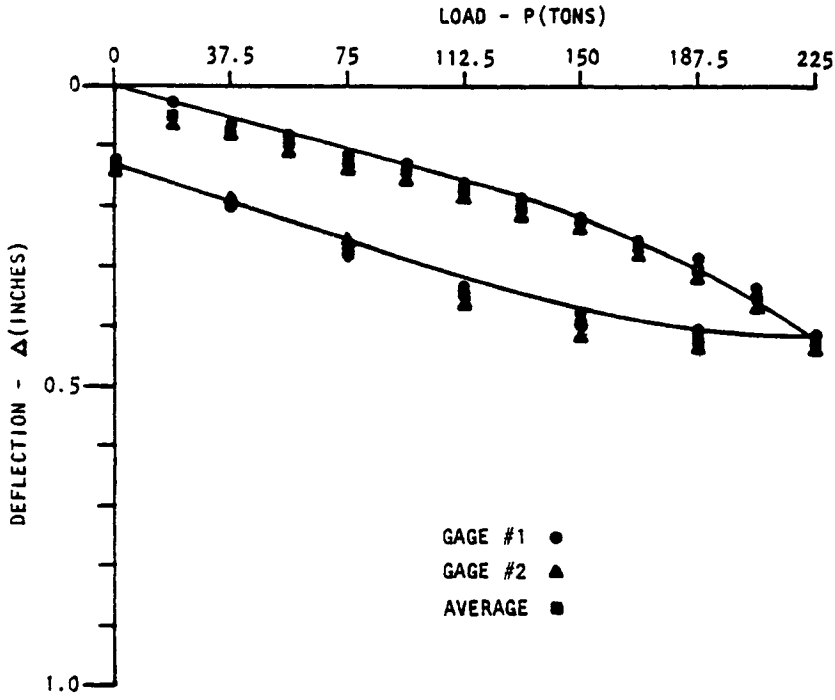
The compression tests were to be carried to three times design load or to "failure," whichever occurred first. The rationale used in selecting this loading was to allow verification of an ultimate capacity equal to or greater than the sum of anticipated dragdown (or negative skin friction) load plus twice design load.

Dragdown loads were estimated to be 30 tons or less per pile and design loads in compression were 75 tons per pile. Thus, the specific goals of the compression part of the pile load test program were: 1) to verify that the pile could sustain at least (30 + 2(75)) or 180 tons, and 2) to optimize penetration requirements so that this could be achieved with the shortest piles possible. To accomplish this, the three test piles were located so as to represent different conditions with regard to penetration into the Yorktown Formation and with respect to characteristics of the overlying material.

Ideally, tension or pullout tests should be carried to three or at least two times design load. The rationale behind this hypothesis is that there are more uncertainties involved in predicting pullout capacity than in predicting compressive resistance. Testing the tension piles at Site B to three or even two times design capacity would have required a fairly elaborate test set up and, more importantly, would have necessitated testing special piles as the structural integrity of the prestressing strands in the production piles limits the capacity to 90 tons, as compared to a design uplift load of 50 tons. Thus, the first tension pile was tested only to 75 tons or 1.5 times design load. The owner and the designer considered demonstration of such capability (i.e, a factor of safety against pullout of at least 1.5) to be acceptable in this case as: 1) the uplift loads are transient, 2) maximum load could

develop only with the simultaneous occurrence of conveyor belt repair and maximum wind conditions, and 3) even with such simultaneous occurrence the design uplift load actually may be only 40 tons.

The results of the first tension test could be interpreted in several ways. One technique would lead to the conclusion that the pile never "failed" in pullout and that the factor of safety is well in excess of 1.5. An extremely conservative approach would lead to the conclusion that it may be as low as 1.4. Because of this uncertainty, it was decided to perform a second tension test; loading the pile almost to its structural capacity.



