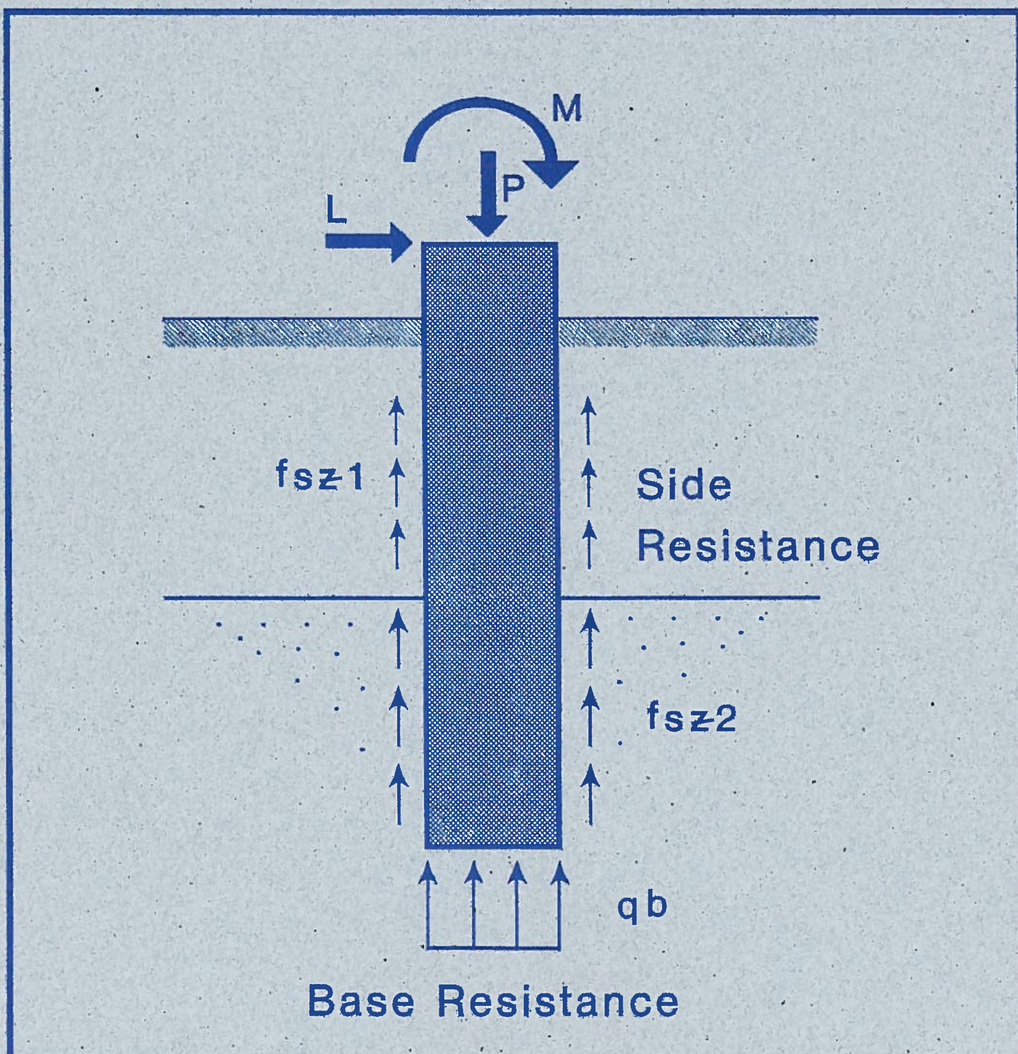
OHIO
RIVER
VALLEY
SOILS
SEMINAR

XXV

RECENT ADVANCES IN DEEP FOUNDATIONS



PROCEEDINGS

OCTOBER 21, 1994

LEXINGTON, KENTUCKY

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ORVSS XXV
Recent Advances in Deep Foundations

CONTENTS i

PREFACE

The XXV Ohio River Valley Soils Seminar (ORVSS) was held on October 21, 1994, at the Campbell House Inn in Lexington, Kentucky. The seminar was principally organized and hosted by the Kentucky Geotechnical Group of the American Society of Engineers. Co-sponsors of the seminar included the Cincinnati Geotechnical Group of the American Society of Civil Engineers, the University of Kentucky Department of Civil Engineering Office of Continuing Education and the Kentucky Transportation Center, the University of Louisville Department of Civil Engineering and the Center for Continuing Studies, the University of Cincinnati Department of Civil and Environmental Engineering, and the University of Dayton Department of Civil Engineering and Engineering Mechanics.

In February, 1994, a task committee was appointed to select a seminar theme and organize the twenty-fifth Annual Ohio River Valley Soils Seminar. The task committee consisted of the following members:

- Daryl Greer - Kentucky Transportation Cabinet
- Scott Murray - FMSM Engineers, Inc.
- Tommy C. Hopkins - Kentucky Transportation Center
- Kevin Sutterer - University of Kentucky
- Larry Snedegar - L. E. Gregg Associates
- Wayne Karem - Law Engineering
- Doug Smith - Kentucky Transportation Cabinet

The time donated freely by these individuals is gratefully acknowledged.

The theme selected for the 1994 seminar was **"Recent Advances in Deep Foundations."** Geotechnical and Civil Engineers are involved daily with the design of deep foundation systems for commercial, industrial, and transportation works. Advances and innovations in deep foundation systems have given engineers a better understanding of how these systems work as well as providing economical and effective alternates to traditional deep foundation systems. ORVSS XXV provided a forum for sharing design techniques and construction experiences regarding deep foundations.

As a means of developing the seminary subject and to encourage participation among the various geotechnical engineers, the seminar committee issued a call for abstracts in early 1994. Additionally, to ensure that the seminar subject was fully explored and developed from a technical viewpoint, some speakers with many years experience in the use of deep foundations were invited to participate in the seminar. As a result of these efforts, nine papers were selected for publication and presentation. Other papers were also submitted for publication.

Calvin Grayson, Director of the Kentucky Transportation Center, and Dr. Don Hancher, Chairman of the University of Kentucky Department of Civil Engineering, welcomed the attendees and officially opened the conference. Dr. F. C. Townsend from the University of Florida provided the seminar's morning "Keynote Address." Dr. George G. Goble of Goble Rauche Likins, Inc., presented the afternoon's "Keynote Address." Dr. B. O. Kuzmanovic gave a presentation of the paper that he co-authored with M. R. Sanchez that was selected for the 1994 ASCE Arthur M. Wellington Prize. The organizing committee members greatly acknowledge Mr. Grayson's, Dr. Hancher's, Dr. Townsend's, Dr. Goble's, and Dr. Kuzmanovic's efforts and give them our warmest appreciation.

The one-day seminar was divided into two sessions. The morning session was presided over by Aubrey May, Principal, FMSM Engineers, Inc. The afternoon session was presided over by Dr. Joseph Hagerty, University of Louisville Department of Civil Engineering. Following the last presentation, all attendees were invited to cocktails and hors d'oeuvres.

Geotechnical equipment and products were exhibited throughout the seminar. Members of the seminar committee sincerely thank exhibitors for their participation. A listing of exhibitors and patrons is provided on the back inside cover of the proceedings.

The seminar committee deeply appreciates the contributions made by the audience, presiding officers, and the authors of papers that were presented at this seminar and published in the proceedings. A hearty thanks must go out to Messrs. Jerry DiMaggio and Christopher Dumas of the Federal Highway Administration for providing information on possible speakers and topics. Also the committee gratefully acknowledges the efforts of Ms. Glenna Vickers with the University of Kentucky Engineering Professional Development, Ms. Ellie Marabello with the American Society of Civil Engineers, Ms. Jackie Overbey of the Kentucky Transportation Cabinet, and the many other people who worked diligently behind the scenes.

Daryl J. Greer
Seminar Chairman

The History of the Ohio River Valley Soils Seminar

This being the twenty-fifth ORVSS, the organizing committee felt it appropriate to include a more detailed summary of its history. This paper summarizes the history based both on official records and personal recollections and impressions. Much of the following information was obtained from the personal recollections of seminar participants, meeting minutes from sponsoring organizations, the history of the Kentucky Geotechnical Engineering Group published in the 50th Anniversary Edition (1986) of the ASCE Kentucky Section Directory (KGEG history by R. Deen and V.P. Drnevich), and of course the printed ORVSS proceedings themselves.

Twenty-five seminars in uninterrupted succession is a substantial feat for a locally organized and operated series. Several of the early participants in the seminar have suggested that it may be the longest uninterrupted ASCE annual continuing education seminar, but inquiries with the ASCE National headquarters only revealed that records of the many such local seminars are not kept. We would prefer to adopt the attitude of claiming the record until proven otherwise. A listing of all the past ORVSS' and the topics are provided in Table 1.

It is noteworthy that with the exception of seed funding provided by the Kentucky Section of ASCE and the University of Louisville for the first seminar, also held in Lexington, ORVSS has been a self-sustaining seminar organized and operated by regional personnel with no National or Section support. The seminar is sponsored by the Kentucky Geotechnical Engineering Group (KGEG), Cincinnati Geotechnical Group, University of Cincinnati, University of Dayton, University of Kentucky, and University of Louisville. ORVSS is held in the Lexington, Louisville, and Cincinnati areas on successive years. The alternation of seminar locations is likely one reason for the continuing success of ORVSS, with geotechnical engineers from each metropolitan area only being responsible for organizing the seminar once every three years. Regardless, the success and continuation of the seminar is a tribute to the geotechnical community in this region, and although the seminar is regionally oriented, it has always featured internationally acclaimed participants and speakers, beginning with the dinner speaker for the first ORVSS in 1970, Ralph Peck.

The first ORVSS was held in Lexington, Kentucky on October 16, 1970, but the events leading up to that first seminar extend at least to June 7, 1968 with the founding of the Kentucky Soil Mechanics and Foundations Group (KSMFG) in Frankfort, Kentucky. In addition to ORVSS, KSMFG, renamed the Kentucky Geotechnical Engineering Group on March 8, 1977, has sponsored numerous seminars and technical sessions over the years and has maintained the rigorous schedule of approximately eight to ten gatherings per year since their founding. Traditionally, at least two honorary lectures are sponsored by KGEG each year, with the lecturers being selected by the University of Louisville faculty in the Spring and the University of Kentucky faculty in the Fall. In fact, prior to its founding, the original members of KSMFG held at least two 1968 meetings that included technical sessions: the first being Soil Problems and Solutions by Nutting Engineers on February 9, 1968, and the second featuring Mobile Drilling on Hollow Stem Augers and New Drilling Equipment held April 26, 1968. A third 1968 technical session on Pile Foundations and Their Applications by Dr. John Heer of University of Louisville was the first of the officially formed KSMFG, held September 13, 1968. Although it was not planned this way, it is appropriate that this twenty-fifth ORVSS be centered on the same topic as the first official KSMFG technical session.

The 1968-69 schedule of KSMFG technical sessions included topics on rock core drilling, Wolf Creek Dam seepage, finite element methods, subsurface investigation, stability and consolidation, and ground freezing techniques. Joint meetings with the Kentucky Section of ASCE featured guest speakers on the study of Moon samples and the Kentucky Highway Soil Exploration Program. Some of these sessions included featured speakers from the Cincinnati metropolitan area, where the Cincinnati-Dayton Soil Mechanics and Foundations Group was sponsoring similar activities. The participation of some of the members of the Cincinnati-Dayton Soil Mechanics and Foundations Group was indicative of a long-running association between the Ohio and Kentucky groups, and played a role in the early development of the partnership between the southern Ohio organizations and universities and the Kentucky groups in sponsoring all of the ORVSS' after ORVSS I.

A review of the KSMFG annual report for 1968-69 indicated that discussion of a continuing education program and planning for a seminar was under way at that time. Woodson (Woody) McGraw, Chairman of KSMFG in its first full year (1968-69), was the Chairman of the Organizing Committee for ORVSS I, which also included Bill Mossbarger (1970-71 KSMFG Chairman) and Joe Hagerty (1971-72 KSMFG Chairman). A summary of the committee members for succeeding ORVSS' as indicated on the proceedings is provided on Table 2.

The Kentucky section of ASCE provided a loan of $350 and the University of Louisville volunteered to print the proceedings for ORVSS I and cover the cost of any loss incurred. The seminar was successful technically and financially, however, as the $350 was repaid along with a $150 donation to the Kentucky Section. The theme of ORVSS I was "Building Foundation Design and Construction" and included Dr. Ralph Peck, Dr. Hagerty's former Ph.D. advisor, as the speaker for the evening dinner. Dr. Peck was honored with the designation of "Kentucky Colonel" and presented with a gift of julep cups for his fine presentation. Attendance for the day's technical sessions for ORVSS I totaled 103, while attendance at the evening dinner/lecture was 149.

In the early years of the ORVSS seminars, the evening dinner session was a separate event from the day's activities. Attendance in the evening sessions often included those who had not been present for the technical sessions, and vice versa. Later, the evening dinner/lecture was included as part of the seminar, while some of the recent seminars excluded the evening dinner/lecture in favor of a social hour to permit an opportunity for catching up with old acquaintances before adjourning to allow those who drove in for the seminar sufficient time to make the long return trip to home. A complete listing of invited dinner speakers for succeeding ORVSS' could not be developed, but a summary list for those known is provided in Table 3.

While the over 200 technical papers of the past ORVSS have always been of high quality and well received, a special part of the seminar is the breaks, lunch session, and evening social hour or dinner. While attendees come from across the continent and even from overseas, ORVSS has always been dominated by local consulting engineers from within a four hour drive of the seminar site, so the interest and knowledge of the group carries a more regional flavor. In addition, unlike the majority of technical seminars, many of the presenters at ORVSS are from a consulting environment where their success or failure is much less dependent on the findings presented, so a more relaxed atmosphere is predominant. One new attendee at the most recent seminar remarked on the closeness and camaraderie among the participants, observing that while many of those present seem well acquainted with each other, the seminar provides the primary opportunity to gather at least once a year in a neutral setting to renew friendships. This aspect of

ORVSS may be as beneficial as any in maintaining state of the art geotechnical engineering in this region. ORVSS provides this region's geotechnical engineers an inexpensive, one-day forum to gather and share ideas and successes among our peers.

While the social flavor of the ORVSS has always been special, the technical content has also been very good. Clearly, the presenters put considerable effort into their papers, and many renowned geotechnical engineers are counted among those who have submitted their work through ORVSS. A review of the past ORVSS papers reveals many excellent works and the prudent geotechnical engineer would do well to review the selection. Although it is suspected that there are a number of complete sets of ORVSS proceedings, the only known complete set in public hands is the set of proceedings in the Kentucky Transportation Research Center library on the University of Kentucky campus. Completion of that set required contributions from the personal library of the late R.C. Deen of the University of Kentucky, an active promoter and participant in the most of the early ORVSS'. A complete listing of papers from past ORVSS' has been compiled and made available during the ORVSS XXV session. The listing can also be obtained from the Kentucky Transportation Research Center library. One of the speakers at ORVSS XX asked those in attendance who had attended all of the ORVSS' up to that time, and no one present spoke out, so unless an attendee at ORVSS XX was out of the room at the time the question was raised, it is unlikely there are any current 'veterans' of all of the seminars.

Attendance for ORVSS over the years has varied from about 120 to 250, with an average of about 200. Attendance was as high as 305 in 1977 (ORVSS VIII). Some interesting observations while reviewing past ORVSS proceedings include editorial comments and summaries of the day's activities by Bob Deen are included in some of the proceedings from 1973 through 1976. The ORVSS logo was designed in 1978, prior to which the acronym ORVSS was not used. Both the logo and acronym caught on immediately. Photographs from some of the ORVSS' over the years are provided on the following sheets.

The Organizing Committee wishes to thank all of those who assisted in the preparation of this history. Of particular note is the contribution of photographs by Vince Drnevich and Joe Hagerty. The assistance of Aubrey May, Vince Drnevich, and Joe Hagerty in researching their records and recollections to assist in the preparation of this history is greatly appreciated.

The Organizing Committee looks forward to preparation of an updated history for the 50th ORVSS with great anticipation!

Table 1. Past ORVSS Locations, Dates, and Topics

ORVSS No.	Date	Topic	Location
I	Oct. 16, 1970	Building Foundation Design and Construction	Lexington, KY
II	Oct. 15, 1971	Earthwork Engineering, Start to Finish	Louisville, KY
III	Oct. 27, 1972	Lateral Earth Pressures	Fort Mitchell, KY
IV	Oct. 5, 1973	Geotechnics in Transportation Engineering	Lexington, KY
V	Oct. 18, 1974	Rock Engineering	Clarksville, IN
VI	Oct. 17, 1975	Slope Stability and Landslides	Fort Mitchell, KY
VII	Oct. 8, 1976	Shales and Mine Wastes: Geotechnical Properties, Design and Construction	Lexington, KY
VIII	Oct. 14, 1977	Earth Dams and Embankments: Design and Construction	Louisville, KY
IX	Oct. 27, 1978	Deep Foundations	Fort Mitchell, KY
X	Oct. 5, 1979	Geotechnics of Mining	Lexington, KY
XI	Oct. 10, 1980	Earth Pressures and Retaining Structures	Clarksville, IN
XII	Oct. 9, 1981	Groundwater: Monitoring, Evaluation, and Control	Fort Mitchell, KY
XIII	Oct. 8, 1982	Recent Advances in Geotechnical Engineering	Lexington, KY
XIV	Oct. 14, 1983	Foundation Instrumentation and Geophysical	Clarksville, IN
XV	Nov. 2, 1984	Practical Application of Drainage in Geotechnical Engineering	Fort Mitchell, KY
XVI	Oct. 11, 1985	Applied Soil Dynamics	Lexington, KY
XVII	Oct. 17, 1986	Natural Slope Stability and Instrumentation	Clarksville, IN
XVIII	Nov. 6, 1987	Liability Issues in Geotechnical Engineering and Construction	Fort Mitchell, KY
XIX	Oct. 21, 1988	Chemical and Mechanical Stabilization of Soil Subgrades	Lexington, KY
XX	Oct. 27, 1989	Construction In and On Rock	Louisville, KY
XXI	Oct. 26, 1990	Environmental Aspects of Geotechnical Engineering	Cincinnati, OH
XXII	Oct. 18, 1991	Design and Construction with Geosynthetics	Lexington, KY
XXIII	Oct. 16, 1992	In Situ Soil Modification	Louisville, KY
XXIV	Oct. 15, 1993	Geotechnical Aspects of Infrastructure Reconstruction	Cincinnati, OH

Table 2. ORVSS Committee Members

Last Name	First	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	# times
Allen	D.													x			x			x							3
Armour	D.																			x							1
Bernauer	M.																						x				1
Bickel	S.																x				x			x			3
Bishop	C.S.										x			x			x										3
Bodocsi	A.			x						x			x			x			x								5
Bowers	M.																					x			x		2
Brodbeck	D.									x																	1
Brown	R.D.																	x									1
Cheeks	J.R.										x																1
Chisolm	G.								x																		1
Coombs	K.D.																				x			x			2
Crawford	K.																						x				1
Deen	R.C.				x	x	x	x	x		x	x		x													8
Doppler	G.E.															x			x								2
Dornoff	D.																					x			x		2
Drnevich	V.P.		x		x			x	x		x			x						x							7
Ebelhar	R.																								x		1
Erdman	F.												x			x											2
Ferrell	J.										x																1
Fetzer	C.												x			x											2
Fields	T.																								x		1
Flaig	J.			x			x			x			x			x											5
Franz	B.																					x					1
Gleason	V.A.						x	x	x																		3
Goettle	R.J.		x	x	x		x			x	x		x			x											8
Gorman	C.T.							x	x	x							x										4
Graves	M.																						x				1
Gray	E.																			x							1
Greenbaum	M.M.					x																					1
Greer	D.J.																									x	1
Hagerty	D.J.	x	x		x	x			x		x	x						x			x			x			10

Table 2. ORVSS Committee Members

Last Name	First	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	# times
Hagerty	P.																							x			1
Harris	S.																								x		1
Heckman	W.																		x								1
Hensey	M.			x			x																				2
Holder	D.																					x			x		2
Hopkins	T.C.							x	x			x								x			x			x	6
Howell	P.													x						x			x				3
Huff	D.D.							x																			1
Hurt	R.													x													1
Jeffers	L.						x	x		x						x											4
Karem	W.																									x	1
Keller	D.																					x			x		2
Lennertz	C.R.					x							x			x			x								4
Lockwood	M.																					x					1
Mathis	H.					x		x	x					x									x				5
May	A.D.				x	x																					2
McClandess	R.															x			x			x					3
McGraw	W.W.	x						x																			2
Miller	E.J.		x					x																			2
Mossbarger	W.A.	x																									1
Munstock	B.									x																	1
Murray	S.																						x			x	2
Nethero	M.			x																							1
Newberry	D.										x																1
Osborn	P.																							x			1
Payne	J.									x			x			x											3
Pfalzer	W.J.																x	x			x		x				4
Ransdell	S.							x			x																2
Rayburn	L.						x	x		x			x			x											5
Roberto	G.									x			x			x			x			x					5
Roelker	R.																		x			x					2
Ruhl	J.																							x			1

Table 2. ORVSS Committee Members

Last Name	First	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	# times
Schmitt	N.G.											x			x			x									3
Schomaker	N.			x																							1
Schuhmann	M.J.														x												1
Slack	F.W.												x			x											2
Smith	D.																									x	1
Snedegar	L.																						x			x	2
Spence	C.																					x					1
Stickney	R.												x			x											2
Storm	J.																			x	x			x			3
Sutterer	K.G.																									x	1
Thacker	B.K.											x			x			x									3
Thelen	D.												x			x			x								3
Ullrich	C.R.								x			x			x			x			x			x			6
Vieth	J.						x			x						x											3
Vogelpohl	T.																		x						x		2
Webb	G.C.															x			x			x			x		4
Zehetmaier	H.						x			x			x			x											4
Zentmeyer	J.																								x		1
Zoghi	M.																								x		1
Zolkowski	D.													x													1

Table 3. Summary of Dinner Speakers for Some Past ORVSS'

Year	Title/Background	Speaker
1970	The Role of the Soils Engineer in Building Foundation Design	Ralph B. Peck
1971		George Sowers
1976	Former U.S.Senator and current Chairman of Board of Island Creek Coal Co.	Albert Gore
1978	The Salvaging of the Abu Simbel Temples	Lennart Berg VBB, Stockholm
1979	Author and Appalachian Historian	Harry M. Caudill
1980		Delon Hampton
1981	Geotechnical Aspects of the Mt. St. Helens Eruptions	Bob Stickney
1983	Measure of Measurements	George Sowers
1984	Construction Methods and Customs in Mainland China	E. Paul Swatek, Jr.
1985	Liquefaction Failure of Tailings Dams Resulting from an Earthquake in Japan	William F. Marcuson III
1986	The Statue of Liberty; Then and Now; Why and How	Edward Cohen
1987	Alternative Dispute Resolution Procedures	Joseph S. Ward
1988	former Research Director of Dynapac	Lars Forssblad
1989	Some Geotechnical Engineering from Around the World	Vince Drnevich

Woody Mcgraw,
Bob Deen,
Ralph Peck,
and Joe Hagerty
at ORVSS-I

George Sowers accepts
a gift from Joe Hagerty
at ORVSS-II

Back: D.J. Barr,
Woody McGraw,
and Don Tupman
Front: Mel Hensey
and Vince Drnevich
at ORVSS-II

Dick Goettle and Joe Hagerty at ORVSS-II

Break time at ORVSS-II

Attendees at ORVSS-VII

Vince Drnevich and Jerry Leonards listen to Albert Gore at ORVSS-VII

Vince Drnevich with the afternoon session's review panel at ORVSS-VII

Thomas O'Rourke, Ed Kinner, Bob Dean and Ernest Selig at ORVSS-XI

COMPARISON OF DEEP FOUNDATION LOAD TEST METHODS

by

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ABSTRACT

Many bridge foundations are supported on deep foundations, of which design and quality control are issues to ensure reliability and economics. Consequently, field load testing is generally specified for most construction projects involving deep foundations with the objectives of verifying design/construction loads and deflections.

Traditionally, the static load test (ASTM D1143) has been the hallmark test used to evaluate deep foundation elements (piles/shafts). However, as capacities have increased, and test sites have become less hospitable (over water), alternative testing methods have evolved. Accordingly, this paper examines several of these alternative methods and their applicability; specifically, (a) static tests (static load test, Osterberg Cell, and scaled-down static tests), and (b) dynamic tests (Wave equation (D4945-89), and Statnamic).

For axial capacities less than 8.9 MN (1000 tons), the conventional static load test is viable. The Osterberg Cell has been used to generate equivalent surface loads of 50 MN (6000 tons) and does not require reaction piles and beams. Wave equation dynamic tests have used drop weights of 20 Mg (20 tons) to mobilize resistances of 30 MN (3400 tons). Statnamic loadings up to 27 MN (3000 tons) are available.

INTRODUCTION

Purpose

A majority of bridge foundations in the United States are supported using deep foundations; i.e., driven piles or drilled shafts. Often, due to uncertainties in design assumptions, site conditions, and construction effects, load tests are specified for verification and quality control. The primary objective of these load tests is to determine the load - deformation response of the pile[1]. To achieve this objective, several different static and dynamic test methods/procedures have evolved for conducting pile load tests; specifically, (1) Static load test (ASTM D-1143), (2) Osterberg cell, (3) Small-scale static load test, (4) Wave equation based dynamic test (ASTM D4945-89), and (5) Statnamic tests. Accordingly, the

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[1] Pile(s) refers to both driven piles and drilled shafts.

objective of this paper is to discuss these load test methods.

Advantages and Disadvantages of Load Testing (FHWA, 1992)

Several advantages offered by load tests are:

- A more "rational" design due to more reliable design values
- Potentially higher design loads due to better knowledge of in situ properties
- Use of a lower more economical "factor of safety"
- Verification of construction/design load.

Several disadvantages cited for performing load tests are:

- Increased costs
- Construction delays
- Limited information on long-term settlement
- Group action effects ignored
- Site variability.

Theory

As illustrated in Figure 1, the ultimate capacity of a pile is composed of (1) Skin friction, and (2) Tip resistance, expressed in Equation 1 and neglecting the weight of the pile.

$$\begin{aligned} Q_u &= Q_p + Q_f - W \\ &= q_p A + f_{su}\, PL - W \end{aligned} \qquad (1)$$

where

Q_u = ultimate axial capacity
Q_p = ultimate tip resistance
Q_f = ultimate skin friction resistance
W = pile weight (usually ignored)
A = pile tip area
PL = surface area
q_p = unit bearing capacity
f_{su} = unit skin friction

To separate these two components, load tests are often instrumented using (1) telltales and/or (2) strain-gages, and Equation 2. In this fashion, load distribution profiles as illustrated in Figure 2 can be obtained. Subsequently, these are developed into t-z curves at various elevations (Figure 3). Appendix A illustrates example calculations for obtaining t-z profiles from sister-bar data.

$$Q = AE\epsilon \qquad (2)$$

where

Q = load at elevation Z_i
AE = pile rigidity
ϵ = cumulative strain

where

ϵ = strain from vibrating wire sister-bar measurements,
or = difference in successive tell-tail dial readings/length

From this approach, it is apparent that the pile rigidity (AE) greatly affects results, and sister-bar measurements over a "stick-up" height are recommended. Also, telltale measurements should be to 0.0001".

STATIC LOAD TEST (ASTM D-1143)

General

Figure 4 illustrates a static load test set-up, which consists of (1) test pile, (2) reaction members, (3) reaction beam, and (4) loading jack, weights, or mechanism (FHWA, 1992).

The test pile usually is the same type and geometry as the piling proposed for the foundation. Installation also should utilize the same equipment as proposed.

The reaction members are to be located not less than 5 diameters or 7 ft (2 m) from the test pile. Obviously, they must be designed to resist the anticipated uplift resistance.

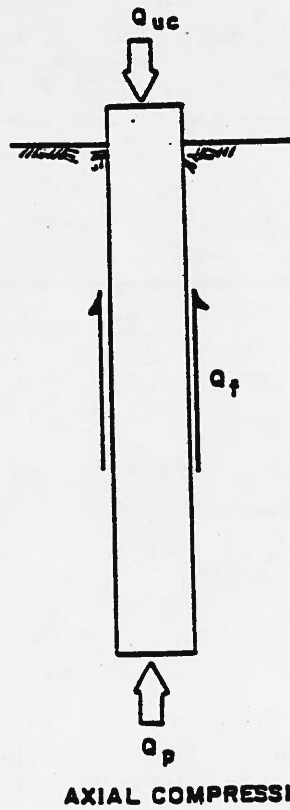

Figure 1. Pile capacity due to compressive load

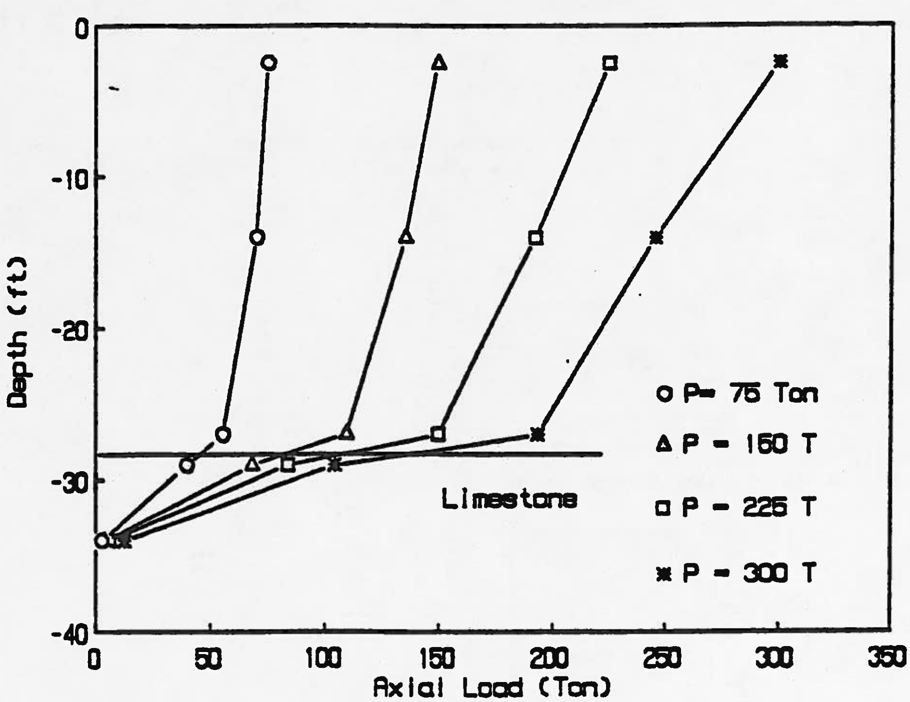

Figure 2. Axial load distributions with depth

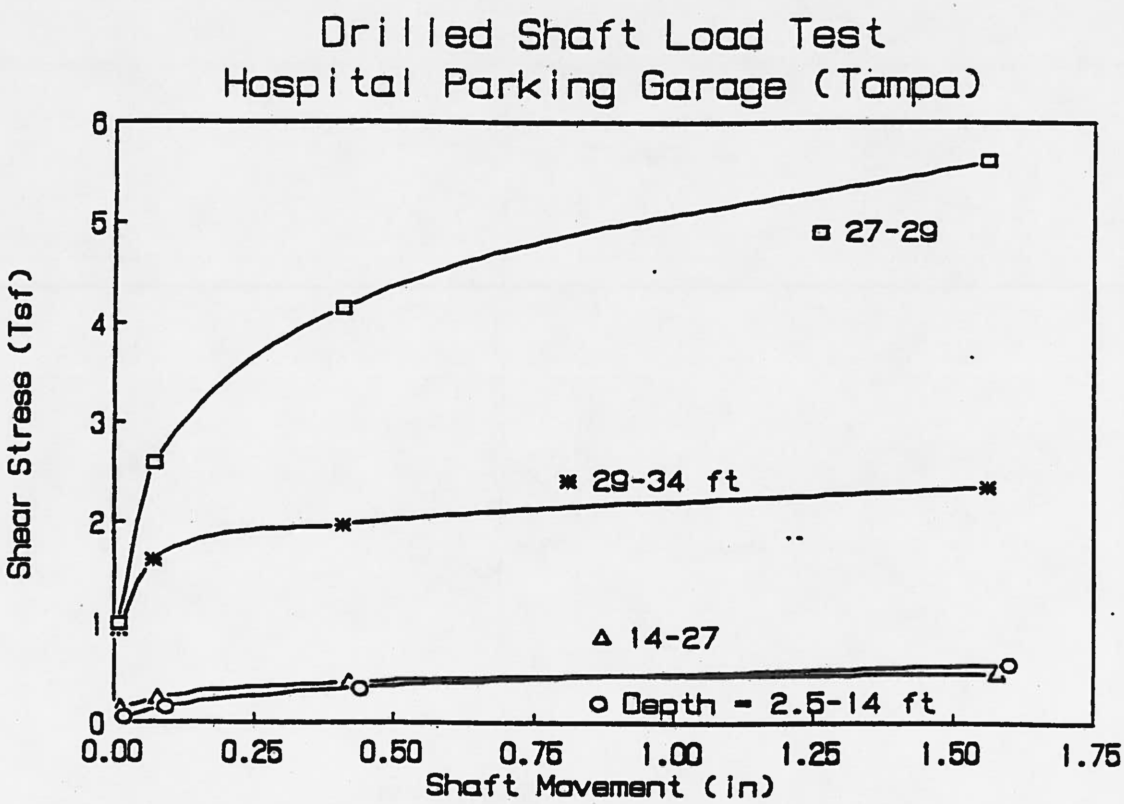

Figure 3. T-Z curves - Tampa hospital parking garage

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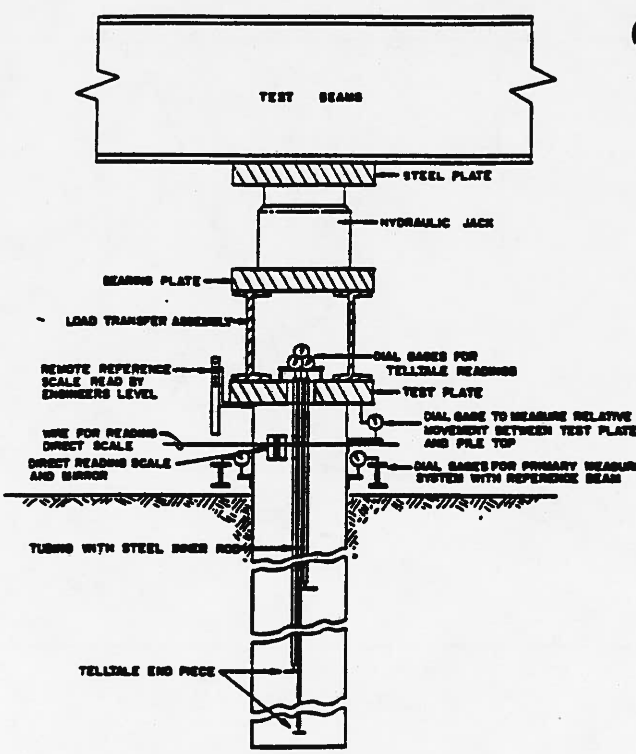

Figure 4. Possible arrangement of instrumentation for measuring vertical movements of pile

Loading is usually to 200 percent of the design load (but preferable to "failure") in increments of 10-15 percent of the design load.

Capacities

Static load tests are usually less than 1000 tons. As shown in Figure 4, this method has the disadvantages of requiring reaction members and beam, measurement instrumentation, and clearances between reaction members and the test pile.

OSTERBERG CELL

General

Figures 5 and 6 illustrate the Osterberg Load Cell, which essentially is a specially designed high capacity hydraulic jack located at the pile tip. As such an upward force is applied to the bottom of the pile, and a downward reaction to the bearing soil. "Failure" of the pile, therefore, will either occur in end bearing OR friction/adhesion. When failure occurs no further load can be applied. The downward tip movement is measured via the hydraulic pressure pipe welded to the jack functioning as a telltale. The upward movement is measured via traditional static load instrumentation (telltale(s), sister-bars).

Since the cell upward force is equal to the downward force, the applied pile load is equivalent to a surface load of twice the applied cell force (less the buoyant pile weight).

One question that arises is whether the ultimate pile resistance obtained by pushing upward is equivalent to that from loading downward, and, if not, what is the magnitude of the difference. Theoretically, for cohesive ($\phi = 0$) soils, there should be no effect on skin friction as the shear stress is independent of the normal stress. However, for granular soils, it is likely that the normal stress is less when pushing upward. Consequently, the shearing stress will be conservatively less than that measured by pushing downward (Osterberg, 1992).

Cell Sizes/Capacities

The latest maximum design of the cell is capable of an internal pressure of 8000 psi, and for a 34-inch diameter size is capable of exerting 3000 tons upward and 3000 tons downward or an equivalent surface load of 6000 tons (50 MN). The minimum size available is 12-inch diameter with a capacity of 400 tons (Osterberg, 1992).

Advantages/Limitations

In my opinion, the O'Cell has these advantages(A)/limitations (L):

(A) Readily duplicates the static load test, but without requiring reaction members and beam. Consequently, construction times and costs are reduced.

(A) Provides equivalent surface loads up to 6000 tons, or 6 times current static load test equipment.

(A) Requires less construction space than conventional test. Thus, it is advantageous for crowded locations or working over water.

(A) Shaft friction or end bearing can be determined without requiring additional instrumentation.

(L) Cell must be installed on the pile prior to driving or on the shaft prior to pouring concrete.

(L) Ultimate failure in *both* friction and end bearing are not normally obtained.

(L) On occasion, ultimate failure conditions are not achieved.

SMALL SCALE STATIC LOAD TESTS

In theory, since the unit skin friction and unit tip resistance are used to calculate pile capacities, these quantities could be determined from small scale tests. However, the

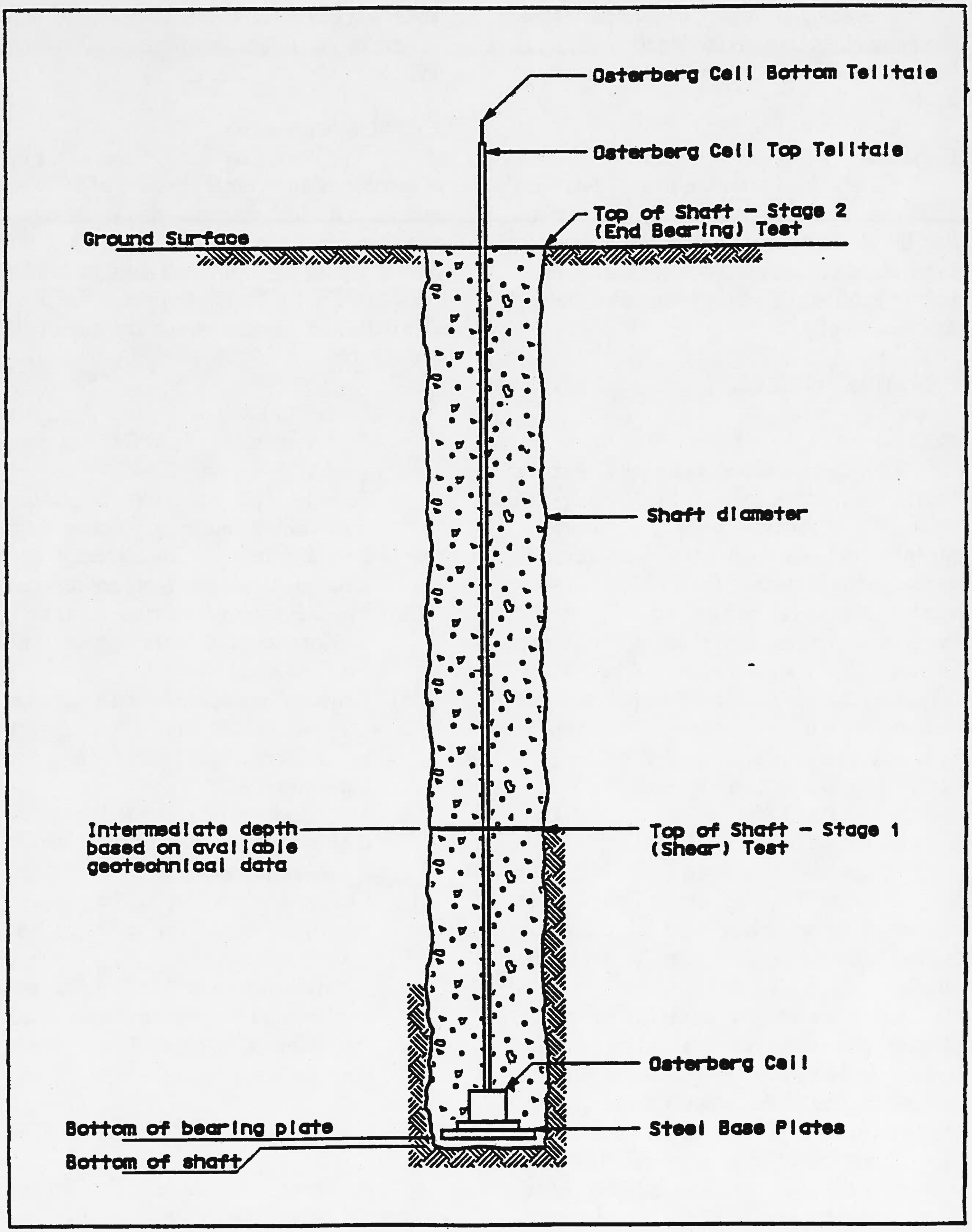

Figure 5. Typical Osterberg cell loadtest staging

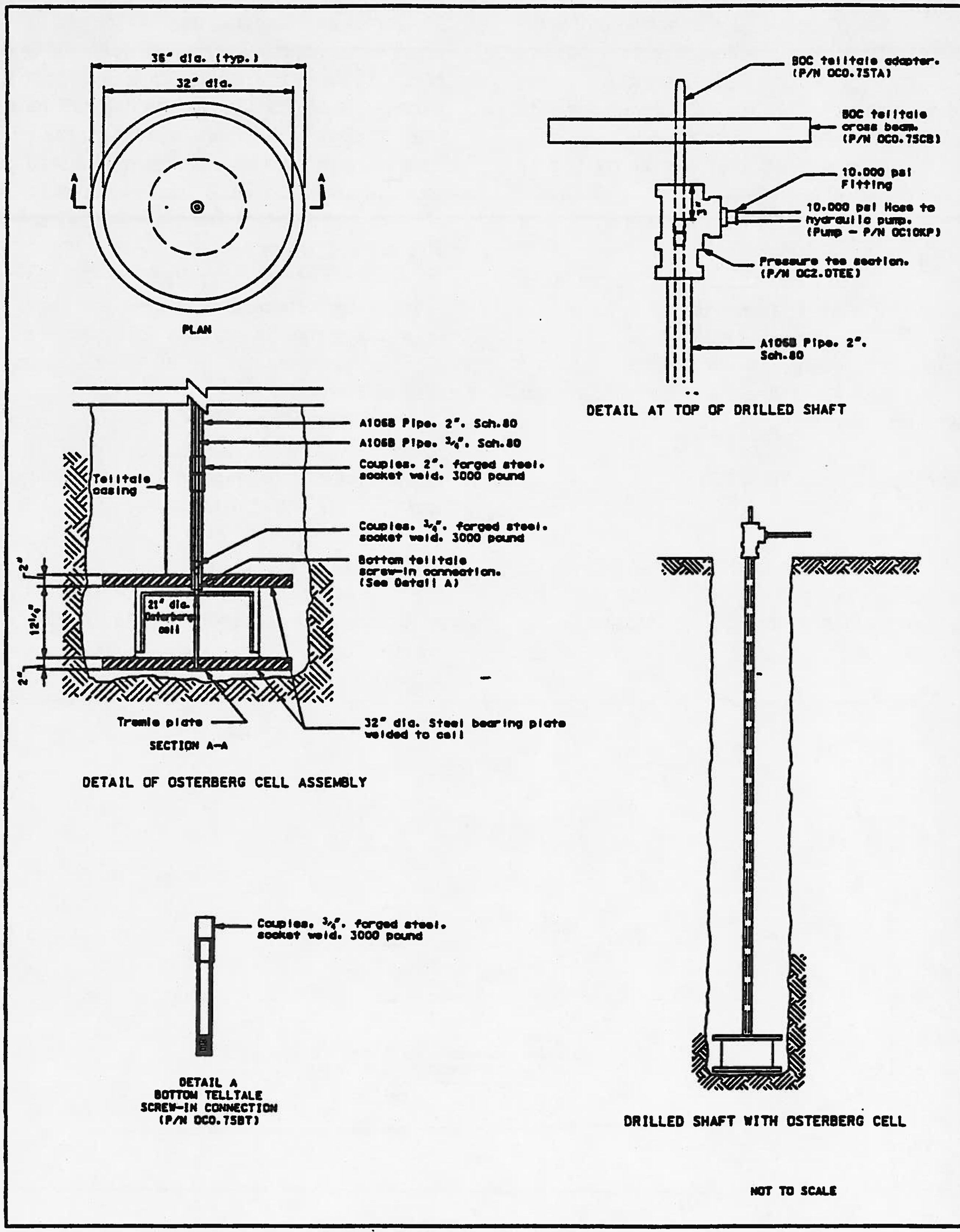

Figure 6. Suggested setup for Osterberg cell load testing device in 36-inch diameter drilled shafts (Loadtest, Inc.)

use of scaled-down piles is not widely practiced. The primary problem is that the zone of influence for skin friction adjacent to the pile, and the stress bulb beneath the pile tip are both a function of the pile diameter. Consequently, only in very uniform soils can the method be considered. One application is the pullout test being used to assess the unit skin friction of rock socketed drilled shafts.

Horvath and Kenney (1979) presented data in Figure 7, which shows that the unit shaft friction decreases with increasing socket diameter. However, for socket diameters larger than 15 inches the effect of diameter appears to be negligible.

DYNAMIC LOAD TESTS

General

Dynamic analyses of piles are methods which predict pile capacity based upon the behavior of the hammer-pile-soil system during driving. The concept being that the pile driving operation causes failure in the pile-soil system. As such, pile driving is analogous to a very fast load test under each hammer blow (Paikowsky et al., 1993). However, the pile must undergo a minimum permanent displacement, or set ($\approx$ 0.1 inch) during each hammer blow to mobilize fully the pile-soil resistance. If there is little or no permanent downward displacement of the pile tip, then the pile experiences mostly elastic deformation, and the full resistance of the pile-soil may not be achieved. Accordingly, the blow count should be less than 120 blows/ft.

There are two basic methods of estimating driven pile capacity based upon dynamic driving resistance: (1) pile driving formulae, and (2) wave equation analysis.

