May 6th

Ground Heave Due to Pile Driving

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SYNOPSIS The factors which influence ground heave due to pile driving outside the construction site are discussed. Elevation survey data are presented for nine case studies in the Boston area where the subsoil conditions consist of an insensitive clay deposit in the range of 60 to 110 feet thick. Curves of heave vs. normalized distance exhibit a trend of increasing heave with increased volumetric displacement ratio. Patterns of ground heave typically occur as radially shaped contours decreasing in magnitude away from pile driving. Building and ground movements observed several years after completion of pile driving indicate that the heave is temporary, and is followed by a net settlement. Eight factors which influence heave due to pile driving are briefly discussed. Pile driving can be designed to minimize or prevent heave by properly planning the methods and sequence of pile installation.

INTRODUCTION

Engineers have recognized that the soil displaced during pile driving causes ground heave. Hagerty and Peck (1971) conclude that heave effects are most pronounced within saturated insensitive clay soils. Based on the results of several case studies, they further state that the volume of surface heave outside the area of pile foundations is equivalent to the volume of approximately 50 percent of the displaced soil. Depending on the proximity of adjacent buildings or surface features, the heave may cause distress and possibly structural damage. It therefore becomes important to estimate the magnitude and patterns of heave outside the construction site in order to preserve the integrity of the abutting features.

Previous studies, including those by Cummings et al. (1950) and numerous discussion papers, Lo and Stermac (1965), Soderberg (1967), D'Appolonia and Lambe (1971), and Vesic (1972), have attempted to explain the factors which contribute to heave. Several of these papers present field data which substantiate that heave effects are related to build-up of excess pore pressure, volume of displaced soil, sequence of driving, and clay sensitivity.

The Boston area presents an ideal setting for investigating magnitudes and patterns of heave due to pile driving. This study area is characterized by a thick deposit of insensitive stiff to soft clay; "Boston Blue clay". This clay underlies much of the city, where many of the modern medium to high rise buildings are founded on deep pile foundations. In many instances, the new buildings are located next to older turn-of-the-century buildings which are typically supported by short wood piles or caissons bearing within the top of the clay.

This paper presents the results of nine case studies involving structures founded on deep piles driven through the clay to end bearing within hardpan or on bedrock. Elevation surveys were made, both during and following pile driving, of adjacent buildings, utilities and streets. These data are presented and discussed in light of the factors which effect heave. The majority of the field data was obtained from projects with which the authors were directly associated.

DATA

The case studies referenced in this paper were all located within Boston or Cambridge, Massachusetts. These projects fall within the Boston Basin, which is dominated by a deposit of marine clay ranging from 60 to 110 feet thick. A typical soil profile at one of the case study areas is shown in Figure 1. For some of the other case studies, the organic soils and/or outwash deposits were absent from the profile and the thickness of deposits varied.

Boston Blue Clay is an insensitive clay deposited from glacial melt in quiet brackish waters. The upper part of the clay is overconsolidated, due primarily to dessication, while the lower portions are normally consolidated. Ladd and Luscher (1965) and Casagrande (1958) and others have documented the typical properties of the marine clay.

Table I summarizes information for each case study. The projects, in general, are buildings ranging from 5 to 40 stories. Foundation pile types consisted of 14-inch or 16- inch square precast prestressed concrete or 12-inch to 14-inch diameter concrete filled steel