ELEVATIONS:
TOP OF PILE - 9.48
GROUND SURFACE - 8.33
PILE TIP - 71.67

FIGURE 3 - TEST RESULTS

PILE #1 - COMPRESSION

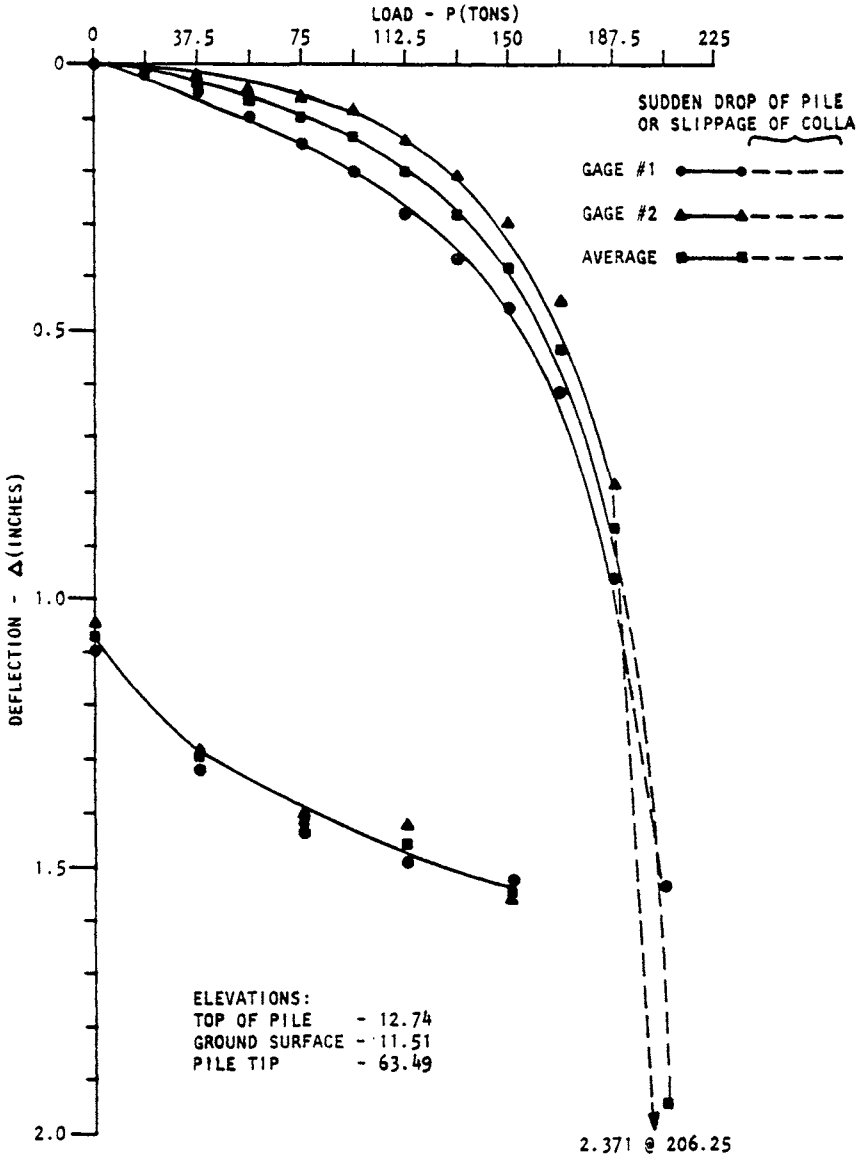


FIGURE 4 - TEST RESULTS
PILE #3 - COMPRESSION

Interpretation of Compression Test Results

Test Pile No. 1 was driven in an area of the site where the depth to the Yorktown varies. Further, in this section the Yorktown is overlain by a granular soil of probably equal competence. Thus, the effective penetration of the pile into the Yorktown or equivalent material was anywhere in the range from 45 ft to 59 ft or more. Static bearing capacity computations, based on the conservative assumption that the penetration was only 45 ft, indicate that the ultimate capacity of the pile should have been about 250 tons. The shape of the load deflection curve shown in Fig. 3 implies that the actual capacity is probably higher and that the actual penetration of the test pile into the Yorktown probably was greater than 45 ft. In reality, it appears that the structural capacity of the pile was exceeded at about 225 tons as concrete at and near the butt failed at around that load. In any event, the test led to the conclusion that a pile shorter than Test Pile No. 1 could be used to sustain dragdown plus twice design load; i.e., to sustain 180 tons.