Pile Driving Formulae

Pile driving formulae, of which the Engineering News Record (ENR) equation is

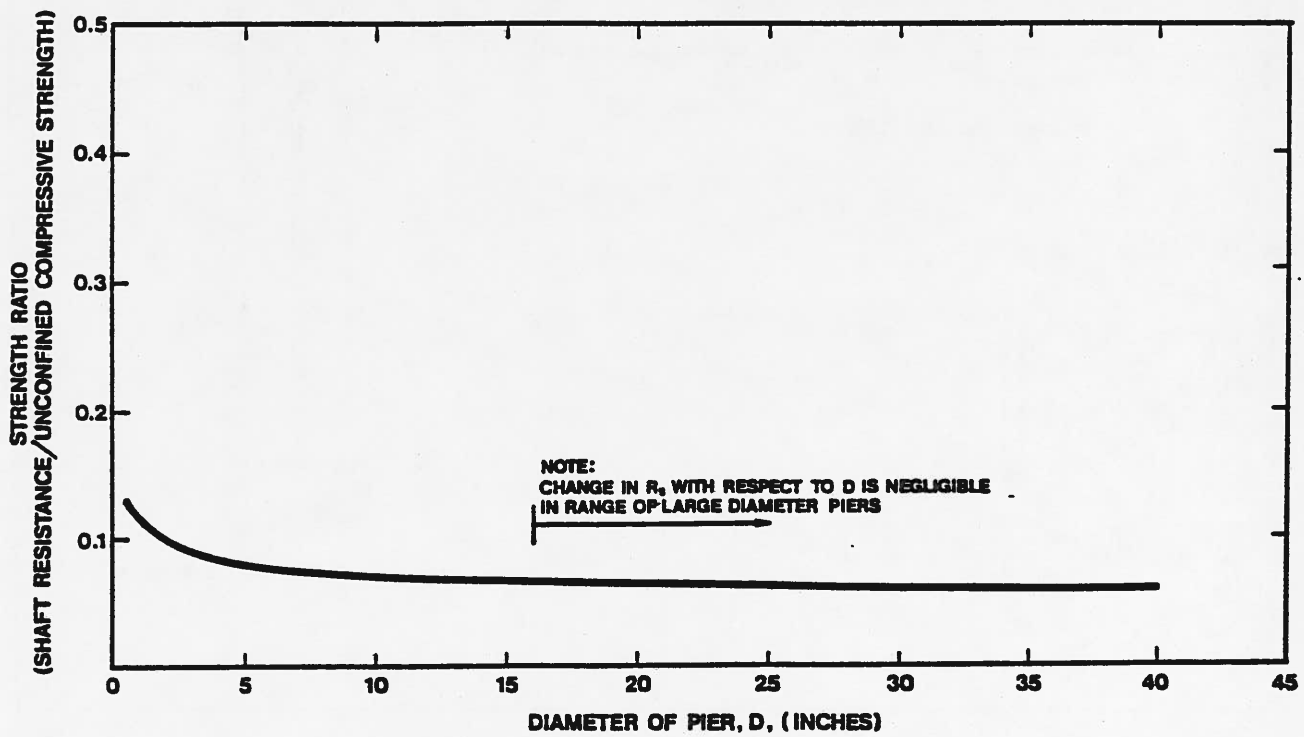

Figure 7. Average strength ratio, Rs, for piers of various diameters

the most famous, are based upon the premise that the total resistance of the pile is related to the work done during pile penetration. Interestingly, 45 state highway departments include a dynamic formula in their foundation specs, of which 30 use the ENR formula (Paikowsky et al., 1993). For example, when dynamic tests are not available, FDOT uses:

Timber Piles

$$R = \frac{2E}{S + 0.1}$$

Concrete/Steel Piles

$$R = \frac{2E}{S + 0.1 + 0.01P}$$

where

R = Bearing Resistance, tons
S = Set, inches
E = energy per hammer blow, ft-tons
P = pile weight, tons

Unfortunately, dynamic equations can be highly inaccurate (Olsen and Flaate, 1967), and a high factor of safety is required (FS = 6 for ENR). To overcome these limitations, wave equation analyses have become increasingly more popular.

Wave Equation Analysis

Although the analysis of 1-D wave propagation began over 100 years ago, (St. Venant in 1867–see Timoshenko and Goodier, 1950) in 1960, Smith developed a numerical model to simulate the dynamic behavior of the hammer-pile-soil system during driving. For background, the "Wave Equation" (WE) model is represented as a series of discrete masses and springs (see Figure 8). The soil resistance is modeled as a spring, slider, and dashpot, which represent the static and dynamic soil resistances, respectively:

$$R_{max} = R_s + R_d \qquad (3)$$

An elasto-plastic soil model is used for the static soil resistance. The distance travelled by the pile toe during the elastic deformation is represented by the soil quake. As the elastic limit of the soil is reached, plastic deformation or set occurs. According to the model presented in Figure 8, point A represents the ground resistance build-up to the ultimate resistance R_u. Plastic failure occurs as the ground resistance has reached its maximum and the pile segment displaces, plastically to point B. The permanent set is therefore equal to the distance OC, which in turn is equal to the distance AB. This static soil resistance is modeled (by Smith, 1960) as a spring, K_s. The dynamic component of the soil's resistance is assumed to be viscous and thus is velocity dependent. This dynamic resistance is modeled by a dashpot, J, parallel to the spring.

The wave equation is used in two general ways: (1) pre-driving and (2) post-driving. WEAP is an example of the former, while PDA/CAPWAP is an example of the latter.

Pre-Driving (WEAP87, FHWA, 1987)

*W*ave *E*quation *A*nalysis for *P*iles (WEAP) is used to: (1) evaluate the suitability of the proposed driving system (including the hammer, follower, capblock, and pile cushions); (2) evaluate the compressive and tensile stresses generated during driving; and (3) determine the driving resistance, in blows/ft, to achieve the pile bearing requirements. Figure 9 presents an example output for illustration.

Performing a WEAP analysis requires several preparatory steps (GRLWEAP manual, p. 8); specifically:

1. Designing the pile length, and estimating the percentages of skin friction and tip resistance.
2. Estimating the potential setup for end of driving (EOD) and restrike (BOR) or (EOR).
3. Estimating the dynamic soil resistance parameters of damping and quake.
4. Selecting a hammer and driving system, and estimating the efficiencies, COR, and cushion stiffness.

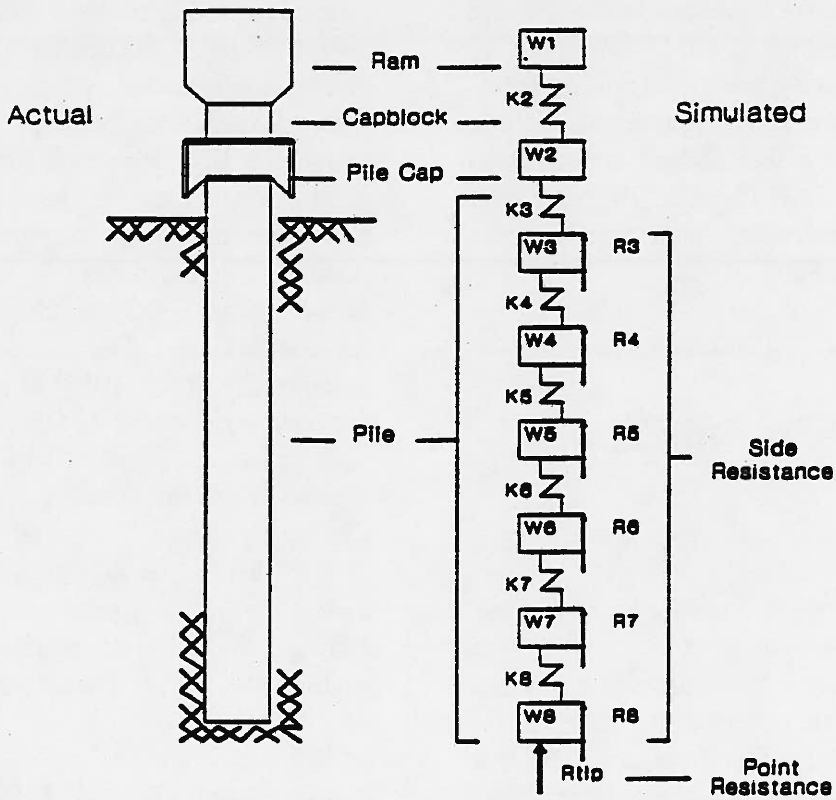

Figure 8a. Smith's model simulating the hammer-pile-soil system for use with the one-dimensional wave equation (Smith, 1960)

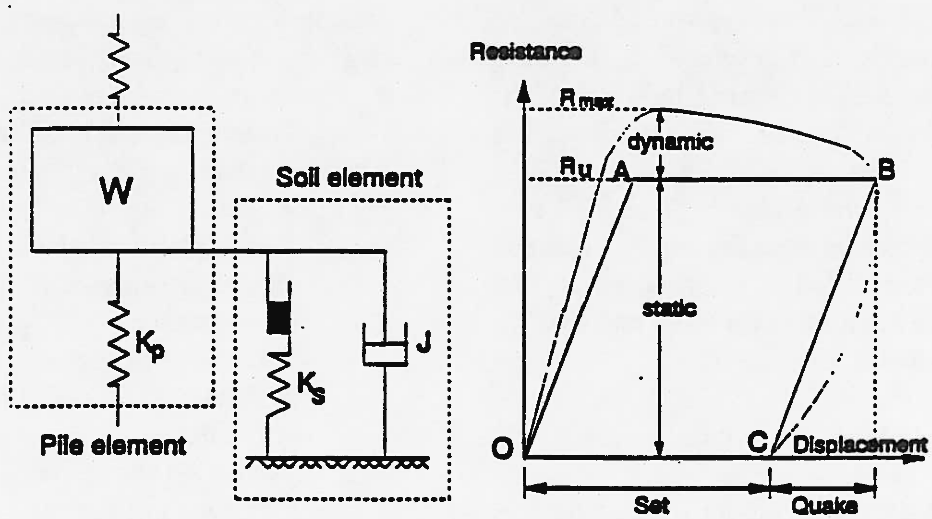

Figure 8b. Soil-pile model and the corresponding elasto-plastic soil resistance-displacement relationship (Smith, 1960)

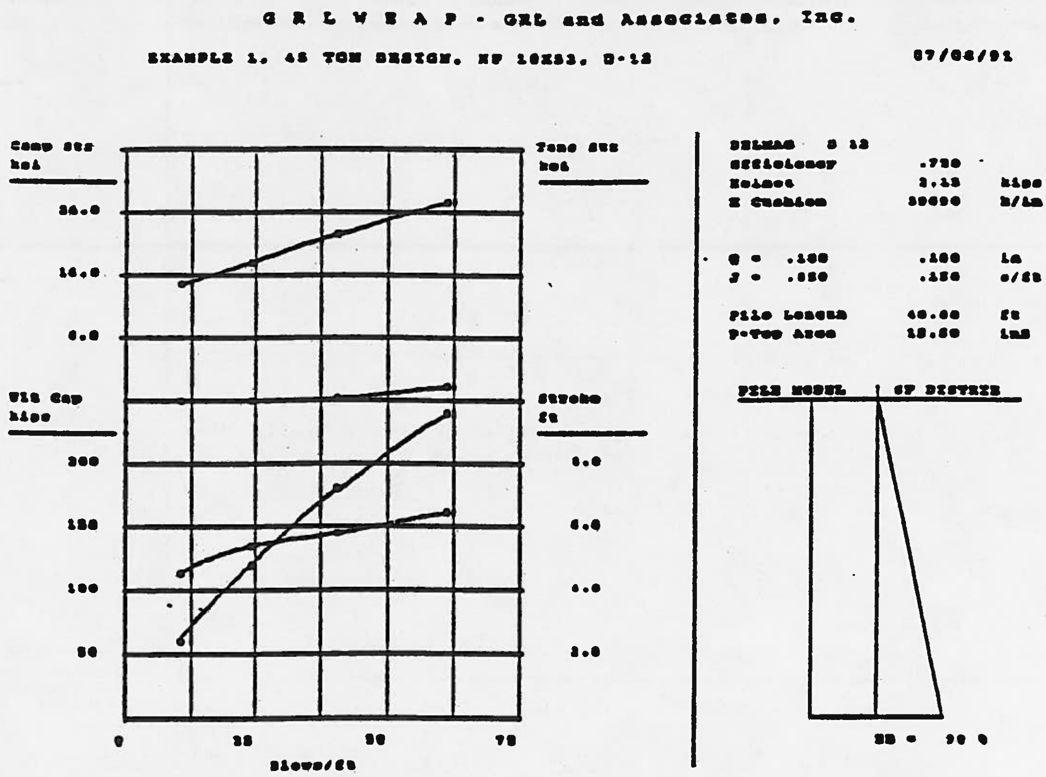

Figure 9. Plotted output for bearing graph results (standard analysis)

Consequently, seven major unknowns are present, in a WEAP analysis to estimate blow count and capacity. Uncertainty of these unknowns generally requires a higher FS (3), than static or post-driving tests:

1. Hammer efficiency
2. Hammer & cushion stiffness
3. Hammer & cushion COR
4. Skin quake
5. Toe quake
6. Skin damping
7. Toe damping

GRL (1992) presented a comparison (Figure 10) of WEAP capacity predictions vs load test capacity, with a mean of 1.12 and variance of 0.17.

Larger differences are usually associated with finer grained soils, due differences between static and dynamic soil properties.

Post-Driving (PDA, CAPWAP)

Post-driving analyses utilize the *measured* force signal (calculated from strain readings), and the *measured* velocity signal (integrated from acceleration measurements) obtained near the pile top during driving. As such, the energy delivered to the pile top is known.

"Real-time" capacity evaluation in the field is attractive for quality control and improved construction efficiency. Consequently, PDA was developed. The PDA capacity estimate is based upon the CASE method (Goble et al., 1970), which is based upon the assumption of a uniform elastic pile, ideal plastic soil behavior, and a simplified wave propagation formulation. The force and velocity measurements taken at the pile top, and a correlation between the soil at the pile tip to a damping

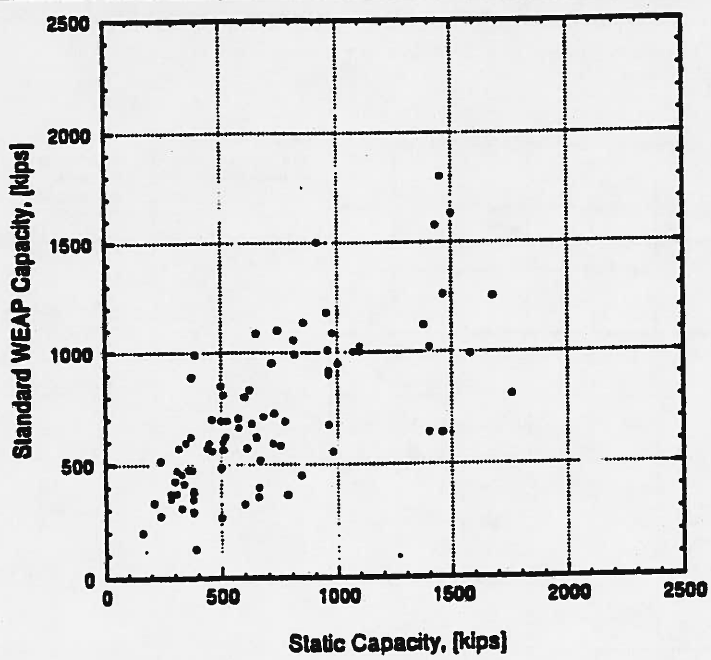

Figure 10. Standard WEAP capacity prediction vs. load capacity

parameter, J_{Case} (J_c) are employed. This latter parameter, J_c, is often the topic of discussion, and Table 1 presents recommended J_c values.

To better estimate the soil dynamic properties, and to be able to provide an estimate of the soil resistance (skin friction and tip), the CAPWAP (Case Pile Wave Analysis Program) was developed by Goble, Likins, and Rausche (1970). The procedure essentially consists of changing the soil model parameters for each pile element until a match is obtained between the calculated and measured force signals (Figure 11 presents an example). Having now obtained values (not necessarily

Table 1. Recommended J_c values according to the soil type at pile tip

Soil Type at Pile Tip	Goble et al., 1975	PDA Manual 1990
clean sand	0.05	0.10 to 0.15
silty sand	0.15	0.15 to 0.25
sandy silt	0.2	-
silt	0.3	0.25 to 0.40
silty clay/clayey silt	0.55	-
silty clay	-	0.40 to 0.70
clay	1.1	0.70 to 1.00

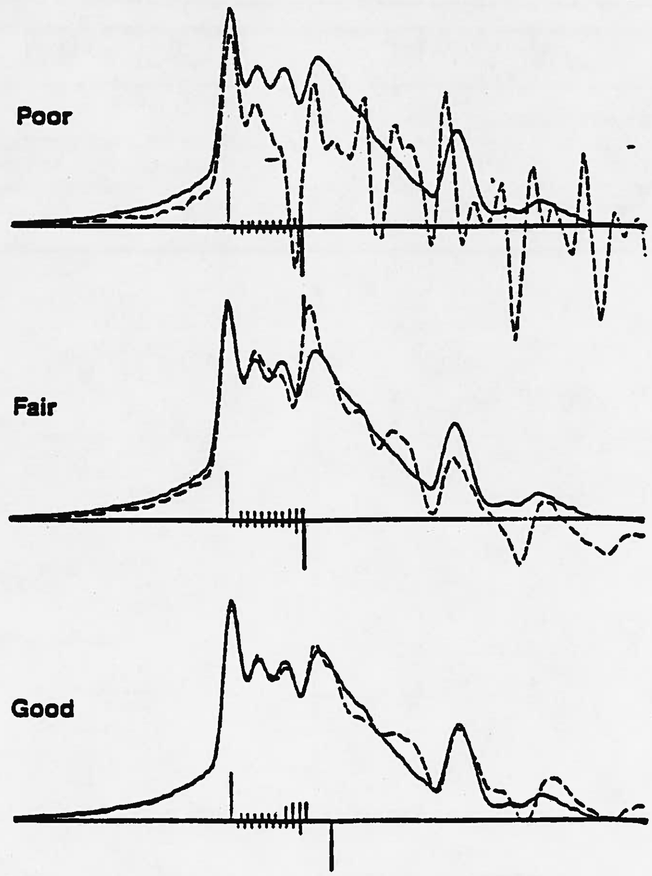

Figure 11. Three trial matches showing poor, fair and good match between measured (solid) and computed (dashed) pile top force

unique) for R_u, J, and q, the pile resistance distribution (Figure 12a), and simulated static load test (Figure 12b) can now be calculated. Since the CAPWAP method uses actual field measurements, the FS (2.5) can be used.

Figure 13 presents a comparison between static load test capacities and those predicted via CAPWAP. Also presented in Figures 14 and 15 are skin and toe quake values from CAPWAP analyses.

STATNAMIC

The same load testing desire as the Osterberg Cell is the impetus for Statnamic; specifically, (1) high capacity piles, and (2) elimination of reaction piles and beam. Statnamic, a junction of the words STATic and dyNAMIC, was developed to provide a more cost effective, less time consuming, and higher capacity method for determining the load bearing capacity of piles. Thus as indicated by the name, Statnamic load testing lies between static and dynamic load testing, and consists of linearly increasing forces of high magnitude with durations over 80 milliseconds an order of magnitude longer than those achieved via conventional drop hammers (as illustrated in Figure 16). The methodology was jointly developed by Berminghammer and TNO-BBC, and is a patented method (Bermingham and Janes, 1992).

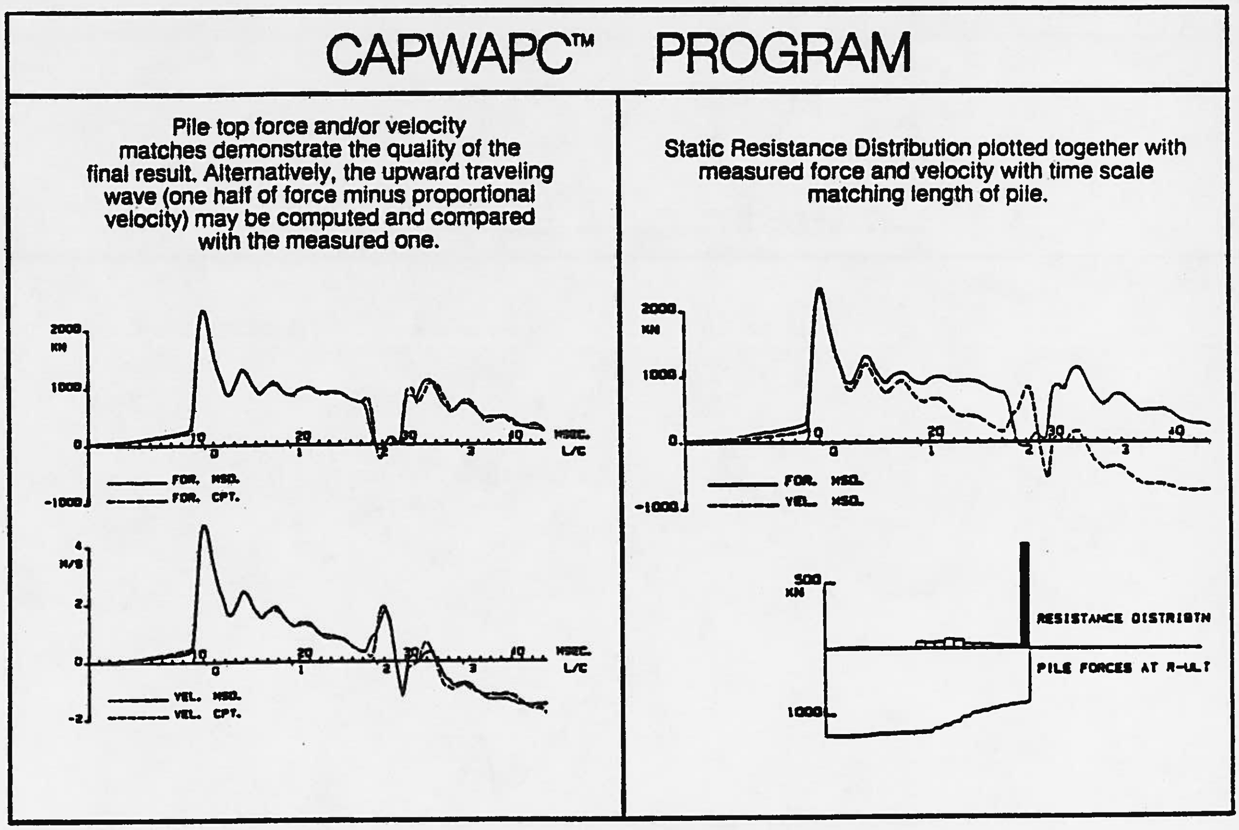

Figure 12a. Pile top force, velocity and static resistance plotted with time

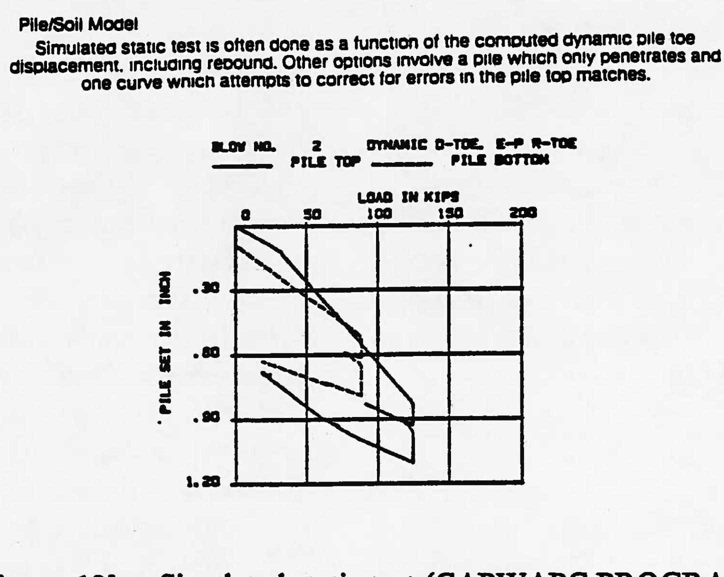

Figure 12b. Simulated static test (CAPWAPC PROGRAM)

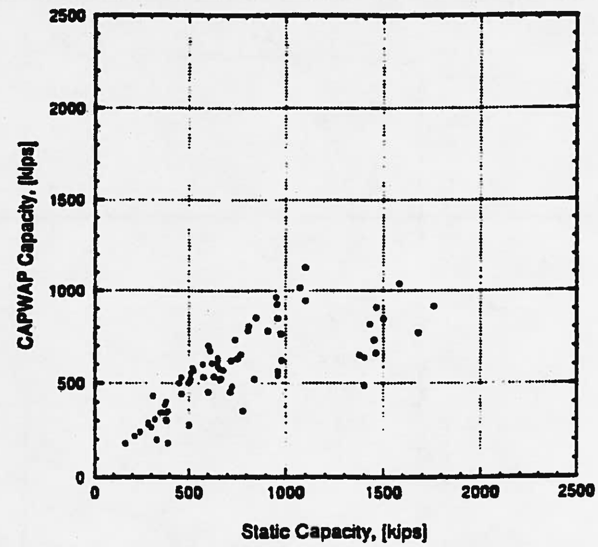

Figure 13. CAPWAP predicted capacities vs. static load test capacity

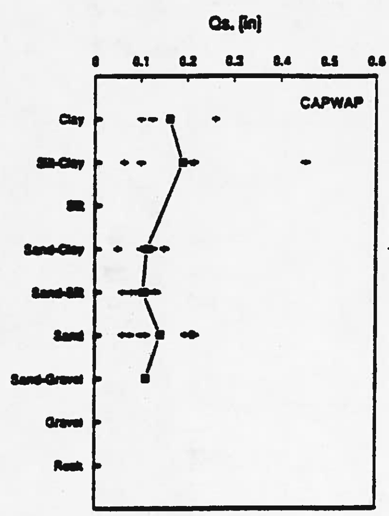

Figure 14. Skin quake values from well correlating CAPWAP analyses

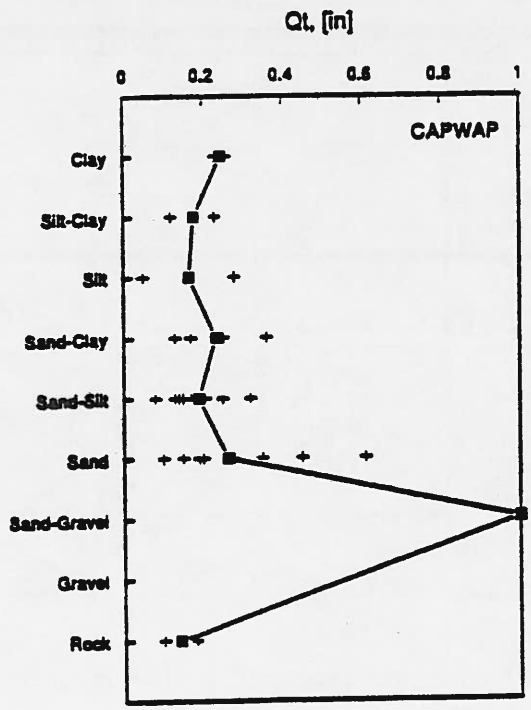

Figure 15. Toe quake values from well correlating CAPWAP analyses

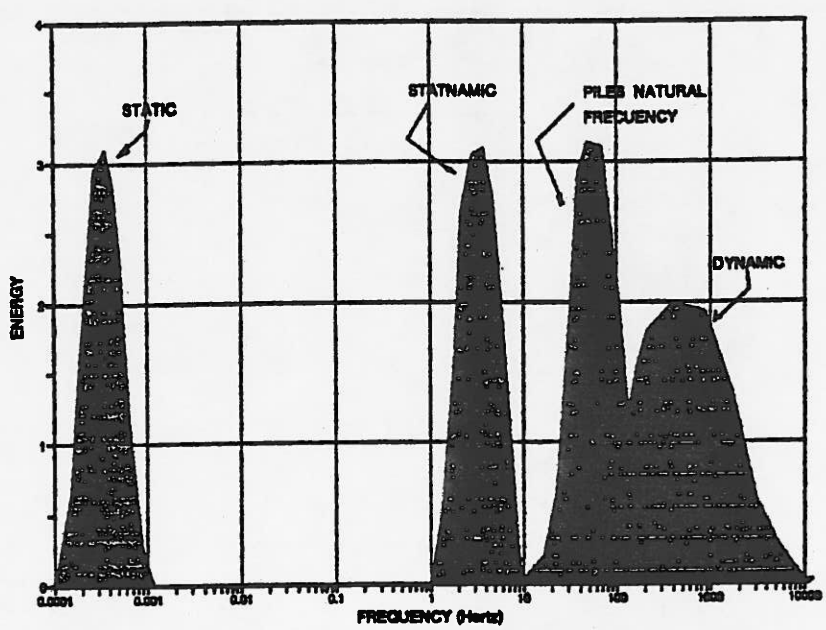

Figure 16. Frequency analysis of typical load methods and typical piles

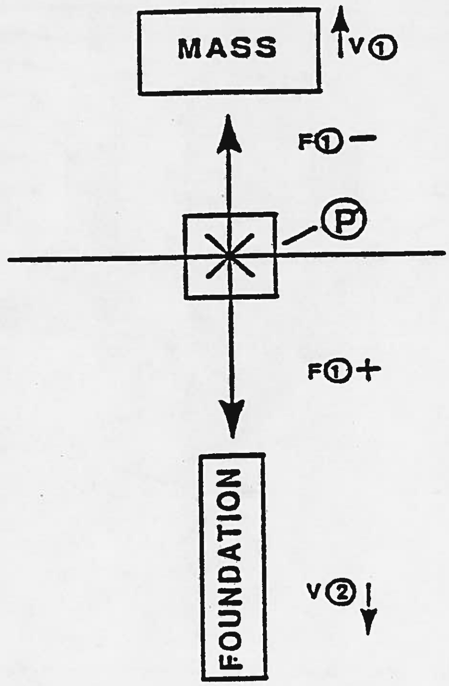

Figure 17. Schematic of STATNAMIC loading event

Statnamic Apparatus

The Statnamic load test consists of a pressure chamber and reaction mass placed atop the test pile. Fuel (or propellant) is ignited within the pressure chamber creating large pressures which drive the reaction mass upward at a high velocity (accelerations of 20 g). Simultaneously, the force driving the reaction mass upward acts equally downward loading the pile top. A schematic diagram of this event is illustrated in Figure 17. The load exerted on the pile top is measured by a load cell, while the displacement is measured by a special laser sensor. Dynamic instrumentation may be installed in the pile and high speed monitors used for capturing t-z information. In this fashion the load distribution is measured, and not calculated as for CAPWAP.

Figure 18 illustrates a cross-section of the device. The pressure chamber consists of a piston bolted to the pile top, covered by a cylinder to which the reaction mass is fixed. The piston contains a cavity for the propellant fuel, an ignition device, the load cell, and pressure transducer. The cylinder contains only a centralized valve and exhaust port. The cylinder rises off the piston before opening the valve, which vents the compressed gasses through a silencer. As the gasses are exhausted, the cylinder and reaction mass continue to rise, eventually clearing the piston, and enter free-flight above the pile. The total height reached may be approximately 2 meters. The loose sand around the reaction weights now flow down over the piston to cushion and "catch" the reaction mass as it falls back after launching.

The required reaction mass for a Statnamic test is about 5-10 percent of the pile capacity. For example, a pile having a test capacity of 2 MN will require a reaction mass of 0.1 to 0.2 MN (10 to 20 tons). The reaction mass can be readily constructed of site available materials; e.g., concrete, and/or steel.

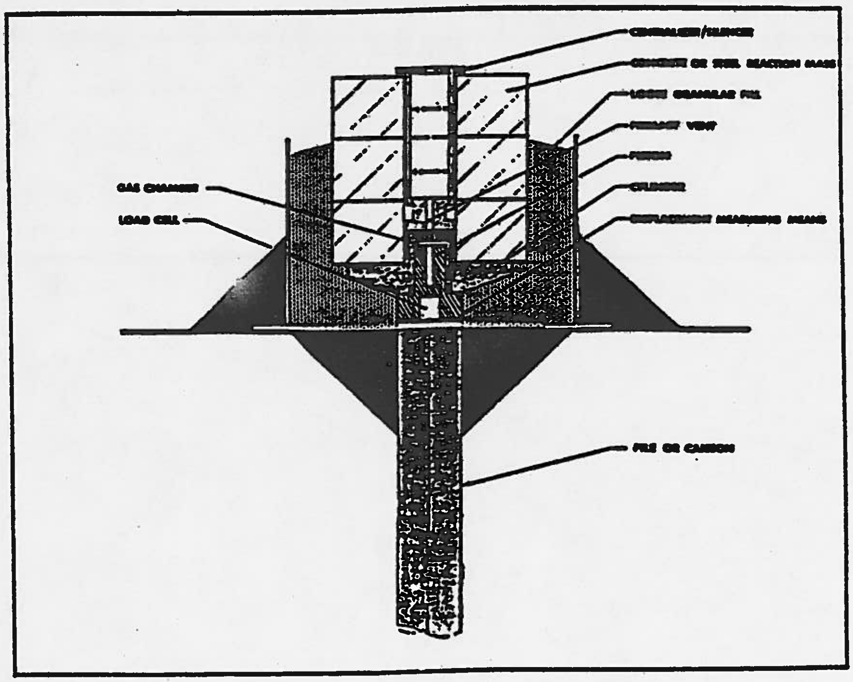

Figure 18. Cross-sectional view of STATNAMIC loading apparatus

Statnamic Characteristics

The method offers:

1. Linear loading characteristics, with accelerations of ≈ 1g as compared with 100 to 1000 g for conventional dynamic tests. Thus the pile behavior is a fast "quasi-static" test, where behavior is no longer dominated by stress wave propagation.
2. Since the test is "quasi-static" tensile stresses are not induced.
3. Battered, as well as, axial pile can be tested.
4. Relative efficient set-up and testing.
5. High capacities can be obtained 0.4 to 30 MN (400 to 3000 tons)

Some limitations may be considered:

1. t-z data requires high speed data acquisition equipment.
2. Dynamic phenomena must be considered. For very stiff soils or piles tipped in rock, the effect is minimal. However, for very soft soils, soil damping can influence results, and a dynamic model used.

Comparison of Statnamic and Static Load Test

Figure 19 presents a comparison between a 18.3 m x 178 mm diam closed-end steel pipe pile founded in interbedded clay, silt, and fine sand. A shale layer existed at a depth of about 19 m. The results show that the Statnamic test was insufficient to achieve static failure, but excellent agreement for load and displacement was obtained for the working stress range. Also shown is that under the Statnamic loading, a "plunging failure" was not observed.

CONCLUSIONS

1. The static load test (ASTM D-1143) is the hallmark for determining the capacity of single piles. Instrumentation allows one to obtain the load-

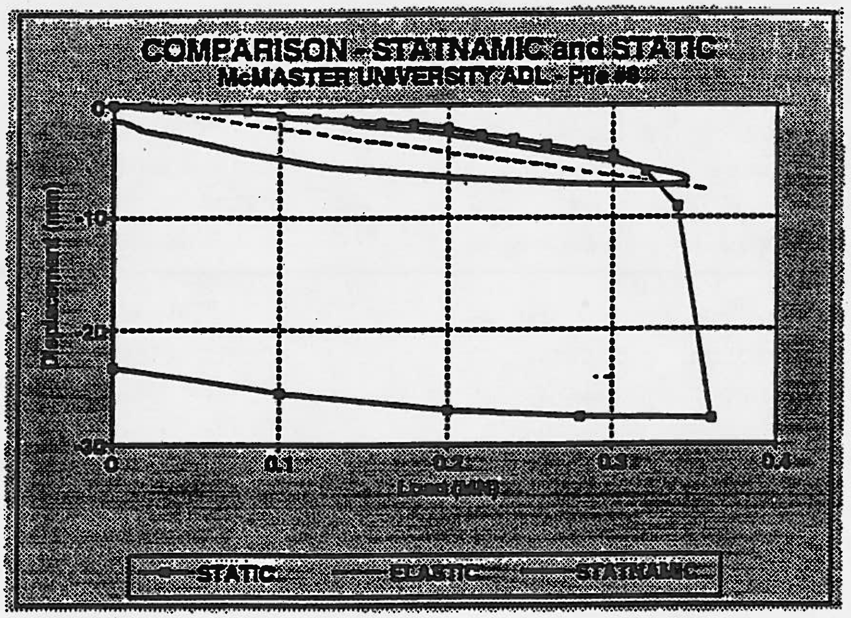

Figure 19a. Comparison between STATNAMIC and static load capacity for a typical pile

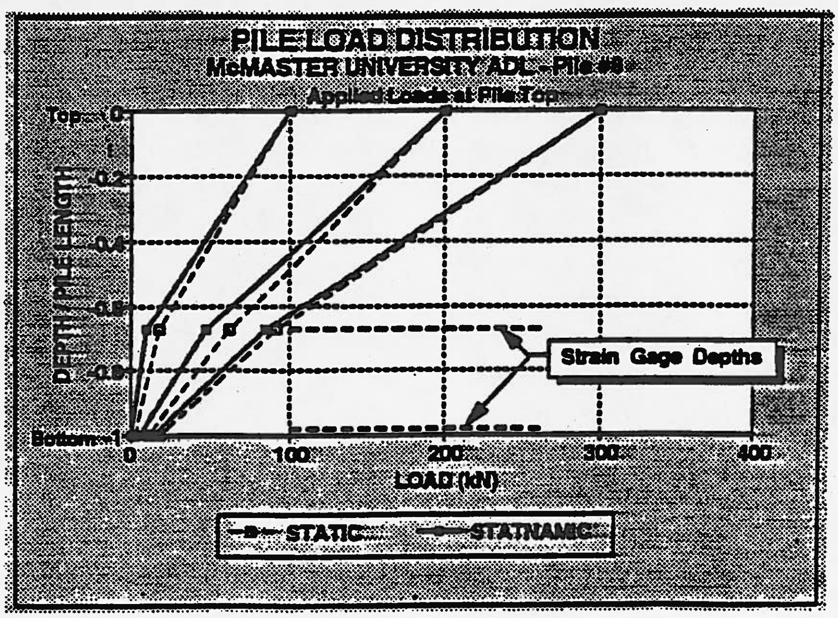

Figure 19b. Pile load distribution using STATNAMIC and static method

shedding response (t-z). However, the test requires reaction members and beam, which limit applied loads to ≈ 1000 tons, and requires considerable construction time for assembly.

2. The Osterberg Cell provides a high capacity static load test with equivalent surface loads to ≈ 6000 tons. The cell loads from the pile bottom, and as such no reaction members are required. Instrumentation allows one to obtain the t-z response. However, the test fails in either end bearing or skin friction. Upward loading in granular soils may produce conservative results.
3. Small scale static load tests are sensitive to scale effects. As such unit skin friction values may be unconservative for pile diameters less than 15 inches, or shaft sockets less than 6 inches.
4. Pre-driving wave equation analyses (WEAP) are designed for determining (a) suitability of driving equipment, (b) compressive and tensile stresses, and (c) bearing resistance in blows/ft. Although estimates are a vast improvement over driving equations, dynamic soil properties, and driving equipment energy are "key" unknowns affecting these estimates.
5. Post-driving wave equation analyses (PDA) are a major improvement over WEAP analyses in that the equipment energy is measured. Thus only the soil properties, particularly damping J_c, must be estimated. This method/equipment is quite successful for quality control, and provides "real-time" capacities. Although primarily used for driven piles where movements of 0.1 inch are obtained, the technique has been applied to drilled shafts using ram weights ≈ 10 percent of the ultimate shaft capacity. For example, rams of 20 tons (20 Mg) have been dropped 13 ft (4 m) (Seitz and Rausche, 1987).
6. CAPWAP analyses offer a viable method for estimating static load tests based upon measured and calculated iterations of soil properties in the wave equation.
7. STATNAMIC testing offers an alternative to O'Cell testing for high capacity piles, with loadings up to 3000 tons (27 MN). However, instrumentation is usually not incorporated for obtaining t-z information.

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APPENDIX A

Example Calculations of Load Transfer from Sister-bars

Given: Load applied at pile top = 150 tons; AE[e] of pile = 3.13E 9 lbs; Perimeter = 8.0 ft

Depth[a] (ft)	Distance (ft)	Strain, E[c] (E-06)	Deformation[d] (inches)	Axial Load[g] (tons)	Load[h] (tons)	Transfer (tsf)
0		(95.867)[f]		150.0		
	2.5		2.876E-3		0.0	0.0
2.5[b]		95.867		(150.0)[f]		
	11.5		1.258E-2		14.7	0.16
14.0		86.470		135.3		
	13.0		1.219E-2		26.1	0.25
27.0		69.795		109.2		
	2.0		1.616E-3		7.7	0.48
29.0		64.888		101.5		
	5.0		2.053E-3		95.9	2.40
34.0		3.561		5.57		
	1.0		4.273E-5			
35.0		(3.561)[f]		5.57		

(a) Location of telltale tips or sister-bar gages
(b) Ground surface
(c) Field sister-bar measurements
(d) Calculated deformations (theoretical telltale measurements) $s = \frac{(\epsilon_i + \epsilon_i + 1)}{2} * L$;
(e) $AE = Q/\epsilon = (150 \times 2000)/95.867E-06 = 3.13E9$
(f) Assumed uniform values
(g) $Q = AE * \epsilon$
(h) Load transfer = $(Q_i - Q_i + 1)/PL_i$; $(150 - 135.3)/8 * 11.5 = 0.16$ tsf

AXIAL RESPONSE OF DRILLED SHAFTS IN INTERMEDIATE GEOMATERIALS IN THE SOUTHEAST

by

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ABSTRACT

Concern regarding scour at bridge foundations has lead to deeper foundation embedment requirements and increased use of drilled shaft foundations for highway bridges in the Southeastern U.S. Very often these shafts are designed to penetrate relatively weak rock materials for which there are little load test data to provide design guidance. Data have been collected from a number of sites in the southeastern states which demonstrate a substantial amount of socket friction in relatively weak rock and dense soils. This paper presents the results of 12 axial tests in Alabama, Georgia, Mississippi, and South Carolina. Although these intermediate materials are often difficult to quantify with respect to in-place strength characteristics, these load test data provide the type of feedback necessary to develop judgement and form the basis for comparisons with proposed design methods.

INTRODUCTION

Drilled shafts are increasingly being used for highway bridge foundations in the Southeastern U.S. Design considerations for scour have led to deeper foundation embedment requirements, resulting in drilled shafts being designed as sockets in relatively weak rock materials. Very little load test data on shafts in these materials are available to provide design guidance. This lack of data has resulted in extremely conservative designs with regard to the mobilization of side friction and end bearing.

In order to provide better design guidance with respect to soft rock materials, the Alabama Department of Transportation (ALDOT) has sponsored a research program to investigate the axial capacity of drilled shafts constructed in weak rocks. These materials can range from very dense/hard soils to weak or soft rocks. By collecting data from tests conducted in materials similar to those encountered in Alabama, design guidance may be developed for use by ALDOT engineers. The results of 12 such tests are presented in this paper.

REVIEW OF DESIGN METHODS

Introduction

Most design methods in use today can be placed in one of four design approaches (Rosenberg and Journeaux, 1976). These are: 1) design based on end bearing only, 2) design based on skin friction only, 3) design based on allowable end bearing with remaining load carried in skin friction, and 4) design based on estimates of mobilized end bearing and skin friction.

The first three approaches place restrictions on the potential socket geometries available to carry a given load by disregarding all or part of the shaft's ability to mobilize one of the components of its capacity. These restrictions can lead to conservative designs since the full load carrying ability of the shaft is not considered.

The fourth approach assumes that the load is carried in both side friction and end bearing in proportions that depend on the actual load transfer occurring between the shaft and the soil or rock. An understanding of the load transfer relationship provides a means of estimating the expected values of mobilized side friction and end bearing. Predicting load transfer can be difficult since the amount of displacement needed at the top of the shaft to mobilize side friction and end bearing are often unequal. A relatively small amount of displacement is need to fully mobilize side friction, whereas relatively large displacements are necessary to completely mobilize end bearing (Osterberg, 1992). This relationship can make it difficult to determine the relative proportions of each component that are mobilized under a given load.

All of these approaches require designing the shaft based on some characteristic of the geomaterial in which the shaft is built. Various methods are based on in-situ strength tests, laboratory strength tests, elastic modulus values, or some combination of these or other characteristics. Yet, it is often difficult to quantify the in-situ strength characteristics of soft rock materials. Most design methods are therefore based on a correlation between unit side friction or unit end bearing and the unconfined compressive strength (q_u) of the rock. These correlations are common because (Horvath and Kenney, 1979): 1) the maximum potential friction at the shaft-rock interface is controlled by the shear strength of the weaker material (usually the rock), 2) the rock strength is dependent on material type, degree of weathering, extent of fractures and joints, etc., and 3) information concerning rock q_u values is readily available or easily obtained.

Methods Reviewed

A number of design methods were reviewed for the project. These include:

Rosenberg and Journeaux, 1976
Pells and Turner, 1979
Williams et al., 1980
Rowe and Armitage, 1987(a), 1987(b)
Reese and O'Neill, 1988
M^{c}Vay et al., 1992
O'Neill, 1993
Mayne, 1993

One early correlation of side friction to rock strength through full scale load tests was presented by Rosenberg and Journeaux (1976). Their method used a correlation of side friction to rock unconfined compressive strength along with load transfer curves developed by

Osterberg and Gill (1973) to select a shaft geometry that mobilized full shaft resistance without exceeding an allowable base resistance. The work of Osterberg and Gill included developing load transfer curves of a rock socketed shaft based on an elastic finite element analysis. They found that the distribution of the load depends on the depth of embedment of the socket, the socket diameter, and the elastic modulus and Poisson's ratio of the rock.

These early elastic solutions to predicting shaft capacity were further refined and expanded by Pells and Turner (1979), and then again by Williams et al.(1980). Pells and Turner presented two different design methods. One method assumes full mobilization of unit side friction and end bearing. The other method uses an elastic load distribution curve to distribute the load between side friction and end bearing depending on the socket geometry and the elastic modulus of the rock.

The work of Williams et al. resulted in detailed design methods based on load tests conducted in a Silurian mudstone in the area around Melbourne, Australia. Methods for designing shafts based on side friction only, end bearing only, or combined side friction and end bearing were presented. These methods involve estimating the ultimate unit values for side friction and end bearing, and then reducing these values according to predicted elastic load distributions. These reduced values are the predicted values for the mobilized unit side friction and end bearing. Ultimate unit side friction predictions are based on rock strength (q_u), while ultimate unit base resistance predictions are based on the elastic modulus of the rock.

Another design method based on elastic analysis and load test data was developed by Rowe and Armitage (1987(a), 1987(b)). Their method is based on satisfying a specified design settlement for an overall factor of safety. Correlations of unit side friction and end bearing to rock strength (q_u) are presented as a part of the method.

The most common design method currently in use in the United States is presented by Reese and O'Neill (1988). They recommend designing the shaft for capacity in either side friction or end bearing based on a computed settlement value. If the calculated settlement is greater than 0.4 inch, the bond between the shaft and the rock is assumed to be broken, transferring the entire load to the base of the shaft. For calculated settlements less than 0.4 inch, the bond is assumed to hold such that little or no load is transferred to the base of the shaft. Both unit end bearing and unit side friction are estimated through correlations to the unconfined compressive strength of the rock.

M^c^vay et al. (1992) presented a method for estimating unit side friction through a correlation to both the unconfined compressive strength and the split tensile strength of the rock. The authors believed that the use of both strength tests more accurately defined the strength of the shaft-rock interface. A database of load tests in Florida limestone was compiled to compare a number of correlations along with the new method.

Two of the most recent investigations of the design of shafts in rock were presented by O'Neill (1993) and Mayne (1993). O'Neill published a summary of preliminary design methods proposed under a research contract with the FHWA. Two methods are presented, one for argillaceous (or clay-based) rocks and one for decomposed (granular-based) rocks.

The method for argillaceous rocks is a modification of the method presented by Williams et al. (1980). The equations and associated graphs of the Williams et al. method have been combined into a series of equations solvable without use of the graphs. Also, a factor has been included to account for the smeared shaft-rock interfaces that can occur in excavations in some rocks of this type.

The method for decomposed rock was developed by Mayne (1993) through a load test program conducted on the campus of The Georgia Institute of Technology. Estimates of unit side friction and end bearing are made with correlations to the effective angle of internal friction and the undrained shear strength of the material, respectively. O'Neill suggests that this method is probably conservative for most decomposed rock.