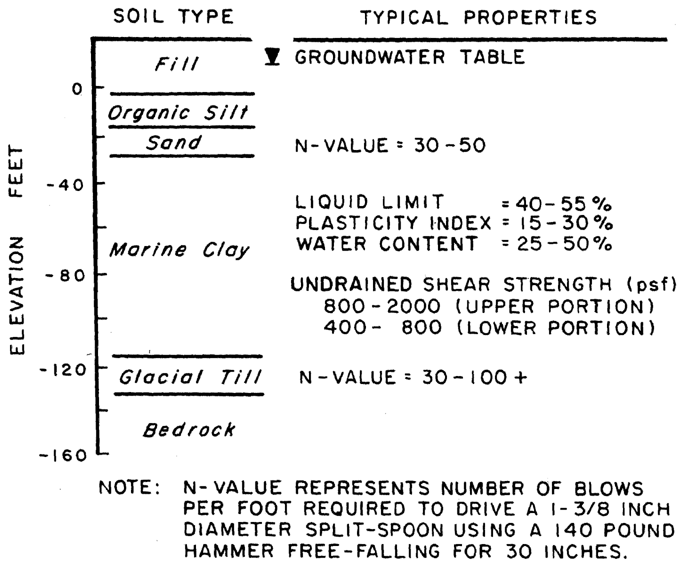

Figure 1. Typical Soil Profile

pipe, of 70 to 175 ton design capacitie Pile lengths varied from 90 to 160 feet. E cavations, as much as 25 feet below grou surface, were made at most of the study are prior to the start of pile driving. In a m jority of the case studies, pile driving w accompanied by preaugering through some or a of the clay deposit.

Adjacent buildings surveyed were typically to 9 stories in height. Surface points mon: tored included streets, retaining walls, d pressed highway ramps, and utility lines.

Pile area ratio and volumetric displaceme ratio are used to quantify pile data. Pi area ratio, expressed as a percent, represent the total cross-sectional area of piles divi ed by the building foundation plan area. Vol umetric displacement ratio, expressed as percent, represents the ratio of the volume c clay displaced during pile driving to the tc tal volume of clay underlying the building' foundation plan area. In calculating volu metric displacement ratio, it is assumed tha the slurry filling the preaugered hole is nc displaced into the ground as the pile is in stalled. This assumption is not generall valid for most, if not, all cases. However it is not possible to quantify the magnitud of this displacement.

Table I - Summary of Case Studies

CASE NO. AND SYMBOL		TYPE OF PROJECT (PLAN AREA, FT^2)	PILE TYPE (AVERAGE LENGTH, FT)	DIAMETER OF PRE-AUGER (IN)	EXCAVATION DEPTH (FT)	PILE AREA RATIO (%)	VOLUMETRIC DISPLACEMENT RATIO (%)	ADJACENT STRUCTURE TYPE	ADJACENT STRUCTURE FOUNDATION
1	○	SUBWAY (Linear Structure 130 ft wide)	14 in PPC (100)	17	28	1.4	0.0	Multistory Bldg.	Caisson
2	△	LOW-RISE BUILDING (28,000)	14 in PPC (105)	None	10	1.2	1.2	3-Story Building 2-Story Garage	Wood Piles Unknown
3	▽	HIGH-RISE BUILDING (22,000)	14 in and 16 in PPC (115)	16 (14 in PPC) 18 - 19 (16 in PPC)	21	5.6	0.9	8-Story Building Soldier Piles Roadways and Sidewalks	Wood Piles -- --
4	□	HIGH-RISE BUILDING (18,000)	14 in PPC (105)	17	12 to 22	5.8	0.4	Bridge Abutment 3-Story Buildings Roadways and Sidewalks	Wood Piles Wood Piles --
5	⧗	LOW-RISE BUILDING (35,000)	12.75 in CFP (105)	13	0	1.4	0.4	6-Story Building Steam Tunnel	Wood Piles Wood Piles
6	⬡	LOW-RISE BUILDING (15,000)	12.75 in CFP (130)	13.4	5	1.3	0.3	5-Story Building	Caissons
7	◇	MEDIUM-RISE BUILDING (17,000)	14 in CFP (120)	15	10	2.3	1.1	Low-Rise Building	Mat on Sand
8	+	MEDIUM-RISE BUILDING (12,000)	14 in PPC (130)	17	10	1.9	0.3	9-Story Building 5-Story Building	Pressure Injected Footings Wood Piles
9	×	LOW-RISE PARKING GARAGE (40,000)	14 in PPC (100)	14	0	0.7	0.4	3-to 4-Story Building	Wood Piles

PPC = SQUARE PRECAST-PRESTRESSED CONCRETE; CFP = CONCRETE FILLED STEEL PIPE

MAGNITUDE OF HEAVE

Data were plotted to determine relationships between the maximum heave and the major factors of distance and plan position relative to the pile driving, clay depth, and volumetric displacement ratio. Data for surface points were plotted separately from points on buildings.