Test Pile No. 3 was driven to a total embedded length of 75 ft or about 40 ft into the Yorktown. As shown in Fig. 4, it sustained a load of 187.5 tons at a cumulative deflection of about 0.5 in. When loaded to about 206 tons, it "failed." Computations indicated that with a penetration of 40 ft into the Yorktown the pile should have "failed" at about 225 tons. This is consistent with the actual "failure" load of about 206 tons and supports the validity of the analytical assumptions and techniques. The results of the test indicated that penetration of this pile was about optimal and that production piles would be driven to similar penetration.

Interpretation of Tension Test Results

As shown in Fig. 5, Test Pile No. 2 sustained 75 tons without continuous pullout but with a deflection of about 0.6 in. During unload, the pile shortened by about 0.45 in. This indicated that most of the deflection noted during loading to 75 tons may have been attributable to "elastic" lengthening of the pile and not to actual pullout. Computations indicated that "elastic" lengthening and shortening of the pile should have been in the range of 0.3 to 0.4 in. or so.

Test Pile No. 2 was driven to a total embedded length of 70 ft in the same general area as Test PIle No. 3. Thus, its penetration into the Yorktown was about 35 ft. Analytical techniques, the validity of which was confirmed by the compression tests, indicated that, with such penetration, the pile should have a pullout capacity of about 129 tons. This, the "elastic" lengthening and shortening computations, and the almost complete rebound on unloading indicated that the pile probably did not "fail" in pullout. However, one question remained: "What did happen as the pile was loaded from 62.5 to 75 tons?" A speculative explanation follows.

During initial loading, resistance is provided by only part of the pile. As load is increased, more and more of the pile must participate in resisting the load. Thus, more and more of the pile is subjected to "elastic" lengthening.

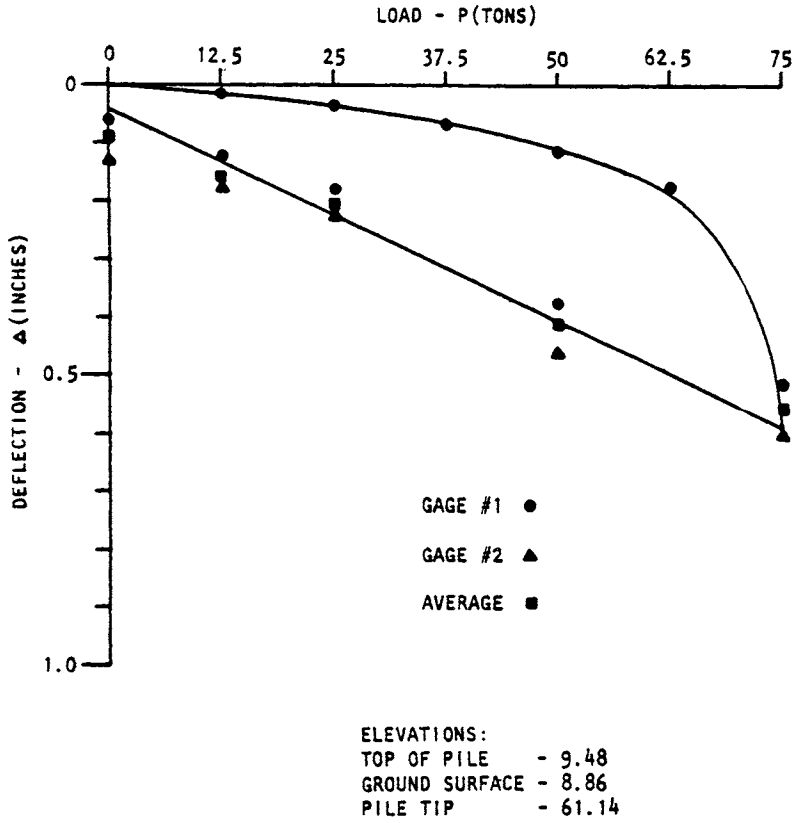


FIGURE 5 - TEST RESULTS
PILE #2 - TENSION

If such transfer of load occurred in a uniform fashion, the components of the load-displacement curve attributable to "elastic" lengthening would be linear. An abrupt load transfer and redistribution could result in an abrupt increase in the length of pile subject to "elastic" lengthening. This would manifest itself as a change in deflection similar to that shown between 62.5 and 75 tons on the load-deflection curve for Test Pile No. 2. The possibility that this occurred is evident in the fact that rebound during unload of a pile in tension is completely attributable to "elastic" shortening. Such shortening of Test Pile No. 2 was only 0.05 in. If the entire pile had participated in both load and unload, the load-deflection curve might appear as a straight line from: zero deflection at zero load to 0.4 or 0.5 in. deflection at 75 tons. It is possible that some of the difference between "elastic" lengthening and

shortening and the abrupt deflection caused by loading from 62.5 to 75 tons is attributable to relief of tension prestress during application of uplift loads.