Summary

The methods reviewed all rely to some extent on an elastic analysis of the rock-shaft interface. All of their predictions of unit side friction yield average values over the length of the rock socket based on peak load transfer. The use of the unconfined compressive strength to characterize the rock strength is also a common feature. Unit end bearing is determined by either an elastic analysis of the controlling settlement or is taken as a simple linear correlation to the q_u value.

A more detailed review of each of the above methods is presented in Thompson (1994).

LOAD TEST DATA

Load tests provide the most potentially reliable method to verify design parameters or methods. The results of a number of axial load tests conducted in soft rock formations in the Southeastern U.S. have been collected for this project. Some of the tests were conducted using a conventional static load test set-up consisting of loading the shaft at the ground surface. Other tests were conducted using the Osterberg Cell loading device (Osterberg, 1989). Table 1 lists the location, geology, test type, and source of each test.

In order to present the data collected, the tests have been grouped into two categories: argillaceous rocks and granular-based rocks. Tables are presented below that compare the results of each test to the predicted values for side friction and end bearing as computed by several different methods. For the tests in argillaceous rocks, predicted values from the methods of Williams et al. (1980), Rowe and Armitage (1987), Reese and O'Neill (1988), and O'Neill (1993a) are given. Predicted values form the methods of Mayne (1993) and Reese and O'Neill (1988) are given for the tests in granular based rocks. M^{c}Vay's method is considered separately.

Table 2 gives the side friction data for the argillaceous rock tests. The value of unit side friction given is the average for the portion of the shaft socketed into the subject geomaterial. The deflection (δ) given is the deflection at which the average side friction was mobilized. Deflections from Osterberg Cell tests are given as positive, representing the upward movement of the shaft.

From these data, it appears that the method of Reese and O'Neill generally underpredicts unit side friction while the other three methods generally overpredict unit skin friction. At both the

Table 1. Load test summary

LOCATION	GEOLOGY	TYPE	REFERENCE
Andalusia, AL	Claystone	Conventional	Bhate Eng. Corp., 1992
Blount Co., AL	Shale	Osterberg Cell	Hwy. Rsch. Ctr., 1994
Montgomery, AL	Very dense sand	Conventional	Brown, 1994
Tuscaloosa, AL	Very dense sand	Osterberg Cell	Loadtest, 1992
Wilsonville, AL	Shale	Osterberg Cell	Loadtest, 1994a
Atlanta, GA	Weath. Granite	Conventional	Mayne, 1993
Coewta Co., GA	Weath. granite	Conventional	O'Neill, 1993
Owensboro, KY	Shale	Osterbrg Cell	Goodwin, 1993
Leake Co., MS	Clay/chalk	Osterberg Cell	Loadtest, 1994b
Mt. Pleasant, SC	Marl	Conventional	Law Engineering, 1991

Table 2. Side friction data for argillaceous rocks

LOCATION	ROCK	AVG f_s (tsf)	δ @ f_s (in)	Predicted f_s (tsf)			
				WILLMS. ET AL	ROWE & ARMTGE	REESE & O'NEILL	O'NEILL
Andalusia Alabama	Claystone	4.8	-0.13	5.7	6.7	3.1	6.0
Andalusia Alabama	Claystone	3.5	-0.61	5.7	6.7	3.1	6.0
Blount Co. Alabama	Shale	>11.5	0.07	8.5	11.7	5.4	7.5
Wilsonville Alabama	Shale	3.2	0.66	3.2	3.1	0.7	4.4
Owensboro Kentucky	Shale	>9.9	0.36	5.8	7.0	3.2	6.1
Leake Co. Mississippi	Clay/ Chalk	1.6	0.18	3.6	3.6	0.9	4.6
Mt. Pleasant S. Carolina	Marl	1.8	-0.15	2.1	1.7	0.2	3.5
Mt. Pleasant S. Carolina	Marl	1.8	-0.10	2.1	1.7	0.2	3.5

Blount County and Owensboro tests, the capacity of the Osterberg cells was reached before the sockets failed. It appears that for both of these tests, all four methods are conservative.

The data for end bearing of the argillaceous rocks is shown in Table 3. The rock strength given for the Wilsonville test is an estimate based on the available SPT blow count. The blow count was used to estimate an undrained shear strength from which q_u was estimated. The end bearing measured at a deflection of 2 percent of the shaft diameter (B) is used as the basis of comparison. Although a legitimate argument could be made that additional end bearing is available at larger deflections, this value represents one that would ordinarily be considered "large displacement" for drilled shafts associated with bridge foundations and most such structures are not capable of mobilizing larger displacements without severe structural distress. Comparisons could be made at any chosen percentage.

Since most design methods predict a maximum unit end bearing as a multiple of the unconfined compressive strength, the ratio of the measured unit end bearing to the rock strength is given. Ratios of 2 to 3 are common for most design methods. The data presented here indicate ratios which are generally well above 3. The Blount County test has a ratio of less than one. It is believed that a large amount of debris was present at the bottom of the shaft excavation when the Osterberg Cell was installed. Most of the measured downward movement of the cell was probably the result of the compression of the debris, resulting in low end bearing measurements (Highway Research Center, 1994).

Tables 4 and 5 present the side friction and end bearing data, respectively, for the granular-based rock tests. The "β" method is the method for designing shafts in granular soils presented by Reese and O'Neill (1988). This method is presented as an alternative to Mayne's method since granular-based rocks weather into granular soils. Some of the tests were conducted in very dense sands similar to weathered rocks.

From these data, there do not appear to be any general trends for either method in regards to predicting skin friction. It does appear from these data that Mayne's method under predicts end bearing while the β-method appears to over predict unit end bearing. These observations are based on unit end bearing values mobilized at shaft deflections of 2 percent of shaft diameter.

Table 6 provides a comparison of side friction values as predicted by M^{c}Vay's method to measured values. Only two of the tests had split tensile data available: Blount County, Alabama and Owensboro, Kentucky. Both of these tests reached the capacity of the Osterberg cells before the socket failed. More data in rocks other than Florida limestone (the material from which the method was derived) are needed to better review this method.

CONCLUSIONS

This project represents an initial step for more reliable predictions of shaft capacity in soft rocks encountered in Alabama. The lack of a large number of tests makes statistical analyses such as linear regression or goodness of fit impractical; however, some trends are recognized in the available data. Additional load tests will be added to the

Table 3. End bearing data for argillaceous rocks

LOCATION	q_u (tsf)	q_b (tsf)	δ=2%(B) (in)	q_b/q_u
Blount Co. Alabama	64.8	42.8	-0.64	0.66
Wilsonville, Alabama	4.5	58.2	-0.64	12.9
Owensboro, Kentucky	23.0	113.0	-1.42	4.9
Leake Co., Mississippi	6.0	8.4	-1.32	1.4
Mt. Pleasant, S.Carolina (Shaft 1)	1.4	13.5	-0.48	9.6
Mt. Pleasant, S.Carolina (Shaft 2)	1.4	10.0	-0.48	7.1

Table 4. Side friction data for granular-based rocks

LOCATION	ROCK	AVG f_s (tsf)	δ @ f_s (in)	Predicted f_s (tsf)	
				Mayne	6
Montgomery Alabama	Dense Sand	1.7	-0.85	1.2	1.3
Tuscaloosa Alabama	Dense Sand	1.6	+1.11	2.2	1.4
Atlanta Georgia (C-1)	Weathered Granite	>3.2	-1.02	2.9	1.6
Atlanta Georgia (C-2)	Dense Sand	0.7	-1.0	0.7	1.4
Coweta Co. Georgia	Weathered Granite	1.4	-1.65	2.6	0.9

Table 5. End bearing data for granular-based rocks

LOCATION	ROCK	q_b (tsf)	δ 2%(B) (in)	Predicted q_b (tsf)	
				Mayne	**6**
Montgomery Alabama	Dense Sand	30.6	-0.60	10.3	36.0
Tuscaloosa Alabama	Dense Sand	34.0	-0.72	22.2	45.0
Atlanta Georgia (C-1)	Weathered Granite	46.4	-0.60	24.6	45.0
Atlanta Georgia (C-2)	Dense Sand	7.1	-0.60	6.3	12.0
Coweta Co. Georgia	Weathered Granite	24.0	-0.72	21.8	45.0

Table 6. Comparisson of M^{c}Vay's method

LOCATION	q_u (tsf)	q_t (tsf)	MEASURED f_s (tsf)	PREDICTED f_s (tsf)
Blount Co., Alabama	64.8	18.3	>11.5	17.2
Owensboro, Kentucky	23.0	4.7	>9.9	5.2

database as they are available in order for further analyses to be made. At present, no single method reviewed herein can be selected as providing a better prediction of shaft capacity than methods in use by the Alabama DOT. With the addition of more tests in the future, a method may be determined as better for the soft rocks encountered in Alabama.

ACKNOWLEDGEMENTS

The authors would like to thank the many firms and individuals that provided data and/or technical assistance for this project. These include Bhate Engineering Corporation, Christian Testing Laboratories, Inc., Law Engineering, Inc., Loadtest, Inc., Long Foundation Drilling Company, The Morris-Shea Bridge Company, Hank Covington (Alabama DOT), Randy Cannon (South Carolina DOT), Dr. Mike O'Neill (University of Houston), and Richard Sheffield (Mississippi DOT).

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A DRILLED SHAFT EXPERIENCE - WILLIAM H. NATCHER BRIDGE OWENSBORO, KENTUCKY

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ABSTRACT

U.S. Route 60/231 is being upgraded to four lanes between Owensboro, Kentucky and Interstate 64 near Dale, Indiana. This route includes crossing the Ohio River via either a steel or concrete cable-stayed bridge. Drilled shafts socketed several feet below the top of bedrock will be used in the foundation systems for all river piers, and drilled shaft alternates are also being developed for land piers and end bents. The foundation systems for the two main, cable-stay towers were designed ahead of other foundation elements to allow partial letting of the bridge construction contract. Construction of these two tower foundations, completed in 1994, will accommodate either the steel or concrete alternate.

Because of the types of geomaterials encountered at the site, and because of the large axial and lateral loads to which drilled shafts in the tower foundations will be subjected, full-scale technique shafts were constructed and load tests were performed on production shafts as part of the foundation contract. In addition, the use of a polymer drilling slurry to reduce degradation of soft shales during excavation of the shafts was evaluated and implemented during construction.

This paper will present project background information; outline the geotechnical exploration conducted and conditions encountered; discuss selection and design of foundation systems for the two main tower piers, and how the selection process was influenced by scour concerns, axial and lateral loading conditions, and Federal Highway Administration directives regarding foundation requirements; detail the installation of technique shafts and load testing of production shafts; and summarize how load test data were used to revise shaft bottom elevations at Piers 8 and 9, and were correlated with available design methods to assist in estimating shaft capacities at remaining river pier locations.

INTRODUCTION

Background

The Kentucky Transportation Cabinet, Indiana Department of Transportation and the Federal Highway Administration are working together to design and construct a new four-lane bridge to span the Ohio River and connect U.S. Route 60 in Daviess County, Kentucky with U.S. 231 in Spencer County, Indiana. The proposed bridge is part of the U.S. 60 relocation project in Kentucky, and the Kentucky Transportation Cabinet is the lead agency for management of the project. The structure has been named in memory of the late U.S. Representative William H. Natcher from Kentucky.

The bridge will be positioned approximately nine miles upstream of Owensboro, Kentucky near river mile 745.6 and will exhibit an end bent to end bent length of approximately 4,500 feet. The bridge will consist of 16 to 18 spans, depending upon which superstructure design alternate is selected. Five of the pier locations will be positioned within normal pool limits of the Ohio River, with the two main tower piers adjacent to the river navigation channel providing a clearance of about 1,100 feet. A vicinity map showing the location of the bridge is presented in Figure 1.

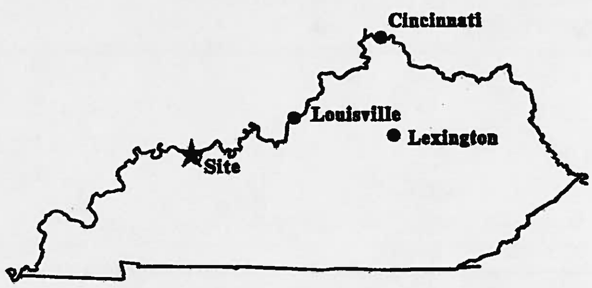

Figure 1
Vicinity Map

Site Description

The bridge will cross the Ohio River at a large meander in the river, and in an area where the floodplain exhibits one of its widest reaches. The width of the floodplain at the bridge site exceeds four miles. Topography on both the Kentucky and Indiana sides of the river is relatively flat, and current land use at the site is predominantly agricultural. Figure 2 presents the overall roadway alignment and bridge crossing on combined portions of the Rockport and Lewisport 7½-Minute Indiana-Kentucky topographic maps.

Geologic Setting

Geologic maps of the Rockport/Lewisport, Kentucky; Maceo, Kentucky; and Vincennes, Indiana quadrangles (USGS 1964 and 1966; and IGS, 1970, respectively) are representative of the regional geology of which the bridge site is a part. Soils at the structure location consist of Wisconsin age glacial outwash deposits overlain by more recent alluvial and overbank soils deposited during and/or following flooding of the Ohio River Valley.

Underlying these depositional soils is bedrock of the Carbondale and Tradewater Formations, representing the Middle Pennsylvanian geologic age and consisting primarily of sandstones, shales and siltstones. Relatively thin zones of coal and underclays, and limestones are occasionally present within these formations.

The William H. Natcher Bridge site lies within the Mississippi Valley Seismic Region and is actually positioned on the eastern edge of the Wabash Valley seismic zone. The bridge could also be subjected to ground motions associated with earthquakes which originate within the New Madrid seismic zones. Figure 3 shows the proximity of the bridge site to these seismic sources.

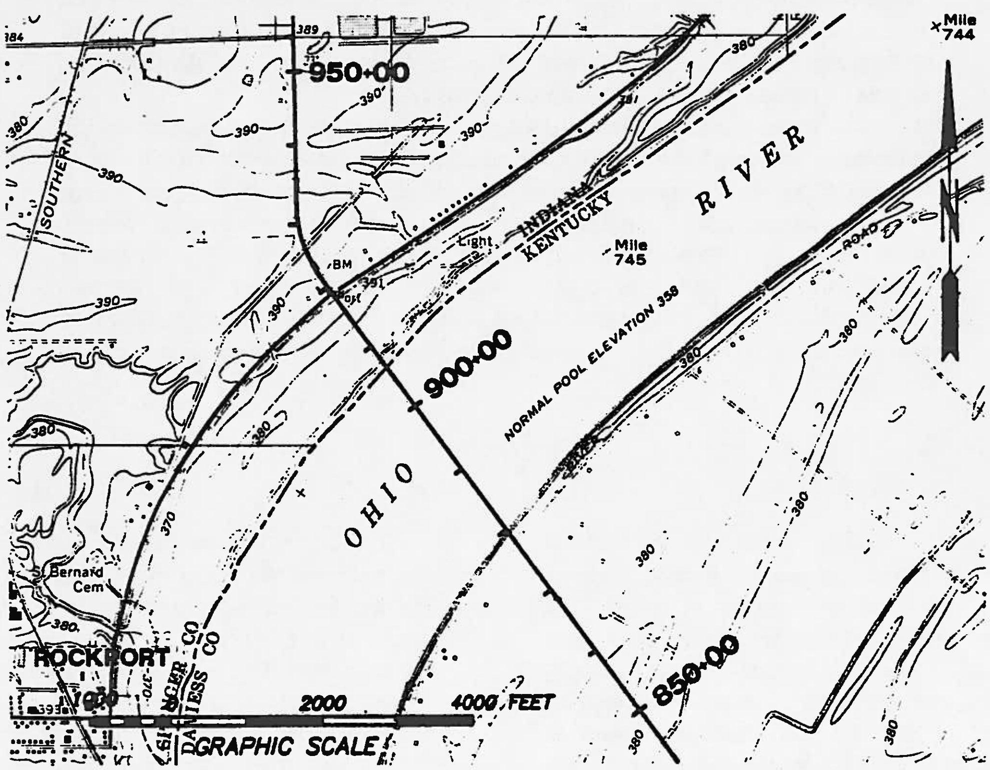

Figure 2

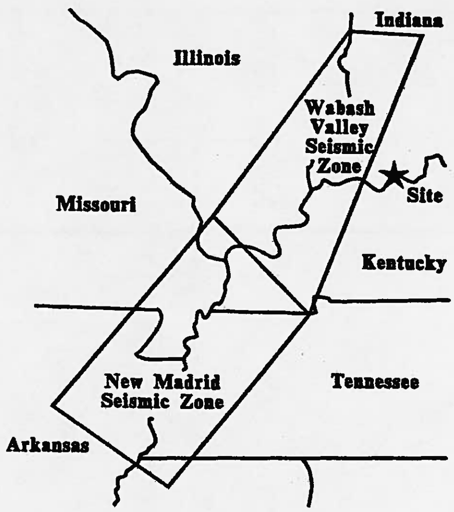

Figure 3
Bridge Site in proximity
to Seismic Zones

GEOTECHNICAL EXPLORATION AND SITE CONDITIONS ENCOUNTERED

General

A preliminary geotechnical exploration was conducted at the bridge site by the H.C. Nutting Company in 1990. The final geotechnical report for the structure was prepared by Fuller, Mossbarger, Scott and May Engineers, Inc. (Fuller, et.al., 1994). The following sections refer to data collected for compilation of the latter report.

Soil Conditions Encountered

Drilling and sampling operations conducted at the bridge site were implemented from April, 1991 through June, 1991. Figure 4 presents the complete boring layout for the bridge. The soils encountered in the land borings drilled on the Kentucky side of the river generally exhibit the profile shown in Table 1.

Table 1
Kentucky Land Borings Soil Profile

Approximate Depths (feet)	Soil Description
0 - 3	Topsoil
3 - 18	Lean to fat clays with sand and sand lenses, moist, medium to stiff
18 - 37	Silty sands with occasional gravel, moist to wet, loose to medium
37 - 112	Silty sands with gravel and gravel zones, wet, medium to very dense
112+	Top of Rock

The same general profile trends shown by land borings conducted on the Kentucky side of the Ohio River are evident in land borings drilled on the Indiana side of the river. Generally, upper deposits of clayey soils with sand lenses and zones grade with depth, into silty sands with gravel and gravel zones.

Table 2 summarizes the typical soil profile encountered at Indiana land boring positions.

Table 2
Indiana Land Borings Soil Profile

Approximate Depths (feet)	Soil Description
0 - 2	Topsoil
2 - 30	Lean to fat clays, moist, medium to stiff
30 - 50	Silty sands with occasional gravel, moist to wet, loose to medium
50 - 105+	Silty sands with gravel and gravel zones, wet, medium to very dense
105+	Top of Rock

Soils at river boring positions consist primarily of silty sands with gravel and gravel zones, and are present in thicknesses ranging from 60 to 80 feet. The upper 20 to 30 feet of these sands typically exhibit loose relative densities, with N-values less than 10 being common in this zone. Below 30 feet, the sands grade into deposits of medium to very dense sands with gravel.

Bedrock Conditions Encountered

Bedrock Present at Land Pier Locations. Bedrock present at Kentucky land boring positions typically consists of an upper unit of gray, fine to medium grained sandstone, 4 to 14 feet thick, above gray to dark gray, silty to sandy shales. The shales extend to the bottom depths of the test borings. A thin coal seam, less than two feet in thickness, is present at depths ranging from about 15 feet to 25 feet below the top of rock at these locations. At boring positions on the Indiana side of the river, the upper unit of sandstone is not present and the shales become more sandy and are more interbedded with sandstone with depth. Increases in sand percentages and amounts of interbedded sandstone also occur with increasing distances north away from the river.

Bedrock Present at River Pier Positions. Bedrock present at river pier positions 5 through 8 is similar to that encountered in the Kentucky land borings, with the

Subsurface Plan

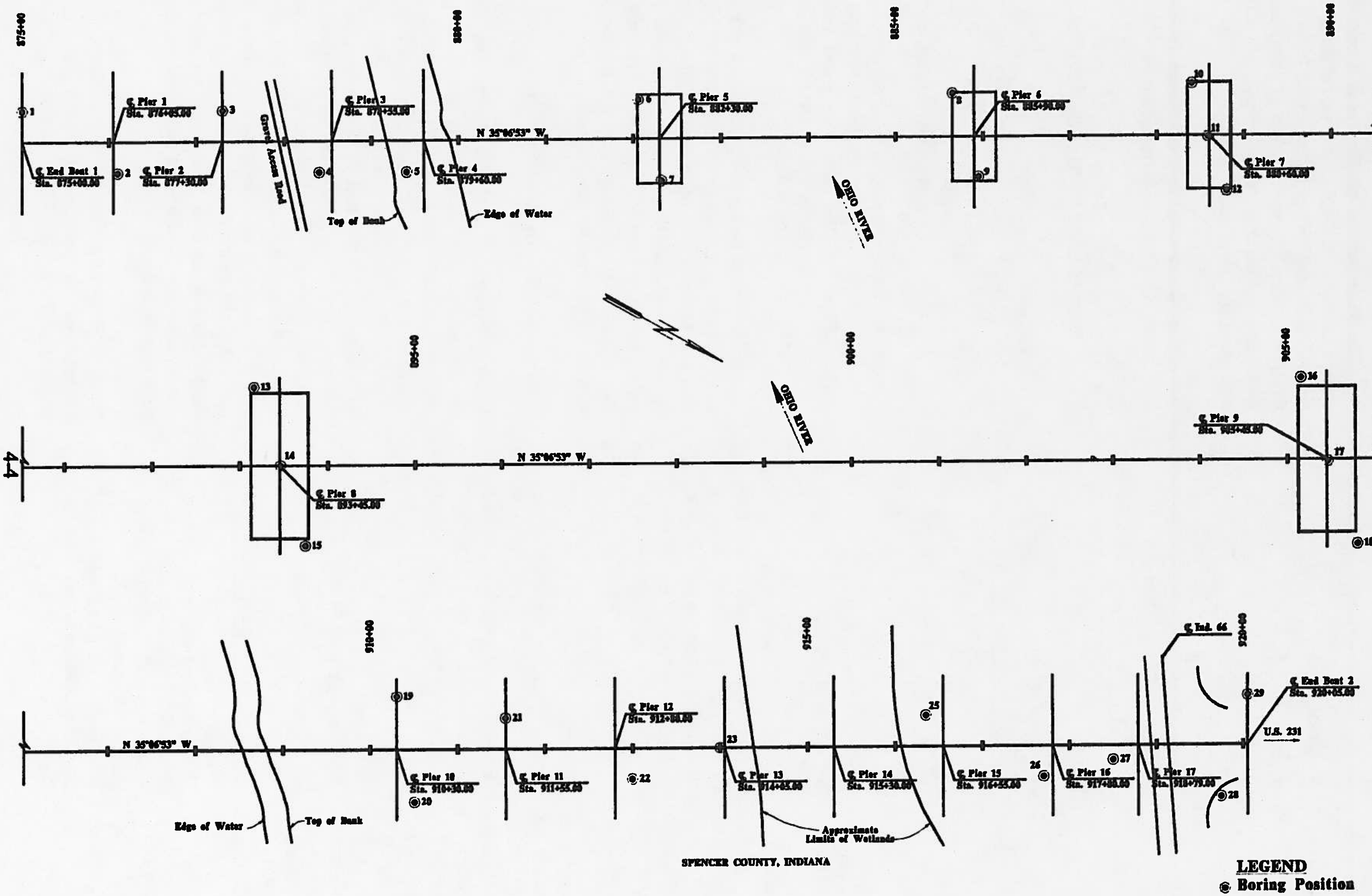

Figure 4
Boring Layout

exception that the upper sandstone unit is not present at the river pier locations. The bedrock at pier positions 5 through 8 consists primarily of non-durable shales to an elevation of about 222 feet (48 feet below the top of rock). Below this elevation more durable shales are present. A thin coal seam and a thin limestone zone are occasionally present at approximate elevation intervals 250 to 254 feet and 229 to 230 feet, respectively. Rock quality designations (RQD) of less than 25, and commonly zero, are representative of the bedrock encountered at these pier positions.

Bedrock conditions at the location of Pier 9 differ significantly from those determined at river piers 5 through 8. Cores obtained from the Pier 9 position consist primarily of interbedded units of shales and sandstones. Even though each boring drilled at the location of Pier 9 was advanced to considerable depths below the elevations at which coal seams were observed in other river borings, the coal seams were not detected in the Pier 9 borings. A coal seam was present in borings performed for all river substructure elements south of Pier 9, and in borings drilled for Piers 10 and 11, located north of Pier 9. Such differences in bedrock lithologies resulted in significant differences in foundation systems for the two main tower piers.

Graphical representations of the soil and bedrock conditions typically encountered at Kentucky and Indiana land borings, and at river boring positions, are shown in Figure 5.

SELECTION OF FOUNDATION SYSTEMS FOR TWO MAIN TOWERS, PIERS 8 AND 9

Initial Recommendations

The drilling and sampling phase of the geotechnical exploration was completed in June of 1991. The Kentucky Department of Highways (KYDOH) was under extreme pressure to begin construction of the project before the year's end. To support an accelerated design process and allow at least a partial letting of bridge construction, Fuller, Mossbarger, Scott and May Engineers, Inc. (FMSM) issued a geotechnical report for the two main river piers on July 12, 1991 (Fuller, et.al., July, 1991). At that time, structural loading information and scour predictions for pier locations were not available from the design team.[1] Initial foundation recommendations included steel H-pile alternates driven to refusal on bedrock, and drilled shafts socketed into rock. FMSM noted in that report that lateral loading (and scour considerations) and/or uplift loading may ultimately control the selection of foundation systems. Lateral load studies were beyond the initial scope of FMSM's work.

Capacities for drilled shaft alternates were estimated using available subsurface data and procedures outlined in Publication No. FHWA-HI-88-042, "Drilled Shafts: Construction Procedures and Design Methods" (O'Neill, et.al., 1988). The procedures presented in that publication recommend that capacities of drilled shafts socketed into rock be based upon either side-load transfer to, or end bearing on rock, depending upon the unconfined compressive strength of the bedrock. Combining side-load transfer and end bearing to estimate ultimate shaft capacities in rock sockets was not recommended. In accordance with FHWA-HI-88-042, shaft capacities at the Pier 8 position were based upon side-load transfer to the rock socket only, because the soft shales at this location exhibit unconfined compressive strengths less than 280 pounds per square inch. Unit side resistance values were estimated using the following equation:

$$f_s = 0.15 q_u \qquad (1)$$

where f_s = ultimate unit side resistance (psi)
q_u = unconfined compressive strength (psi)

Ultimate shaft capacities at Pier 8 were then estimated using :

$$Q_s = \pi D L f_s \qquad (2)$$

where Q_s = ultimate side resistance
D = diameter of rock socket
L = length of rock socket
f_s = unit side resistance

A safety factor of 3 was recommended to determine allowable capacities unless full-scale load testing was conducted.

Again, utilizing procedures outlined in FHWA-HI-88-042, shaft capacities at Pier 9 were based upon end bearing only because $q_u > 280$ psi (see Figure 5). FMSM recommended an allowable end bearing capacity of 50 tsf. FMSM noted that the shafts may need to be socketed below the RDZ to develop sufficient resistance to uplift and/or lateral loadings.

1 Design Team. Haworth, Meyer and Boleyn, Inc., of Frankfort, Kentucky - Lead consultant and concrete alternate design; Parsons Brinckerhoff Quade & Douglass, Inc., of Louisville, Kentucky - steel alternate design.

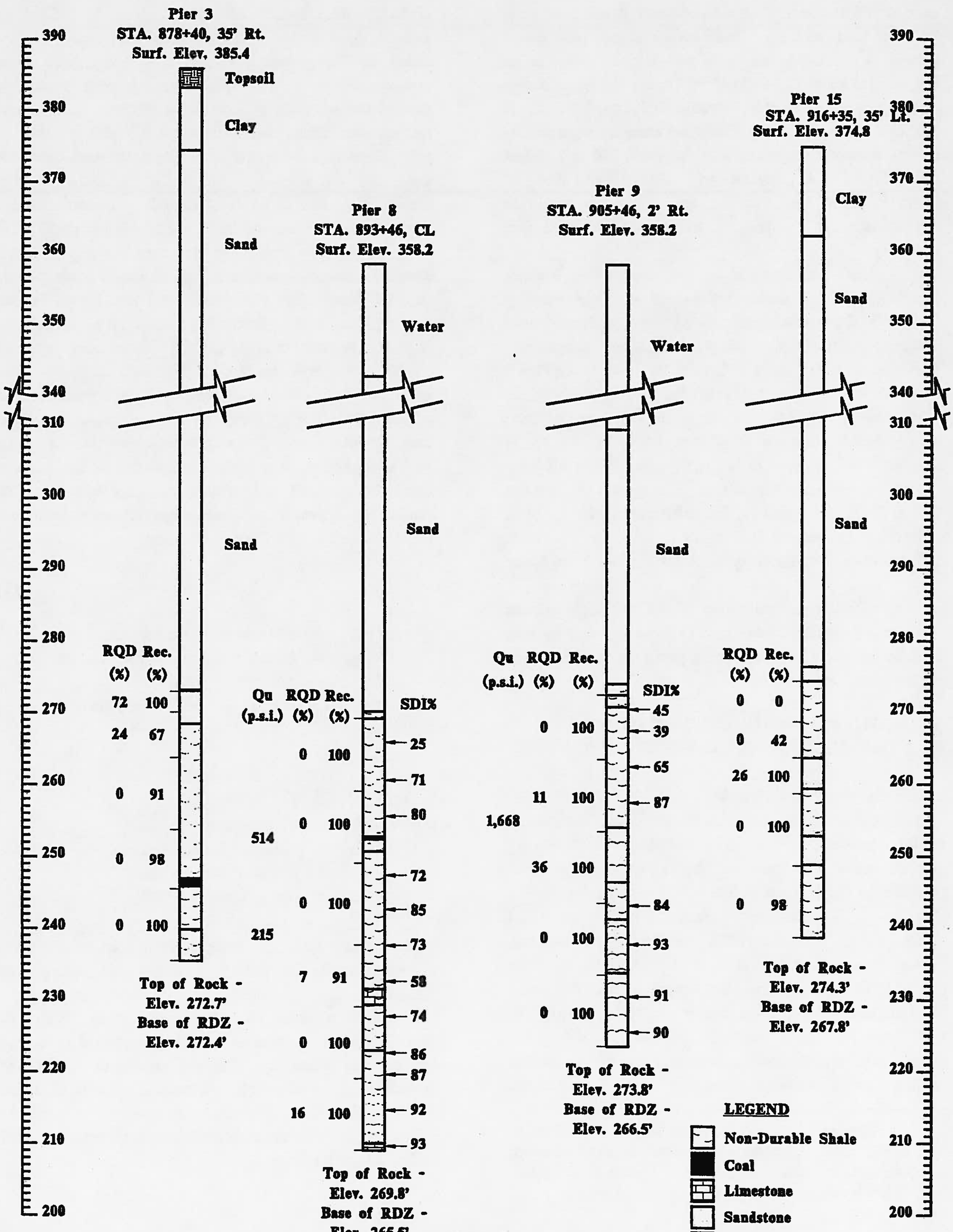

Figure 5
Logs of Borings

FHWA Design Directives

Following completion of the geotechnical report for the two main river piers, FHWA issued design directives in September of 1991 that would ultimately make selection of the foundation systems for Piers 8 and 9 a relatively easy task. The design team was directed to:

- Utilize the state-of-art, two-dimensional finite-element computer model FESWMS (Froehlich, 1989) to re-evaluate hydraulics at the bridge site and update predicted scour depths at substructure element locations.

- Locate bottoms of footings or foundation seals no higher than the lowest scour elevation, or at the lowest streambed elevation (Thalweg), whichever is more conservative.

- Abandon the use of countermeasures such as rip-rap or scour mats to reduce scour at pier locations for new bridges.

- Design all foundations to fully support the bridge structure for 100-year and 500-year flood events with full considerations for ship impact, hydrodynamic loads, compressive and tensile loads, and seismic criteria (without the use of protection dolphins).

- Base drilled shaft capacities on a combination of side-load transfer and end bearing, and calculate capacities for a design allowable settlement (even in rock).

- Use state-of-the-art computer design methods such as COM624 (Sullivan and Reese, 1980) to conduct lateral load studies for foundation systems.

All of this had to be accomplished in time to let a November, 1991 construction contract for the foundations and portions of the piers for the two main towers.

Hydraulic Evaluation of Floodplain and Predicted Scour Depths

Very important pieces of information still needed to finalize selection of foundation units for the tower piers were predicted scour depths at these locations. Hydraulic evaluations of the river and scour conditions at proposed bridge foundation positions were initially performed during conceptual and preliminary project phases using HEC-2 and WSPRO analyses (Burgess & Niple, Limited, Hazelet & Erdal, and Balke Engineers, 1991). However, as a result of the previously discussed FHWA design directives, the Kentucky Department of Highways retained Parsons, Brinckerhoff, Quade and Douglass, Inc. (PBQ & D) to perform a much more detailed evaluation of the floodplain at the bridge site. PBQ & D, with assistance from Dr. Froehlich, then Associate Professor of Civil Engineering at the University of Kentucky and co-developer of the FESWMS-2DH computer program, developed a finite element network covering the entire width of the floodplain at the bridge site and extending both 4.5 miles upstream and downstream of the planned structure location. PBQ & D was directed to evaluate many alternatives. The study included the entire roadway section positioned within the floodplain, not just the bridge over the Ohio River (Parsons, et.al., 1992).

Obviously, such an undertaking could not be accomplished within the time frame necessary to meet a November, 1991 construction letting date for Piers 8 and 9. PBQ & D issued a scour analysis report for the project in June of 1992, and KYDOH issued updated design scour depths in November of 1992.

Although final design scour depths were not available until November of 1992, KYDOH met the planned November, 1991 letting date for Piers 8 and 9 by allowing design to proceed using preliminary scour estimates. On September 27, 1991, FMSM received a preliminary design scour elevation of 278.5 feet for the two main pier locations from Haworth, Meyer and Boleyn, Inc. (HMB). Figure 6 presents a graphical representation of estimated scour depths (1991) at the two main tower pier positions.

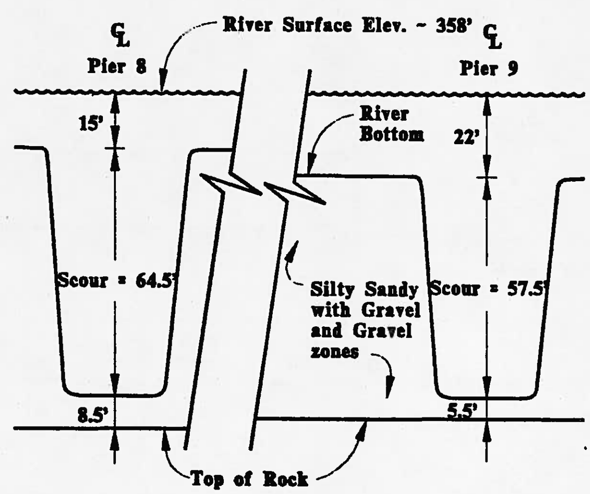

Figure 6
Schematic of scour depths at Pier 8 and Pier 9

Foundation Design

Based upon these estimated scour depths and upon anticipated structural loads, it was apparent that drilled shafts socketed significant depths below the top of rock would be required to support the two main tower piers. The task then was to determine shaft capacities and estimate shaft bottom elevations. At this same time, HMB also supplied FMSM with the conceptual foundation configuration, shown in Figure 7, and the design information provided in Table 3.

FMSM's scope of work was expanded, and based upon the conceptual foundation configuration and the design loading information provided, additional axial and lateral load studies were conducted for the drilled shafts utilizing methods outlined in FHWA-HI-88-042, AASHTO's 1992 Interim Specifications, and the computer program COM624. Both side-load transfer and end bearing values were calculated and combined to estimate ultimate shaft capacities, as directed by FHWA.

Table 3
Summary of Design Information for Drilled Shafts at Piers 8 and 9

Maximum Axial Service Load	-	1,425	tons
Maximum Uplift Service Load	-	300	tons
Maximum Lateral Service Load	-	75	tons
Maximum Tolerable Settlement	-	3	inches
Shaft Stem Diameter	-	6	feet
Rock Socket Diameter	-	5.5	feet

The predicted capacities and estimated shaft bottom elevations are summarized in Table 4. Bottom elevations of shafts were selected to satisfy both axial and lateral load requirements. Group efficiency factors were considered in estimating shaft capacities.

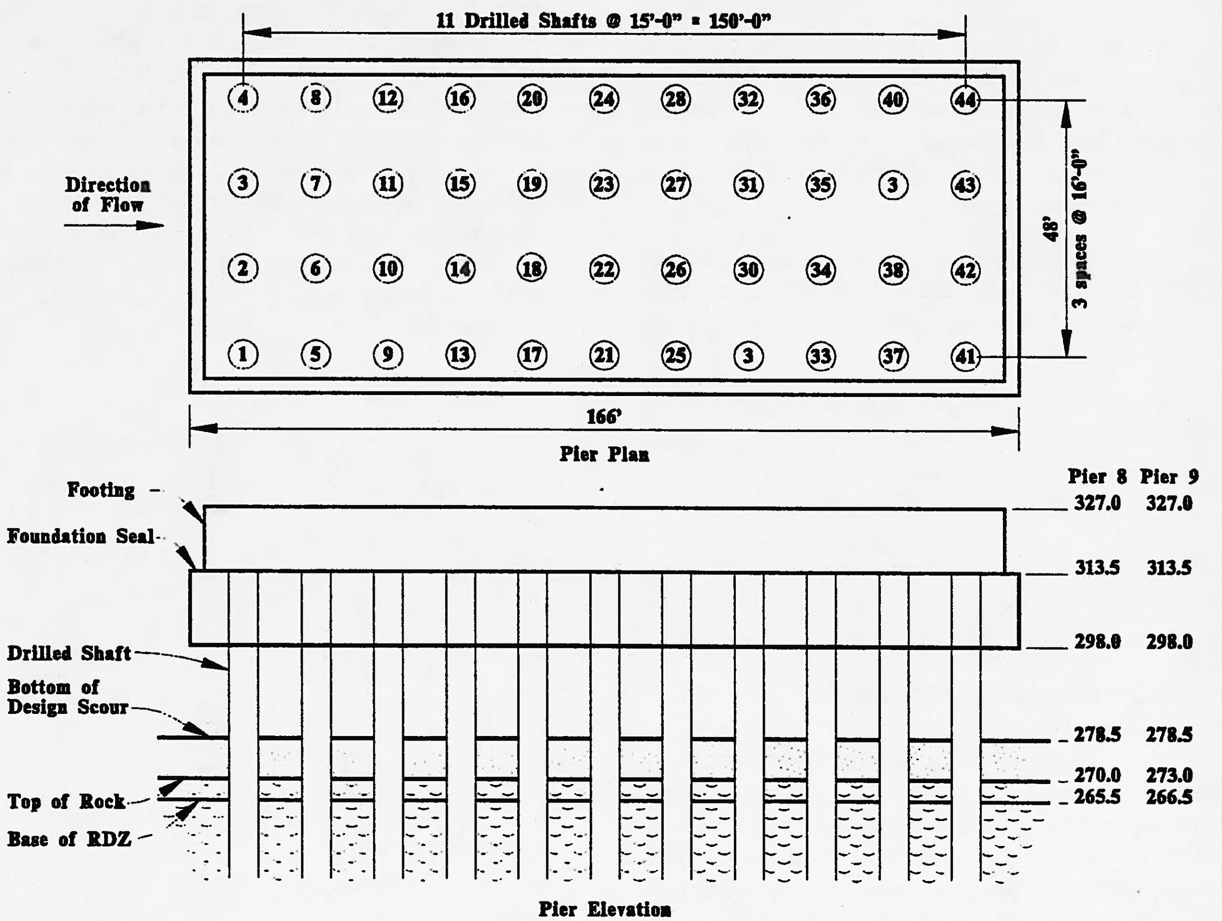

Figure 7
Conceptual Foundation Configuration Piers 8 and 9

Table 4

Summary of Predicted Shaft Capacities and Bottom Elevations at Piers 8 and 9

Pier No.	Diameter of Rock Socket (feet)	Bottom Elevation of Shaft (feet)	Estimated Ultimate Axial Capacity (tons)	Estimated Ultimate Uplift Capacity (tons)	Maximum Lateral Service Load (tons)	Predicted Lateral Deflection of Top of Shaft (inches)	Predicted Settlement at Service Load (inches)
8	5.5	234.0	3,750	925	100	1.1	1.0
9	5.5	251.5	5,875	1,325	120	0.9	0.5

All values are for single shafts.

The results of this evaluation were issued in a letter report dated October 14, 1991 (Fuller, et.al., 1991). Based upon this information, a final rock socket diameter of six feet was selected. Final design, construction and bid documents were completed for the two tower pier foundations, and the construction contract was awarded to National Engineering and Contracting Company of Strongville, Ohio, for approximately $14.5 million, in November, 1991.

CONSTRUCTION OF TECHNIQUE SHAFTS AND LOAD TESTING OF PRODUCTION SHAFTS AT PIERS 8 AND 9

Additional Subsurface Exploration During Construction

Because of delays in obtaining Coast Guard permits, actual work did not begin in the river until the summer of 1992. One of the first operations for construction involved obtaining additional rock cores. Six core borings were drilled by the contractor at each pier location instead of the one core boring per shaft normally required by KYDOH. Fewer cores were collected because of difficulties of working in the river, the relative uniformity of the subsurface materials at each pier location, and the grouping of a number of shafts in a relatively small area. Graphical representations of the additional rock cores are shown in Figure 8.

The cores obtained at Pier 8 appeared to be more weathered than those recovered during design, particularly within the zone of shale below the coal seam and above the first limestone layer. This shale zone was clayey, slickensided, very soft, and swelled considerably in the core barrel. The cores obtained at Pier 9 verified the anticipated conditions at that location.

Slake durability index (SDI) tests and jar slake tests were performed to determine the durability of the shales. The jar slake test consists of placing a piece of core in a beaker of water and observing how the core breaks down over a 24-hour period. The jar slake test evaluation assigns a number from one (complete breakdown to flakes or mud) to six (no change) depending on the degradation of the core. In general, the slake durability test results obtained by KYDOH indicated lower SDI values than the results obtained during design, and suggested the shales would be less durable than expected. At Pier 8, the SDI values were much lower for the shale zone immediately below the coal seam. A possible explanation for the differences in SDI values could be that the cores tested by KYDOH had a much longer time to desiccate prior to slake durability testing than did those tested by FMSM. Cores collected by FMSM were protected by plastic wrap to maintain in-situ moisture conditions until laboratory testing could be conducted.

Revision of Shaft Bottom Elevations Based on Review of Construction Cores

Although no additional unconfined compressive strength tests were performed using cores obtained during construction, visual examination of the cores and the results of the SDI and jar slake tests prompted KYDOH to lengthen the technique shafts and load test shaft at Pier 8. The initial design placed the shaft tips in the shales below the coal seam but above the first limestone layer. Because

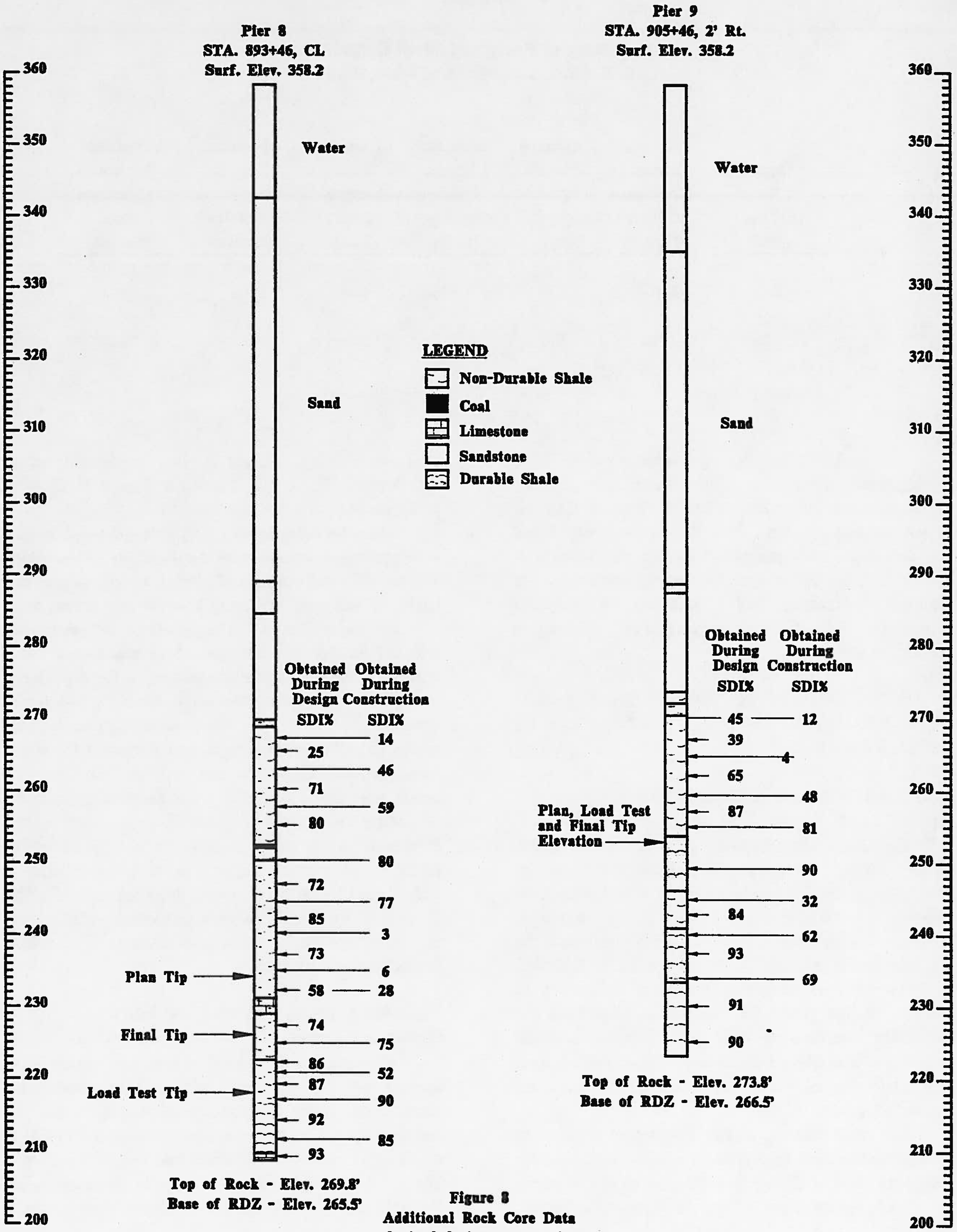

Figure 8
Additional Rock Core Data
obtained during construction

the shafts were to be constructed in the wet, sloughing and degradation of the shale could lead to difficulty in keeping the excavation clean and significant sidewall softening could occur. These factors could have negative effects on the load-carrying capacity of the shafts.

The bottom elevation of the load test shaft at Pier 8 was also lowered because the load test was to be conducted on a production shaft, and a failure of such a load test could have required partial redesign of the foundation system. As a result, the technique and load test shafts at Pier 8 were proposed to be lengthened about 15 feet resulting in a 47-foot rock socket. This placed the shaft tips below the first limestone layer into a more durable shale. No changes in plan shaft tip elevations were made for Pier 9.