Patterns of heave adjacent to pile driving are proportional to the depth of the bottom of the clay layer. To account for variations in clay depth for the cases analyzed, heave was plotted against a normalized distance. A normalized distance was calculated by dividing the distance between the reference point and the pile driving by the average depth to the bottom of clay.

The magnitude of neave along a line parallel to the site is related to the plan position with respect to the central portion of the pile driving. Data were plotted separately according to zones A, B and C as defined in Figure 2.

Figure 3 is an example of heave vs. normalized distance plots. This figure applies to points within zone A on buildings. There is a general trend of increasing heave with increasing volumetric displacement ratio. However, there is substantial variation such that a family of curves for varying volumetric displacement ratios could not be developed. A variety of factors which affect heave response are noted in the following section.

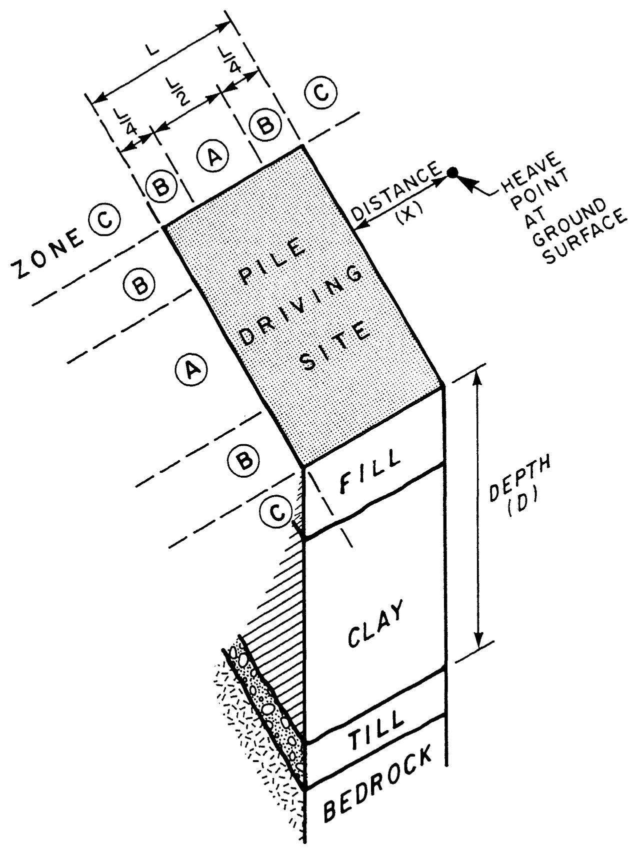

Figure 2. Definition of Zones of Heave

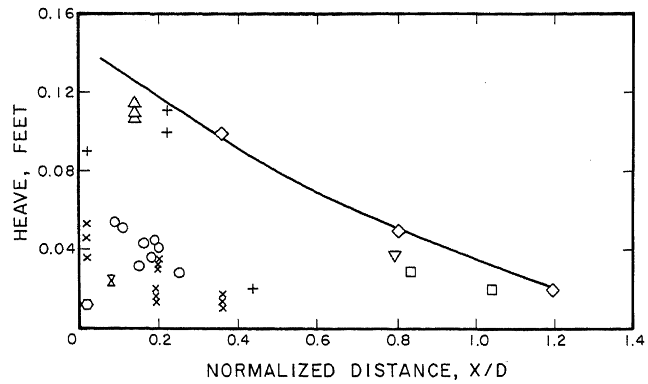

Figure 3. Heave vs. Normalized Distance for Buildings in Zone A

The heave vs. normalized distance plots apply to the following conditions, which are common to the case histories studied:

1. The typical Boston area soil/rock profile.
2. Volumetric displacement ratios up to a maximum of 1.2 percent.
3. Heave of buildings ranging from three to nine stories in height and founded near the top of the clay layer on footings, caissons or short friction piles; or heave of surface structures.