The foregoing hypotheses indicate that Test Pile No. 2 did not "fail" at 75 tons and that its capacity probably is somewhat higher. Thus, the factor of safety against pullout failure probably is greater than 1.5 To check this, another pullout test was performed; this one on Test Pile No. 1, which had been subjected to a compression test but had not "failed." The results, shown in Fig. 6, seem to confirm the hypotheses, in that a load of 87 tons was held with a gross pile deflection of less than 0.5 in. The deflection, net of what is probably "elastic" lengthening and shortening, was about 0.15 in.

Based on the results of the tension tests, it can be concluded that the factor of safety against pullout "failure" for piles driven to an embedment of about 40 ft in the Yorktown Formation is at least 1.5 and possibly 1.8 or higher.

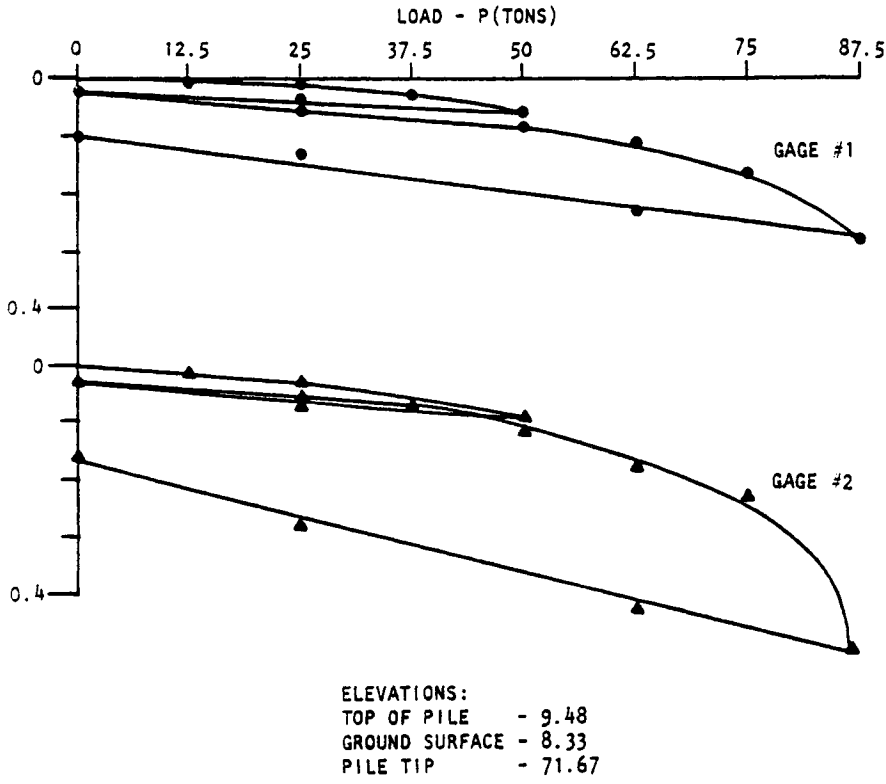


FIGURE 6 - TEST RESULTS
PILE #1 - TENSION

Recommendation and Conclusions

Based on the test results, the writers recommended installation of production piles driven to tip elevations corresponding to 40 ft penetration into the Yorktown. The piles were so installed and are functioning satisfactorily.

Based on the data, interpretations and opinions presented herein, it can be concluded that:

- Excess positive pore water pressure developed during driving a sampling spoon can result in very low SPT values and thus misleading interpretations with respect to soil strength.
- Similarly, like pressures developed during pile driving can have a profound effect on pile performance and can cause great differences between short and long-term capacities.
- Static bearing capacity theory can be used with great reliability to predict the long-term capacity of piles.
- For cases where short-term capacity is of significant concern, static theory could be employed if data such as piezometer readings are available with respect to excess pore water pressures developed during pile driving.