Slurry Evaluation

Because of the potential of the shales to degrade during shaft construction, it was proposed that KYDOH evaluate the use of drilling slurry to retard slaking of the shales. Slurries from several different manufacturers were obtained and mixed according to the typical ratios proposed by the manufacturers. The slurry types evaluated were as follows:

- Polymer (dry granular and emulsion)
- Bentonite (with and without polymer additives)
- Attapulgite
- Water

Four representative samples obtained from different depth intervals within the same boring were placed on a No. 10 sieve. The cores were separated by cardboard dividers placed in the sieves. This setup was repeated for each type of slurry evaluated. The sieve and cores were then immersed in buckets filled with slurry. The samples were observed over a 24-hour period and evaluated in the same manner as for a jar slake test.

Water served as a control upon which to base the performance of the slurries. In general, after 15 minutes the cores immersed in water were beginning to slake. After one hour there was considerable degradation, and after 24 hours the cores exhibited a jar slake value of one. Attapulgite performed no better than water, and bentonites without additives only marginally better (jar slake value of one to two). Attapulgites usually require agitation to remain in suspension, and the mineral settled out of suspension very quickly during this study. Bentonites remained in suspension, but they produced a thick coating on the cores.

Polymer slurries, with one exception, produced little or no slaking (jar slake values of four to six), even after 24 hours of exposure. Bentonite slurry with polymer additives performed as well as polymers alone (jar slake values of six), but still left a thick coating on the cores. No significant difference in the protection against slaking could be distinguished between the different types of polymer slurries.

It appears that the polymer in the slurry is attracted to the clay particles in the shale. The polymer bonds to the clay particles and creates a barrier against water infiltration. This barrier is thin and does not inhibit the bond between the shaft concrete and the shale. In contrast, the clay particles in the mineral slurries are not attracted to the similarly charged clay particles in the shale. As a result, an effective barrier against water infiltration is not developed. Any mud cake that may build up on the shale could adversely affect the bond between the concrete and the shale. Figure 9 shows the degrees of slaking of soft shale samples after being immersed in water and in a polymer slurry for 24 hours.

Polymer slurries may have disadvantages over mineral slurries in some instances. The polymer chains break down when they come in contact with concrete; therefore, the polymer slurry does not interfere with the contact between the concrete and the sidewall. Many of the polymer slurry producers claim the slurries are biodegradable and non-toxic; therefore, they may have fewer disposal problems than mineral slurries. Some polymer slurry manufacturers have products that cause the polymer to "break" or fall out of suspension so that the waste slurry can be disposed of more easily.

As a result of these tests, KYDOH required the contractor to use polymer slurry when constructing the shafts. The contractor chose a dry granular polymer slurry.

Technique Shaft Construction

KYDOH added two technique (non-production) shafts to the contract after the decision to require slurry was made. Technique shafts were to be constructed in the same manner as the production shafts but they would not be used in the final structure foundations. Discussion was initiated during design whether or not to add technique shafts, but environmental concerns limited excavation to areas inside a cofferdam in order to reduce the transport of excavated materials downstream. Technique shafts would have to be constructed outside the cofferdam because of the close shaft spacing; therefore, construction of technique shafts was not planned during design. During construction, however, it was determined that a permanent

Shale samples after being immersed in water for 24 hours.

Shale samples after being immersed in polymer slurry for 24 hours.

Figure 9

casing used in the installation of a technique shaft also served the same purpose as a cofferdam, so technique shafts were added.

The technique shafts would provide the contractor a chance to refine his operations prior to construction of the production and load test shafts. They would also give KYDOH inspectors an opportunity to observe the operations and determine the amount of inspection effort that would be necessary. Because one technique shaft would be constructed at each pier, it would also give all participants an opportunity to evaluate the differences in the bedrock at each pier location.

Construction of the technique shafts began in December, 1992, with driving of permanent casing at a location outside the cofferdam. The overburden materials (sands and gravels) were removed either with an airlift or by augering. If augering was used, slurry was introduced prior to beginning excavation. In this case, the slurry held the non-cohesive materials together for easier removal. In the case of airlifting, the slurry was added just prior to beginning rock excavation.

Excavation of the rock socket was generally performed using rock augers, although the contractor would later use a core barrel to excavate the limestone layer for the production shafts. Initially, excavation rates were extremely slow indicating that about one week with 24-hour daily production shifts would be required to construct one shaft.

Mechanical failure of the contractor's drill unit stopped construction of the first technique shaft at Pier 8 with less than 15 feet of an approximately 47-foot long rock socket being constructed. A delay of about a week occurred until the drill unit could be repaired. Polymer slurry was maintained in the excavation during the delay. Prior to resuming construction of the technique shaft, samples of the rock socket bottom and sidewall were obtained using a split-spoon barrel attached to the bottom of the drill kelly bar. Samples of the bottom and sidewall materials showed that less than ¼-inch of degradation had occurred.

Prior to final cleaning of the excavation, a wire-brush cleaning tool was used to remove loose material from the rock socket sidewall. The wire-brush cleaning tool consisted of an open-ended bucket with wire cables protruding out the sides. The tool was lowered into the hole, rotated at a moderately high speed, and slowly raised out of the excavation. Final cleaning was accomplished with a muck bucket.

One of the properties of the most popular polymer slurries is that they do not hold particles in suspension. With a large column of slurry, a significant amount of sediment would accumulate on the bottom of the excavation after the first cleaning pass. Several cleaning passes were occasionally necessary to meet bottom cleanliness specifications prior to placing concrete.

Concrete placement was conducted through the slurry by tremie or by pump line, depending upon the situation, to the bottom of the shaft excavation. As the slurry was displaced by the concrete, the slurry was recovered for reuse or disposal. Concrete was placed to about one shaft diameter above plan shaft top elevation for purposes of removing contaminated or deleterious concrete. The overpour was then removed by an air lift to final grade. The floating concrete plant brought to the site had a small capacity (two cubic yards per load) and was initially plagued with mechanical problems which caused difficulties in meeting the required four-hour concrete placement time.

Technique shaft construction at both pier locations did not proceed smoothly or within some of the specification requirements. However, KYDOH decided to allow the contractor to proceed with construction of production and load test shafts. The contractor would construct one production shaft at Pier 8 prior to constructing the load test shaft at that location. He would then proceed to Pier 9 and construct a load test shaft at that position.

Pullout Tests

Two pullout tests were also conducted at each pier location. In general, a pullout test consists of drilling a 5.5- or 7.5-inch diameter hole and filling it with a reinforced grout plug. A dywidag bar extends from the plug to the surface. The plug is then dislodged by a center-hole jack pulling on the dywidag bar. The resistance to pulling out the plug is then used to estimate the unit skin friction of the full-sized shaft. Cores obtained from drilling the hole are usually subjected to laboratory testing for correlation between rock strength values and unit skin friction results.

One 5.5- and one 7.5-inch diameter pullout test was performed at each pier location by personnel from LOADTEST, Inc., and the University of Florida (Townsend, et. al., 1993). The pullout tests were located as close as possible to the planned full-scale load test shaft. Twelve-inch diameter casings were driven to near the top of bedrock. Overburden material was removed, and a double-tube core barrel was used to form a 5.5-inch diameter hole and obtain a 4-inch diameter core. The polymer slurry selected for use on the project was used in drilling the core. Each 7.5-inch diameter pullout tests included a miniature Osterberg load cell at the base. To complete the holes for the miniature Osterberg cell tests, a 7.625-inch diameter tricone bit was used to enlarge the holes after the 4-inch diameter cores were obtained.

Grouting the holes at such great depths and under water presented some difficulties. A piece of stovepipe was placed around the plug reinforcement and filled with grout prior to placement in the hole. Once on the bottom, the stovepipe was removed to allow the grout to flow into contact with the sidewalls of the plug.

Results from the pullout tests were inconsistent with expected results (Townsend, et. al., 1993). Pier 8 results were somewhat higher than the expected shear values, while Pier 9 unit shear results were very low. Procedures used in constructing the pullout tests may have affected these results. The stovepipe apparatus for placing the grout may not have provided enough head to scour the sidewalls and produce good contact between the sidewalls and the grout.

Slurry mixing problems or slurry breakdown may have occurred at Pier 9 and significantly affected the pullout tests. During construction, large spherical nodules of polymer were noted during cleaning of a test plug hole.

These nodules could have resulted from improper mixing of the dry granular slurry. A dry granular slurry requires a hydration period, and the slurry must be agitated during this time or the granular particles will settle out of the mixture. If the granular particles settle out during the hydration period, the polymer chains will intertwine during hydration, forming nodules. Agitation disperses the polymer evenly throughout the mixture and prevents the intertwining of the polymer chains. Improper agitation and dispersion of the polymer throughout the slurry produces a product that is less likely to provide protection against slaking of the shales. This effect was noted in the "softening" of the cores obtained for the pullout tests at Pier 9.

Load Test Shafts

The load test shafts were constructed in the same manner as the technique shafts. When cleaning of an excavation was complete, the rock socket diameter was measured using a wireline caliper tool. This information provided an assessment of the roughness of the socket sidewalls, as well as data for refinement of the load test results. A typical caliper printout for Pier 8, Shaft 43, is shown in Figure 10. The method of constructing the shafts, as well as the differences in the durability/ hardness of the bedrock encountered, are primarily responsible for the variations in socket diameter.

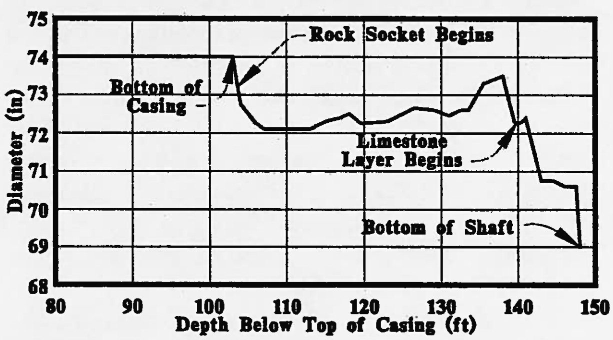

Figure 10
Caliper Results for Load Test Shaft 42, Pier 8

(after Townsend, et.al., 1993)

An Osterberg load cell was attached to the reinforcing cage along with strain gages and telltales. The cage was carefully lifted to avoid damage to the instrumentation and placed in the excavation. Because the top of the shaft was below the water surface, dywidag bars were attached to the top of the cage so that the cage could be supported from the surface during concrete placement. The Osterberg cell was then grouted in place, and the remainder of the shaft was filled with concrete using a pump line.

The load test was performed and documented by LOADTEST, Inc. (Goodwin and Schmertmann, 1993). Once the shaft concrete gained adequate strength, testing of the shaft began. The Osterberg load cell used for this project had the capability of providing 3,000 tons each in shear and end bearing for a total capacity of 6,000 tons. Assuming a factor of safety of two and taking into account group efficiency factors, it was estimated that about 4,200 tons was required from the load test in order to verify that the shafts had adequate capacity to support the design axial loads.

Telltales were used to determine settlement of the bottom of the shaft, top of shaft movement, and compression of the shaft. Strain gages placed at various locations along the reinforcing cage enabled measurements of the load transfer from the shaft to the rock socket with depth. Thermistors were also used to allow measurement and compensation for any temperature induced strains.

Both load tests were conducted to the maximum allowable capacity of the load cells. The bottom of the load cell at Pier 8 experienced significant downward movement, but little upward shaft deflection was observed (Figures 11 and 12). At Pier 9, only minor bottom settlement and upward shaft movement were noted (Figures 13 and 14). However, based upon review of the deflection curve for the Pier 9 test results, it appears that the shaft was approaching shear failure when the capacity of the load cell was reached. Using the end bearing- and shear-deflection curves from the load tests, an equivalent top load settlement curve was constructed (Figures 15 and 16). These indicated that the shafts had more than adequate capacity at the as-built depths.

Using the strain gage results and the adjusted shaft diameters from the caliper tests, the load transfer and unit shear resistances were calculated (Tables 5 and 6). Because the top of shaft movements were relatively small and side shear failure was not apparent, it is likely that these unit shear resistances are somewhat less than maximum.

After completion of the load tests, the void left by expansion of the load cell was grouted so that the shafts could be used as part of the final structure.

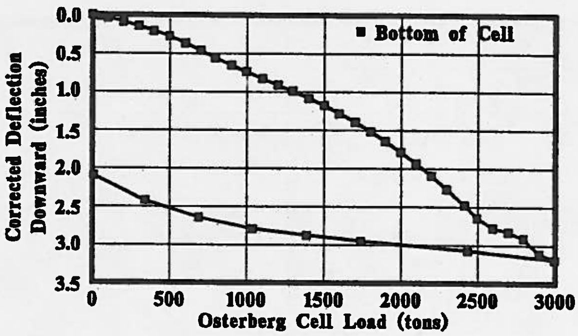

Figure 11
Bottom of Cell Deflection,
Pier 8, Shaft 43

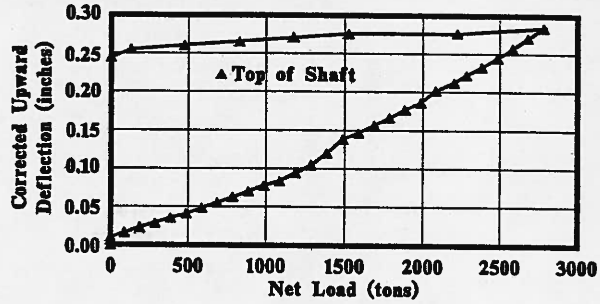

Figure 12
Top of Shaft Deflection,
Pier 8, Shaft 43

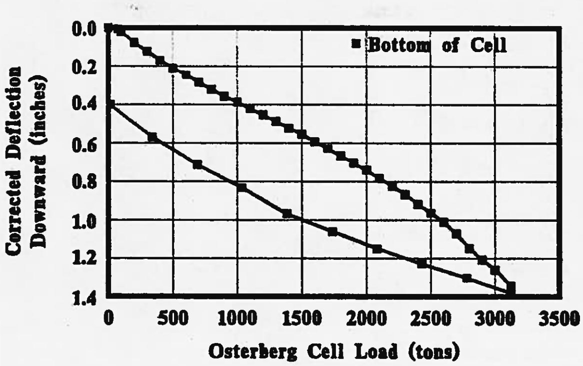

Figure 13
Bottom of Cell Deflection,
Pier 9, Shaft 42

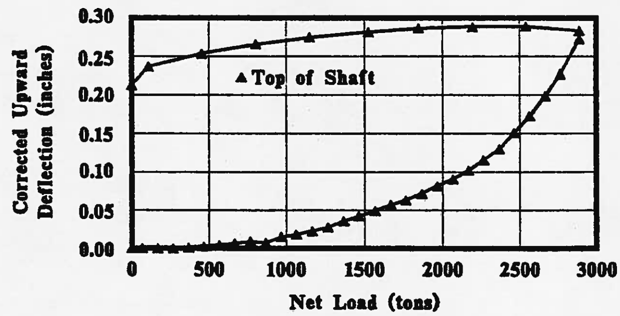

Figure 14
Top of Shaft Deflection,
Pier 9, Shaft 42

Figures 11-14 (after Townsend, et.al., 1993)

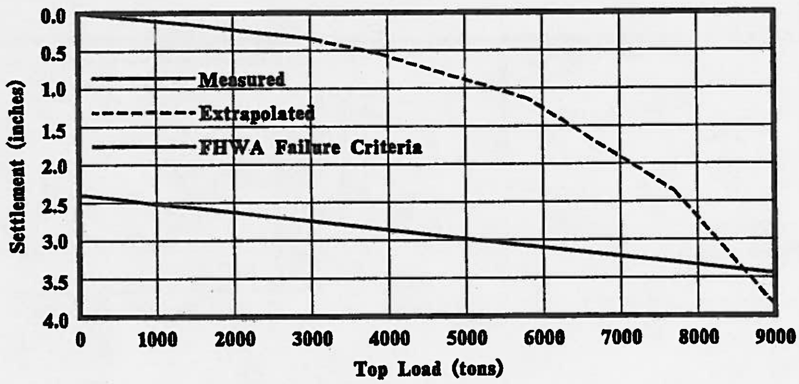

Figure 15
Equivalent Top Load Settlement Curve,
Pier 8, Shaft 43

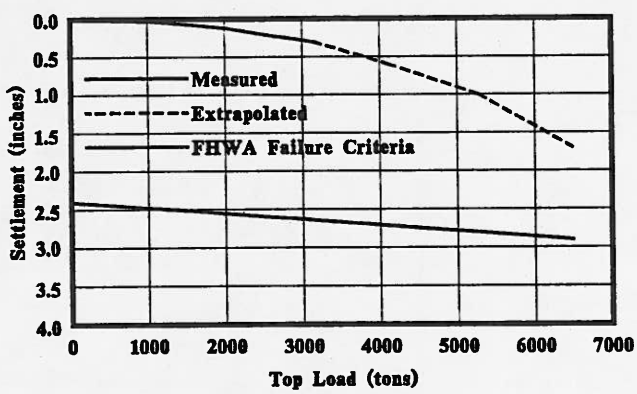

Figure 16
Equivalent Top Load Settlement Curve,
Pier 9, Shaft 42

Figures 15 and 16 (after Townsend, et.al., 1993)

Table 5
Unit Shear Resistances for Pier 8, Shaft 43

Elevation (feet)	Cell Load (tons)	Weight of Shaft (tons)	Net Cell Load (tons)	Unit Shear Resistance (psi)
297.8	19	36	0	
				0
277.5	75	82	0	
				1
267.6	123	105	18	
				24
262.0	316	118	198	
				24
251.5	681	142	539	
				44
237.5	1,542	174	1,368	
				51
227.9	2,225	195	2,030	
				66
222.5	2,717	206	2,511	
				88
220.2	2,998	211	2,787	

Table 6
Unit Shear Resistances for Pier 9, Shaft 42

Elevation (feet)	Cell Load (tons)	Weight of Shaft (tons)	Net Cell Load (tons)	Unit Shear Resistance (psi)
297.8	-5	135	0	
				0
285.6	19	163	0	
				0
274.0	-4	189	0	
				0
270.2	14	198	0	
				70
266.5	560	206	354	
				59
262.7	874	214	660	
				183
259.3	1,725	221	1,504	
				143
255.3	2,509	229	2,280	
				205
253.1	3,126	233	2,893	

Adjustment of Shaft Bottom Elevations Based on Load Test Results

The results of the load test conducted at Pier 9 indicated the shafts would provide more than adequate axial capacity at the plan tip elevation. However, shaft bottom elevations were not adjusted from plan tip elevations because the design rock socket length of 15± feet per shaft was necessary to accomodate lateral loading.

The load test information from Pier 8 was used to adjust the final shaft tip elevation at that location. The unit shear resistance information indicated that the shafts probably would provide adequate capacity at the plan tip elevation, however, end bearing for the plan shaft tip elevation would have to be estimated instead of based on an actual load test result. KYDOH was also concerned that the load-settlement characteristics of a shaft placed at plan tip elevation could be less than desirable.

The final selected shaft bottom elevation for Pier 8 was nine feet below plan tip elevation, instead of the 15 feet selected for the load test shaft. This placed the shafts below the first limestone layer into a more durable shale. While this tip elevation would provide more than adequate capacity, the location of the tip in a more durable shale was intended to provide better constructability and load-settlement characteristics. Plan, load test, and final tip elevations are shown versus the graphical representation of the rock cores on Figure 8.

Remainder of Tower Pier Foundation Construction

The addition of slurry, technique shafts, and deeper shafts at Pier 8 led to a significant increase in costs. The cost per foot for the shafts increased from $600 per foot to $1,162 per foot with a major portion of this cost being the use of slurry. The addition of technique shafts resulted in a net increase of about $256,000 to the project, while the additional shaft footage added about $220,000 to the cost.

High water on the Ohio River has caused the contractor several delays. This was the case soon after the completion of the load tests. When the contractor resumed work in the late Spring of 1993, it was with greater confidence, smoothness, and production. Production at Pier 9 eventually was about 1 shaft per day (two 12-hour shifts), with production at Pier 8 improving to about one shaft every 1½ days. Shaft construction was completed late Summer of 1993. At the time of this writing, construction of the two main river piers is near completion.

CORRELATION OF LOAD TEST RESULTS WITH CURRENTLY AVAILABLE METHODS TO ESTIMATE SHAFT CAPACITIES

General

The results of drilled shaft load testing at Piers 8 and 9 were utilized to refine preliminary shaft capacities at other substructure element positions. The conditions of the bedrock encountered at remaining river pier locations were very similar to those observed at Pier 8. An attempt was made to model the load test results at Pier 8, using an updated version of the "SHAFT1" computer program (Reese and Wang, 1990). If a model could be developed to accurately predict the unit shear and end bearing values observed during load testing, then it could reasonably be used to assist in refining shaft capacities at other river pier locations.

The Model Developed for Pier 8

The "SHAFT1" computer program provides methods for estimating the capacity of a single drilled shaft subjected to axial loading. The computation procedures are based principally on the methods outlined in Publication No. FHWA-HI-88-042. The "SHAFT1" program contains several options for selecting the type of subsurface materials to model. These include 1) granular soil, 2) cohesive soil, 3) clay-shale, and 4) rock. Because of the low unconfined compressive strengths exhibited by samples of bedrock obtained from the Pier 8 position (see Figure 5) and because of the rock's shale composition, the "clay-shale" option was selected in developing the computer model. Using this option, the following input parameters are required to define the properties of the clay-shales:

- depth interval
- undrained shear strength, S_u
- shear strength reduction factor, α
- unit weight, γ

Other input parameters for the program include the length and diameter of the shaft stem and rock socket, elastic modulus of the concrete, and area of reinforcing steel. Once all required parameters are defined, the program will generate the following output data per foot of shaft length:

- estimated volume of concrete
- ultimate side resistance, Q_s
- ultimate base resistance, Q_b
- total ultimate resistance, Q_u
- load-settlement relationship

Basically, the only variable items were the undrained shear strengths and the shear strength reduction factors. It was expected that if these values could be well-defined the model should provide results reasonably close to the observed values of unit side resistance and end bearing obtained from load testing.

The first step implemented was to correlate the depth intervals within the model to those defined during load testing. Figure 17 provides a schematic of the load test set up and defines the intervals used in the "SHAFT1" computer model for Pier 8.

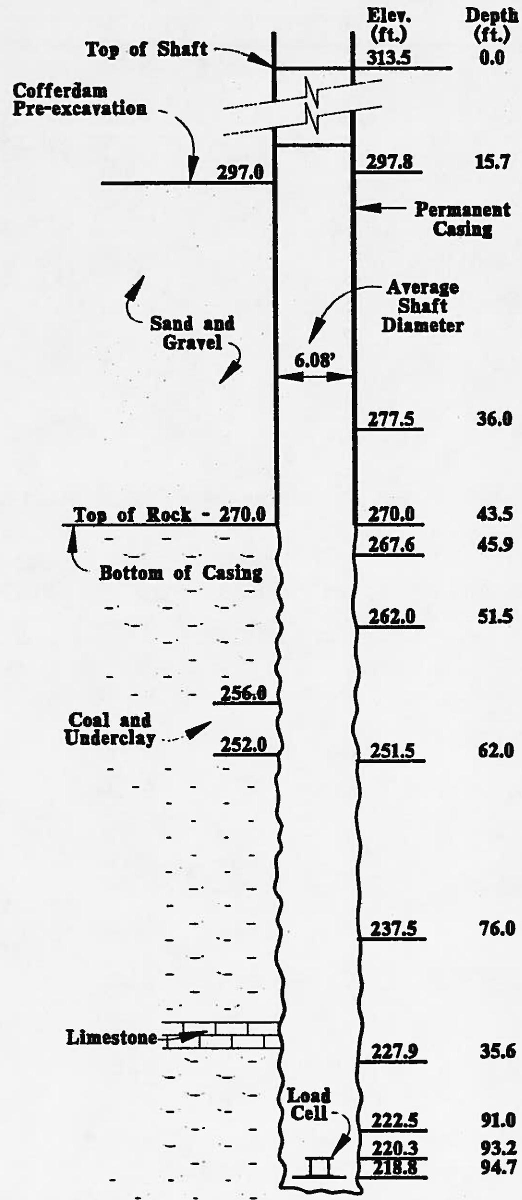

Figure 17
Osterberg Cell Load Test Setup, Pier 8, Shaft 43

The second step required estimation of the undrained shear strengths for the individual test intervals to input into the program. Because the load test results provided unit side resistance values, f_s, it was necessary to use these data to "back-calculate" unconfined compressive strengths, q_u. These back-calculated q_u values were compared to data obtained during the initial geotechnical exploration to determine if reasonable correlations were resulting. Unconfined compressive strength values were back-calculated using the following equation:

$$q_u = f_s / 0.15 \quad (3)$$

where q_u = unconfined compressive strength (psi)
f_s = ultimate unit side resistance (psi)

Undrained shear strengths were then estimated by:

$$S_u = \tfrac{1}{2} q_u \quad (4)$$

where S_u = undrained shear strength
q_u = unconfined compressive strength

The resulting q_u and S_u values are summarized in Table 7.

Table 7

Summary of Unconfined Compressive and Undrained Shear Strength Values Back-Calculated from Unit Side Resistance Values for Pier 8

Depth Interval Below Top of Shaft (feet)	Elevation Interval (feet)	Unit Side Resistance Value Obtained from Load Test, f_s (psf)	Back-Calculated Unconfined Compressive Strength, q_u (psi)	Undrained Shear Strength, S_u (psi)
45.9 - 51.5	267.6 - 262.0	24	160	80
51.5 - 62.0	262.0 - 251.5	24	160	80
62.0 - 76.0	251.5 - 237.5	44	293	146
76.0 - 85.6	237.5 - 227.9	51	340	170
85.6 - 91.0	227.9 - 222.5	66	440	220
91.0 - 93.2	222.5 - 220.3	88	587	293

Table 8 presents the unconfined compressive strength data obtained from rock samples collected at Pier 8 during the initial geotechnical exploration.

Table 8
Summary of
Unconfined Compressive Strength Data from Initial Geotechnical Exploration for Pier 8

Elevation Interval (feet)	Unconfined Compressive Strength, q_u (psi)
251.7 - 251.3	514
249.4 - 249.0	320
237.8 - 237.4	216
238.6 - 238.2	300

As can be seen from a review of Tables 7 and 8, back-calculated q_u values correlate reasonably well with available laboratory test data. The undrained shear strength values presented in Table 7 were then input into the model along with other necessary data. The only variables remaining were the strength reduction values to be used for each defined interval of rock socket. The shear strength reduction factor, α, will vary depending upon the subsurface materials present and the construction techniques used to install the shafts. It was expected that these factors would need to be adjusted through an iterative process to achieve correlation of unit side resistance values observed during load testing with those predicted by the computer model. AASHTO's 1993 Interim Specifications provide guidance in selecting α-values for clays (After Reese and O'Neill, 1988), and indicate that the trend in the shear strength reduction value is inversely proportional to the undrained shear strength of a clay. Because the rock at Pier 8 was being modeled as a clay-shale, an initial α-reduction value of 0.3 was selected for each defined interval.

Results of Computer Analyses

Computer analyses were then conducted using the SHAFT1 program and the developed drilled shaft model. The results are presented in Table 9.

The total side resistance predicted by the "SHAFT1" model for the interval between 267.6 feet and 220.3 feet was approximately 2,807 tons. The net shear load observed during load testing within this same interval was approximately 2,787 tons, or less than one percent difference. Further iterations and manipulations of the strength reduction factors were not warranted.

At first thought, it would appear that such close correlation should have been achieved because of using undrained shear strengths back-calculated from load test results. However, even small differences in shear strength reduction values will result in significant differences in ultimate side resistance values. Obviously then, selection of the correct subsurface material type to analyze, and selection of α-reduction factors are critical to developing a reasonable model.

A second "SHAFT1" analysis was conducted to target a total shear capacity of 3,800 tons and end bearing of 1,500 tons. This would provide correlation between the computer model and values estimated from load testing for an equivalent top load at one inch of settlement. The undrained shear strengths were increased approximately 33 percent, and these values were used to predict ultimate shaft capacities. One inch of settlement was selected by the writers to represent ultimate capacity because of concerns associated with settlement of drilled shafts in soft shales. The resulting shear strengths were correlated with similar bedrock materials at other river pier locations and used to assist in estimating shaft capacities and recommended shaft bottom elevations. Additional correlations were conducted for load test data from Pier 9 with similar results.

Final design of the remainder of the bridge is underway and is expected to be complete in 1995.

Table 9

Summary of "SHAFT1" Correlation with Load Test Results at Pier 8

Elevation Interval (feet)	Undrained Shear Strength, S_u, used in SHAFT1 Analysis (psi)	Shear Strength Reduction Factor, α, used in SHAFT1 Analysis	Q_s obtained from SHAFT1 Analysis (tons)	Unit Side Resistance, f_s, calculated from SHAFT1 Results (psi)	Unit Side Resistance, f_s, estimated from Load Test Results (psi)
276.6 - 262.0	80	0.3	184.58	23.97	24
262.0 - 251.5	80	0.3	346.08	23.97	24
251.5 - 237.5	146	0.3	846.44	43.96	44
237.5 - 227.9	170	0.3	673.34	51.00	51
227.9 - 222.5	220	0.3	490.21	66.01	66
222.5 - 220.3	293	0.3	266.44	88.06	88

SUMMARY AND CONCLUSIONS

An extremely accelerated design schedule was implemented to meet a November, 1991 letting date for partial construction of the two main tower piers. Because of this schedule, refinement/optimization of foundation designs for Piers 8 and 9 was not possible, and concerns over constructability and the high predicted shaft capacities predicted prompted installation of technique shafts and full-scale load testing. Results of load testing indicated that the shaft capacities predicted during design were very reasonable. Shaft tip elevations at Pier 8 were lowered approximately nine feet, however, because of constructability issues and long-term settlement concerns. No adjustments to Pier 9 shaft bottom elevations were made. The use of a polymer slurry during construction of the shafts significantly reduced slaking and degradation of the soft shales and helped protect the integrity of the foundations.

Computer models were developed that showed strong correlations with load test results and were used in refining shaft capacities at remaining substructure element positions. Considering the types of bedrock present at the site, and the construction techniques utilized to install drilled shafts in rock, a shear strength reduction factor, α, of 0.3 appears to be appropriate for use in estimating shaft capacities. It is the opinion of the writers that caution and sound engineering judgment should be exercised when deciding whether or not it is appropriate to combine side-load transfer and end bearing in rock sockets to develop ultimate shaft capacities, particularly in soft shales. Combining skin resistance and end bearing in such materials may result in prediction of allowable capacities at which load-settlement characteristics of the shafts may be undesirable.

ACKNOWLEDGMENTS

The writers extend their appreciation to the staff of the Kentucky Department of Highways' Geotechnical Branch and to the staff of Fuller, Mossbarger, Scott and May Engineers, Inc., not only for their assistance in preparation of this paper, but also for their efforts during design and construction of the two main tower pier foundations.

Special thanks are given to National Engineering and Contracting Company of Strongville, Ohio; LOADTEST, Inc., of Gainesville, Florida; Dr. Michael O'Neill of the University of Texas at Austin; Mr. Bill McKinney Kentucky Department of Highways' Resident Engineer - William H. Natcher Bridge project; and Mses. Shawn Black and Cheryl Lingle of Fuller, Mossbarger, Scott and May Engineers, Inc.

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QUALITY CONTROL FOR JET GROUTING

by

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ABSTRACT

Quality assurance and control for jet grouting is not a simple issue. Unlike ground anchors, soil nails or generic micropiles, very few capacity-oriented tests are available for jet grouted elements and even fewer are used. Alternatively, the industry relies heavily on preproduction test programs and rigorous quality control measures. So it is disconcerting to find that there is little consensus in the industry as to what constitutes an appropriate approach to these issues and what is the relative worth of some very basic quality oriented measures. The goal of this paper is to present a variety of different possibilities related to this problem in hopes of providing owners and first-time users with a full range of testing and inspection options.

INTRODUCTION

After struggling for U.S. acceptance for nearly a decade and a half, jet grouting seems to finally being coming into its own as a widely-considered, ground-improvement option. This special form of mix-in-place concreting has taken on an increasingly important role in structural and environmental installations. During its initial U.S. introduction the technique faced a variety of problems and wide-spread rejection due to problems associated with inconsistent quality. In the last two year many municipalities and federal agencies have conducted their first jet grouting installations. If full acceptance of this technique is to occur, clarifying the key elements of good quality control and quality assurance are of utmost importance. This is especially so since with the recent expansion of jet grouting into new applications and untried geographic regions, an increasing number of owners are finding themselves as first-time designers, supervisors, and quality control managers of a technology on which there is relatively little literature available. The critical problem is that many of the standard quality assurance tests applied to related ground improvement techniques are not regularly done for jet grouting. Many are seemingly inappropriate and others simply physically impossible. This forces a greater emphasis on good installations processes and more rigorous quality control. Additionally, much of the literature for grout-based ground improvement techniques has focused on corrosion prevention. Due to a general omission of steel reinforcing elements, this is not a concern in jet grouting.

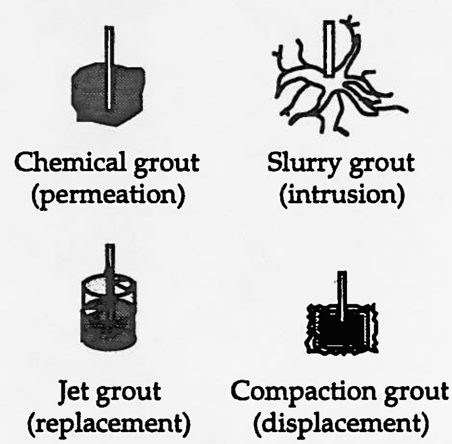


Figure 1. Types of grouting techniques (after Winterkorn and Pamukcu, 1990 after Welsh, 1986)

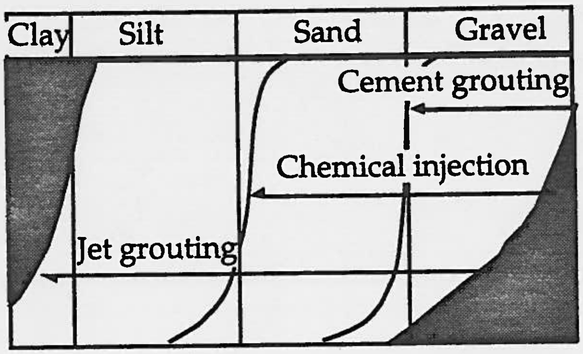


Figure 2. Typical ranges of gradation for different grouting techniques (after Winterkorn and Pamukcu, 1990 after Welsh, et.al., 1986)

BACKGROUND

The first U.S. jet grouting installation was in 1980 (Welsh and Burke, 1991), but within a few years the procedure picked up a very bad reputation. Despite claims that the automated machinery alleviated the need for highly trained personnel, many had disappointing first experiences (Grand, 1993). Some projects were improperly or carelessly installed (Micciche, 1993a), and early work for the Bureau of Reclamation indicated only moderate success (Anon, 1986). Trade articles cited lack of performance and consistency. With titles such as "Jet Grouting Doesn't Cut It" (Anon, 1986) and "Jet Grouting: Snail's Pace of Adoption" (Andromalos and Pettit, 1986), it is not surprising that industry-wide acceptance was not high. Welsh and Burke (1991) record a seven-year hiatus between the first jet grouting projects and the next major installations. This was in large part a result of concerns with quality. In general, jet grouting has faced exceptionally slow adoption. This can be seen both in comparison to the acceptance of other soil modification techniques and in the volume of U.S. jet grouting projects compared to the quantity of installations in Europe and Asia. The tremendous upsurge in jet grouting throughout the world since the late 1970s (Flick, et.al., 1992; Trevisani, 1992; and Silence, 1992) has not been matched in the U.S.

Only in recent years, with a new emphasis on QAQC procedures has this begun to change. Computer-generated data from the jet grouting rigs has significantly helped with acceptance. Even within this past year a real change in attitude has been reported (Micciche, 1993a). Along with this changed attitude has come a significant market expansion. Several government agencies are trying jet grouting for the first time. The New York City Metropolitan Transit Authority conducted a feasibility-oriented test program in late 1993 and now has three full-scale projects in construction. The National Park Service

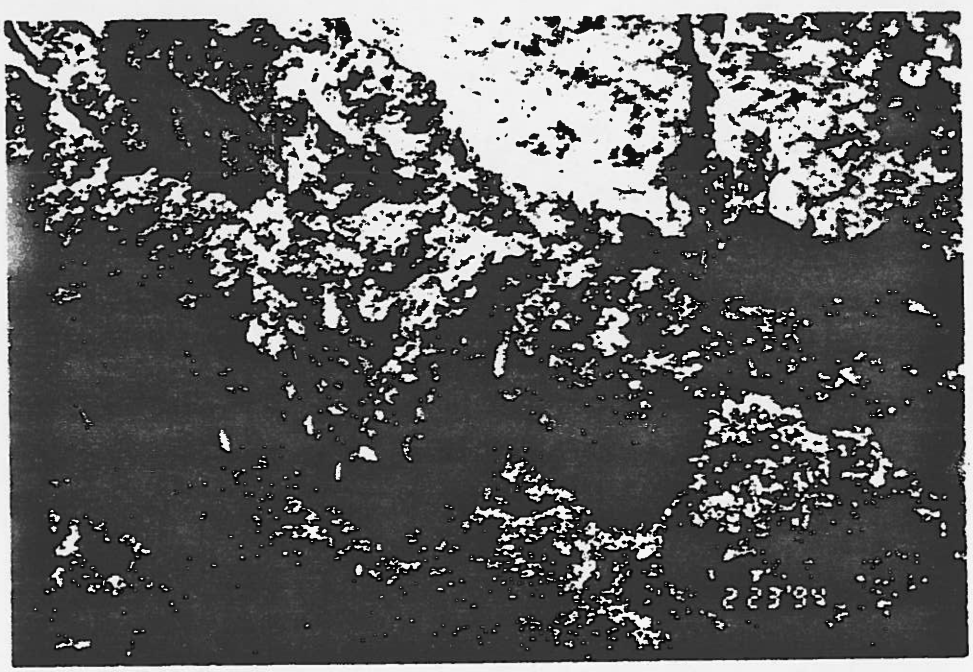


Figure 3. Soilcrete

installed its first jet grouted columns in early 1994, and after a successful initial experience is evaluating the process for other sites. To date the largest documented jet grouting contract is 1.8 million dollars (Andromalos and Gazaway, 1989).

DESCRIPTION

Jet grouting also known as replacement grouting (Munfakh, et.al., 1987) has been described as a "a high velocity erosion process" (Kauschinger, et.al., 1992b). The resulting product, generically referred to by the trade name Soilcrete (Andromalos and Gazaway, 1989), may be considered a mix-in-place concrete that uses the drilled soil as the aggregate portion of the concrete (Figure 3). American jet grouting evolved from a multi-rod system that was introduced in Japan, around 1970. Grout, water, and air are introduced through pressurized rods. Soilcrete is a fully-mixed, in-situ material capable of piling formations in almost any ground conditions (Hayward, 1993a), even below water (Parry-Davies, et.al., 1992). Jet grouting is the newest major form of grouting (Gallavresi, 1992) [Figure 1] and is capable of treatment in the widest range of soils (Figure 2), so it is not surprising that it can be used for a large variety of applications, including underpinning existing structures threatened by subsidence, seepage control, cofferdams, and subsidence limitation over tunnel excavation sites (Andromalos and Pettit, 1986 and St. Simon, 1993a).

Begun as a waterproof barrier, jet grouting now offers an alternative to conventional grouting, chemical grouting, deep slurry trenching, proprietary underpinning systems, and compressed air and freezing for tunneling (GKN 1993a and Gallavresi, 1992). Jet grouting can also be installed prior to dewatering to minimize settlement (Burke, et.al., 1989a). The effect of jet grouting on the soil is to either increase its strength or decrease its permeability (Munfakh, et.al., 1987).

There are three major variations on jet grouting: a single rod, double rod and triple rod options (Figure 4). They are distinguished by the number of material delivery rods. Each of the different variations injects high-pressure liquids (e.g., water, air, and grout) through various rods to cut or mix the soil in place (Burke, et.al., 1989a and Kauschinger, et.al., 1992b). After the drill has advanced to the proper depth, injection of the components begins as the rods are slowly withdrawn (Burke, et.al., 1989a). The process cuts through the soil, displaces the fines, and mixes the soil with the

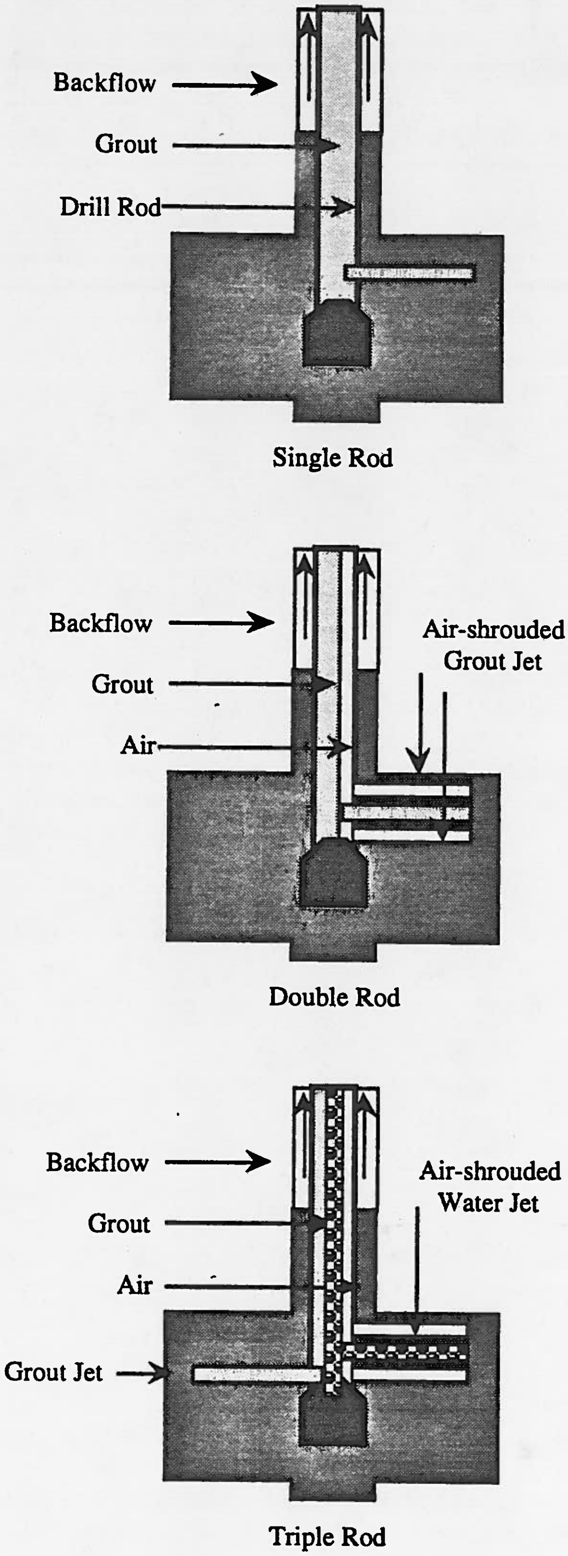


Figure 4. Single, double, and triple rod systems (after Steiner, et.al., 1992 and Burke, et.al., 1989a)

grout. The result is a fairly constant radius of treated ground (St. Simon, 1993a).

QUALITY

Quality is an element which must be committed to from the first act of a process and carried through at each stage. This begins with an adequate geotechnical investigation, is enhanced by a test program, continues with regular material checks and installation monitoring and ends with sufficient performance testing. A proper site investigation, acceptable material handling and storage, good installation practices, common problems and potential remedial measures, as well as environmental impacts are all elements of this process.

Since each site and application is different, quality control and performance measurement need to be appropriately specified for each project. Additionally, it is important to establish site specific tolerances for each installation (Delta, 1993). Potential contractual controls, industry standards, and the requirements of a proper test program are key administrative elements.

PRECONSTRUCTION CONTROLS

Prior to the commencement of construction, owners can take many steps to ensure the quality of the final installation. These include a thorough geotechnical investigation, thorough specifications, and a rigorous test program.

Geotechnical Investigation

With jet grouting so much of the performance is controlled by in-situ conditions (which are beyond the installer's control), instead of by the grout, steel, or other manufactured materials. Testing and a good geotechnical investigation are, therefore, vital. Littlejohn (1990) states,

> The ground is one structural component of the...system, and the importance of a good quality site investigation cannot be overstated. Lack of adequate information on the ground remains the most common cause of...failures.

This is a widely held opinion throughout the industry (GKN, 1993a and Lefroy-Brooks and Hooley, 1992). Welsh (1993b) sites insufficient or inappropriate geotechnical investigation as one of the largest problems regularly encountered, with site geology and soil interpretation being integral to proper analysis (PTI, 1986). A thorough investigation should include hydro-geological information as well (Winterkorn and Pamukcu, 1990) as standard soil analysis. The most common preliminary approach to data collection is through boreholes (Bruce 1989b; Littlejohn, 1990; and Winterkorn and Pamukcu, 1990). Emphasis should be placed on taking a sufficient number of samples to adequately represent variations across the site (Winterkorn and Pamukcu, 1990). Samples should be taken across the range of the anticipated installation, every 15 to 30 m depending on soil uniformity (PTI, 1986 and Bruce, 1993e), but exploration should not be limited to the area designated for treatment. It is important to include adjacent zones. These surrounding areas may influence the stability of adjoining structures (i.e., excessive settlement) or influence grout injection through the loss of material due to open channels (Winterkorn and Pamukcu, 1990). SPT readings should be made at every 1.5 m to at least 3 m below the expected depth of installation (PTI, 1986), and samples should be taken of each soil stratum (Littlejohn, 1990), laterally, as well as vertically. Gallavresi (1992) lists soil density and water content as being of particular importance.

Goals. The purpose of the samples and subsequent testing is multifold:

1. Standard soil gradation and classification data.
2. Shear strength.
3. Compressibility.
4. Density.
5. Groundwater level and moisture con-tent.
6. Granulometry of granular soils.
7. Atterberg limits of cohesive soils.
8. Permeability.

(after Gallavresi, 1992; Bruce, 1993e; Juran and Elias, 1990; and PTI, 1986)

Tests. For many parameters both in-situ and laboratory tests are available. Too often cost and accuracy become competing factors in test selection. In certain cases there will be the need to use both field and laboratory methods for the purpose of a preliminary analysis and then as an attempt at in-depth correlation.

Soil Classification. The most fundamental step is soil identification. It is basic to all treatment decisions. Checking for organics is particularly crucial (Soliman and Munfakh, 1988), especially since many organic compounds retard the setting of cement-based mixtures (Winterkorn and Pamukcu, 1990). The cohesiveness of the soil is also primary to treatment decisions. There are a variety of in-situ testing options. The most reliable are borehole sections equipped for constant-head pumping tests and piezometers in the surrounding ground. Permeability will also need to be known where excavations are scheduled beneath the level of the ground water.[1]

Density and Strength. Soil strength can be determined in the field through energy methods (e.g., SPT) or in the laboratory through traditional direct shear and triaxial tests (Littlejohn, 1990). Static cone penetrometer tests may be used to test in-situ density and consistency for certain soils (Gallavresi, 1992). Where this is not possible, ASTM offers a battery of familiar laboratory tests. Bruce (1993e) suggests the pressuremeter as an uncommon but potentially useful tool to obtain radial stress-strain data, which provides information on the load transfer mechanism in the soil. The accuracy of laboratory tests must be carefully considered in relation to actual soil density, water content, and loading conditions (Bruce, 1993e).

Ground Water. In addition to soil testing, a hydro-geological assessment should be made (Gallavresi, 1992). Groundwater conditions can be measured with stand-pipes or more sophisticated piezometers (Bruce, 1993e).