Plots of maximum heave vs. normalized distance for both surface points and building points are shown on Figure 4. The effect of vertical stress from buildings in offsetting heave movements is clearly shown. At a given distance, heave of surface points is approximately twice the heave of buildings. Maximum heave nearest the site ranges from about 0.14 feet to 0.32 feet for buildings and surface points, respectively.

There were few data points to define the heave vs. distance relationship for surface points in zones A and B at normalized distances greater than 0.5. Consequently, over this range, the curves were drawn to correspond to the trends shown for the other plots on the figure.

The greatest distance from a pile driving site at which measureable heave occurs is of interest. For the cases studied, the maximum normalized distance at which measureable heave was recorded ranged from about 1.2 to 1.5. Broms (1981) and D'Appolonia (1971) have noted that the lateral limit of heave beyond the construction site in a homogeneous clay stratum is approximately equal to the depth to the bottom of the clay layer. This assumption corresponds to a normalized distance of 1.0. The soils which overlie the clay have the effect of spreading the lateral limits of ground heave.

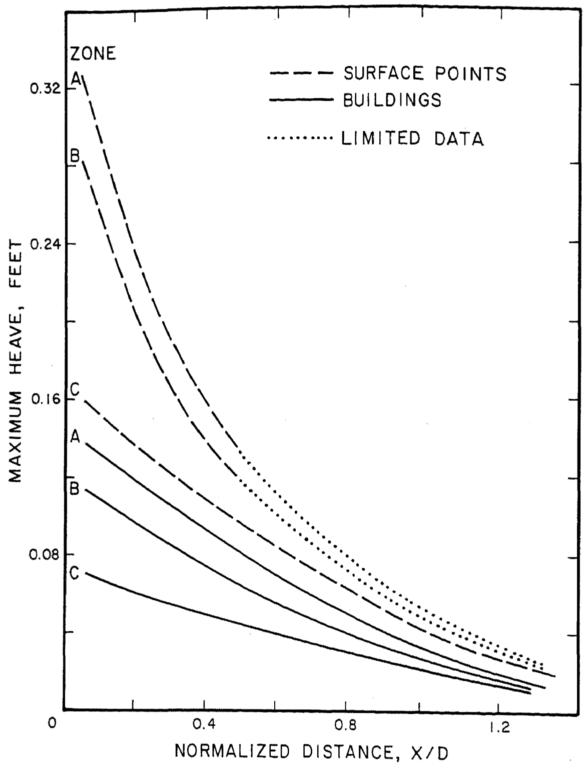

Figure 4. Maximum Heave vs. Normalized Distance

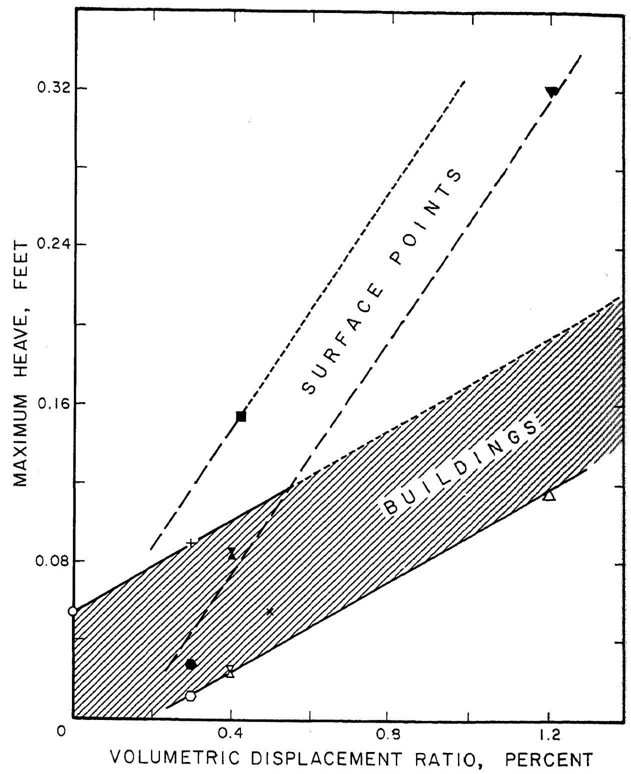

Figure 5. Maximum Heave Within Ten Feet of Pile Driving vs. Volumetric Displacement Ratio

Figure 5 shows relationships between maximum measured heave within ten feet of the construction site and volumetric displacement ratio for surface and building points. For case 1, heave occurred despite an indicated volumetric displacement ratio of 0.0. This may indicate that the augering was not in fact extended to the full depth of the clay, or that auger slurry was displaced into the ground during pile installation.

Potential for damage resulting from movements is generally correlated with angular distortion. Angular distortion is the differential movement between adjacent points divided by the distance between the points (or the slope of the movement profile). Figure 6 is a plot of maximum angular distortion versus normalized distance. A maximum angular distortion of 0.0011 was recorded. This value is approximately one-half the angular distortion of 0.002 noted to be the safe limit for buildings where cracking is not permitted (Bjerrum, 1963).

An example of ground heave contours due to pile driving is shown for case 3 in Figure 7. As shown in Weber (1978), heave contours are radially shaped. In case 3, heave decreased in all directions away from a maximum adjacent to and opposite the mid-point of the pile driving. Driving was directed from west to east. The effect of this sequence is seen in

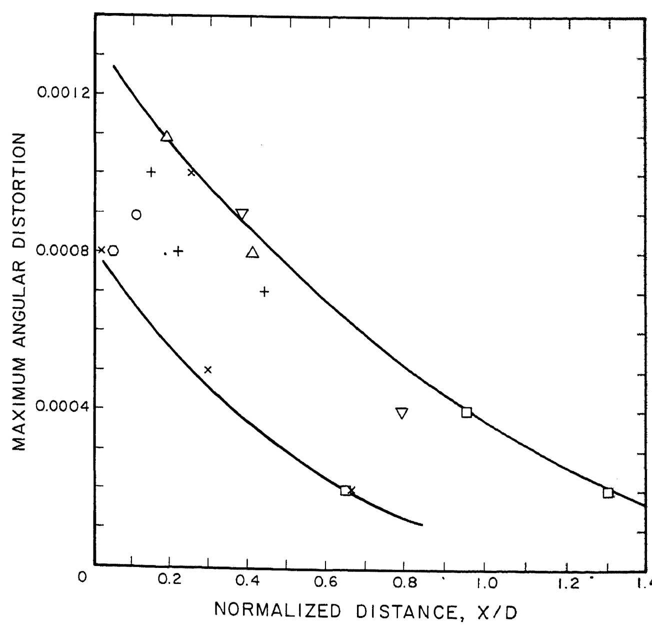

Figure 6. Maximum Angular Distortion vs. Normalized Distance for Buildings

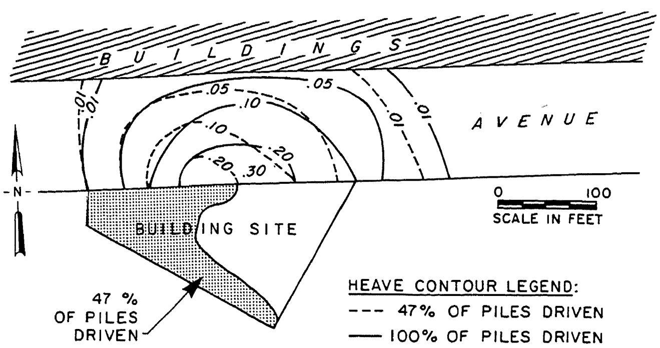

Figure 7. Contours of Surface Heave for Case 3

greater heave opposite to and east of the eastern end of the site, compared to the heave opposite the western end. Also, driving over the western half caused 0.05 feet of heave at a distance 100 feet north of the driving. Driving over the easterly end of the site caused little or no heave to the west. Thus, the first piles driven provided a barrier and deflected subsequent heave toward the east.