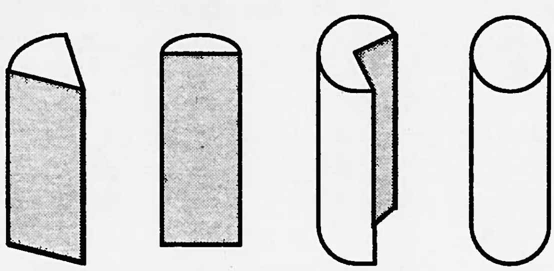


Figure 5. Varying geometries of jet grouted columns

Summary. Although very basic, general causes of destabilization should also be evaluated: nearby excavations, particularly in urban areas; installations using vibration-inducing machines; lowering of water tables (consolidation); changes in natural water levels; earthquakes; landslides; and weathering-based degradation (Lizzi, 1980). Assessments of all of these factors should be included in a geotechnical investigation. A solid, preconstruction investigation is in many ways preemptive (Burke and Brill, 1992). Thoroughly done, it may also uncover unexpected toxic waste prior to contractual obligations by the owner.

Contractual

Establishing good contractual procedures for new technologies is not easy. There is a fine distinction between protecting the owner and creating an unworkable site environment due to excessive testing or extreme performance controls. Properly trained personnel, appropriate equipment, good material handling and installation practices, adequate test programs, final performance testing, and remedial measures are all key elements to a successful project. Trying to establish these items contractually is particularly difficult in the absence of appropriate building codes. To date there are no formal codes addressing jet grouting.

Since jet grouting has traditionally been used for projects already facing difficulties, either due to issues of accessibility or structural sensitivity, it is essential that constructability proposals accompany bids. The owner should retain the right to reject bids on the basis of technical qualifications, as well as price (Hayward, 1991a).

Specifications. Most specialty contractors have quality control specifications. Some are used entirely as internal documents, while others become the basis for design-build projects. Typically, the specifications define the construction tolerances, such as the number of millimeters (per meters in depth) out of plumb a drill hole may be. The specifications will also identify all required submittals. These may include shop drawings showing anticipated loading during construction (including transitory loads due to the installation equipment), schematics of the connection details between the excavation and the support system, or the submission of design calculations. The specifications may also require submission of working drawings for the grout plant, a detailed explanation of the grout mixing and injection techniques, proposed borehole location, drilling and installation methods (including sequencing), as well as the grout mix design (including the source of the mix materials) [Hayward, 1991a and Hayward,

[1] See Winterkorn and Pamukcu (1990) for additional options for geotechnical investigations.

1991b]. Another issue is whether an independent testing lab will be brought on board, and if so, as an expense to whom. Specifications should allow the contractor and the engineer sufficient freedom to adapt the details of the specified procedures to meet actual conditions (Nonveiller, 1989).

The specifications should also detail warrantee expectations. Typically this will include repair or replacement, for a period of one to two years, of any structural damage caused by the contractor's installation or due to poor performance under the prespecified working loads (Hayward, 1991c). In addition to all of the technical matters, a standard set of specifications should outline bidder prequalifications, schedule or accessibility constraints, and payment method.

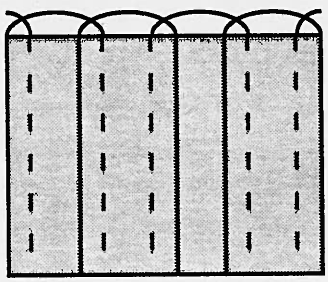


Figure 6. Portion of a cutoff wall comprised of interconnected, 180° columns

Submittals. In the field of ground improvement, it is not unusual for a contractor to generate the design either as part of the original plan or as a value engineering alternative. A contractor generated design should specify micropile type, materials, location, installation method, verification testing, and design assumptions (Hayward, 1991a), as well as the dimension (length and diameter), degree of inclination (Winterkorn and Pamukcu, 1990) and whether the piles will be 360° in circumference or some portion of that (Figures 5 and 6). The extent of overlap between piles should also be specified. Other factors which may be explicitly written or may be at the contractor's discretion, guided only by performance requirements, are the injection rates and pressures, the grout properties (liquid, transition, set), and the stages of the installation sequence (Winterkorn and Pamukcu, 1990). The construction methods and equipment play key roles in determining what strengths or permeabilities are achieved (Munfakh, et.al., 1987). The details of the work should be submitted in a process very similar to a traditional shop drawing submittal and review.

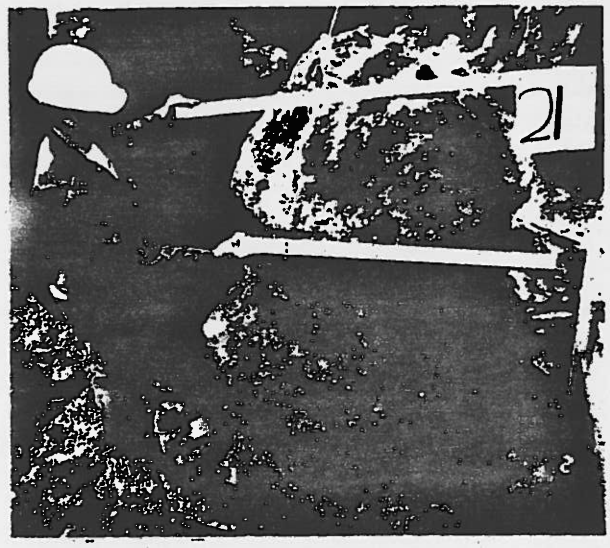


Figure 6. Typical pile (GeoCon, 1994a)

Testing Program. Quality control is the periodic testing which occurs as part of the installation process. Quality assurance is testing done after the element is complete and should be considered a form of performance testing. Both are crucial components to a good installation, but there is one additional type of testing which must be considered. This is the formal preproduction, test program which includes the full-scale execution and testing of large sections of jet-grouted soil. Given the wide variability of soil and other geological conditions, the large number of factors which may influence engineering behavior, how intimately the performance of a jet grouted element is connected to the soil/jet-grout interaction, and the relative dearth of empirical information, preproduction test programs are critical. The specifications describing a test program should outline the quantity of soilcrete to be tested, the type of testing, and the capacity to which it is to be tested. All of these items are very site specific and provide a lot of valuable information.

Test programs for jet grouting play a more critical role than for most other types of ground improvement and are thus widely recommended (Micciche, 1993a and Winterkorn and Pamukcu, 1990). Preproduction testing gives the best sense of the pile's actual geometry and capacity (Burke and Brill, 1993). It establishes the anticipated diameter (Figure 6) which then allows the actual capacity of each pile to be back calculated (Welsh, 1993b) from the minimum observed width (Andromalos and Gazaway, 1989). The minimum diameter is extremely important as it also determines the required spacing between holes. This is essential since there is a problem relating the diameter and strength of the column versus the soil characteristics and the selected construction method (Jamiolkowski, 1992). During a test program various liquid injection pressures can be tested on conduit materials to establish maximum contact pressures and minimum distances to prevent damage to utilities (GeoCon, 1994b).

It is not unusual for extensive field trials to be carried out (De Paoli, et.al, 1989). For a large job, there may be as many as 30 test columns (Andromalos and Gazaway, 1989). The test program can also be quite intricate, as in the case of a historic Japanese building where a replica foundation was made to check attachment details (Shibazaki and Ohta, 1982). A test program is also used to determine pressure and distance thresholds to prevent damage (Gazaway and Jaspers, 1992). With a good preproduction program, remedial measures can be avoided (Welsh, 1993b).

During a test program, a variety of materials as well as installation configurations should be tried (Andromalos and Gazaway, 1989, Ichihashi, et.al., 1992). This may include several different configurations of water/cement ratios and installation pressures to test for relative performance (Parry-Davies, et.al., 1992). Delta Drilling (1993) is a proponent of experimentation with water/cement ratios by weight of 0.6 to 1.4 and notes that any proposed admixture should always be included. Important grouting parameters including the number of nozzles (Andromalos and Gazaway, 1989), pressure, flow rate, jet nozzle diameters, rotation rate, withdrawal rates (Gazaway and Jaspers, 1992 and Munfakh, et.al., 1987), and air and water injection velocities (Burke and Brill, 1992) are needed. During the preproduction program these elements are either verified or optimized.

The most important element of a test program is that it matches as exactly as possible the materials and methods of the production elements. This includes using the same drilling equipment and procedures, the same installation pressures, grout mix, and any proposed admixtures. The testing must also occur in the same ground conditions. The test program will give the most efficient combination of pump energy, orientation, and displacement speed (Steiner, et.al., 1992). A correlation between compressive strength and the water/cement ratio is also desirable, as well some quantification of the relationship between installation method, soil type, and resulting column diameters for particular operating parameters. The water jet pressure and withdrawal rate appear to be the most influential parameters in the column's final diameter (Bruce, 1988b). Measurement of the volume of ejected material versus the material in place is helpful for the quality control program (Gallavresi, 1992) and for establishing a waste disposal plan.

The testing can be done in individual locations with the consolidated material excavated for analysis, or in test pits, which are used to check permeability (Figures 7 and 8), but it is more effective to form and excavate a series of contiguous, grouted columns at various spacings to establish final design spacing (Parry-Davies, et.al., 1992). The goal is to have uniformly shaped piles with adjoining members being intimately locked to each other (Parry-Davies, et.al., 1992), where necking due to dense layers can be checked (Burke, et.al., 1989a). Ideally, the test sections are constructed in the same subsurface conditions as the final columns and in a location where excavation, observation and sampling can occur the next day (Burke, et.al., 1989b). When this is not possible, coring at the interstices of several columns is often done. If this approach is embraced all the columns (usually three in a group) should have the same installation parameters, grout composition, and be installed with at least a day's curing time between them (Burke and Brill, 1993). Columns with observation pipes located at specific distances from the jet grouting can be constructed. In this case, the jet grouting must be modified until connection between the various observation pipes is achieved. Confirmation of this phenomena will occur with a visible vibration of the observation pipe and an appearance of spoil at this location when the jetting nozzle passes that point (Burke, et.al., 1989b). Instrumentation can be embedded in the piles to better assess loading response (Flick, et.al., 1992), and dilatometer soundings may be used to estimate the compression modulus for settlement analysis (Burke, et.al., 1989a). Depending upon the results of the test program, the engineer may require modifications in the jet grout production to achieve the desired outcome (Hayward, 1991a). In the test program, if the jet grouting should not adequately perform, the design and/or installation methods should be modified, and a new installation (reflecting the revised design) will need to be tested. If the design was contractor generated, the cycle of test, revise, test should occur without additional expense to the owner (Hayward, 1991c). Otherwise it can be considered a design error.

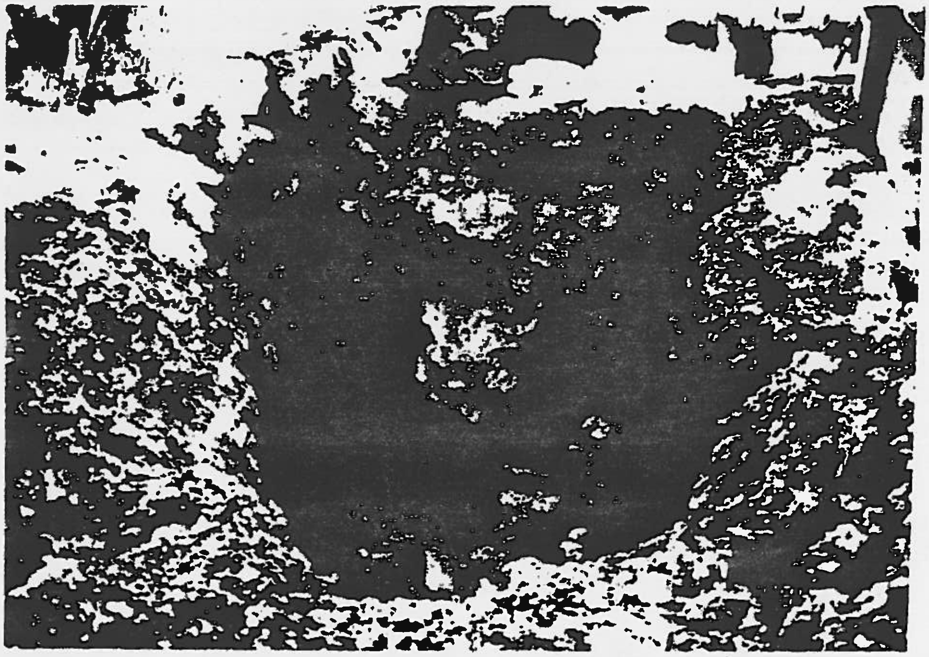


Figure 7. Test pit

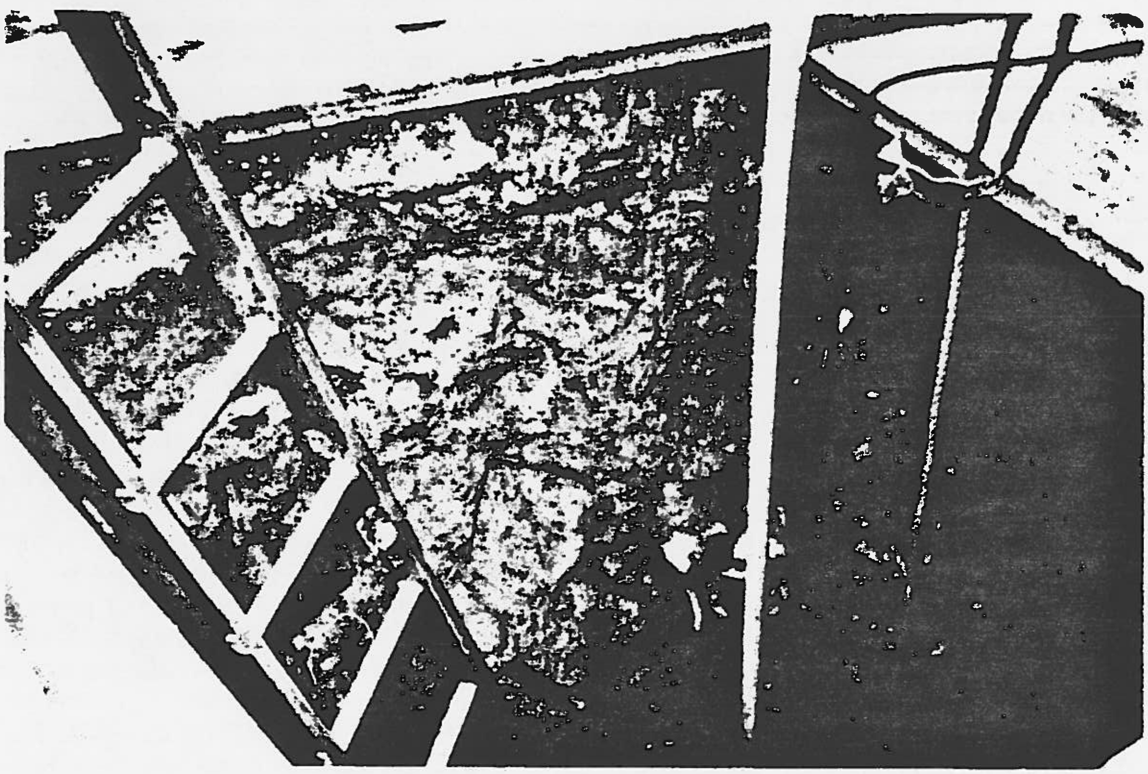


Figure 8. Jet pile excavation

The test program provides verification of design assumptions (Bruce, 1993e), an opportunity to modify and adjust the design if needed, and an ever growing body of performance data essential for the proliferation of new technologies. The test program also permits a fine tuning of the design. For large-scale installations this could mean significant savings, if the specified design is found to be overly conservative.

If a jet piling project is particularly small and relatively straight forward, a test program may be foregone in the name of cost but must be replaced by a highly conservative design approach (Gazaway and Jaspers, 1992). The parameters in Table 1 were used for such a situation.

Table 1. Conservative design parameters as an alternative to a test program (Gazaway and Jaspers, 1992)

Jet nozzle diameter (range)	1.8 to 2.2 m
Grout pressure	41.4 MN/m^2
Rotation Rate	1 rpm
Lift Rate	0.3 m/min

Parry-Davies, et.al. (1992) propose a non-excavation oriented method for treated zone estimation. First, estimate the volume of spoil and assume that the remainder of the grout fills the voids. Second, assume that the voids constitute 50 percent of the total volume consolidated. The diameter of the circle plotted in plan view constitutes the calculated area of treatment for each jet grouted hole. This is compared by drilling tertiary holes. When grout injection is found to be excessive, the tertiary holes can be modified with washing, jetting, and blowing, alternating between flushing water and short bursts of compressed air, followed by tremieing grout into the hole. When using a prewash, dilution was not been found to be a problem (Parry-Davies, et.al., 1992).

Kauschinger, et.al. (1992a) offer a mass-balance, field-based method to estimate the size of jet grouted bodies. This procedure can only accommodate single-fluid systems. The presence of additional fluid systems would greatly increase the complexity of this approach, which is already based on several assumptions. The first of which is that the soilcrete column is homogeneous. In truth it may vary as much as 30 percent. The second premise is a uniform resultant geometry, without permeation or hydrofracture. This can often be controlled by high quality installation, but it may not occur. The third assumption is a simplification to ignore any drainage induced by the column's own weight. The fourth supposition is that the cuttings sampled at the surface are representative of cutting produced at the point of injection. This is a problematic unless the pile is relatively short (Kauschinger, et.al., 1992a). Jamiolkowski (1992) notes that this mass balance approach gives a good, general order of magnitude, with variations being largely dependent on sampling.

Personnel. Regardless of the type and quantity of tests specified during and after construction, if installation is not done by an experienced company with a qualified crew, the results will not be favorable. Burke, et.al. (1989b) note, "Experience of the Contractor is paramount in a successful jet grouting project." It is therefore advisable that the prequalification of the firm and the personnel be clearly called out in the specifications or the contract. Ideally the firm would have a minimum of three (Delta, 1993) to five years of jet grouting experience (Hayward 1991a and Hayward, 1991b), and the key personnel would have a minimum of one to two years of experience in jet grouting installation with the proposed company (Hayward, 1991c and Bruce, 1993c). This will prevent a company unfamiliar with jet grouting from simply buying expertise to get a job. In the bid papers a brief description of each project and an affiliated reference should be included (Hayward, 1991c). Nonveiller (1989) remarks that only qualified organizations with specialized personnel and modern equipment should be engaged in drilling (Nonveiller, 1989).

Key personnel includes the engineer, drill operators, and on-site supervisors (Hayward, 1991c). Each should possess several years of experience in the design and construction of jet grouting, particularly for any engineer supervising installation (Hayward, 1991c). The engineer of record should be licensed (Commonwealth, 1984), preferably in the state in which the project is being executed, and consultants or manufacturer's representatives should not be permitted as substitutes for on-site engineers (Hayward, 1991c). Focht and Drash (1985) specifically cite the importance of knowledgeable on-site supervision and monitoring. All personnel should be prequalified prior to commencement of work (Kauschinger, et.al., 1992b), and substitution of the approved personnel should not be permitted without proper notification to and assessment by the owner (Hayward, 1991c). Kauschinger (1994) describes the recipe for a successful project as being a "marriage of quality control and good experience".

As a final note, when specialty consultants are brought in as independent experts, they must be permitted to do their work. Kauschinger (1994) cites failure to do so as an obstacle to effective on-site progress.

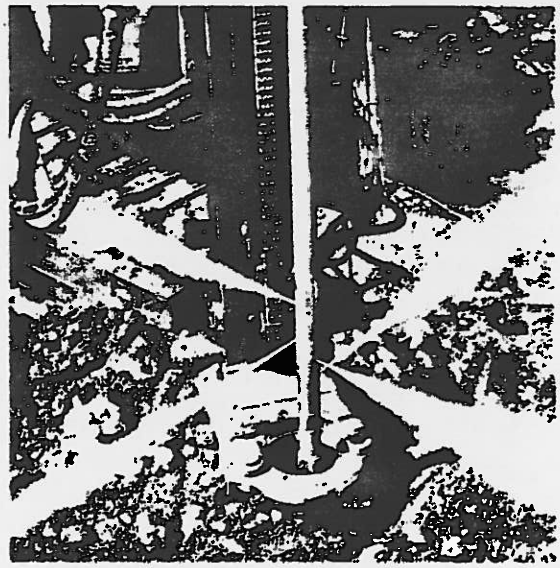


Figure 9. Single system installation rod (GeoCon, 1994a)

MATERIAL STORAGE, HANDLING, AND SELECTION AND EQUIPMENT MAINTENANCE

The quality of a final product cannot be superior to its individual elements. This begins with the actual materials and includes their storage, handling and preinstallation preparation. Proper use and upkeep of all machinery is also essential to this process.

GROUT

Grout is the most important component to jet grouting (St. Simon, 1993c), since it must be sprayed through a very small nozzle under tremendous pressure (Figure 9). This process begins with submittals and continues through daily testing. Proper selection of cement type, water percentage, admixtures, aggregate and even mix time are crucial to a successful and economic installation. Suitable batching and mixing equipment and mixing parameters are critical, and often overlooked, components. Good quality grout starts with proper material storage. This primarily consists of preventing hydration by keeping the cement dry. Prior to production, samples of all grout material, identical in source, composition, and properties to the materials proposed should be submitted for approval. At a minimum 11.35 kg of cement and admixtures adequate for 75.7 l of grout should be provided (Gazaway and Jaspers, 1992). Catalog cuts and manufacturers' information for admixtures (including bentonite) need to be submitted (Delta, 1993). Mill certificates for all cement and bentonite should submitted within eight hours of each shipment received (Delta, 1993), and all cement should conform to ASTM C150.

Storage. Cement storage must prevent moisture damage. Cement should not be stored directly on the ground or anywhere else where it may be subject to significant moisture absorption. Caking due to moisture absorption, lumps or foreign matter, and compaction due to stacking bags over ten high must be avoided (Delta, 1993). Prior to use, the cement should be checked for lumps or any other indications of hydration (Hayward, 1991c). Alternatively, the cement may be screened through a standard 100-mesh screen to ensure that premature hydration does not occur (Delta, 1993). Handling of admixtures should follow a similar course, with primary attention paid to manufacture's recommendations. Bulk cement is permitted, as well as cloth or paper bags that incorporate plastic or rubber vapor barriers (Delta, 1993). Moisture-resistant paper sacks shipped in dated, sealed containers, and handled and stored to avoid absorption of moisture are actually recommended (Delta, 1993). Admixtures which have exceeded the manufacturer's recommended shelf life should not be used (Delta, 1993). The most common admixture in jet grouting is bentonite. It must conform to the requirements of API Recommended Practice 13A, with a minimum yield of 99 barrels per megagram. Nonveiller (1989) recommends that the bentonite have a liquid limit greater than 400 percent and contain more than 90 percent particles finer than 6.68 mm. Like cement, bentonite which has become caked due to moisture absorption should not be used (Delta, 1993).

Select. When using a cement-based process, the first step is cement selection. A Portland based cement, 20,670 kPa (clinker based) is used. Some have commented that the appropriateness of the material for jet grouting is not as good as European cement because the extent of fineness in the element is not as high (St. Simon, 1993c). In terms of the jet grouting industry, there is a gap in U.S. concrete manufacturing between the rough of the Portland cement and the micro- or submicrofine cements. A product in between has been described as perfect for jet grouting (St. Simon, 1993b). A Portland Cement of Type I, II or a I/II mix complying with AASHTO M85 grouting (Hayward, 1991c) is generally selected. Only if a highly accelerated set-up time is needed is a Type III chosen, since it is significantly more expensive (Micciche, 1993a). Generally Portland Type I is the most economical and is therefore used when possible (Welsh, 1993b). Material cost is a major issue considering that it takes 7.4 to 37.3 kg of cement per meter of treated ground (Anon, 1962). The grout mix constituents and composition are selected to meet the specific requirements of strength and permeability and have significantly less-restrictive criteria than those that apply to conventional permeation grouting (Gallavresi, 1992).

To ensure high quality grout and to prevent any chemical reactivity, the water should be relatively clean and free of harmful amounts of oils, salts, alkalies, acids, or organic matter (Winterkorn and Pamukcu, 1990 and Delta,1993). This desired quality of water is commonly referred to as potable (Hayward, 1991a and Delta, 1993). Nonveiller (1989) proposes owners require water quality certificates, although this is rarely done. Aggregate is never used. That component comes from the in-situ material. The final set of ingredients are the additives. Bentonite is a common additive in soilcrete (Perry, 1993 and GKN, 1993b). It can reduce drainage effects in granular soils when permeability control is the main concern and high strength is not required (Gallavresi, 1992). Pulverized or powered, premium-grade natural sodium cation bentonite is preferred (Delta, 1993 and Hayward, 1991a). It is important that the specific chemical composition of any admixture always be checked for compatibility with the cement and the soil. A case history by Pitt & Rhode (1984) notes the existence of several commercial Portland cement retarders which proved ineffective in delaying set. In general, it is recommended that no additive containing more than a total of 0.1 percent (by mass) of chlorides, sulfides or nitrates be used (Littlejohn, 1990).

REINFORCEMENT

Steel reinforcement is not often used with jet piles (Ryan, 1993), usually only as a shear key (Perry, 1993). When included, it is composed of either a high-tensile plain-threaded bar, steel tube, hollow anchor bar, or a combination of plain bar within a steel tube (St. Simon, 1993a). In Europe, high tensile steel is used (St. Simon, 1993b). In the US, Grade 60 steel bars are recommended (Delta, 1993). The reinforcement adds some bearing capacity and can provide resistance to lateral and uplift loadings (St. Simon, 1993a).

WATER

Although potable water is preferred, in some cases salt water can be used, as long as there is no steel present and a slower initial set time can be tolerated (Parry-Davies, 1992). This is permissible because without a steel element there is no chance for corrosion.

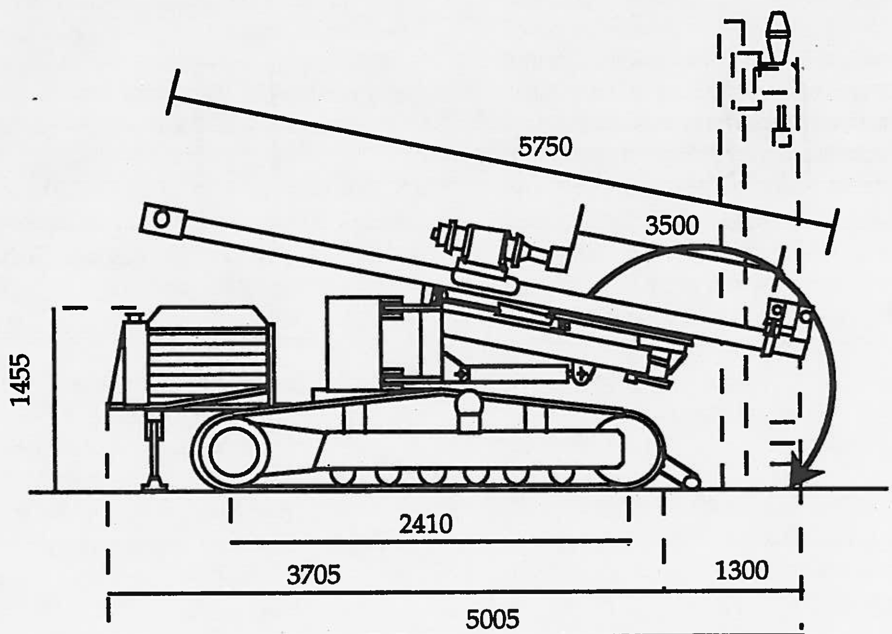
EQUIPMENT

The equipment needed for a jet grouting operation is less than simple. It requires a full sized grout plant, all necessary hoses and connectors, and the jet grouting rig. The entire operation may take on the upwards for a day to install (Perry, 1993), $50,000 to mobilize (St. Simon, 1993b), and four tractor trailers of equipment for relocation (Perry, 1993).. Backup equipment should be kept on-site to provide uninterrupted grouting in case of mechanical breakdown, malfunction, or clogging. Spare parts and equipment should be readily available to keep "down time" to a minimum (Delta, 1993).

Submittals. Prior to work the contractor should submit all proposed drilling equipment, including the manufacturer and model number, performance criteria, range of operation and accuracy, and catalog cuts (Delta, 1993).

Grout Plants and Mixers. The grout plant should be automated (Perry, 1993) and include a pump, a mixer with its associated lines, valves, gauges, and regulating devices (Delta, 1993). The two and three fluid system plant also includes the compressed-air and water circuits. A high pressure pump is provided in the water circuit, a medium-pressure pump (up to 12 MPa) for the cement grout injection, and the air is supplied by a compressor (usually delivering 24 m^3/min at 1.2 MPa) [Gallavresi, 1992]. A high-speed colloidal mixer capable of operating at 1,500 to 2,000 RPM (Delta, 1993 and St. Simon, 1993d) is required. Use of a high speed colloidal mixer will decrease the likelihood of bleed (PTI, 1986) by ensuring full initial mixture and is almost uniformly regarded as necessary to the furnishing of high quality grout mixtures (GeoCon,

Figure 10. Grout plant



All measurements in mm

Figure 11. Jet drilling rig (after Pachiossi, 1994)

1994c). The goal is to produce a grout free of lumps and undispersed cement (Hayward, 1991a). A special heavy duty pump, capable of delivering water or grouts at pressures in excess of 60 MPa is also required (Gallavresi, 1992). The rotary speed of a mixer is an essential element to proper production (St. Simon, 1993c), and must therefore be periodically verified.

Grout components are measured and mixed either continuously or in batches (Delta, 1993). Grout plants are fairly complicated operations (Figure 10), and may be equipped with internal quality control measures, specifically, automatic weighing and volumetric systems to insure exact proportioning (Winterkorn and Pamukcu, 1990). Automated batching and mixing systems provide more consistent mixes and allow quick and efficient changes in mix designs (GeoCon, 1994c). There are many ways to check the machinery. Pumps can be outfitted with pressure gauges to monitor grout pressures. All gauges must be capable of pressure measurement to twice actual expected grout pressure or a minimum of 1033.5 kPa (Hayward, 1991a). They must be designed to exclude grout from the pressure tube (Delta, 1993), and all pressure gauges and water flow meters must be periodically calibrated (Delta, 1993). Each pressure gauge should have a gauge saver, and all instrumentation should be protected from clogging, vibration, and shock (Delta, 1993). Additionally, all grouting machinery must be equipped with controls to permit accurate and continuous variation of grout pressures and low rates (Delta, 1993).

It is important that the system is sufficiently large and rapid to provide complete jet grouting for a single hole without interruption (Delta, 1993). Each jet grouting rig may require 5 to 8 m^3/hr of grout (Gallavresi, 1992). Perry (1993) notes that running out of grout or mechanical breakdown is the easiest way to ruin a jet column. For small jobs a mobile pumping station can be employed (Hayward, 1993b), but regardless of the equipment selected, working pressures up to 138 MN/m^2 are often required (Andromalos and Gazaway, 1989). Typically, the same machinery is used independent of whether the material to be grouted is soil or rock (Munfakh, et.al., 1987).

Use of computer-aided devices as monitors and controls for field grouting operations is increasing in popularity (Bruce, 1988a). Automated grout injection monitoring systems can maintain grout pressure and flow rates within preset limits and can provide either local or remote readout, as well as recording of injection parameters (GeoCon, 1994c).

Drill Rig. Gallavresi (1992) notes that the selection of a rig that enables operation with a single rod or very long units is advantageous not only to speed up drilling but, more importantly, to minimize interruptions during the injection phase. Any operation (such as rod-handling, in particular) that causes an interruption of flow under pressure may involve the risk of clogging the nozzles. Depending on which rod system is selected there is a slight modification in the equipment. The use of compressed air requires an inner and outer set of rods, with an annulus of about 5 mm (Kauschinger, et.al., 1992b). Regardless of rod variation, a specially manufactured drill is used in conjunction with a high-capacity pumping unit to deliver the liquid(s) at the appropriate volumes and pressures (Burke, et.al., 1989a).

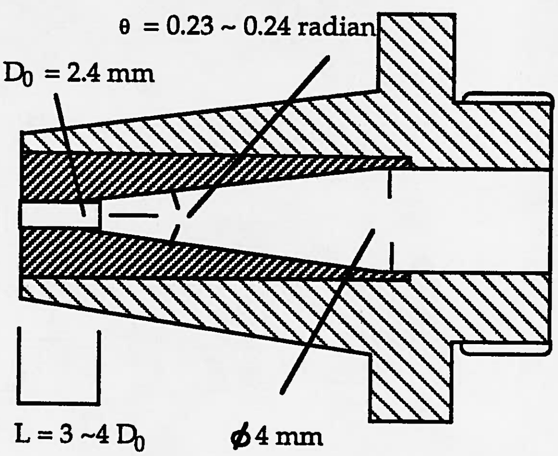


Figure 12. Jet nozzle head
(after Shibazaki and Ohta, 1982)

Nozzles. The jet nozzles (Figure 12) must be kept in good working order. As clearly seen in Figures 13 and 14 erosion of the jet rods is a real possibility. The jet nozzles are of vital importance in eliciting the maximum energy out of the jet water (Shibazaki and Ohta, 1982), and must therefore always be kept in good working order, including timely replacement. Damaged rods will lead to a significantly inferior product. In the case of the triple rod system, unless air is blown out uniformly, the high-velocity jet of water cannot be encapsulated in the air (Shibazaki and Ohta, 1982). Proper positioning of the nozzles in a multirod system is crucial. Otherwise, the soil hydraulically excavated by high-

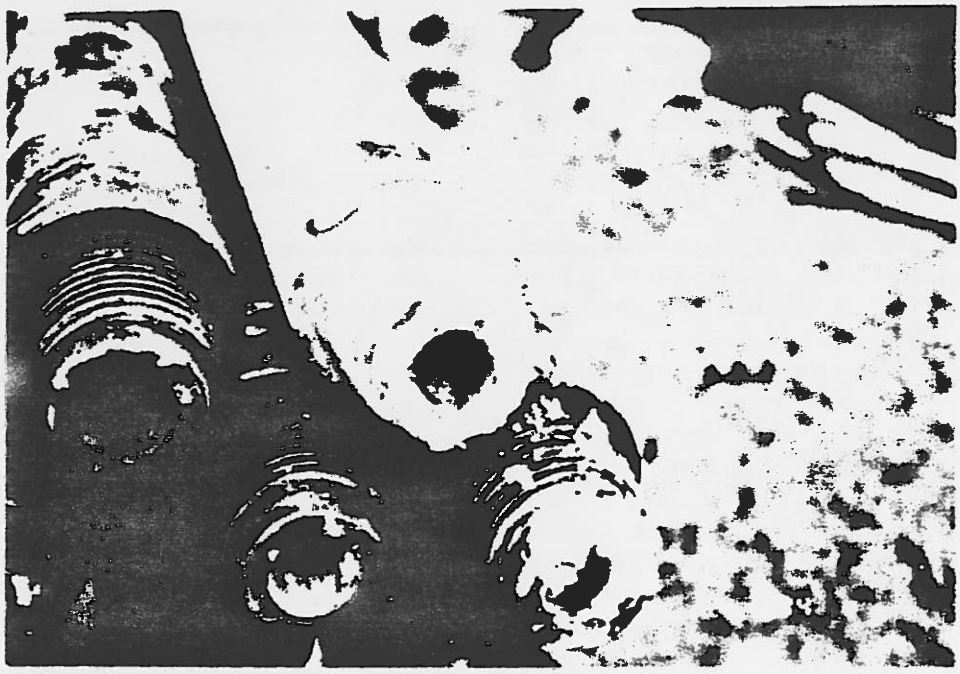


Figure 13. Jet rods after some and wear and tear (1)

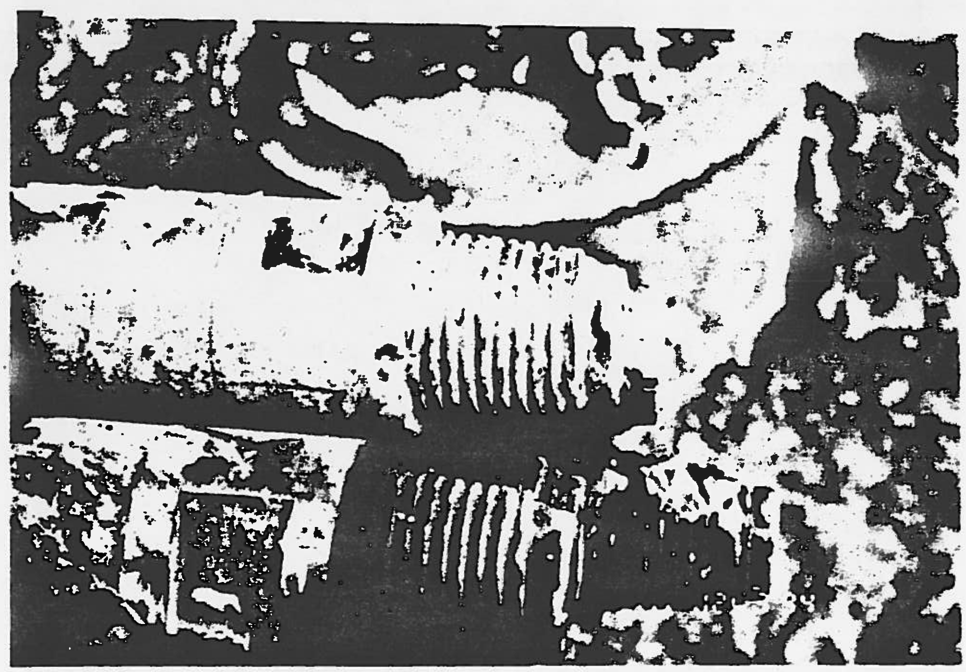


Figure 14. Jet rods after some wear and tear (2)

pressure water will remain puddled and in the system, particularly if it is clay. This will defy it from being completely discharged completely at the surface. To prevent this, the grout must be jetted in a stream-line in the lateral direction. In this way, the puddle can be completely substituted by the cement grout (Shibazaki and Ohta, 1982).

The nozzles are modified as to their extent of rotation in order to create different geometries (GKN, 1993b). Nozzle energy and grout flow are functions of the jet nozzle diameter and grout pressure. Nozzle type depends on the equipment used and is generally selected by the specialty contractor (Munfakh, et.al., 1987). It is not uncommon to use as many as four jets within a 1.8 mm to 2.2 mm diameter range (Gazaway and Jaspers, 1992, and Delta, 1993). The nozzle used for water expulsion is very fine in comparison to the much larger grout nozzle (GKN, 1993a). Parry-Davies, et.al. (1992) note that a 2 mm diameter nozzle has proven optimization for the water nozzle, with the water and air rods located above the grout rod by 150 to 400 mm.

INSTALLATION

With jet grouting, the installation is more intricately wedded to the final performance than to the constituent materials. This distinguishes jet grouting from many other ground improvement techniques. It is for this reason that good installation methods must be outlined and followed. Since standard tension and compression load tests cannot be easily or regularly done for verification proper installation and strong quality control measures are vital. The installation process may begin with clearing and grubbing the site and may end with reseeding, but regardless of what steps occur in between, each must be done with great care as to not jeopardize performance potential. The purpose of this section is to highlight aspects of construction that may be considered good practice, to identify common problems, and outline QAQC procedures.

PRECONSTRUCTION

Prior to the commencement of any construction, the contractor's plan of work should be established either by contract or by submittal. It should describe all anticipated aspects of construction (Hayward, 1991c), including any anticipated difficulties. The following is a compilation of potentially applicable submittal requirements directly related to installation:

1. Location and orientation of each grouted column.
2. Type and size of each column.
3. Capacity.
4. Equipment, including grout plant lay-out.
5. Grout design mix and mixing and injection techniques.
6. Sequence of installation, including phasing and scheduling.
7. Guidelines to maintain drilling alignments, both horizontally and vertically.
7. Plan to accommodate low headroom or nearby obstructions.
8. Proposed measures to overcome under-ground interferences, including necessary documentation and protection.
9. Anticipated equipment loads on decking or adjacent ground during construction.
10. Testing criteria including allowable deforma-- tions and test procedures.
11. Connection details.
12. Criteria for implementing remedial procedures.

(Hayward, 1991c and Delta, 1993)

Equipment. A properly located and adequately sized staging area are essential (Munfakh, et.al., 1987). Delta (1993) recommends the requirement of shop drawings showing a schematic diagram for all equipment layout and water and grout flow. The equipment's location should not cause conflict with other construction activities or induce delays (Munfakh, et.al, 1987). A grouting plant has a potential pumping reach of 152.5 m. If the grout plant is thoughtfully located so that it is in the center of the operation, a span of 305 m radially (152.5 m in any direction) can be achieved (Perry, 1993).

DRILLING

Good drilling is the first step to good jet grouting. The drilling process is intimately connected to the success of the jet grouting process. Jet piling installation occurs in two motions. In the first, the equipment drills down to the final depth of the planned grouting. In the second, the equipment is gradually withdrawn while the area is simultaneously grouted. The drilling portion is highly integral to the element's ultimate performance because it involves the actual preparation of the aggregate. The rotating jets destroy soft soil formations (Andromalos and Pettit, 1986). The high-speed water jet works in both the horizontal and vertical directions during soil excavation to cut up the in-situ material (Munfakh, et.al., 1987).

Although the standard drilling procedure for jet piles is done without a casing, in unstable ground or where large in-situ obstructions exist, casing (Delta, 1993) and predrilling are options. Predrilling can be done with a down-the-hole hammer with a 139.7 mm bit through a 152.4 mm hole (Perry, 1993). Drills and bits must be of suitable type and size and must be kept sharp (Delta, 1993). This is especially important since straightness and verticality are controlled by the drilling equipment (Perry, 1993 and Welsh, 1993b). The most important factor in drilling is straightness. Tolerances range from site to site depending upon the presence of nearby structures (Figure 15), the proximity of utilities and right-of-way issues. For conventional duplex and single rod drilling methods when used horizontally, the target alignment tolerance is one percent of the drill hole length (Delta, 1993). The same is true for vertical installations (Steiner, et.al., 1992).

Nonveiller (1989) identifies the main causes of uncontrolled borehole deflection:

- Inclined or not steady fastened drill rig.
- Use of bent drill rods
- Selection of rotational speeds and pressures inappropriate for the soil/rock character and quality.
- Imprecise centering of the rig or drill after equipment relocation.

Nonveiller (1989) concludes that the hole deflection can be largely reduced by careful drilling. It is important to keep to within the tolerances because the potential drilling deviation, in conjunction with the pile's expected diameter, determine the column spacing (Steiner, et.al., 1992).

Predrilling decreases the chance for error which might arise with a drill string (Perry, 1993). When predrilling is done a prewashing technique, which may improve performance, can be tried. De Paoli, et.al. (1989) report a case history in peat where a three rod system was compared in an extensive test program. Actual diameters were correlated with the injection pressure, the discharge, the rate of fluid ejection, the power leaving the hole, and the specific energy per linear meter of the column or per cubic meter of column. Prewashing permitted a fifty percent decrease in required energy for an equivalent diameter. For the same amount of applied energy, the column diameter increased from 1.5 to 2.2 m (De Paoli, et.al., 1989).

In cases where jet grouting is to be done through existing footings, coring is done prior to the set-up of the grouting rig (GKN, 1993b), and in especially difficult soil conditions, where wood, cables, boulders and other obstructions are anticipated, holes can be predrilled with a diamond drill without casing and with a down-the-hole hammer (Steiner, et.al., 1992; Parry-Davies, et.al., 1992; Perry, 1993; and Welsh, 1993b). Drilling equipment may be rotary or rotary percussion (Delta, 1993). Use of hammers or blasting is generally prohibited (NYSDOT). If utilities are anticipated, extra care must be taken. Uncovering them through hand excavation is the recommended procedure (NYSDOT).

Another alternative in predrilling is the use of an air-powered core, drill mounted on a conventional air-track drill rig (Burke, et.al., 1989a). A cased or uncased borehole is first drilled with circulation either of water or bentonite mud to the required depth. A string of three way rods fitted at the bottom with a jetting tool is then lowered into the casing. When the casing has been wholly or partly withdrawn, the injection phase is started by revolving and drawing up the monitor. The procedure is comprised of fracturing the soil and removing its finest particles by air-water jets just before the injection of grout (Gallavresi, 1992). When drilling through existing structures, a 150 mm diameter cores is typical (Burke, et.al., 1989b and Burke and Meffe, 1991), and a meter of plastic pipe may be inserted into the predrilled core to help keep the hole clean of waste grout from adjacent columns (Burke, et.al., 1989a). In organic or highly fine-grained material, a single rod drill system without casing is sometimes employed (Delta, 1993). As with all techniques the drilling fluid must be compatible with both the drilling method and the soil conditions (Delta, 1993), but unlike those ground improvement techniques that derive the load-carrying capacity from skin friction, use of bentonite is not a problem with the jet grouting (Perry, 1993). Whenever drilling occurs care must be taken to protect the newly installed, as well as, the preexisting environment. The most important factor in protecting new work is allowing sufficient grout set up [Commonwealth, 1984 and Bruce, 1993e]. Littlejohn (1990) warns that the actions of drilling and grouting in cohesive soils cause stress changes within the ground which cannot be accurately modeled by either an effective stress or total stress analysis. Of vital importance in cohesive deposits is the time during

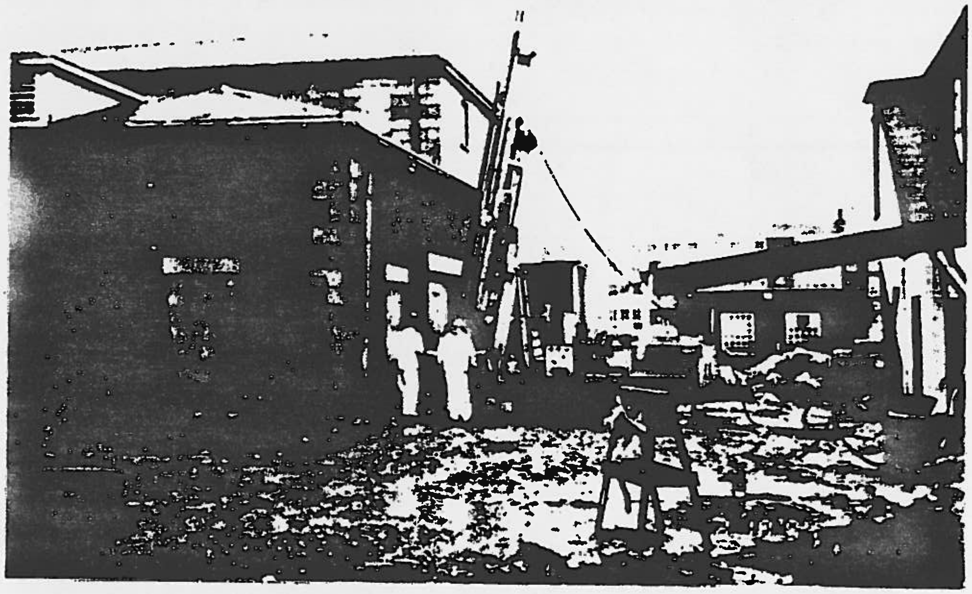


Figure 15. Jet grouting immediately adjacent to a structure

which drilling and grouting take place; water softens the clay, and within only a few hours the load bearing capacity is significantly reduced (Littlejohn, 1990). Since the grouting should immediately follow the drilling, this should not be a problem. Installation, including casing placement and removal, should be done with non-vibratory and non-displacement methods, when possible. In the vicinity of subways, tunnels, and underground utilities (Hayward, 1991a and Gazaway and Jaspers, 1992), drilling should be performed by rotating or oscillating casing and applying a static vertical load (NYSDOT). When grouting through concrete piles caps, there is the possibility that pressure may build-up from beneath. To avoid this, drilling holes through each existing pile cap prior to the start of any grouting is recommended (Parry-Davies, et.al., 1992 and Perry, 1993).

In a triple-rod system, the air around the perimeter and water in the center do the actual excavation (Welsh, 1993b). The sheathed water jet (Figure 16) erodes the soil while the air dislocates it upward out of the jet pile (Munfakh, et.al., 1987). By displacing the cuttings, thereby clearing the spray path by up to one meter, the concentric collar of compressed air enhances the cutting action of the water jet (Hayward, 1991a and Parry-Davies, et.al., 1992). The air curtain allows the jet water to maintain a pressure similar to one open air (Anon, 1974) and works particularly well below the water table (GKN, 1993a). Depending on the lift and injection parameters selected, partial or complete soil removal can be achieved (GKN, 1993a). The fluid rods are contained within a pipe and are frequently 101.6 mm in diameter (Flick, et.al., 1992).

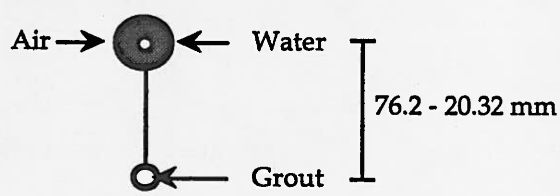


Figure 16. Flow is key (after Perry, 1993)

The size of the actual drill hole is dependent on which jet grouting variation is employed. The different variations control the amount of in-situ material that will be replaced or used as aggregate. When little soil replacement by grout is anticipated as in a single or double rod system, a borehole of the same size as the grout rods is used (final column diameters range up to one meter). When a large amount of soil replacement is expected, as is usually the case in the triple rod system, the jet grouting is carried out through an oversized borehole that allows the slurried spoil containing the eroded soil to exit the hole (3 m diameter columns are not uncommon) [Munfakh, et.al., 1987 and Bruce, 1988a]. Flick, et.al. (1992) describe a 178 mm diameter drill hole created to accommodate a 102 mm diameter pipe. Ichihashi, et.al. (1992) report that a 140 mm diameter drill hole is sufficiently large to produce an improved body of soil 2 m in diameter.

should be used only if they serve a specific function such as improving workability or durability, reducing bleed or shrinkage, or increasing the rate of strength development (Hayward, 1993c). Additives should always be used sparingly and according to the manu-

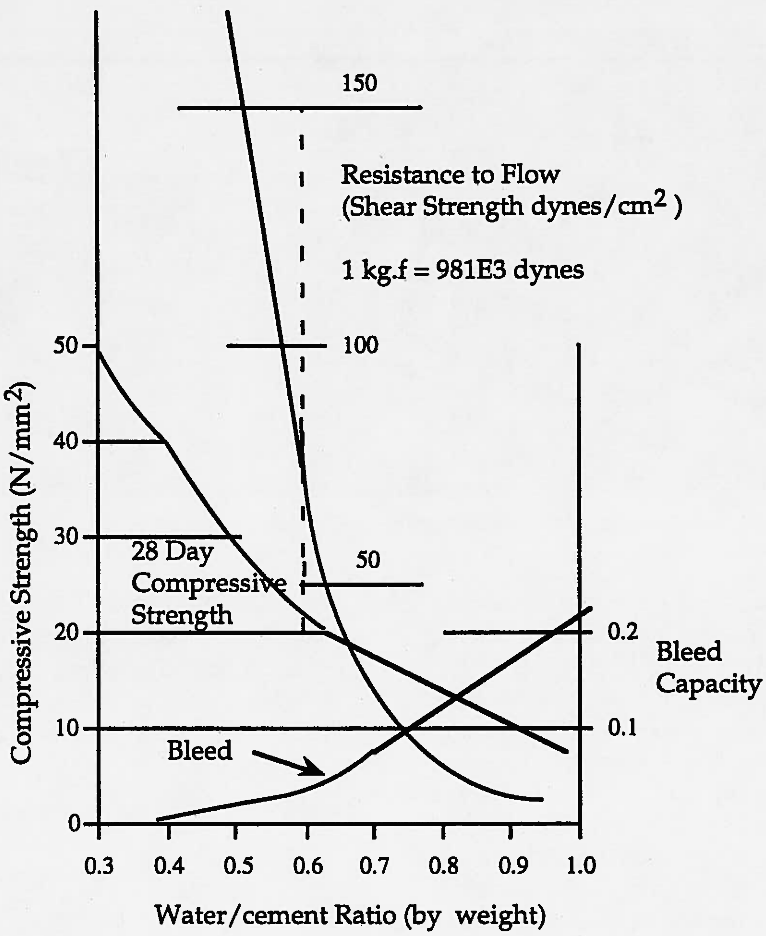


Figure 17. Effect of water content on grout properties (after Littlejohn and Bruce, 1977)

GROUT DESIGN MIX

The mix design is a neat grout with additives. Grout plasticity is crucial since central plant mix may be over a hundred meters from the point of application (Micciche, 1993a). For this reason, admixtures in jet piles are common (Welsh, 1993b). This may include plasticizers (Micciche, 1993b) or fly ash in a 1:10 by weight ratio with the cement (Munfakh, et.al., 1987). A plasticizer allows the grout a sufficient pumpability to permit transport over 30.5 meters without increasing the water content, which would compromise strength (Bruce, 1993a). Littlejohn (1990) advises that admixtures facturer's recommendations (Hayward, 1991c and PTI, 1986). Bruce (1991) warns that "a little additive goes a long way", and that any mix involving an additive should be thoroughly tested on site, for both its fluid and set properties. The methods of proportioning and mixing should result in a precise and easily measured dosage thoroughly and uniformly mixed into the grout (Delta, 1993).

The need for good plasticity is also reflected in the relatively high water to cement ratios. A range of 0.6 to 0.7 by weight is common (Micciche, 1993b and Gallavresi, 1992), but may range from as little as 0.55 to as much as 1.4 (De Paoli, et.al., 1989;

Micciche, 1993b; Munfakh, et.al., 1987; Winterkorn and Pamukcu, 1990; Andromalos and Pettit, 1986; Bruce, 1988b; and Delta, 1993), with cement contents between 1,270 and 1,451 kg/m^3 (De Paoli, et.al., 1989 and Gallavresi, 1992). The most critical decision for grout is the water content since is directly related to strength performance (Figure 17). The compressive strength is inversely related to the water content (Bruce, 1993a). As an example, if there are no additives or aggregate, a water/cement ratio of 0.45 will achieve a 28-day strength of about 24.115 MPa (PTI, 1986). Strength is highly dependent upon need and is, therefore, entirely site specific.

The water/cement ratio also controls the amount of bleed (Bruce, 1991). Bleed is the phenomenon of water separation from the rest of the mix. When bleed occurs a layer of water will form at the top of any grouted element (PTI, 1986). The importance of bleed is not fully agreed upon. Jefferies (1992) remarks that it is possibly the most important phenomenon; St. Simon (1993b) disagrees. Due to the hydration process, excess water results in increased shrinkage. Significant shrinkage results in the build up of tensile stresses. If the level of stress exceeds the tensile capability of the system, cracking will result (Winterkorn and Pamukcu, 1990).

The other manner in which shrinkage occurs is the result of water bleeding into the surrounding soils (Burke, et.al., 1989a). In permeable granular soils, much of the injection water may be expected to be drained out both from soil and grout, whereas in a cohesive soil of low permeability, poor or no drainage is likely (Bruce, 1988b). Water content needs to be adjusted accordingly. Using a small amount of bentonite as an additive will help this problem (Bruce, 1988b and Munfakh, et.al., 1987). The bentonite is used to decrease permeability. The addition of two percent bentonite to the grout mix can help to reduce shrinkage during curing to an inconsequential amount (Burke, et.al., 1989a). The use of bentonite does, however, come with a decrease in strength and must, therefore, be used with care (Bruce, 1988b).

Overall, the grout should be a relatively, non-shrink material (Welsh, 1993b). Mix viscosity should be low to promote uniform treatment to the greatest extent (Bruce, 1988b). An optimum mix for one site consisted of approximately 17 percent cement and 9 percent bentonite (Gazaway and Jaspers, 1992) It is also possible to use chemical grouts for jet grouting, although this is not standard practice (Munfakh, et.al., 1987).

Once a design mix is submitted and accepted, it should not be changed without formal notification and approval by the supervising engineer (Delta, 1993).

EQUIPMENT

The first element to proper installation should focus on the fluid lines. There is no standard practice for selecting which passage-way carries the air, water and grout, except that the drilling related fluids (air and/or water) must be located above the point where grout is injected (Kauschinger, et. al., 1992 and Parry-Davies, et.al., 1992) {Figure 16]. A recommended distance is 400 mm (Parry-Davies, et.al., 1992), except in sands, where tests have shown that placement of the grout nozzle 150 mm from the water and air nozzles to achieve maximum effectiveness. A 180° orientation between the grout nozzle and the air and water nozzles has been determined to be optimum. Any closer orientation results in increased grout wastage (Parry-Davies, et.al., 1992).

The air curtain allows the jet water to maintain the same pressure as it would in open air, but if the amount of air pressure is excessive the resulting turbulence will disturb the water jet (Anon, 1974). There must be continuous flow of cuttings from the nozzle up to the surface. This insures that the pressure head at the point of injection is only due to the weight of the cuttings. When jet grouting, extreme caution must be exercised to prevent the vent hole from clogging (Marcuson, 1992). If the path is restricted, then the pressure head at the point of injection becomes the same as the pump pressure and flow halts. If the restriction is not cleared by pressure blowing the soil from around the drill rods, severe ground heave, and lateral soil movements will occur, in addition to production stoppage (Kauschinger, et.al., 1992). At fluid output of 130 to 145 liters/min (Perry, 1993). The possibility of inducing damage during a rod clogging is a serious consideration. Hydro-fracture occurs mostly when there is an interruption of the flow of cuttings to the surface (Kauschinger, et.al., 1992a). In a double rod system if the pathway is not kept open, the mechanism essentially becomes single fluid grouting. Kauschinger, et.al., (1992) warn, "Problems with the annulus clogging may arise when the double system is used to jet grout under limited overhead room, which requires uncoupling of the drill rods." Independent of the number of fluid lines, the jet nozzles must be sized to allow an even flow of the

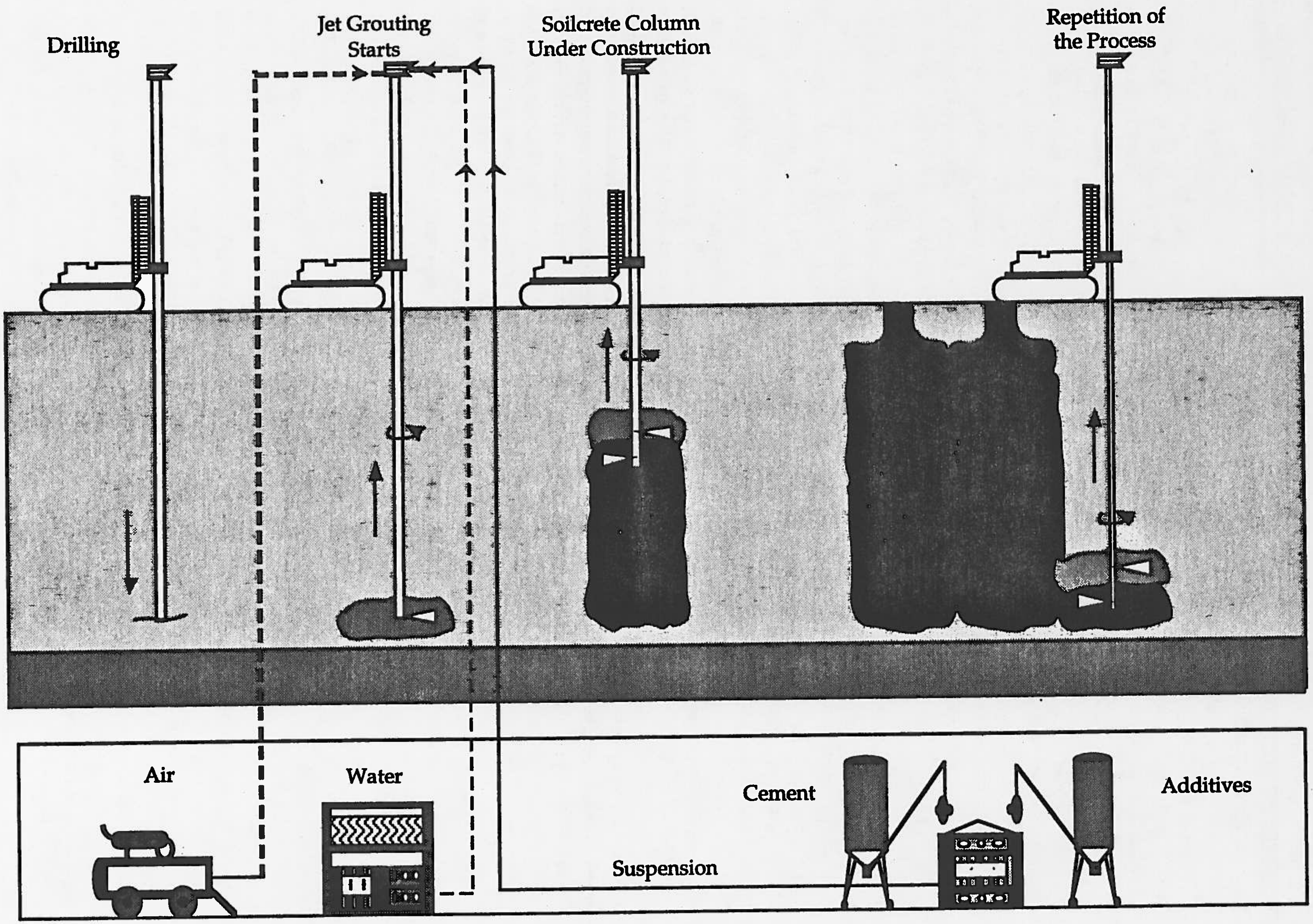


Figure 18. Jet Grouting Plant Layout and Installation Sequence (after GKN, 1993a)

water or grout at a sufficient pressure to erode the in-situ soil (Munfakh, et.al., 1987). As clay content increases, grouting pressure must be increased and/or withdrawal rate decreased for a given jet grouted volume (Munfakh, et.al., 1987).

Any grouting condition such as flow rate, pressure, grout set time, or viscosity which would cause mechanical breakdown, undue wear and tear, damage, or clogging of the equipment must be avoided (Delta, 1993). Changes in soil conditions can be taken into account and accommodated for in the selection of these parameters (Kauschinger, et.al., 1992). As with most ground improvement techniques, some mild design modification may be anticipated in the field (Hayward, 1991a). Welsh and Burke (1991) advise a high level of control must be built into the pumping unit to ensure consistent flow rates and to record the volume of grout placed.

INSTALLATION

Jet grouting is usually done toward the beginning of construction as a protective measure, but it can also be used as a remedial measure later in construction to control excessive settlement, instability or inflow of water (Munfakh, et.al., 1987).

Drilling with air or water jets is the first step (Flick, et.al, 1992). Once drilling reaches the final depth, the jet grouting begins with a continuous rotation at the lowest point (Figure 18). Depending upon the number of rods and the soil, jet piles can be installed at nearly an angle and in a variety of geometries (Welsh, 1993b and St. Simon, 1993c). The installation process is summarized in Figure 19.

The method excavates a cavity by the action of a horizontally-oriented high velocity air/water jet, from a grout pipe that is rotated at a controlled rate. The cavity is then filled with grout (Winterkorn and Pamukcu, 1990). This occurs for a period of five minutes without any withdrawal. This process creates a single enlarged bulb to ensure a good base. A competent base is particularly important, since unlike most other deep foundation alternatives, jet-grouted columns act primarily in end bearing (Micciche, 1993a and Welsh, 1993b). Then, concurrently with the water and air, the grout is injected for an additional amount of time until the grout begins to emerge at the top of the drill hole (Parry-Davies, et.al., 1992). After grout emerges, the withdrawal begins. Pumped through horizontal nozzles, the grout mixes with the soil as the drill bit is withdrawn (Munfakh, et.al., 1987 and Andromalos and Pettit, 1986). The fines are flushed from between the coarse-grained material (>25 mm) and are replaced by grout (Parry-Davies, et.al., 1992). In material predominated by rock and boulders, the process is identical with the water jet removing the fines from the voids of the larger pieces and replacing those with grout (Parry-Davies, et.al., 1992).

Grout. For those ground improvement techniques which use grout, various codes and authors offer some guidelines for installation:

1. Add all cement, plasticizers, and other additives prior to mixing.
2. Prevent the presence of air in the grout lines, by expelling all air and checking that the suction circuit is airtight (Littlejohn, 1990).
3. Do not draw down the level of grout in the supply tank below the crown of the exit pipe, during grouting (Littlejohn, 1990).
4. Select sufficiently large equipment to permit continuous grout placement (Hayward, 1991a; Nonveiller, 1989; and PTI, 1986).
5. Select a mixer capable of continuous grout agitation (Hayward, 1991a).
6. Ensure the exclusion of any foreign matter during grout placement full breadth (Commonwealth, 1984 and BOCA, 1990).

When dealing with a heterogeneous soil, proper bonding at the interface of the soil layers is extremely important. For soil-cement mixtures it has been shown that in multiple-layered, soil-pavement structures that retardation of the hydration process of the soil-cement mixture within the first two to seven days of curing significantly improved the bonds between the layers and resulted in a subsequent increase in shear strength (Winterkorn and Pamukcu, 1990). Cure time is defined by PTI (1986) as the time it takes for the material to reach 80 to 90 percent of its final strength. The cure time is very important because a newly placed grout-based element must be given sufficient time to set up prior to additional construction work in the area. Figure 20 shows the relationship of cure time versus strength for three different water/cement ratios. Permitting sufficient setup time may have a significant effect on production rates and, therefore, on the cost. Grout and any reinforcing in the grout should remain undisturbed until the grout has cured (PTI, 1986). This statement may also apply to connection methods and premature loading of the element.

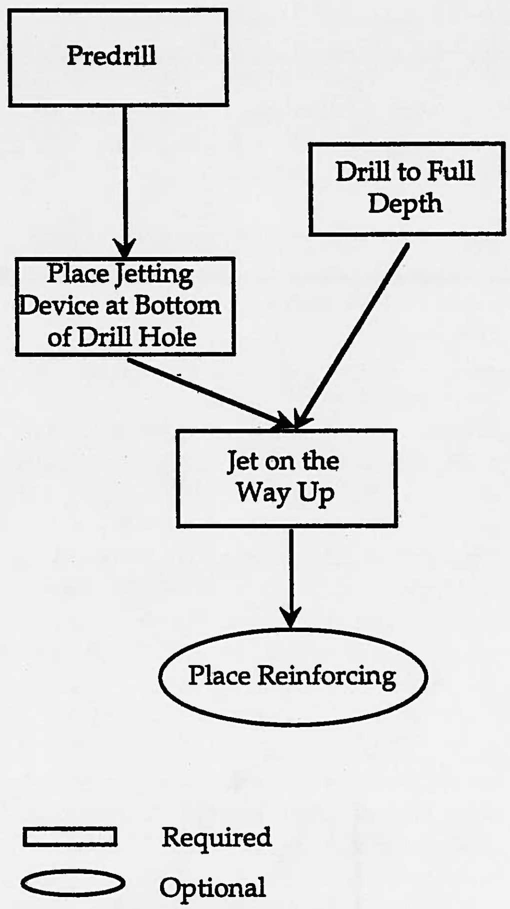


Figure 19. Installation process summary

Grout injection pressures can conveniently be monitored by attaching a pressure transducer to the injection stand-pipe via a gauge saver in exactly the same manner as one would attach a Bourbon gauge (Jefferies, et.al., 1982).

Throughout the process, a positive grout head needs to be maintained. This permits installation in high water table conditions (Burke, et.al., 1989a). Use of a removable or sacrificial casing can be helpful when placing under water (Kauschinger, et.al., 1992b and Flick, et.al., 1992). The temporary material must be appropriate for shredding during the jetting process. Thin-walled, PVC pipe has been used with success (Delta, 1993). The end product is a continuous, fully stabilized soil and grout zone (GKN, 1993b). Where predrilling occurs, the material is removed prior to grouting (Welsh 1993b), and grouting begins immediately after the drilling is completed from the bottom up in a manner almost coincident with conventional installation processes (Steiner, et.al., 1992). Grouting always starts at the lowest point or the point furthest from the ground surface.

Grouting across the project may be conducted in several stages with inactivity between each stage, but there may not be an interruption in the installation of any single pile (Delta, 1993). If for some reason this is impossible the jet grouting should resume beneath the level that grouting was previously completed (Perry, 1993). Preventing lapses in grouting may be the most important statement on good installation techniques. Radio contact with the grout plant should be maintained to avoid this problem (Perry, 1993).

In predrilled holes, grouting begins at the location furthest from the point of entry, progressing backwards in a manner that the grouting pipes are progressively shortened or removed during withdrawal (Delta, 1993).

Once the installation process is complete, frequent column topping off with grout will offset any grout bleeding into surrounding soils and minimize structural deformations (Burke, et.al., 1989a and Delta, 1993). It is common to permit a minimum of one hour at the end of the work day for this activity (Burke, et.al., 1989a). To accomplish this, a plastic pipe is placed in the core hole of the column (Burke, et.al., 1989a). By maintaining a positive head on the jet grouted column, there will be minimal soilcrete shrinkage, particularly at the critical point where the column meets the under-side of the foundation being underpinned (Welsh and Burke, 1991).

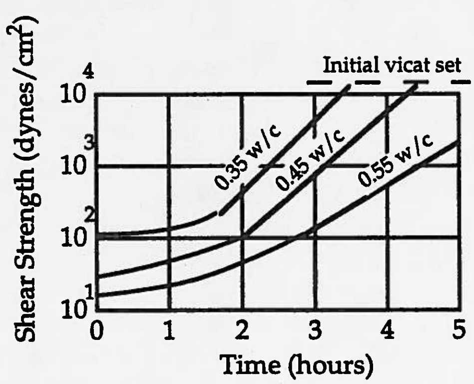


Figure 20. Setting Time for Type 1 Grouts (at 18° C) [after, Littlejohn, 1990]

Parameters. There are many factors which effect the final size and strength of a jet-grouted column. One which cannot be changed is the soil type. Table 2 shows the influence of soil type on the column's final diameter.

Table 2. Range of Jet Column Diameters

Diameter (m)	Soil Type	Source
0.4 - 0.8	Clays	Kauschinger, et.al., 1992b
0.5 - 1.2	Sands and Gravel	Kauschinger, et.al., 1992b
0.6 - 0.8	Gravel	Steiner, et.al., 1992
0.6 - 0.8	Unspecified	Andromalos & Pettit, 1986
≥ 0.8	Loose Silt and Sand	Steiner, et.al., 1992
1.0 -1.33	Non-Cohesive	Ryan, 1993
≤ 1.5	Stiff Clay	Munfakh, et.al., 1987
2.0	Unspecified	Gallavresi, 1992
2.0	Sand	Flick, et.al. 1992
4.0	Non-Cohesive	Parry-Davies, et.al., 1992
≤ 8.0	Boulders	Parry-Davies, et.al., 1992

The final diameters are also strongly effected by a variety of variable installation parameters. The most influential of which are considered to be water/grout pressure and withdrawal rate. Table 3 shows the influence of these parameters on the volume of soil treated. Figure 21 demonstrates the influence of pressure on various soil types.

Table 3. Installation parameters in relation to treated volumes (Munfakh, et.al, 1987)

Soil	Water/Grout Pressure (MPa)	Withdrawal Rate (mm/min)	Grouted Volume (cm/m)
Soft Rock	29.273 - 48.919	22.86 - 93.98	0.138 - 0.165
Dense Sand	29.273 - 48.919	30.48 - 116.80	0.147 - 0.174
and Gravel	34.450 - 41.340	304.80	0.285 - 1.800
Medium	29.273 - 48.919	132.08 - 231.14	0.166 - 0.239
Dense	19.981 - 36.517	99.06 - 500.38	0.129 - 1.525
Sand	38.584	914.40	0.184
	30.316	398.78	0.276
	29.273 - 48.919	500.38 -1198.88	0.083 - 0.597
	5.512 - 6.890	398.78 - 599.44	0.202 - 0.992
Loose Sand	29.273 - 48.919	231.14 - 287.02	0.220 - 0.248
Clay and	29.273 - 48.919	309.88 - 386.08	0.239 - 0.266
Silt	19.981 - 39.962	99.06 - 500.38	0.129 - 1.525
	30.316	398.78	0.276
	29.273 - 48.919	500.38 -1198.88	0.064 - 0.533
	5.512 - 6.890	398.78 - 599.44	0.073 - 0.395

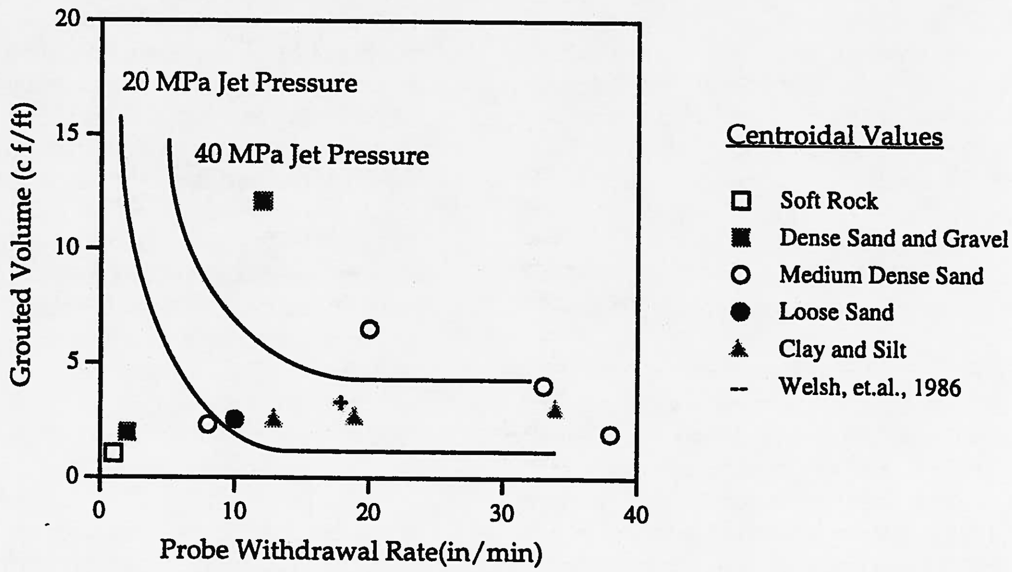


Figure 21. Probe withdrawal rate effects on jet grouted volume (after Munfakh, et.al., 1987)

In general, the range of variables regularly employed are shown in Table 4.

Table 4. Standard installation parameters (Delta, 1993)

Rotation Rate	5 - 20 rpm.
Withdrawal Rate	0.15 - 0.61 m/min.
Injection Flow Rate	57 - 303 l/min.
Injection Pressure	13.78 - 51.68 MPa (at the jet)

When three fluids are used, each is placed at a different pressure. Table 5 shows relative installation pressures.

Table 5. Various fluid placement pressures (after Bruce, 1988a and Parry-Davies, et.al., 1992).

Fluid	Placement Pressure
Air	7 - 12 bar
Grout	15 - 25 bar
Water	170 - 600 bar

A selection of smaller steps (e.g., 12.5 mm instead of the standard 25 mm) results in a more uniform section (Miyasaka, et.al., 1992). Rates also depend on whether the columns are primary or secondary. Parry-Davies, et.al. (1992) report an increase of nearly 20 percent in the withdrawal rate for the secondaries.

The withdrawal rate tends to be between 0.152 and 0.610 m/min. (Parry-Davies, et.al., 1992, Andromalos and Pettit, 1986, Gazaway and Jaspers, 1992 and Delta, 1993). It is this parameter which largely controls radius of influence (Gallavresi, 1992). Some other variables are the rotation speed which varies from 1.3 to 20 rpm (Parry-Davies, et.al., 1992; Gazaway and Jaspers, 1992; and Delta, 1993) and the injection flow rate, which ranges from 56.775 to 302.8 liters/min (Delta, 1993 and Anon, 1974).

Placement. The system is most often designed as a series of interconnected columns of various lengths and geometries to create a soilcrete mass (Burke, et.al., 1989a). A common configuration uses 180°, which is done by omitting rotation during withdrawal (Bruce, 1988a). Jet columns are installed in predetermined locations and according to a pre-specified sequence which can be outlined in the contract documents (Burke, et.al., 1989a). The collar of all grout holes should be located within 76.2 mm of the designated location (Delta, 1993). Collars for grout holes should control the flow of water into the access excavation both at the collar/wall contact and at the collar/drill rod contact. Leakage or flow past or through the collars should be controlled so that no adverse impacts result, including but not limited to, groundwater drawdown outside the excavation, excessive pumping requirements from inside the excavation, or the piping of soils into the excavation. Sleeves, gaskets, boots, skirts, and similar devices may be necessary (Delta, 1993).

Regardless of which rod system is selected, the installation of interconnected jet columns follows roughly the same process. The jet piles are installed on a center-to-center spacing based on the anticipated diameter of the element plus a certain margin added for overlap. The extent of overlap is dependent on final performance criteria, and for permeability requirements, it is especially critical. Overlap and redundancy are also necessary in case of deviation from the boring axis during installation (Steiner, et.al., 1992). A 0.762 m, center-to-center spacing is a common distance (Gazaway and Jaspers, 1992).

As with the required amount of overlap, the installation sequence of a group of jet piles is heavily influenced by the application, specifically settlement or permeability tolerances predominate. The sequencing of the work is very important as it may strongly influence the amount of subsequent settlement occurring during the construction process (Burke, et.al., 1989b). The sequencing plan must maintain sufficient load-bearing under all supported structures at all times (Burke, et.al., 1989a)[2]. The order of installation for a multirow underpinning will be done to provide the maximum support and stability for the structure. This usually means starting with the one that is furthest out and working towards the underside of the building (Micciche, 1993b). In this case, a first row may be installed right up next to the building with a batter of up to 20 degrees (Burke, et al 1989a) [Figures 22 and 23]. A second row may be installed from virtually the same starting point but without any inclination. The other approach to the double row wall is to alternate from front to back to permit maximum installation without disturbing columns in the curing process (Anon, 1962 and Perry, 1993b). This allows the grout to set before being subjected to disturbance by adjacent jet grouting (Anon, 1962 and Parry-Davies, et.al., 1992). Adjacent elements should never be done sequentially

[2] See Winterkorn and Pamukcu (1990) for a more in depth discussion of soil-cement.

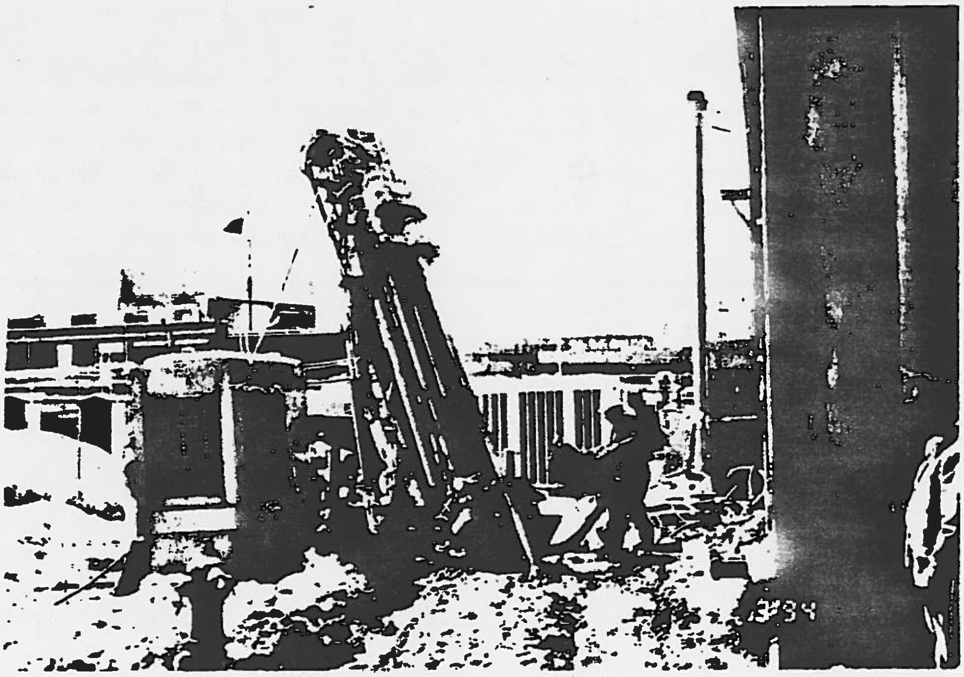


Figure 22. Jet pile installation

Figure 23. Jet pile installation close up

(GKN, 1993b). A double row of jet piles is common, but in some cases a third row may be required (Welsh, 1993b). When third row is employed, often the center one is saved for last (Steiner, et.al., 1992).

Through careful planning, "grouting oneself into a corner" or having to stay on a job for extra days waiting for adjoining columns to cure can be avoided (Perry, 1993). If a second row of jet columns is employed, installation should be varied between the rows (Anon, 1962) [Figure 24]. GeoCon (1994c) describes this as the 'split-space' method and notes that it is done in a predetermined order. Nonveiller (1989) defines 'split spacing' as installation of a first set of holes at a distance which exceeds the expected average each of the injected grout. The next set of holes is then injected in the middle of the spacing between the primaries. The sequence is repeated again with the tertiaries in the middle of the spacing, between the se-condaries. In some cases a triple row is designed. In that case, the center row is often saved for last (Parry-Davies, et.al, 1992) to serve as a complementary check on the soilcrete quality (Steiner, et.al., 1992). The issue of sequencing is extremely important. Nonveiller describes it as the "dominant influence of the end result."

To increase production, the installation sequence may alternate between the rows. In one case every fourth hole represented the primary holes and was drilled at 2.6 m centers (Parry-Davies, et.al., 1992). The secondaries were then drilled midway between the primaries. When the tertiary holes are treated, many of the primaries and secondaries should already be in contact with each other. This should be evident during the drilling; many of the drilled cuttings for the tertiary holes should consist of fragments of rock and grout (Parry-Davies, et.al., 1992).

Coordination with the excavation and excavation supports (if any) is important to prevent any grouting during active excavation (Delta, 1993) and or nearby columns (Micciche, 1993b).

One must check that there are no voids being created under the rear edge of the footing (Figure 25), which might occur from the inclination of jet pile under the interior portion of footings (Burke, et.al., 1989a). The same problem can occur around utility banks or other underground obstacles. Pressures and lifting rates need to be varied near existing utilities and pipelines to eliminate damage or ensure full closure (GeoCon, 1994b). Much slower rotation and lift rates need to be utilized adjacent to large diameter

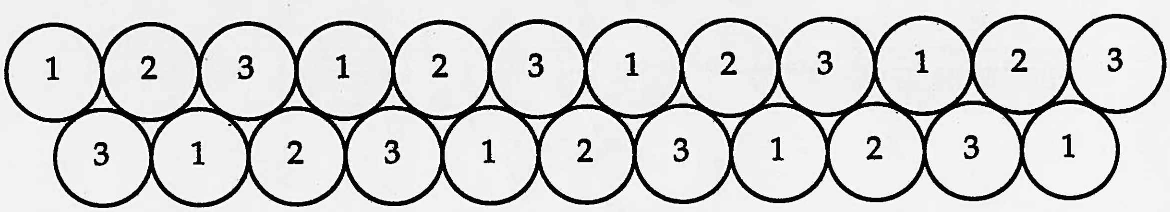


Figure 24. Potential installation sequence for underpinning

pipes, in order to assure closure beneath them. In areas of close proximity to small and fragile conduits, column spacings should be tightened and rotation and lift rates increased in order to optimize closure while preventing damage. In rare instances, the pressures are also decreased to as little as 34.5 MN/m^2 for short periods in the immediate vicinity of particularly sensitive conduits to further reduce the likelihood of damage (Gazaway and Jaspers, 1992). In the case of obstructions, the position of jet piles have been relocated up to 0.3 m without harming the final product (Flick, et.al., 1992).

A minimum distance of 4.6 m is needed between consecutive installations in sands (Perry, 1993). Although a soil-cement mixture begins to gain strength six hours after mixing [in contrast to a neat grout which takes two to five hours (Steiner, et.al., 1992)], the earliest adjacent columns can be jet grouted is 8 hours (Steiner, et.al., 1992), which is considered its initial jet time (Welsh and Burke, 1991). A 24 hour period is preferred (Perry, 1993 and Welsh, 1993b), since final set is between 13 and 15 hours (Burke and Welsh, 1991). For those elements curing less than 24 hours, no jet grouting within 2.1 m is permitted (Delta, 1993). Usually the rig is moved two column diameters away for the next installation. This prevents damage to the first column as during initial curing (Anon, 1962).

Reinforcement. Once the required length is grouted, steel reinforcing can be installed. Reinforcement is unusual and only included when required.

Given the limitations of the jet grouting equipment, the reinforcement cannot be added during grouting. In fact, nothing can be added to the jet grout mix during installation unless it is mixed in at the grout plant (St. Simon, 1993c and Welsh, 1993b). If reinforcement is used, it should be inserted in the center of each column to the column's full length (Delta, 1993). This must be done prior to hardening (St. Simon, 1993a), usually within two to three hours (Furth, 1994b).

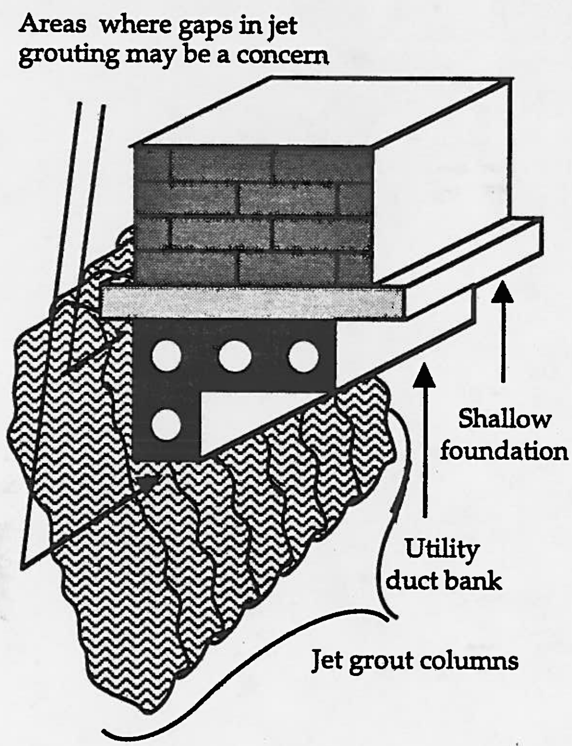


Figure 25. Areas of potential voids with jet grout

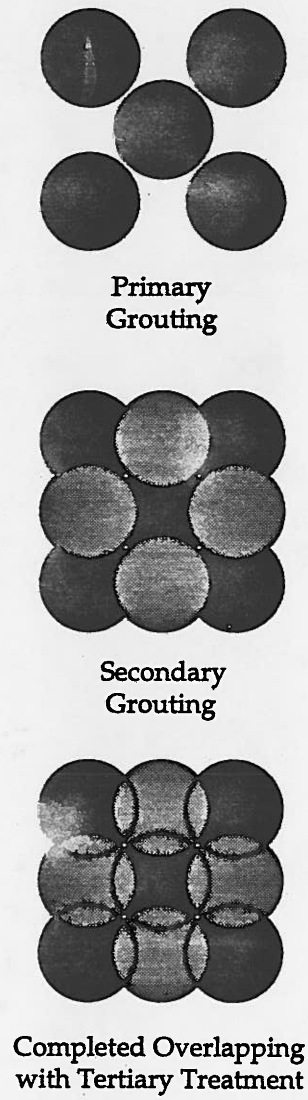


Figure 26. Typical block treatment pattern (after Gazaway and Jaspers, 1992)

Redundancy. As the jet grouting process takes place below ground with little or no direct control it is necessary to include redundancy in the design (Steiner, et.al., 1992). A major problem is deviation from the boring axis during installation (Steiner, et.al., 1992). To combat this Hayward Baker (1991b) recommends a minimum overlap of the greater of 1/8 of the column's diameter or 152 mm. In addition to overlapping adjacent columns, an additional row can be used to assist in providing sufficient redundancy and to prevent excessive settlement (Ichihashi, et.al., 1992) [Figure 26].

Attachment. Installation can occur without any direct connection between the structure and the jet column. In this case, the structure rests on the treated soil, without being anchored to it (Figures 27 and 28). On the other hand, it is possible to make the footing and the wall a single, continuous system, although this is not yet done in the U.S. (Welsh, 1993b). Attachment is often done with a reinforcing element to serve as a shear key (Flick, et.al, 1992 and Perry, 1993).

Current practice for installation through an existing footing includes dropping a PVC pipe down through the footing and filling it with the grout.

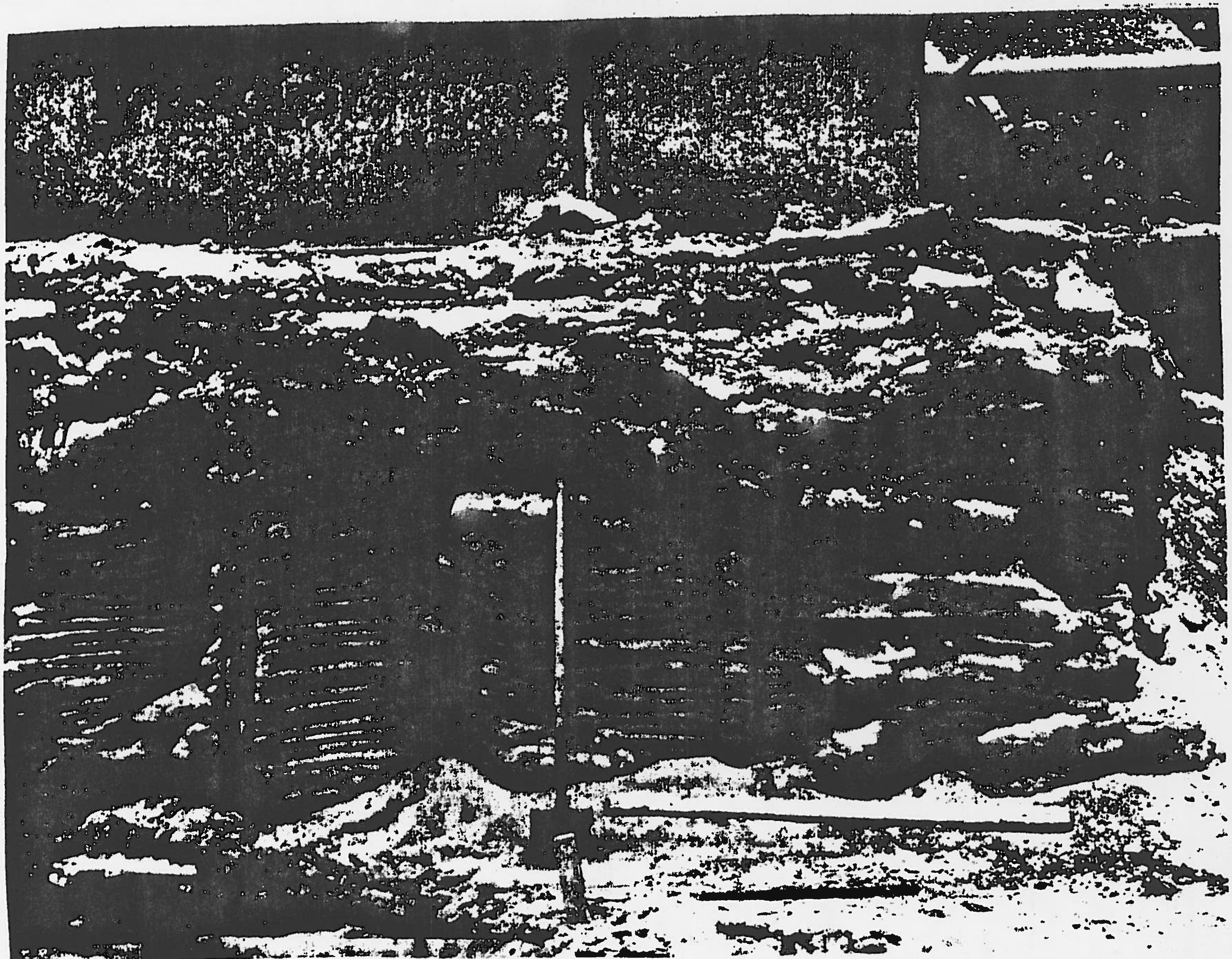


Figure 27. Half columns used as underpinning--note the drill annulus and individual water jet "ribs" and structure-free attachement method (courtesy of GKN)

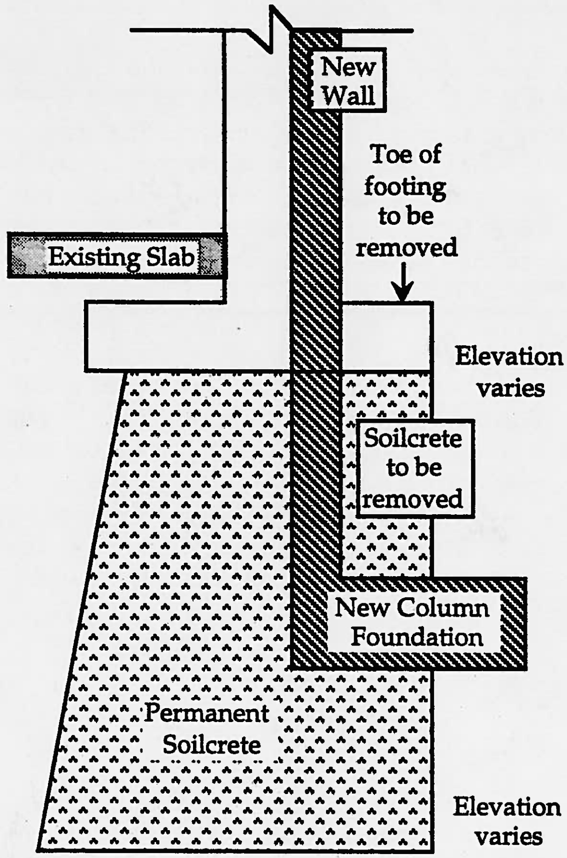


Figure 28. Underpinning installation (after Burke, et.al., 1989a)

This creates hydrostatic pressure so if some shrinkage occurs compensation can occur. After the curing is complete, any excess grout is jack-hammered away. The connection method for the outside is largely the same (Welsh, 1993b). When used in bedrock with heavy anticipated loads, a shear key connection is sometimes used.

Record Keeping. For each element the following should be recorded:

1. Size of grout hole.
2. Length of pile.
3. Location of grout hole.
4. Installation start and stop time.
5. Grout mix data (including the proportions).
6. Air, water, and grout jet pressures.
7. Grout rates for each column.
8. Withdrawal rates.
9. Any unusual occurrences including grout escape or ground heave

Automation has come to play a large part in jet grouting. Once the pressure and withdrawal speed are set the installation process and monitoring are all automated (GKN, 1993a and Hayward, 1991a). Automated rigs can keep track of much of the previously manual record keeping. Additionally, many drilling machines use computer controls for varying the amount of rod rotation while grouting (Kauschinger, et.al., 1992b), which would be necessary in case of an organic or highly cohesive layer (Perry, 1993b). Where difficult or undesirable soils (cohesives and organics) are encountered, it is possible to remove an extra percentage of the material to be replaced by grout through the process of "double cutting" (Burke and Brill, 1992). Double cutting, as seen in Figure 29, is where the withdrawal process is interrupted by the redrilling of the area. This forces the equipment back below the less suitable material allowing for additional removal during recutting and additional replacement during the second withdrawal. The results are significantly better since the amount of undesirable material is significantly less than if the process was only done once (Welsh, 1993b). Single cutting is the standard, as well as the less expensive, approach (St. Simon, 1993a).