FACTORS INFLUENCING HEAVE

The magnitude and pattern of ground heave due to pile driving in insensitive clay are influenced by numerous factors, including the following:

1. Volume of displaced soil. Heave magnitude is directly proportional to the volume of clay displaced by the piles. As a first approximation, the total volume of heave beyond the site limits may be assumed equal to one-half the displaced soil volume (Haggerty and Peck, 1971).

2. Thickness and depth of clay layer. The lateral extent of ground heave in insensitive clays is approximately equal to the depth of the bottom of the clay layer.

3. Pile installation procedures. The effectiveness of pre-augering can be a significant factor in controlling displacements. When wet augering is used, the auger hole size must be large enough to permit slurry to be displaced from the hole as the pile is inserted. Displacement of slurry into the ground instead of onto the ground surface, can also be controlled by reducing the rate of pile placement into the hole. Collapse of overlying granular soils into the annulus between the pile and the auger hole prior to complete placement of the pile may prevent return of displaced slurry as the pile is placed. The augering method must be effective in removing soil materials from the hole. Dry augering in clay to greater than a critical depth may result in inward squeezing of clay and subsequent disturbance, as reported by Broms (1979).

4. Existing vertical stress. Heave of adjacent ground is inversely proportional to the level of in-situ vertical stress. Buildings and other above grade structures will experience less heave compared to the ground surface.

5. Pile driving sequence. Ground heave is increased in the direction toward which piles are driven, and reduced adjacent to the location of the piles first driven.

6. Clay sensitivity. Greater heave is experienced in insensitive clay. Consolidation during driving reduces heave in sensitive clays.

7. Excavation depth. With increasing depth of foundation excavation (and working level of pile driving), heave outside the site decreases and heave inside the site increases, as noted by Broms (1981). This factor is based on the same principle as item 4 above.

8. Granular soil layers. As reported by Hagerty and Peck (1971), penetration of piles through alternating layers of clay and coarse-grained soils will result in substantially reduced heave compared to the response in a homogeneous clay.

SETTLEMENT AFTER HEAVE

Soil displacements and excess pore pressures which result from pile driving will cause consolidation settlements in a clay mass. The magnitude and time period of settlement will depend on the degree of disturbance and on the clay properties.

It is likely that the total net movement following heave may result in net settlement. For the four case studies reported by D'Appolonia and Lambe (1971), which include cases 5 and 6 reported herein, net settlement on the order of 1.5 inches developed in 1 to 3 years following the end of construction. Somewhat smaller settlements were shown by Weber (1978).

Other available data substantiate the above findings. Consequently, the heave generated during pile driving is only a temporary condition. Until more data exist to permit better evaluation, it is recommended that, for projects similar to conditions analyzed herein, the net settlement resulting from pile driving be projected to be equal to the anticipated heave.

Figure 8 shows the movements in some of the case studies for both surface and building points following completion of pile driving. The data show that settlement continues as much as two years following completion of pile driving. The rate of settlement decreases with time and ranges in magnitude from 1/4 to 1/2 inch per year.