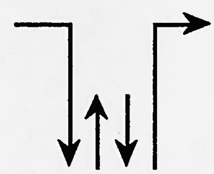


Figure 29. Double cutting process

Summary. Although the issues surrounding jet grouting are generally the same regardless of the number of delivery rods, each system has its particular advantages. Most of these characteristics are related to applications-related benefits, but the triple rod system possesses two particular installation related benefits. It is the least likely system of the three to pressurize the subsurface and heave the ground, since the water-soil-air mixture is of lower density and easily flows up the drill annulus (Burke, et. al., 1989a), and it causes the least amount of abrasion to the equipment because of the large amount of soil removal (Steiner, et.al., 1992).

QUALITY ASSURANCE

Quality assurance must occur at every step, from beginning to end. Kauschinger (1994) identifies it as the key to the successful adoption of the technology.

DRILLING

Quality control can only be obtained through regular checking of the materials, the installation procedures, and the final product. This checking must be done with each stage of installation. For drilling this process begins with establishing bore-hole verticality. To verify verticality of bore-holes there are special survey tools. A drill string and sacrificial (PVC) casing can be used to assess alignment deviation (Delta, 1993). When first introduced into the U.S., the procedure was done with two guide holes (Anon, 1974). Magnetic survey tools, applicable for vertical holes and gyroscopic tools, which are ideally suited for inclined boreholes with tight drilling tolerances. The advantage of a gyroscopic tool is that it can be used within the drill rods, thereby minimizing delays, but cost for operation and maintenance of such equipment are major obstacles (Bianchi and Bruce, 1992). For many ground improvement techniques an optical theodolite is an accurate and relatively inexpensive alternative, but because it is a line of sight instrument, it cannot be used in curved holes to any great depth or when water or another fluid is in the drill hole (Bianchi and Bruce, 1992). Given the installation requirements of grouting as part of the drilling process for all but predrilled holes, this is not usually an option with the jet grouting.

To ensure proper installation a proper drilling record should be kept. The following lists possible elements for inclusion (some of which are only applicable in case of predrilling):

1. Location of drill hole and depth
2. Drill hole diameter
3. Depth of bedrock.
6. Drilling rate
7. Type of equipment.
8. Verticality.
9. Problems encountered.
10. Date and duration.

One of the most effective methods for checking quality is the continuous monitoring of grout placement. Gallavresi (1992) gives a detailed description of Paperjet, one version of this type of system. It continuously records the impulse propagation velocity during drilling and grouting, thereby permitting real time quality control of the process. It is an energy-based approach which generates sufficient information to determine the soil profile along the vertical axis and the energy expended in construction. It records pressures, flow rates, and volumes. All variables are converted to be represented in terms of specific energy. The specific drilling energy relates to soil characterization and is based on physical parameters playing a specific role in the drilling phase. The excavation of a unit volume of soil or roc requires a certain energy, which consists of two parts. The first one is the work done by the thrust F (kN) on the drilling tool, having the same section A (m^2) of the hole, for a unit of downward displacement and is dimensionally considered a pressure, since the thrust for a unit displacement, divided by the excavated soil volume, corresponds to the pressure over the bottom of the hole. The second contribution is the work done by the torque T (kN • m) of the power swivel, which is a function of the rotational speed S [rev/s] and of the rate of penetration V (m/s).

Consequently, the specific drilling energy is expressed by the following equation:

$$E_A^F = \frac{2\pi S \cdot T}{A \cdot V} \ [kJ/m^3] \qquad (1)$$

In the case of rotary-percussive drilling used in rocks, downward and upward acceleration is required. The specific energy for grading is dependent upon the following main parameters: grout pressure P (MPa), grout flow rate Q (m/h) and withdrawal speed V_t (m/h). It can be expressed as the following:

$$E_s = \frac{P \cdot Q}{V_t} \left[\frac{MJ}{m}\right] \qquad (2)$$

When the triple-rod system is used, the jet grouting energy is calculated, adding together the energy of water and grout jettings. The air is omitted for simplicity. Transducers are placed on the jet grouting rig, while the flowmeters are integrated with the injection equipment. An important advantage is that the system conducts real time monitoring which is useful in case of mechanical malfunctioning (e.g., a drop of pressure or of flow rate or clogging of the nozzles). The rig-operator will be able to immediately detect problems on the graphic display panel and make the necessary corrections (Gallavresi, 1992). Bruce (1988a) proposes that with such a system potentially dangerous conditions (e.g. sand runs) can be closely predicted, and that the generated geological log has a high factor of accuracy which can incorporate the influence of water and inclination into its analysis.

GROUT

There are many aspect to grout testing. These include the water content, compressive strength, bleed, viscosity, and density. The extent and method of testing for several of these items is not clearly agreed upon in the industry. The following paragraphs present the some of the alternatives.

Specific gravity. Water content in grout is most frequently checked by specific gravity. Burke, et.al. (1989a) recommend it be done at least twice a day. This can be done with a hydrometer (Perry, 1993). Delta Drilling (1993) stipulates compliance with ASTM D-4380. A common specific gravity for ground improvement elements is 1.895 (Bruce, 1993b). This equates to about a 0.5 water/cement ratio (Table 5). By monitoring the water/cement ratio during grouting, one can ensure that the grout is being prepared according to the agreed upon mix design (Bruce, 1991).

Table 5 Calculated Specific Gravities of Neat Water/Cement Grouts (Littlejohn and Bruce, 1977)

Water/Cement Ratio	Specific Gravity
2.10	0.3
1.95	0.4
1.84	0.5
1.74	0.6
1.67	0.7
1.61	0.8
1.56	0.9

Checking the specific gravity through the density is an extremely quick and inexpensive test (Winterkorn ad Pamukcu, 1990). The Baroid Mud Balance (Figure 30) is a favored measure (Bruce, 1991 and Littlejohn, 1990),.

Alternatively the slump may be checked. In ground anchorage installation, a 203 to 254 mm slump would be typical (PTI, 1986). For standard concrete, the slump should be between 102 and 152 mm (BOCA, 1990).

Bleed. Bleed should be kept in the range of five to seven percent monitoring over a couple of hours will show how well the cement is mixed. Checking the bleed should be done two to three times a day (St. Simon, 1993c). Bleed can be checked with a 1000 mL graduated cylinder (75 mm in diameter) [Littlejohn, 1990].

Viscosity. A Marsh cone or flow cone measures viscosity as a function of grout flow time (Figures 31-33), giving a visual indication of grout production consistency and fluidity (Bruce, 1991 and Littlejohn, 1990). The test measures the time elapsed for a certain volume of grout to pass through a cone and should be done in accordance to API RP 13B (Delta, 1993). It is used to evaluate grout consistency. Given its speed (less than a minute) and ease, it is a popular test (St. Simon 1993a and Winterkorn and Pamukcu, 1990). Otherwise a viscometer may be used (Nonveiller, 1989).

Cement content. The cement content, as indicated through the density, influences the amount of shrinkage. Higher cement contents increase the thermal coefficient of expansion of a soil-cement system, which renders it more susceptible to temperature variations but reduces volume change and, thus, shrinkage. Increased cement content also increases water absorption capacity and reduces the overall permeability of a system (Winterkorn and Pamukcu, 1990). The target value should be between 1.6 to 1.65 (St. Simon, 1993c). Testing in accordance with API Std. 13B for bentonite and cement content is recommended for each batch (Gazaway and Jaspers, 1992).

Strength. Grout samples are an important and integral part of good quality jet piling (Welsh, 1993b). The most common test is similar to a concrete cylinder break. Compressive strength is checked through the crushing of 100 mm grout cubes (Figure 34) [Bruce, 1993c and Winterkorn and Pamukcu, 1990] or 150 mm high, 75 mm diameter cylinders (Perry, 1993 and Burke, et.al., 1989a). Unconfined compressive grout strength is determined according to ASTM C-109 at 3, 7, 14, and 28 days (Delta, 1993 and Littlejohn, 1990). Compressive strengths depend on the design specifics for a particular application but tend to range between 350 KPa and 14,000 KPa (Munfakh, et.al., 1987). Seven day tests are considered the most crucial (Delta, 1993), with anything less being suspect to too significant of a variation. At seven days a jet column will have approximately the following composition: soil 55 to 60 percent, cement 10 to 15 percent, and water 25 to 30 percent (Steiner, et.al., 1992). For grout-based soil improvement how often cubes are taken varies widely throughout the industry. Some practitioners propose testing of each batch (Bruce, 1993c), others daily (Littlejohn, 1990), and some only two to three times a week (Hover, et al., 1989).

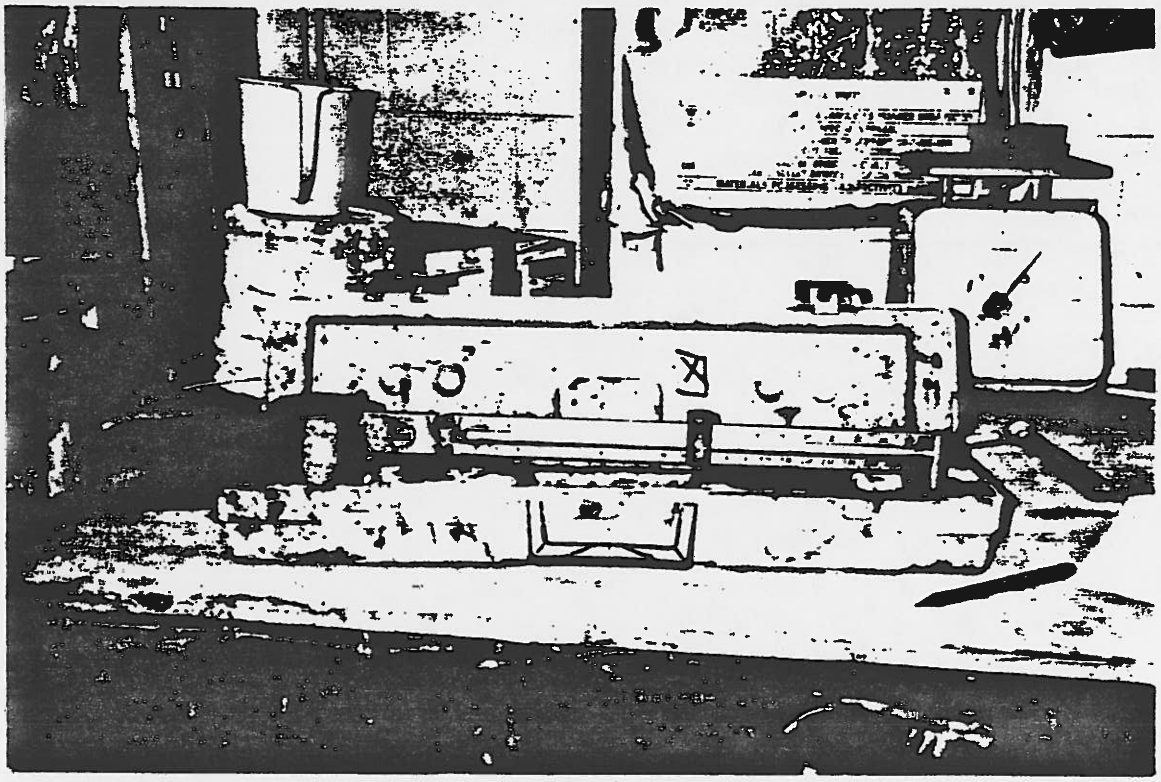


Figure 30. Density testing equipment

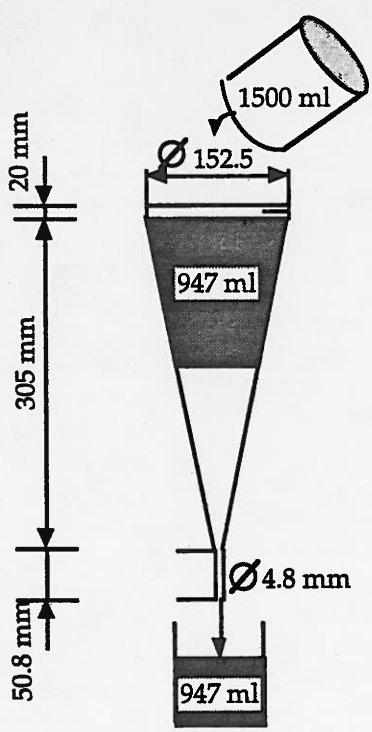


Figure 31. Marsh flow cone for viscosity measurement (after Nonveiller, 1989)

Figure 32. Viscosity testing

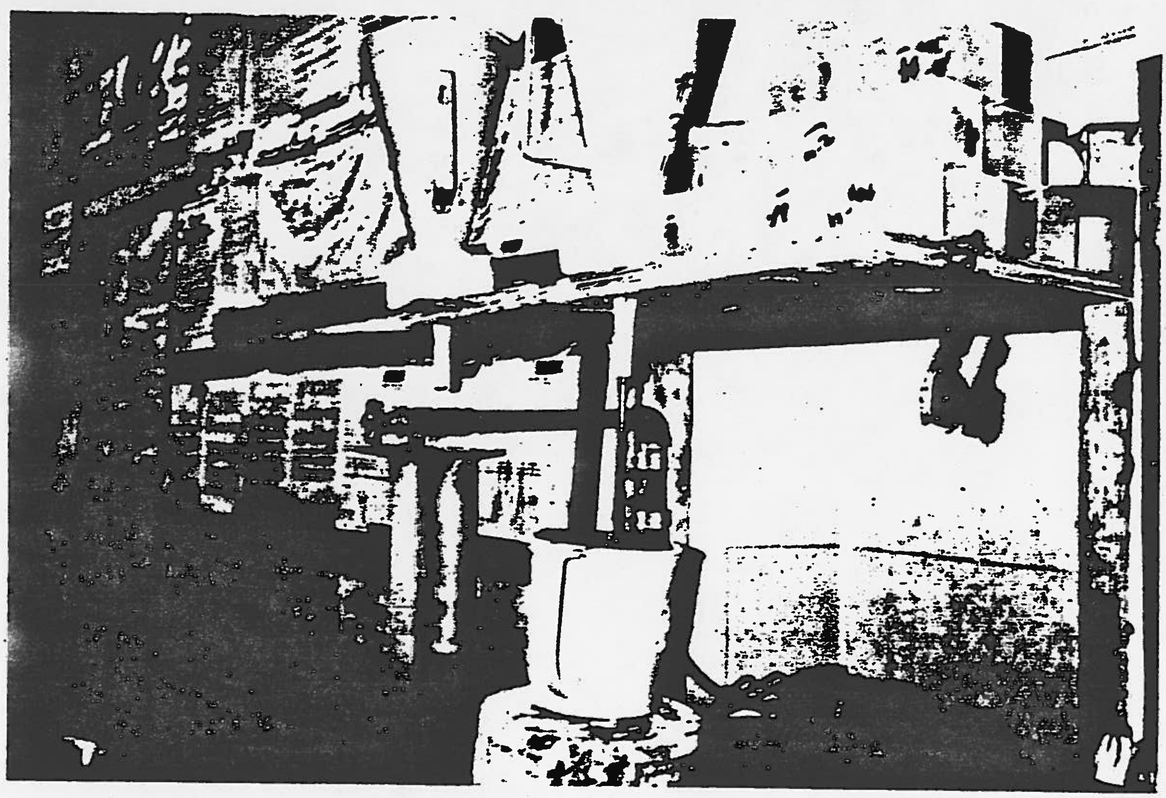


Figure 33. Viscosity testing

Fresh grout, cuttings, and hardened, in-situ soilcrete are all cement products generated during the jet grouting process. A comparative set of compression tests were run by Kauschinger, et.al, (1992) on all three materials. The results are contained in Table 6. Fresh grout had an average strength of 0.64 kg/mm^2, while the cuttings were slightly stronger (0.70 kg/mm^2). However, the in-situ column was much harder than the injected grout or cuttings. After seven days, the soilcrete had strength comparable to concrete (1.78 kg/mm^2), with an increase of 25 percent after 100 days of curing (Kauschinger, et.al., 1992).

will give an absolute worst case scenario (St. Simon, 1993c and Burke, et.al., 1989a). Four to eight waste samples should be taken per production shift, with an average expected strength of 9.89 MPa after seven days (Burke and Meffe, 1991). Unfortunately, this method is not foolproof. Kauschinger, et.al. (1992) report a difference of up to 700 percent in 100 day compressive strengths for two jet piles which had shown only a 30 percent strength difference in the fresh grout and cuttings. It is for this reason that field tests should also be made during construction to periodically verify the design (Munfakh, et.al., 1987). Although high strength is usually the desired out-

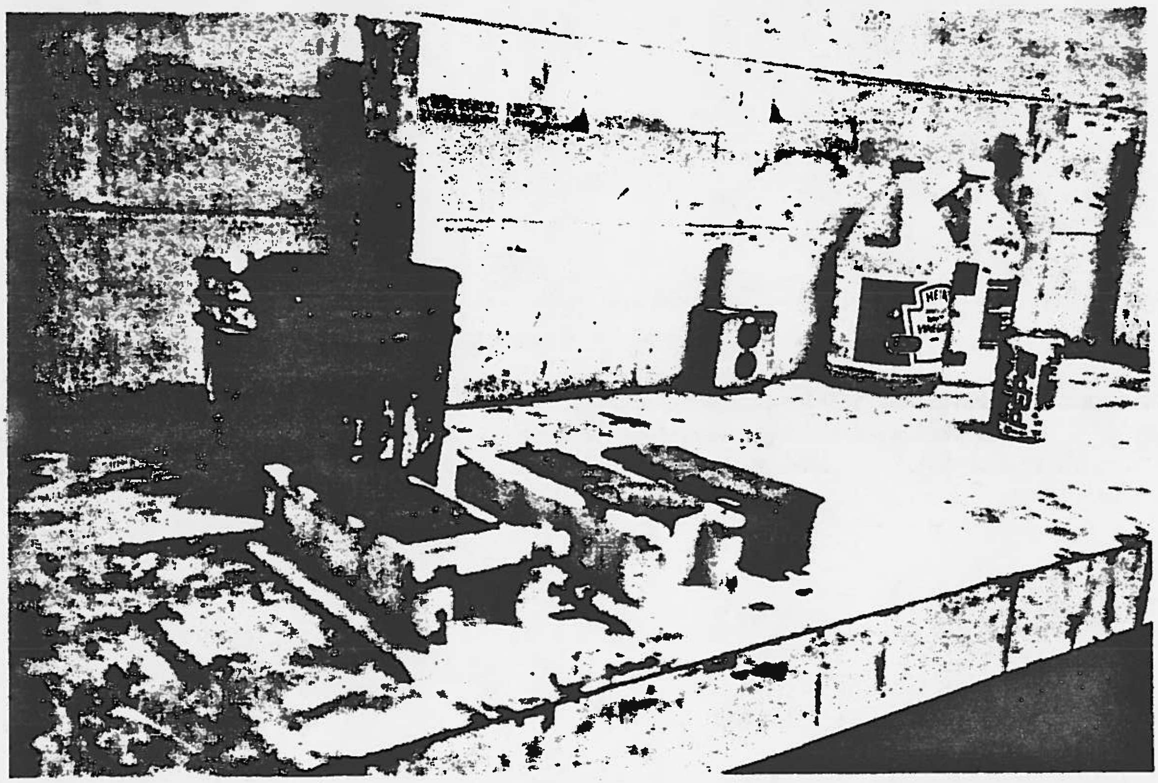


Figure 34. Grout gubes setting up prior to compression testing

Wet samples from the actual jet pile columns are poured into molds and cured in the lab (Burke and Brill, 1992) in a procedure similar to taking grout cubes. Compression results of 27.56 to 34.45 MPa are not uncommon. Miyasaka (1992) proposes a value as low as 1.22 MPa for non-structural elements. The other cement based product produced during installation is spoil which should be evaluated in terms of quantity and quality (Gallavresi, 1992). A 28-day compression strength of the spoil

come, a soil/grout mix can be too hard, which will hinder excavation; a reduction of the cement contentis usually the answer (Kauschinger, et.al, 1992).

The grout return can be seen as representing what is not being left in the ground. As such some describe it as a valuable monitoring tool (Perry, 1993) with a loosely inverse relationship between the diameter of the jet grout column and the cement content of the spoil (De Paoli, et.al., 1989). Alternatively, it may be seen as generally of representative the process, a kind of worse case scenario. Yet others (St. Simon, 1994b) contend that the grout strength is unimportant without knowledge of its location.

decrease in physical capacity (Kauschinger, et.al., 1992). Yet, jet grouting in clays tends to result in clumps of clay being incorporated within the soil-crete, which can create weak zones (Kauschinger,

Table 6. Unconfined compression strengths for soilcrete, cuttings, and fresh grout used on MARTA project (Kauschinger, et.al., 1992)

Column Identification	In-situ Soilcrete Days	q_u	Cuttings Days	q_u	Fresh Grout q_u	Injected Cement kg/m	Material Grout l/m
TC - 3	7	178	7	0.70	0.64	270	335
TC - 3	100	223	7	0.70	0.64	270	335
C - 2	100	62	7	0.51	0.45	210	320
C - 3	100	25	7	0.38	0.32	203	320

Notes: Age is in days; values of q_u are in kg/mm^2

To minimize some of the controversy sampling of the wet soilcrete is preferable and now possible. New testing equipment can take soilcrete samples at different depths within the column itself (Welsh, 1993b, and Burke and Brill, 1993). The new sampler is piston-based (Ryan, 1993). It provides a significant advancement in testing flexibility and accuracy. Instead of having to wait for the jet column to set-up before coring can occur (to prevent damage to the column), these internally taken samples can go though a whole battery of lab tests at 2, 3, 5, 7, 14 and 28 days (Welsh, 1993b). This type of testing combines the speed benefits of laboratory testing while more closely representing actual field conditions; it is not however perfect. The removed samples will not be as strong as those cured in the ground because the pressure, temperature, and moisture will not be as ideal for curing as what is below ground (Ryan, 1993).

Soilcrete can be tested in-situ by coring the treated material (Welsh, 1993b and GKN, 1993b).Coring, however, cannot be done until one is certain that the coring process will not disturb the pile's inherent set up strength (Welsh, 1993b). Parry-Davies, et.al. (1992) propose a 28-day strength test. If a problem is not discovered for an entire month, or if construction is held during this period, delays, and scheduling problems would inevitably result. Coring also has the disadvantage of being costly (Parry-Davies, et.al., 1992), and since it is in essence a destructive test, only a limited number of cores can be abstracted from a given area without causing a et.al., 1992a). This is caused by clay being difficult to pulverize (Winterkorn and Pamukcu, 1990). Clay-based deficiencies can only be discerned through coring.

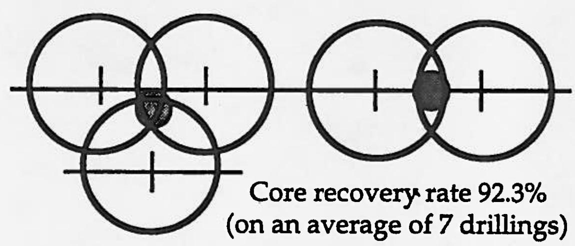


Figure 35. Sampling locations (after Miyasaka, et.al., 1992)

Coring. Capacity is usually measured in terms of unconfined compressive strength (Burke, et.al., 1989b). Starting seven days after the first group of columns are placed soilcrete coring can safely begin in order to verify quality, continuity, and strength characteristics (Andromalos and Gazaway, 1989). If core sampling is performed at the center of overlapping columns (Figure 35), the adhesion effects can establish the success of the process (Miyasaka, et.al., 1992) and the extent overlapping can be determined (Gazaway and Jaspers, 1992), as well as strength and permeability (Burke and Brill, 1993). Core boring is the most common test (Shibazaki and Ohta, 1982). It will show if the joints between the columns are visible (Steiner, et.al., 1992). If semi-continuous cores are not readily obtainable, the coring

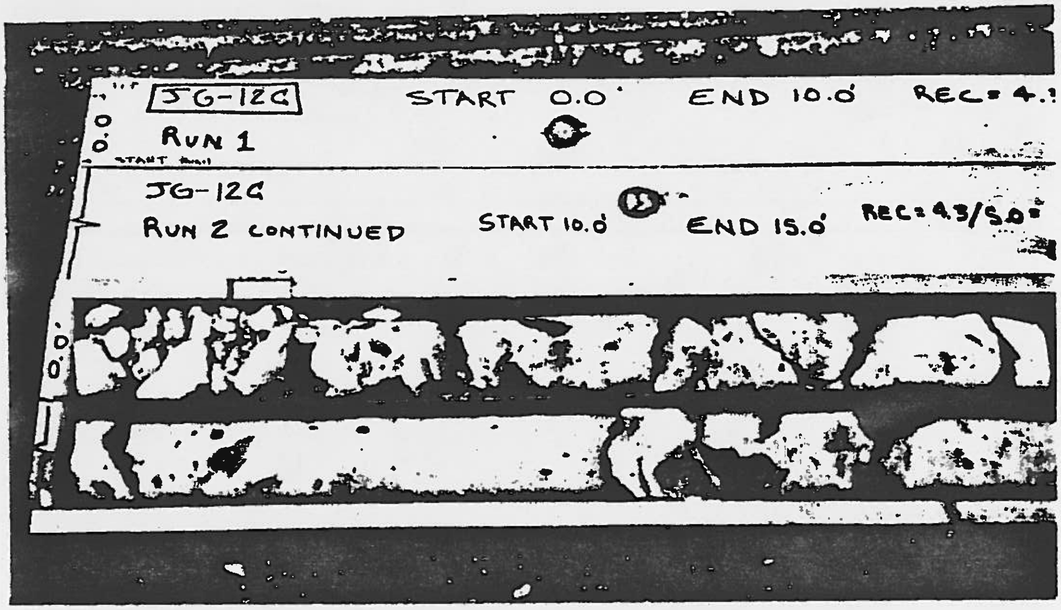


Figure 36. Jet pile cores (1)

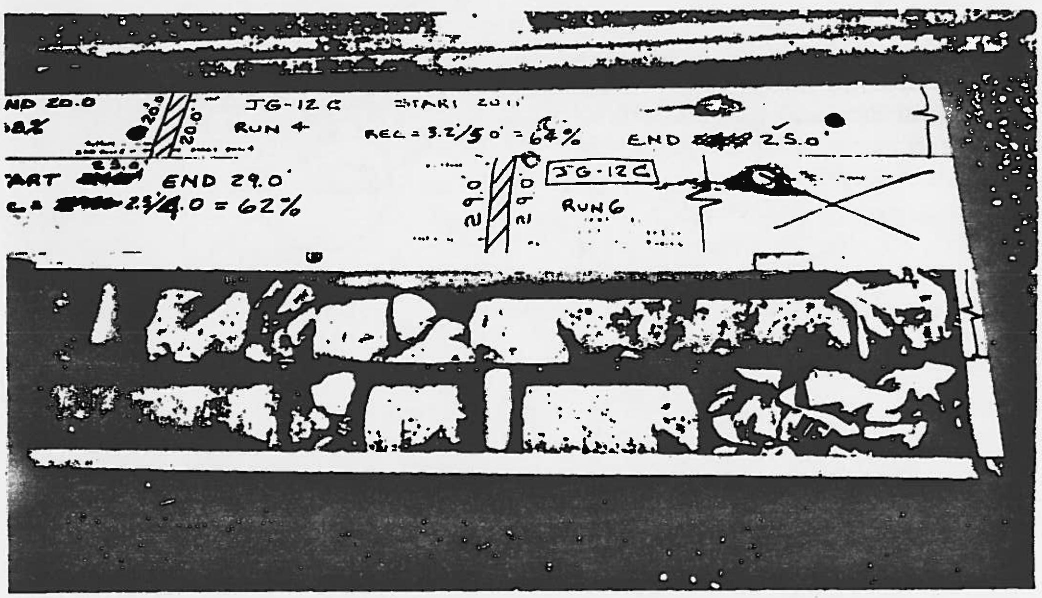


Figure 37. Jet pile cores (2)

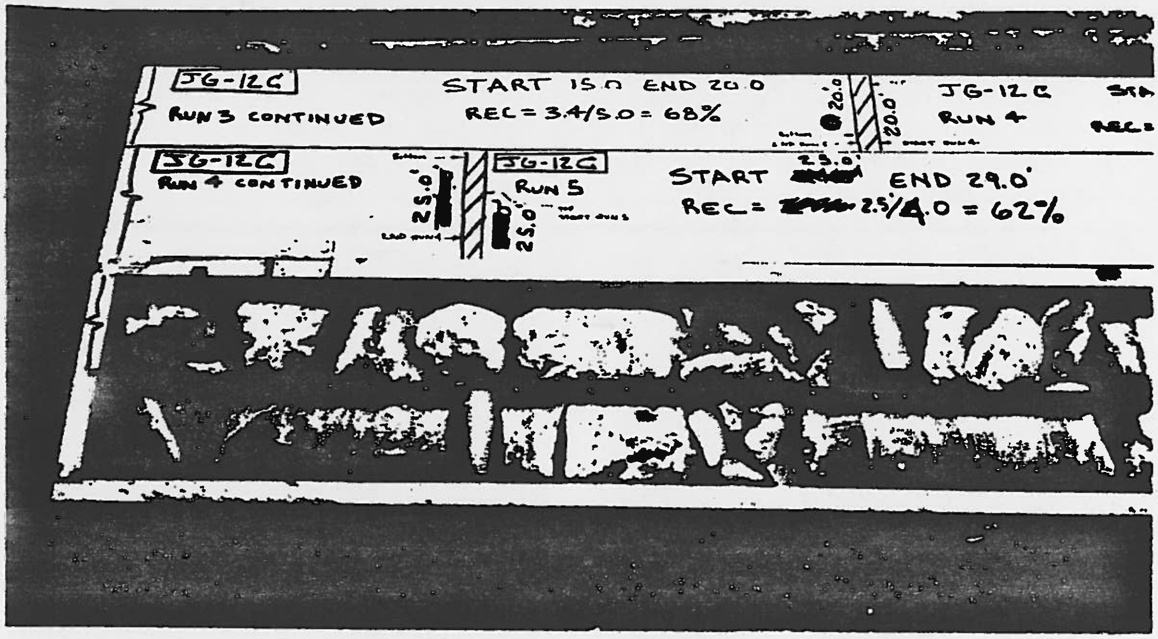


Figure 38. Jet pile cores (3)

method must be altered (Andromalos and Gazaway, 1989). Coring can be done at an inclination (Parry-Davies, et.al., 1992), and coring may also be done during the testing program (St. Simon, 1993c) in the horizontal direction [Figure 39]. Similar to borehole soil samples, variations through the the pile's depth or breadth are seen, depending upon how the sample is taken (Figures 36-38). The most preferred direction of coring is a matter of some debate. Horizontal coring is only possible when an excavation related scenario exists. It also establishes cores for an other than axial loading scenario. Conversely, vertical coring had the disadvantage the deviation may be occurring without any indication at the surface, thereby providing a false reading. Figure 39 depicts the two approaches.

Workability or Curing. In addition to strength characteristics, the grout must be workable and/or pumpable over extended distances, although there are other instances where quick set up time is a premium. Setting time for cement products is determined by the Vicat test and is measured as a function of strength in relation to time (Nonveiller, 1989), as previously seen in Figure 20.

Permeability. For some applications, permeability is important. Permeability testing can be performed in accordance with US Army Corps of Engineers EM-1110-2-1906. Anticipated results should yield permeabilities of less than 1 x 10^{-6} mm/sec. These results should be consistent with those obtained during the bench scale testing (Gazaway and Jaspers, 1992). A packer test may also be used to measure permeability (Micciche, 1993b). Alternatively, permeability can be confirmed according to ASTM D-5084 at 28 days (Delta, 1993). ASTM C39 and ASTM D2938 are also recommended tests (Hayward, 1991b).

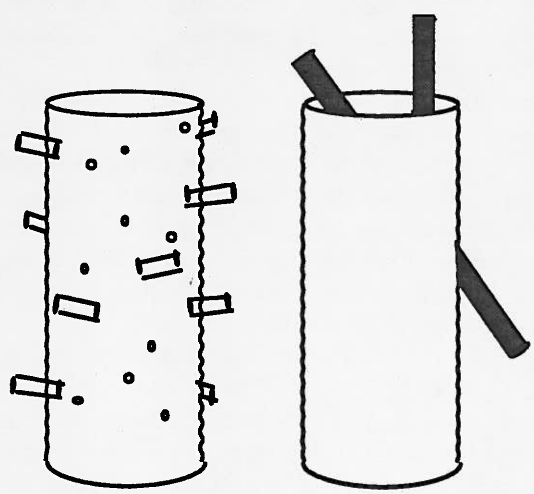


Figure 39. Different approaches to coring

Other tests. A cheap and important check is visual monitoring. Visual observations are useful in establishing the existence and nature of grout refusal (waste) [Gazaway and Jaspers, 1992 and Burke, et.al., 1989b] and changes in the system behavior, particularly in discerning the presence of utility lines (Gazaway and Jaspers, 1992). If there is a decrease in spoil, ground heave may be occurring. Miyasaka (1992) proposes a visual check of mixing at 85.4 percent for "visual core recovery rate". Recent electron microscopy tests on thin sections by Halliburton showed 30 to 40 percent of the original soil in the jet grout treated sections (Kauschinger, 1992c). The remaining 60 to 70 percent must be pushed out of the system in the spoil or into the ground heave.

Additional items to check for are pH, set times, and temperature [Winterkorn and Pamukcu, 1990]. A pocket penetrometer can be used to check initial and final set times (Bruce, 1993c). Temperature is an important, yet little discussed factor (St. Simon, 1993c and Nicholson, 1993c). Like concrete, the temperature of the mix and of the surroundings (soil and air) may have a strong impact on the hydration process and consequently, on the grout's ultimate strength and performance. The temperature of the constituent materials, particularly the water will greatly influence the grout temperature. The New York State Department of Transportation limits the placement of concrete to a 10.0 to 32.2 C° range, with the water having a temperature of 1.67 to 32.2 C°. Since all grout placement is beneath the ground, the allowable temperature range may be slightly greater. The minimum air content is listed at three percent.

A number of the Portland Cement Association's (PCA) "shortcut" tests for soil-cement are also used for soilcrete. These basically consist of sieve analysis, moisture-density tests, cement content, and compressive strength. Although there is quite a bit of similarity between soil-cement and soilcrete, they are not the same (Furth, 1994b). In general soil-cement only contains sufficient cement to produce a hard and durable construction material with only enough moisture to satisfy the hydration requirements of the cement and soil, as well as sufficient lubrication for the compaction of the mixture to a high density (Winterkorn and Pamukcu, 1990). Soilcrete is a fully integrated mixture of the various components. The more full-scale testing for soil-cement is not done for soilcrete, in large part because the tests require a 28-day curing period.

Control. There are a variety of devices which increase control over a grout based project.including grout pump regulators. Jefferies, et.al. (1982) outline a procedure for selecting transducers for localized grout monitoring. Specifically noted is that small signals are not compatible with long cables and electronic components are not compatible with mud and water (Jefferies, et.al., 1982). Hermetically sealing the transducers is the recommendation.

Alternatively, electromagnetic flow meters and digital pressure gauges may be used to better control grout volume. These can be set at a predetermined elevation or attached to an electronic warning system (Neely, 1991). Grout consumption is recorded in "sacks per foot" (Bruce, 1992b). The measurement of the rate of discharge and of the pressure gives a good picture of the grouting operations.

Record Keeping. Before any grouting begins, the condition of the equipment should be checked and the correct grout mix must be verified to ensure that it is the approved one (Munfakh, et.al., 1987). During the grouting operation, the following items should be recorded:

1. Mixer type.
2. Installation equipment.
3. Water/cement ratio.
4. Additives .
5. Grout pressure.
6. Cement type.
7. Free expansion..
8. Shrinkage..
9. Bleed.
10. Setting time.
11. Compression strength samples.
12. Volume of all grout takes.
13. Installation equipment.
14. Pile dimensions and locations.
15. Installation procedures.
16. Any difficulties or special circumstances encountered.

Automation. Presently there is an Italian developed instrumentation system which displays in real time numerically and graphically the full injection characteristics of each pump as a function of pressure and flow rate versus time. It also provides a printout summary of each grouted element (including volume, maximum and average pressures, flow rates and time) [Perry, 1993]. Jefferies, et.al., (1982) claims that the real time display of grout pressure versus flow rate is of great assistance because the pressure can be increased monotonically from a small value over a period of 2 or 3 minutes. Bruce (1988a) contends that such information is also valuable for more precise design and as a basis for payment. The system may offer the benefits of reduced manpower and decreased installation time through the optimization of ground pressure (Jefferies, et. al., 1992).

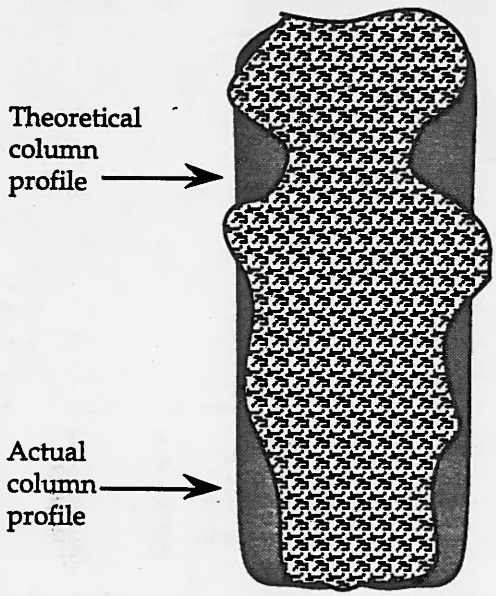


Figure 40. Uncertainty of grout placement

Location. Computer monitoring affords the benefit of ensuring consistent and repeatable grout placement (Welsh and Burke, 1991) and a check against hydraulic fracture. Yet, one of the largest disadvantages with the technology is that despite all of the computer feedback there is no actual assurance of where the grout is (Ryan, 1993), or any changes in the strength, the Young's modulus or the permeability (Marcuson, 1992). The grout take is known, but not the actual placement (Gallavresi, 1992) [Figure 40].

There are no direct methods to verify the size of each soilcrete column formed during production (Kauschinger, et.al., 1992a). Additionally, short of excavation, there is no easy way to judge a particular diameter at a particular level (Ryan, 1993) [fig. 4.3-21]. PVC markers placed at varying distances from the center of the column have been used with some success (Welsh, 1993b). PVC checks are done through sound (Steiner, et.al., 1992). This procedure is also useful if a full preproduction test cannot be run due to limited site access or sensitivity issues (Welsh, 1993b).

Another technique to verify diameter uses a large caliber log (umbrella) to be inserted into the liquid jet grouted column (Steiner, et.al., 1992). As an extreme measure, jet columns are redrilled to qualitatively assess grout continuity (GKN, 1993b).

Table 7. A standard set of testing parameters (De Paoli, et.al. (1989)

Water/cement ratio	0.7 - 0.85 (by weight)
Bulk density	1.424 - 1.487 kg/m3
Bleed	2 - 5 %
Marsh viscosity	32 - 37 sec.

Assessment. Tests that permit quality assessment prior to installation are more valuable since they permit immediate reaction in case of anomalies (Littlejohn, 1990 and Winterkorn and Pamukcu, 1990). Unfortunately, unlike other factors, compressive strength values may only be established 48 hours after the sampling (Winterkorn and Pamukcu, 1990). This is problematic for quick response. In-situ, grouting tests are beginning to be developed, but none are yet in use.

At a minimum the water/cement ratio, density, bleed, and viscosity should always be checked. De Paoli, et.al. (1989) offer a standard set of parameters in Table 7.

OTHER

In addition to grouting quality, general monitoring needs to occur. For this many standard geotechnical tools can be used. Gallavresi (1992) and Winterkorn and Pamukcu (1990) recommend inclinometers to check horizontal soil displacement, piezometers to record pore pressure build-up and dissipation, and datum points to check vertical soil displacements with reference to a fixed point.

Performance Evaluation. Given the wide variety of soil conditions and installation procedures it is virtually impossible to create a sufficiently accurate lab replication for jet grouting. It is for this reason that in-situ verification is critical. On-site acceptance testing may include proof loading, load transfer analysis and serviceability checks. Any performance criteria for both load capacity and allowable settlement should be pre-specified in the contract documents.

Conventional performance tests of cyclic loading, tensile pull-out, and creep tests are not usually done since they are not perceived to be a problem (Ichihashi, et.al., 1992).. Most of the verification is done through the quality control measures previously described. Not only is performance testing difficult and sometimes physically impossible, it is often considered unnecessary due to the high factor of safety built into the design process. This safety factor may be as much as five or six (St. Simon, 1993b) since the critical factor is often settlement and not load capacity. Although the jet grouting procedure is fairly new to the U.S., much information is available overseas both in terms of actual site monitoring and laboratory data (Micciche, 1993a).

Load Test. A load test systematically applies and records loads to foundation elements (Figure 41), while measuring and recording the corresponding foundation movements (O'Rourke and Kulhawy, 1985). It is primarily a measure of load-carrying capacity. "A load test permits the direct measurement of foundation resistance to loads affected by the actual construction procedures and soil conditions which prevail in the field. It provides a relationship between foundation forces and displacements so that design loads can be chosen for acceptable levels of deformation. A properly documented load test results in valuable reference data which can be used to improve analytical models and to clarify soil strength and stress parameters." (O'Rourke and Kulhawy, 1985). Load tests are achieved through reaction

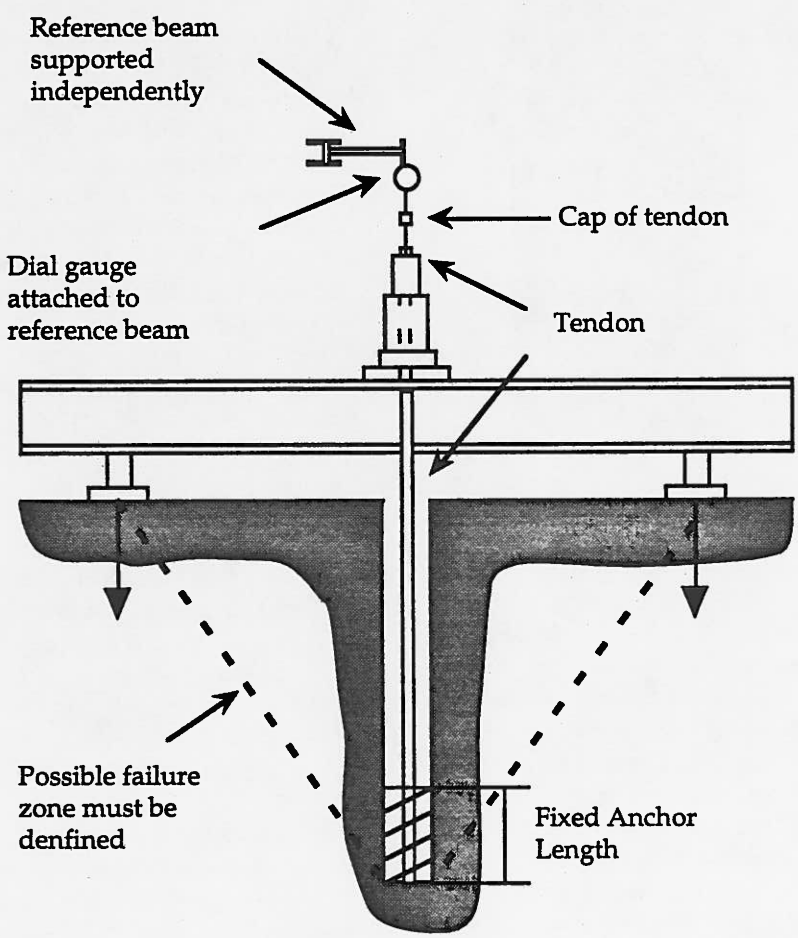


Figure 41. Remote loading of an anchorage (after Littlejohn, 1992 after ISRM, 1985)

1985). Unfortunately, this is not always possible. Grouted columns should may be rejected due to inadequate capacity, damage, improper location, misalignment, or failure to meet other installation criteria. When this occurs, design modifications must occur, and the new variants must then be tested. If the problem is determined to be the soil which cannot be easily replaced, load-bearing capacity must be downgraded or a more cement-intensive version of the jet grouting with additional cutting or additional columns needed.

The results from a test program or regular production may indicate a need for the modification of standard operating procedures or even of the specifications. This may included a reduction in the depths of the lifting steps. A truncated or inverted truncated cone (Figure 42) is rectified by initially rotating the grouting lance for a short period, without extraction, to enable the sand to be brought into suspension. Subsequently grout injection can commence (Parry-Davies, et.al., 1992). If the treated columns are not intimately locked into each other reduction of the center to center spacing will ameliorate the problem (Parry-Davies, et.al., 1992). Alternatively, the lifting speed can be reduced. Shibazaki and Ohta (1982) report a diameter increase of fifty percent with a lift speed reduction of an equivalent amount. The other remedial technique is to install an additional column adjoining the one already installed (Welsh, 1993b). By reducing the lifting speed by half, the grouted column was made larger than three meters in diameter.

Figure 42. Truncated and inverted truncated cones

In organics, clays, and other cohesive materials, the area is sometimes double cut (Figure 29) [Perry, 1993]. This means that the jet column experiences removal and replacement, and then the procedure is repeated. This serves to remove significantly more fines than if the process was only done once (Welsh, 1993b). As a modification, the jetting rods may be lifted in steps of 12.5 mm instead of 25 mm of standard, thus aiming at a more uniform section (Miyasaka, et.al., 1992).

If "overbreak" of the soil mass resulting due to construction activities, traffic, or poor mixing at the surface is discovered, the grout at the surface can be manually removed and replaced with a low permeability fill to maintain the integrity of the completed unit (Gazaway and Jaspers, 1992). Where permeability is the foremost objective, an increased amount of bentonite will solve the problem but at the loss of some strength (Perry, 1993). In the case of hydraulic fracture, a decrease in the pressure will help (Jefferies, et.al., 1982).

PROCEDURAL IMPACT

With most projects there is a strong need to minimize the visual and other environmental impacts during and after construction (Ford and Chartres, 1992).

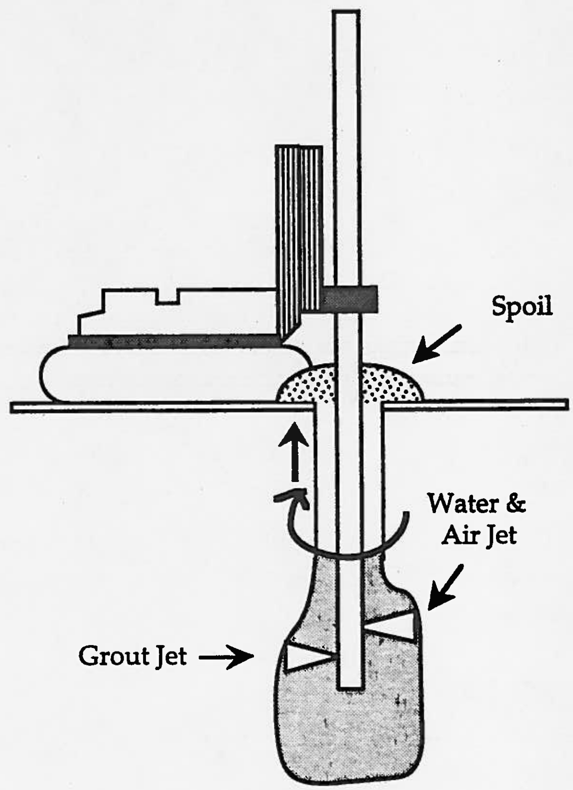


Figure 43. Jet grouting operation using triple fluid system (after Bruce, 1988a).