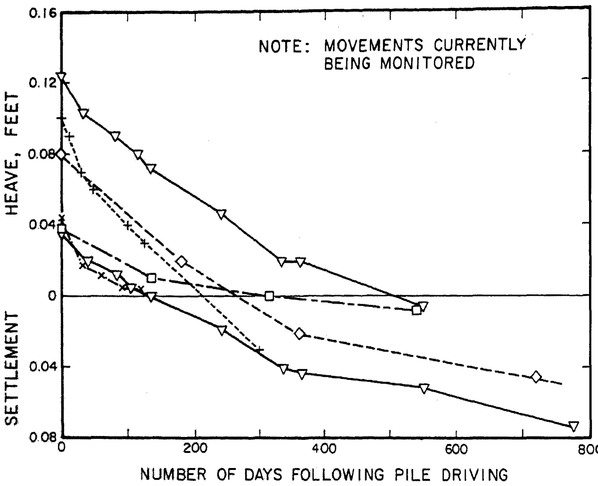

Figure 8. Movement Following Completion of Pile Driving for Selected Points

SUMMARY AND CONCLUSIONS

Heave of structures near construction sites caused by pile driving through thick clay deposits is investigated. Data are presented for nine cases of measured heave of buildings and surface structures adjacent to pile driving sites in the Boston area. Data plots relate heave of low-rise buildings and ground surface to volumetric displacement ratio, and plan position with respect to the driving.

Information on angular distortion of buildings, patterns of heave, and long-term settlements after completion of pile driving is presented. Factors which influence the magnitude and pattern of ground heave, relating to site and subsurface conditions and pile installation methods are summarized.

The ground heave relationships presented in Figures 4 and 5 are useful in assessing potential heave effects due to pile driving under similar subsurface conditions. Pile driving can be designed to minimize or prevent heave by properly planning the methods and sequence of pile installation. A quantitative assessment of the impacts of pile installation variables on the magnitude of ground heave is beyond the scope of the paper.

Ground heave is a temporary condition. Data confirm that dissipation of excess pore pressure and consolidation of the clay following pile driving results in a net settlement. Additional data is required to determine, if possible, relationships between heave and long-term settlement.

REFERENCES

Bjerrum, L. (1963), "Allowable Settlements Structures", Proceedings of European ference on Soil Mechanics and Found Engineering, Vol. 2, p. 135-137.

Broms, B. (1979), discussion to a paper titled,"Pore Pressure of NC Soft Silty Clay Around Driven Displacement Piles' Appendino et al., Recent Development the Design and Construction of Piles 179-180.

Broms, B. (1981), Precast Piling Practice. Thomas Telford, Ltd., London, England.

Casagrande, A. and L. (1958), Report to Me & Eddy on Foundation Investigation for dential Center in Boston, Vol. 1, pp. 1

Cummings, A. et al. (1950), "Effect of Dri Piles into Soft Clay", Transactions, Vol. 115, p. 275-349.

D'Appolonia, D. and Lambe, T. (1971), "Per mance of Four Foundations on End Bea Piles", Journal of the Soil Mechanics Foundation Division, ASCE, Vol. 97, S 77-93.

Hagerty, D. and Peck, R. (1971), "Heave an Lateral Movements Due to Pile Driv Journal of the Soil Mechanics and Fo tion Division, ASCE, Vol. 97, SM11 1513-1531.

Ladd, C. and Luscher, U. (1965), Engineeri Properties of the Soils Underlying the Campus, MIT Department of Civil Engi ing, Cambridge, MA.

Lo, K. and Stermac, A. (1965), "Induced Po Pressures During Pile-Driving Operati Proceedings, 6th International Confe on Soil Mechanics and Foundation Engi ing, Montreal, Vol. 2, p. 285-289.

Soderberg, L. (1967), "Consolidation Theor Applied to Foundation Pile Time Effe Geotechnique, Vol. 12, p. 217-225.

Vesic, A. (1972), "Expansion of Cavities i Infinite Soil Mass", Journal of the Mechanics and Foundation Division, Vol. 98, SM3, p. 265-290.

Weber, R. (1978), "Foundation Response Cau by Disturbance of Clay", Journal of Geotechnical Engineering Division, Vol. 104, GT5, p. 583-592.