BUILT ENVIRONMENT

Grout-based procedures are messy operations that can cause heave of adjacent structures and utilities. Consequently, protection of the built environment, particularly adjoining facilities is important (Munfakh, et.al., 1987). Ground move-ment may be caused by settlement as well as heave (Burke and Meffe, 1991), and encroachment on future permanent structures or right-of-way limits must also be avoided (Central, 1991).

SPOIL

The other major area of impact is spoil handling and disposal. Prior to commencement of work, a plan should be submitted to measure and manage grout flow (GKN, 1993a). Collection, transportation, storage, treatment, and disposal of spoil need to be addressed (Delta, 1993). With up to fifty percent of the mixed grout becoming waste product in need of disposal (Burke, et.al., 1989a and Kauschinger, et.al., 1992b), the issue of spoil may be the greatest limitation to this technology (Figures 43 and 44). Although not an insurmountable issue, environmentally acceptable, and economically viable disposal must be adequately preplanned (GKN, 1993a and Delta, 1994c).

Kauschinger, et. al. (1992) provide design aids in estimating the spoil handling required. They propose to measure the amount of cuttings ejected from the borehole during several trial fields and production work. The percent outflow of cuttings is the ratio of outflow to grout volume injected. Comparative tests amongst different rod system resulted in 50 percent spoil for single fluid systems in contrast to over 60 percent for a triple system (Kauschinger, et.al, 1992). The majority of the single and triple fluid data show a strong tendency to cluster around 0.20 to 0.40 m^3 of spoil generated per cubic meter of soilcrete formed (Kauschinger, et.al., 1992).

All return flow should be collected at the hole collar or point of entry of drill or grout rods into the ground. Grout discharge should be prevented from flowing down the side of or ponding in the bottom of the excavation (Attwood, 1987) or into storm drains (Hayward, 1991a). Unless a specified purposed can be designated for the low strength capacity of the spoil, the material should not be disposed of on site (Perry, 1993a and NYSDOT). Proper planning permits the utilization of the cement rich spoil for items which need low strength concrete (Kauschinger, et. al., 1992 and GKN, 1993a) such as walls for additional lateral restraint (Flick, et.al., 1992). Where use of the spoil is possible, overall cost of the project will be lower (Kauschinger, et.al., 1992). Unfortunately, this is not foolproof. In a recent case at Yale University, the spoil was deposited in an area which was later designated for excavation (Perry, 1993a). The relatively small amount of money saved by avoiding hauling and disposal charges was more than lost in the subsequent excavation of the newly deposited concrete mass when it had to be removed for an unrelated building project.

Disposal must be in accordance with all federal, state and local codes, rules, and regulations, and permits, as well as being in accordance with

Figure 44. Jet grout spoil

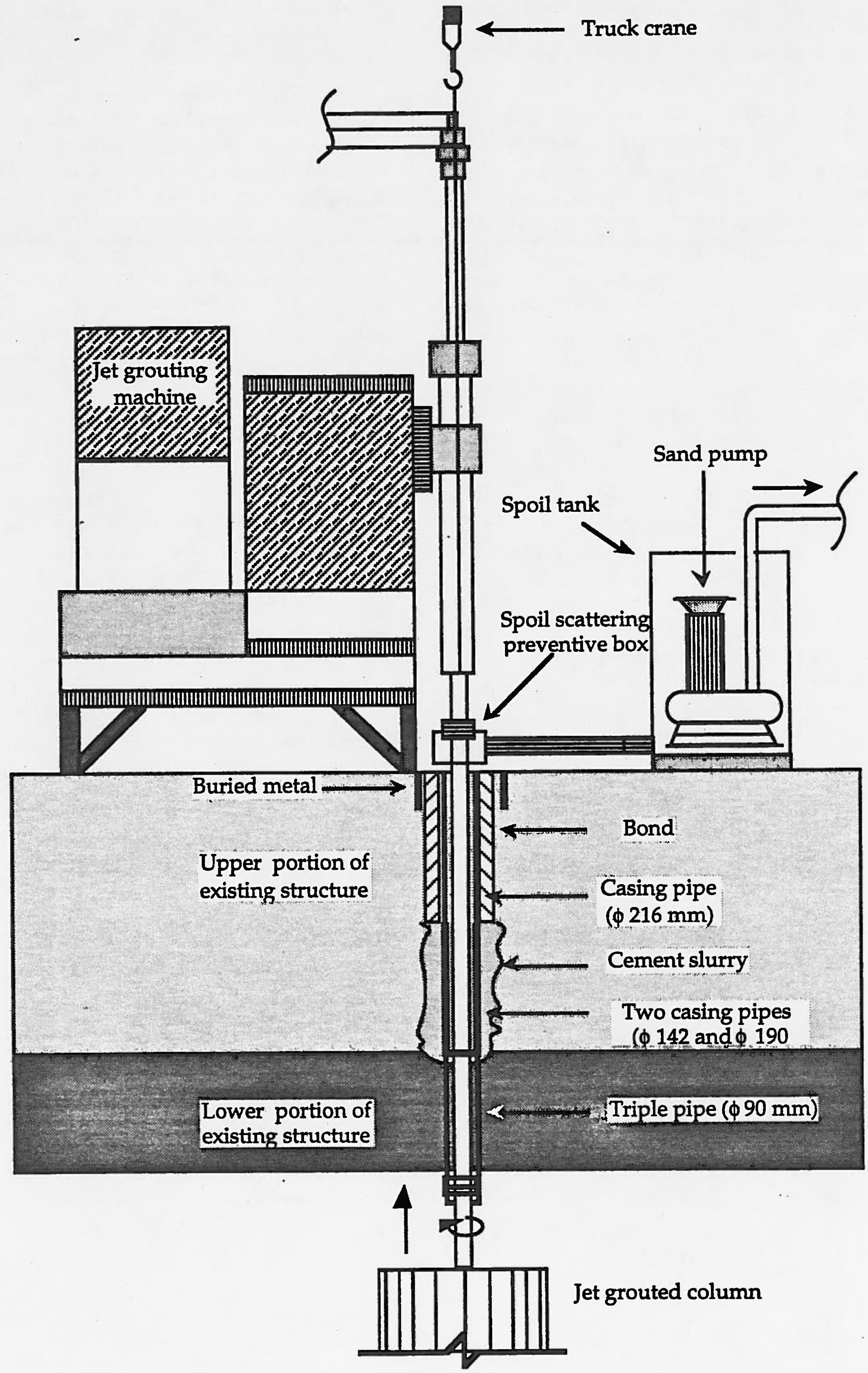


Figure 45. Guide pipe and recovery of spoil (after Ichihashi, et.al., 1992)

provisions of the contract specifications. To prevent the spoil from being discharged onto the ground during drilling, a tank can be attached to the drill hole to create a fully closed circuit (Ichihashi, et. al., 1992) [Figure 45]. As final notes, if grouting is carried out through a pavement, extensive patching or replacement may be required (Munfakh, et.al., 1987), and as-built drawings show-ing the location of each jet-grouted mass, its length and orientation should be provided upon completion of the project (Hayward, 1991c).

CONCLUSION

There currently exists a more than adequate set of quality related practices for jet grouting, both in terms of installation and testing. Unfortunately they are neither well known nor well established. Following this loosely held set of guidelines would represent a major improvement to jet grouting reliability throughout the industry. With a strong commitment to a quality installation on the part of the contractor and a knowledgeable inspection staff on the part of the owner to oversee quality control processes, jet grouting will continue to grow as an innovative solution to many deep foundation and environmental problems.

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ACKNOWLEDGEMENTS

This paper was made possible through a research grant by Dames and Moore through the American Society of Civil Engineers. The author would also like to recognize and thank Joe Welsh, Allen Furth, and Joe Perry of Hayward Baker, Romano Micciche of Delta Drilling, Patrick St. Simon of G.I.T., Donald Bruce of Nicholson Construction, Joseph Kauschinger of Ground Engineering Services, Chris Ryan of GeoCon, and Bernard Grand of Slattery for their contributions.

PILE DRIVING - AN INTERNATIONAL STATE-OF-THE-ART

G. G. Goble[1]

A. Introduction

Over the past 40 years there has been a continual evolution of pile driving technology so that today as a result of this development the industry has been radically changed. These changes have occurred because of an evolution in piles, driving equipment - particularly hammers, and the engineering of the entire operation. Pile driving hammers have become larger, at least partially due to developments in offshore pile driving. Piles and pile capacities have also become larger and, in addition, changes have been made to take advantage of high strength pile material and new concepts for pile types and installation procedures. Finally, analytical studies combined with the ability to make accurate computer simulations of pile driving operations and to make measurements of the event has supported the development of a general understanding of the pile driving process. This capability opens the door to orderly, further developments in the area. In this paper, these developments will be reviewed and summarized and projections will be made for further developments. North American practice will be emphasized but an effort will be made to discuss current practice from an international point of view.

B. Background

At the end of World War II the dominant hammer type in the United States was the air/steam powered hammer of the type developed by the Vulcan Iron Works. At that time there were other manufacturers of similar machines in addition to the Vulcan Company. This hammer had a stocky ram and the motive fluid driving the machine was either compressed air or steam, but probably most commonly steam. The motive fluid lifted the ram and then it was allowed to fall free on the downstroke. These machines had a stroke of 0.9-1.0 meters and delivered about 60 hammer blows per minute. Double-acting hammers of a similar type were also available and also powered by compressed air or steam. In this context, the reference to "double acting" refers to the general class of hammers in which the ram is powered on the downstroke as well as the upstroke. The double acting hammer was attractive to contractors because it operated at a higher speed, 100-140 blows per minute, than the single acting hammer.

Single acting air/steam hammers were rated by the manufacturers based on the potential energy in the ram at the top of the stroke and they were considered to operate at a constant stroke. The procedures used to rate hammers were quite direct, were

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generally accepted by engineers and rated energy became the standard approach used to specify hammers. Double-acting hammers could also be rated in a similar way if the downward acting force is taken into account.

At this time, drop hammers were still widely used in the United States. Abroad their use was even more prevalent. One can hypothesize as to why the use of the powered hammer was more widespread in the United States than abroad. Surely, one reason was the effect on union work rules in the United States. Here, in the industrial parts of the country, a pile crew of six or seven was generally a union requirement. As labor became more expensive it was natural for the contractor to seek higher productivity by using larger equipment and, to some extent, larger piles. The Raymond Company was a major factor in the American market. They brought in large equipment with many innovative changes and in the period after the end of the war they were dominant in pile construction in the United States and to a lesser extent abroad. In the United States, they operated throughout most of the country.

During this time the skid rig began to disappear and was replaced by large cranes with fixed leads and steam generators mounted on the rear. This came to represent pile driving on large jobs in the United States.

The most dominant pile types at this time was the timber pile and the steel H-pile. In addition, some proprietary types such as the Raymond Step Taper concrete filled, mandrel driven pile were common, particularly on large jobs. The use of reinforced concrete piles that were usually precast on the job site occurred in the United States and was more widespread abroad. Steel pipes were also used and they were usually filled with concrete after driving.

Pile capacities were generally low. Timber piles were commonly loaded to 20 tons working load in a practice dominated by the New York City area where their use was very common. For other pile types, a design load of 50 - 60 tons was common and would have been considered to be reasonably high.

The engineering of pile foundations was almost completely an art at this time. Efforts had been made to predict pile capacity from subsurface information but the results were not encouraging. In fact, the engineering of pile foundations is generally credited with motivating the development of the Standard Penetration Test (SPT), (Fletcher 1965). Again the influence of a predecessor company to the Raymond Company was important. In the immediate post war years, soil mechanics offered explanations for phenomona observed during pile driving and the hope that sometime it may be possible to reliably predict pile capacity from subsurface information. However, at that time, pile capacity was generally determined at the job site from one of the many dynamic formula that were commonly used. This describes conditions that existed in other parts of the world. In the United States, the Engineering News formula or some minor modifications was the most common.

C. Current State-of-the-Art and Its Development

I. Engineering

a. Static Performance Prediction

In the years since the Second World War an immense effort has been devoted to the development and proof of methods that would reliably predict static pile capacity from subsurface investigation information. A large number of methods have been developed and tested but A widely accepted solution has not been found. A minimally complete discussion of this topic is not within the scope of this paper.

b. Performance Prediction by Dynamic Methods

1. Discrete Dynamic Model

Three major applications of dynamic methods affected the engineering of pile driving practice during the period under consideration. First, and probably most important, was the development of discrete Wave Equation analysis by E. A. L. Smith of the Raymond Company (a computer simulation). Second, analytical work on the closed form solution of the wave equation (the second order, partial, differential equation) with particular reference to pile driving applications, produced results that provided explanations for what happens during pile driving impact. Third, developments in dynamic field instrumentation made it possible to obtain force and velocity measurements of pile driving impacts and process them for use in support of the analytical procedures described in the second case above and, further with the quantitative results from the discrete Wave Equation, to directly generate soil resistance characteristics. These three developments will be examined in greater detail.

The Wave Equation method of analysis was developed from initial concept all the way to practical application by Smith (1951, 1957, 1960) in a remarkable achievement. This solution may have been the first application of the digital computer to a civilian engineering problem (communication with Schiffman (1988)). The model that he used is shown in Figure 1.

It is appropriate to discuss the computational procedure that Smith developed to make use of this concept. When the program is executed it is necessary to provide the various input quantities to describe the hammer, pile and the driving system. Most of these quantities are necessary to describe mechanical characteristics and they can be determined quite directly using appropriate tests.

In dealing with the soil, Smith understood that the problem is more difficult. He selected an elastic-plastic spring and a linear dashpot as the soil representation as shown in Figure 1. Thus, the soil resistance to penetration is defined as a function of displacement and velocity only. To satisfy the needs of this model, three "soil" constants must be supplied. Smith suggested values for two of these constants, the displacement where the static resistance changes from elastic to plastic (named the Quake, q) and the damping constant. In the computational procedure, he left the total capacity to be determined and only required that the static soil resistance distribution be provided. Total capacity is then incremented in some orderly fashion to generate a curve, called a bearing graph, that relates capacity to blow count (the inverse of penetration). Thus, it was possible to observe blow count in the field and obtain a prediction of capacity, or the opposite, to select the blow count required to obtain the required capacity. With this computational procedure the pile driving process becomes sort of a soil investigation technique.

When viewed from today it seems more logical to enter all of the soil variables as well as the pile and driving system quantities and then generate a predicted driving record as a function of depth of penetration. This approach to evaluation has become known as a driveability analysis. Had Smith taken that route it is likely that his work would not have been as well-received since our lack of knowledge of soil properties would have been much more obvious.

The above description has only mentioned the treatment of the velocity dependent portion of the soil resistance in a very limited way. The Smith definition of dynamic soil resistance (damping) is more complex than is described here. A more complete discussion of this topic is contained in another paper in this conference (Rausche et al, 1994).

Today Wave Equation analysis is standard in North America having almost completely displaced the dynamic formula. In other parts of the world, its use is growing. In offshore applications, Wave Equation analysis has been widely used for several years and today is used almost universally both in verifying hammer selection and also in estimating pile capacity.

The first generally available Wave Equation program was developed at Texas A and M University with support from the Texas DOT and The Federal Highway Administration (FHWA) and has been known as the TTI Program (after the Texas Transportation Institute)(Samson et al 1963, Hirsch et al 1976). This program became available in the early 60's and has been widely used.

In 1974, the FHWA sponsored the development of the WEAP Program to deal with problems that had been observed with the analysis of driving with diesel hammers (Goble and Rausche 1976). Development of this program has continued and a large user base has been generated (Goble and Rausche 1981, Goble and Rausche 1986, GRLWEAP 1991). Work was also done in this area by Rempe (1975).

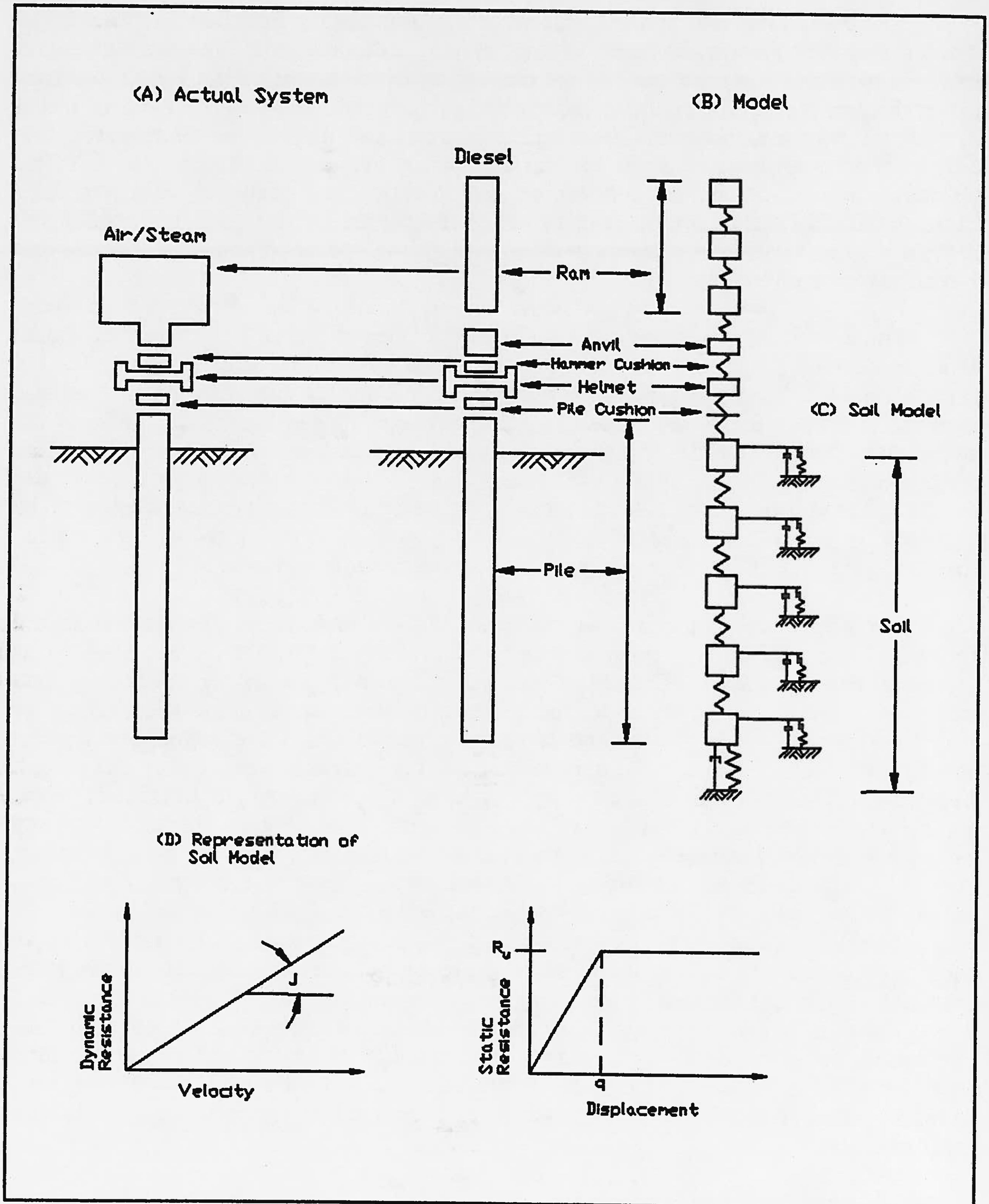
(A) Actual System
(B) Model
Diesel
Air/Steam
Ram
Anvil
Hammer Cushion
Helmet
Pile Cushion
(C) Soil Model
Pile
Soil
(D) Representation of Soil Model
J
Dynamic Resistance
Velocity
R_u
Static Resistance
q
Displacement

Figure 1

The parameters that must be entered to perform a Wave Equation analysis can be divided into four groups, hammer, driving system, pile, and soil. The most extensive work on hammers has been done in the continuing development of the WEAP program (several references). The resulting models and performance parameters have been tested extensively with measurements, both on the pile and also directly on the hammer. The only hammer performance variable that cannot be well defined today (1994) is the efficiency which, of course, depends on the operation and condition of a particular hammer. This cannot be predicted in advance but there are hammers available today that contain instrumentation to measure performance during operation and have controls that can adjust the hammer operation.

The driving system parameters relate to the hammer and pile cushion stiffnesses. Hammer cushion parameters have been well-defined for manufactured materials and the parameters have been extensively tested against pile driving measurements. For pile cushions (on concrete piles), plywood or other wood loaded across the grain is the dominant cushion material. At the begining of driving, both the stiffness properties and the thickness will change rapidly and these changes should be better quantified for use by the practicioner. The force-deformation unloading characteristics must also be provided by the Wave Equation user and this parameter is more difficult to define. However, the results are not very sensitive to variations in its values.

The pile model as originally proposed by Smith consisted of discrete masses and springs. This model gives good results when properly used in most applications as proven in extensive tests. Care must be used in selecting the proper integration time increment but that can be accomplished quite successfully on an automatic basis. For long piles, errors are generated and sometimes the front of wave reflections are not handled very well. However, the hammer and driving system fit very well in this model. Another approach has been suggested by a group of Dutch engineers. In this approach, the pile is modeled as a series of uniform cross section, continuous elements with any cross section changes concentrated at the element boundaries. Since the wave is known to propagate unchanged in a uniform rod, and since the transmitted and reflected forces can be determined at the element boundaries the wave propagation calculation becomes only a bookkeeping process. This approach to wave propagation analysis will give an excellent representation of the wave, again as proven by measurements. However, rules for hammer and driving system modeling have not been developed for the continuous model. The use of a hammer "signature" force-time record has been proposed but it does not include the effect of the reflected wave on the driving system and hence, the total input wave. For diesel hammers, the stroke is dependent on the soil resistance and, therefore, the results obtained by using a force-time input will be particularly unsatisfactory.

The inclusion of residual stresses in the driving analysis can be considered as part of the pile modeling process. This problem was first analyzed by Holloway in an examination of the critical depth concept. A modified approach that emphasized the affect of residual stresses on driveability was presented by Hery and this tool has been available for some time in the WEAP program. It can be shown that residual stresses are important in many driveability evaluations but the method has been used only infrequently in land applications. Engineers involved in driveability predictions for flexible (steel) piles driven with large shaft resistances should gain experience with a residual stress analysis. It will probably improve the accuracy of the analysis and its inclusion in driveability analyses will often produce important differences in the results.

The remaining topic for review is the soil model. As may be expected this topic is certainly the most difficult. In summary, one can conclude that there has been little actual change in practice since the initial work of Smith. His model contains three constants; ultimate resistance (or resistance distribution), quake and the damping constant (see Figure 1). Ultimate resistance remains difficult to evaluate and as indicated above the topic is beyond the scope of this paper. A very large literature exists on this topic and it cannot be discussed here. However, as was noted above, total soil capacity can be obtained from the Wave Equation analysis if a resistance distribution is entered, a bearing graph calculated, and the observed blow count used to determine the total pile capacity. The bearing graph can be shown to be quite insensitive to the distribution of resistance.

In his original work, Smith recommended that a value of 2.5 mm be used for the quake in all cases. With the availability of measurements and signal matching techniques cases were found (Likins 1983) where different values had to be used to represent field observations. After extensive experience, it can be concluded that the 2.5 mm value is satisfactory for all shaft quakes. The toe quake can be estimated as a function of the pile toe diameter from the expression D/120, where D is the effective pile toe diameter. There are some cases where toe quakes larger than this value are observed and this phenomonon is apparently a soil property. Fortunately, large quake not associated with pile toe diameter is only infrequently observed.

The most serious soil modeling problems are associated with the last of the three parameters, the dynamic resistance. These values are important since modest changes can produce a substantial effect on the results. Coyle and others in the research team at Texas A and M have shown, quite conclusively in exprimental studies, that the dynamic resistance is not linearly related to the velocity and this work has been confirmed by others. The question is examined in detail in a paper by Rausche et al (1994) and will not be discussed further here. The work of Coyle is very important in applications other than Wave Equation analysis. In any case of soil penetration, the non-linear soil response causes large dynamic resistances at low velocities.

A more general question regarding the treatment of the soil has been the suggestion of several researches that the soil model should be substantially changed. This suggestion has also been examined in the paper by Rausche et al (1994) and will not be discussed further here. Certainly the matter of a proper soil model has not been settled at this time.

2. Continuous Dynamic Model

During the time that the discrete pile model described above was developing in the United States another approach was under study in Europe. In this approach, the solution of the one dimensional wave equation was used directly to investigate the pile driving problem. The earliest and most extensive work was done by Fischer at Uppsula University. It has been published extensively. Fisher used the wave equation solution to develop graphical methods that were practical and useful in explaining many pile driving phenomona. This work influenced many of the applications that have been made since the First Stress Wave Conference in 1980.

Some simple results of one dimensional wave mechanics can be applied to understand pile driving. First, there is a proportionality between particle velocity and force in a one dimensional wave traveling along a pile. This relationship can be stated as

$$F=\frac{EA}{c}v$$

where E is the modulus of elasticity of the pile material, A is the cross sectional area of the pile, and c is the velocity of particle motion in the pile during the passage of a wave. Thus, the peak force is directly related to the impact velocity of the ram. This force may be modified by the affect of the cushion and the helmet. The cushion will have the affect of reducing the peak force and delaying it in time and the more flexible the cushion the greater the force reduction and delay. The helmet mass will have the same effect as the cushion and again as the mass is increased the peak force is reduced and delayed.

The affect of the magnitude of the ram mass is to modify the rate of decay of the impact force. This decay rate is related to the ratio of the pile impedance to the ram mass as shown in Figure 2. Thus, it can be seen that if the pile impedance (The ratio of the pile mass to the ram mass can also be used if an appropriate correction is made for the differences in elastic modulus and wave speed.) is large compared to the ram weight the decay after impact will be rapid. On the otherhand, if the ram is heavy compared to the pile impedance and if a high toe force reflects a large compression force, this compression force will arrive back at the pile top while the hammer is still introducing force into the pile. When this force is reflected at the ram together with that due to the remaining ram velocity a force larger than the initial impact will be generated that will induce penetration of the pile in the second wave transmission. This case can only happen if the ram weight is very large compared to the pile. Therefore, it is concluded that the mobilization of higher resistance forces and penetration in hard driving can be most effectively accomplished with a higher impact velocity than with a heavier ram. Of course, there are limits to this concept since a very light ram would have a very short period of high force and this could further be strongly affected by the cushion and helmet.

If it is assumed, that the particle velocity in the pile is usually strongly affected by the ram impact velocity, then it is possible to make some estimates of desirable hammer design characteristics.

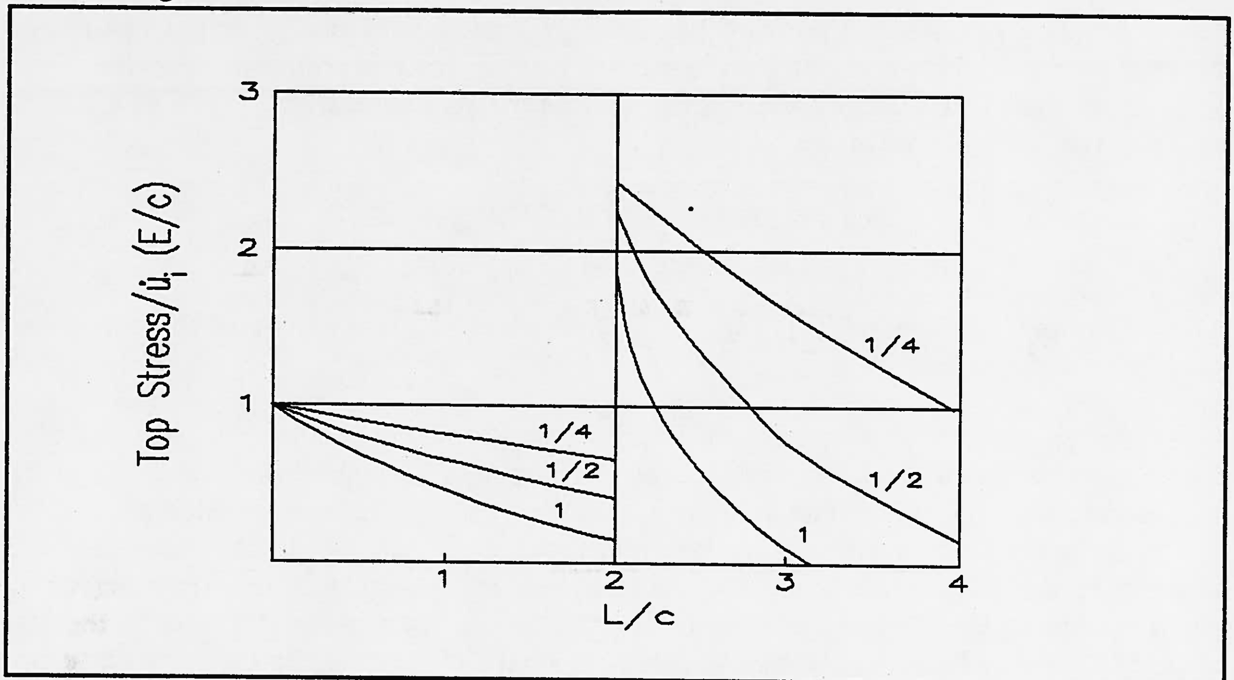

Figure 2

3. Dynamic Monitoring

Another advancement that had a large influence on piling practice was the development of the technology for making routine measurements of force and acceleration under the pile driving hammer. The first measurements of the stress wave during pile driving were made at the British Building Research Station in 1938. In the early 1960's, the Michigan Highway Department performed an extensive testing program on pile driving with the purpose of verifying and evaluating pile driving hammer performance. In this program, they made the first measurements of acceleration during pile driving. These measurements made it possible to calculate the energy delivered to the pile top. Between 1964 and 1976, The Ohio Department of Transportation and the Federal Highway Administration conducted a research project at Case Western Reserve University that emphasized force and acceleration under the pile driving hammer. A large volume of measurements were collected on piles that were also load tested.

The routine processing of these measurements can yield a great deal of information on the pile driving operation. Quantities such as maximum force, maximum delivered energy, bending stresses, maximum reflected tension, energy delivered to the measurement point, etc. can be obtained and displayed in real time during the driving operation. This information can be very useful in monitoring and controlling the pile driving and in detecting problems.

A large volume of literature has been generated with the principal effort being devoted to the development of simple methods of capacity prediction. For the case of piles of uniform cross section the total force that resists penetration, given as a function of time, has been shown to be

$$R_T(t)=\frac{1}{2}[F(t)+F(t+2\frac{L}{c})]+\frac{EA}{2c}[v(t)-v(t+2\frac{L}{c})]$$

where L is the pile length, c is the velocity of wave propagation, and the other terms have been defined above.

An example of a measurement is shown in Figure 3 together with the calculated total resistance force. The method of calculation of the resistance force was derived from consideration of one dimensional wave mechanics. Only the most limited assumptions had to be made (uniform cross section, modulus, mass density, linear behavior, etc.). Therefore, this expression gives the value of the resistance to penetration active at a given instant in time. The question of interest to the foundation engineer is the static capacity. Two questions must be answered; at what time should the capacity be selected from the available time dependent values given and how should this result be modified to deal with the fact that the result is velocity dependent. These problems have been studied extensively. In general, the time selected usually starts at the point of maximum impact force although in some cases a later time is used if a larger displacement is required to mobilize the resistance force (large quake).

The determination of the velocity dependent component of the resistance to be removed from the total resistance has proved to be a difficult problem. In early work, it was assumed that the total resistance at the time of zero velocity represented the static resistance. Later, the dynamic resistance was assumed to be proportional to the toe velocity. Data from cases where both dynamic measurements and static load test results were available were used to generate values of a "damping constant". Those values were found to be dependent on the soil type at the toe. A number of other methods have been proposed and found to work well in particular cases. This problem is not a closed topic today.

A more fundamental approach to the determination of static capacity from dynamic measurements, known as CAPWAP (CAse Pile Wave Analysis Program) was developed by Rausche. This signal matching approach is described in Figure 4. The pile is modeled by a series of continuous discrete elements and the soil by the Smith Model.

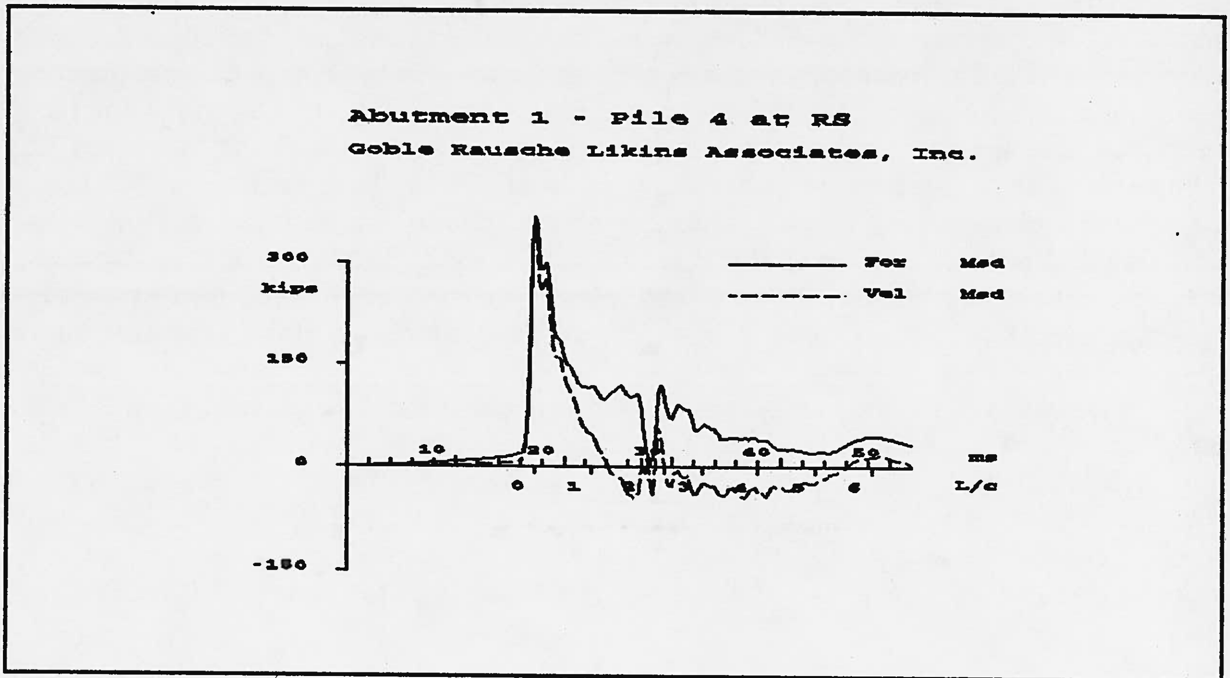

Figure 3

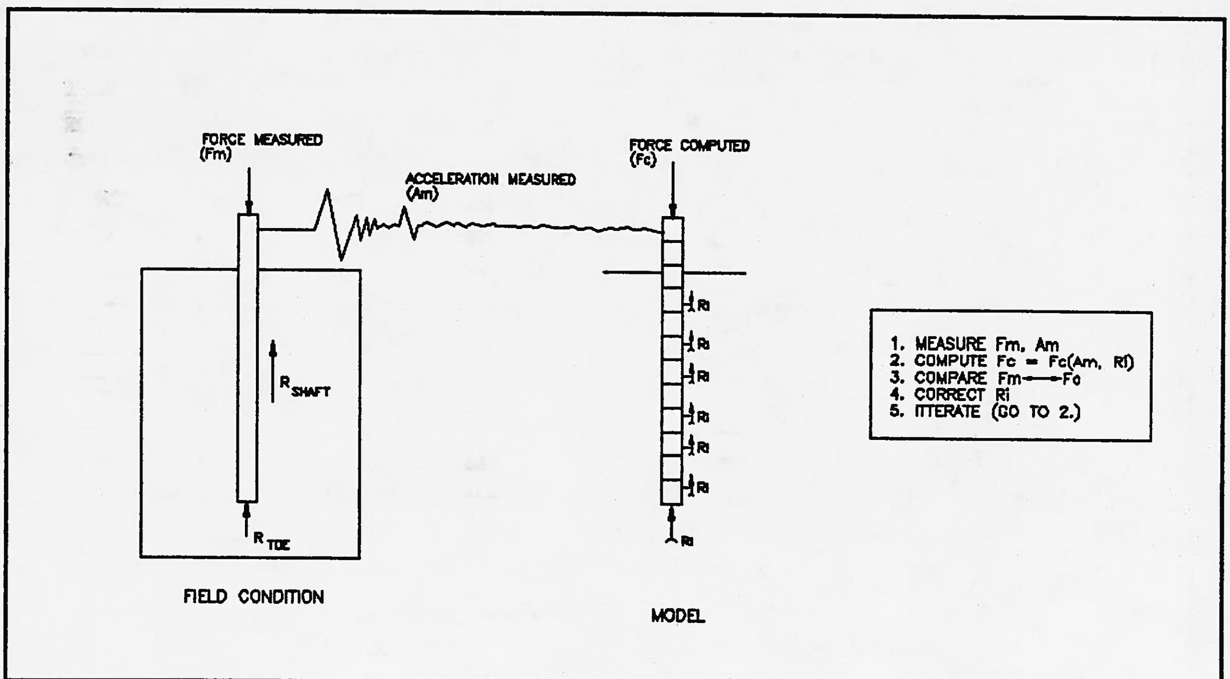

Figure 4

Modifications have been made in the standard Smith model to allow different loading and unloading stiffnesses, different loading and unloading capacities, radiation damping, a gap in the toe soil resistance, and a soil mass at the toe. In the computational procedure, the measured velocity is induced at the pile top, a set of values is assumed for the soil elements, and the force at the top is calculated and compared with the measured force. The values for the soil elements are adjusted to obtain the best possible match between calculated and measured force. Alternatively, the force can be input and the velocity calculated or the wave up-wave down record can be used. When the best possible match has been obtained a simulated static load test can be run on the static part of the result. An example of a measurement, the match, and the simulated static load test curve is shown in Figure 5.

Questions have frequently been raised regarding the uniqueness of the CAPWAP solution. The mathematical uniqueness has been proven by Rausche et al. Uniqueness was also studied by Fellenius in a more practical way. Four different measurement sets were sent to 15 different engineers who are regularly involved in CAPWAP analysis. Each one made the analyses and submitted the results. A comparison of all of the results showed that good uniqueness was achieved. An example of the results is shown in Figure 6.

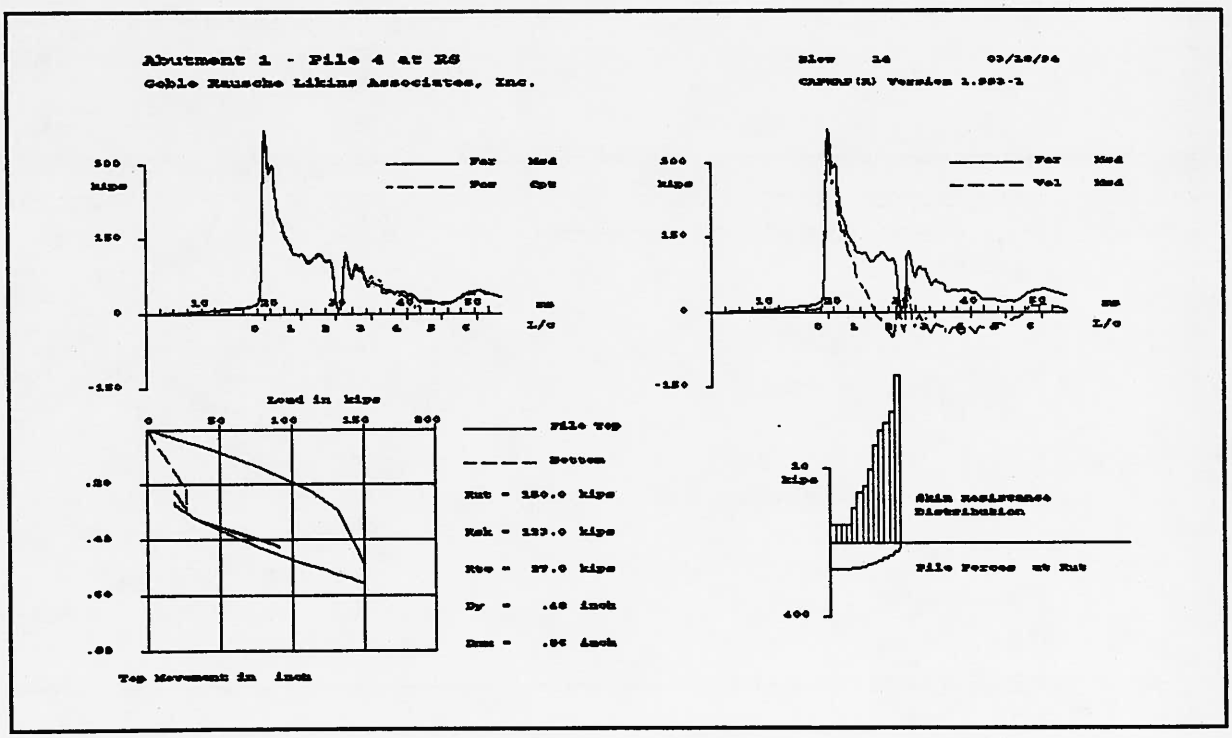

Figure 5

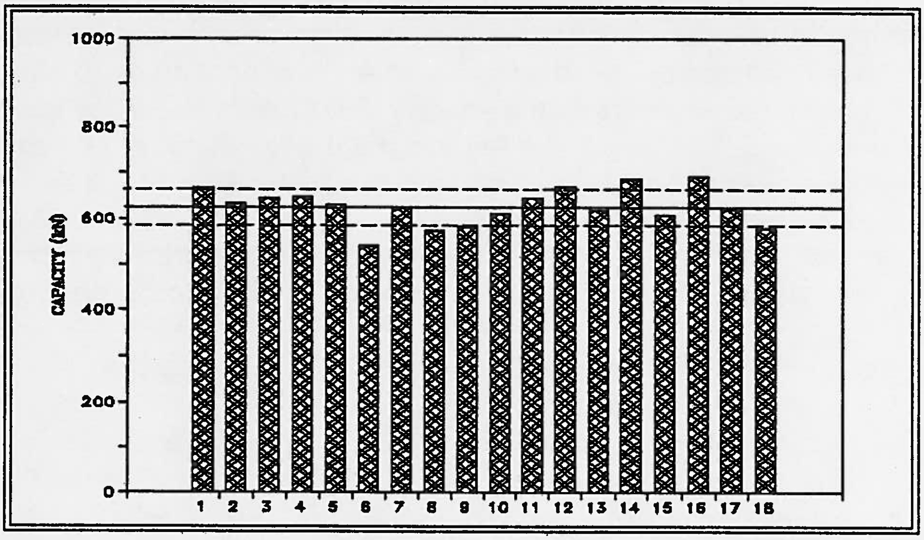

Figure 6

II. Hammers

In the period of reference here two external factors influenced the state-of-practice in the use of pile driving hammers in the United States. The first was the construction of the Interstate Highway System and the second was the offshore industry. The Interstate program caused a large growth in bridge construction capability in the United States. The regionally diverse nature of the program was such that there was a demand for pile driving capability over much of the country. Many small general contractors took on pile driving jobs that earlier would have probably turned to a speciality contractor such as Raymond. At the same time, many well-trained Raymond superintendents, looking at the large number of talented people that were above them in the organization chose to start their own business. The diesel hammer was ideal for their operation. It was light and could be handled by a much smaller crane than the equivalent air/steam hammer. The power was part of the machine so a large compressor was unnecessary and, therefore, in union areas the crew could be one man smaller. This hammer with a set of swinging leads constituted an inexpensive, flexible system that made it possible for a small contractor with little overhead to be competitive with the large specialty piling contractors particularly on small jobs.

The diesel hammer was developed in Germany before the war by the DELMAG company but was only brought into the United States after the war. This device can be described as a two cycle, internal combustion engine in which the ram serves the function of both ram and piston. In operation, the ram stroke will vary depending on the pile resistance. It is single acting with the top of the ram usually visible at the top of the stroke. It is sometimes also referred to as an "open end" hammer. Since the stroke is variable the speed of hammer operation will vary from aboout 40 to 50 blows per minute. Currently, several companies are manufacturing these hammers. They are available with strokes up to almost 3.5 m and generally the trend over the past 20 years has been to produce machines with continually increasing maximum strokes. The result has been higher and higher impact velocities. However, it is incorrect to assume that the impact velocity can be calculated from the stroke and free fall kinematics. After the ram passes the ports further a increases in velocity is impeded by the precompression force under the ram in the combustion chamber. This characteristic is modeled in the WEAP wave equation program and the hammer stroke is calculated for each case.

Diesel hammers have also become larger with passing time. In the sixties, a commonly used hammer might have had a ram weight of 11,000 to 22,000 kN. Today much larger machines are used. Currently the largest Diesel hammer has a ram weight of 100,000 kN.

Some diesel hammers have throttles that can be set for a metered amount of fuel for each stroke and there are several available throttle settings. It is possible to run the hammer at reduced throttle settings for cases where larger strokes may damage the pile. This capability can be of use in controlling stresses, particularly in the case of concrete piles.

Since the stroke is variable depending on resistance the peak energy is also variable. This makes rating of these hammers quite difficult and the various manufacturers do not use a standard procedure. The forces of competition have caused the ratings to be inflated to meet the requirements specified in particular job specifications. For external combustion hammers, the rating can be done in an orderly and obvious fashion and it is more important since it is usually used to calculate the stroke and the impact velocity for Wave Equation analysis. In the case of diesel hammers, the characteristics of the hammer are entered into the wave equation analysis and impact velocity and stroke are determined during the analysis. The diesel hammer manufacturers would provide a service to foundation designers if a reasonable and equitable rating procedure were developed. This will not be a simple task.

The performance of Single Acting diesel hammers can be checked by observing the stroke which can then be compared with the stroke obtained from Wave Equation analysis. The observation of stroke has been made with a "Jump Stick" that is attached to the upper barrel of the cylinder has stroke marks visible from the ground. This device is undesirable since it can become a dangerous missle if it breaks loose. A device is also available that will measure the stroke by timing the interval between hammer blows and calculate the stroke.

One diesel hammer manufacturer has eliminated the hammer cushion in his driving system, claiming greater efficiency of energy transfer and improved productivity.

Double acting or closed end diesel hammers are also available. These machines use a passive air cushion in a closed chamber as a spring at the top of the ram. Actually the design is more complicated since there are valved connections between the combustion chamber and the upper chamber that use the upper chamber pressure to improve scavenging and hence combustion. This design shortens the stroke and increases the speed of operation to the range of about 80 blows per minute. One manufacturer has produced a hammer that can be converted from closed to open end operation by removing a cap on the top of the cylinder.

The primary manufacturer of double acting diesel hammers is the International Construction Equipment Company (ICE). Their hammers use atomized fuel injection and due to this characteristic their performance will be quite different than single acting hammers that have impact atomization. The atomized fuel injection machines inject the fuel prior to impact in an atomized state. The fuel begins to burn almost immediately while injection continues. Since combustion takes place before impact this cushions the blow somewhat and increases the stroke. One advantage of this hammer is that it will run in very easy driving conditions where the single acting diesel would stop. This advantageous characteristic is achieved at the cost of a somewhat reduced impact velocity.

In the off shore environment, hammers became much larger, particularly during the oil boom of the late 70's and early 80's. Hammers of the traditional vulcan type became available with ram weights up to 1,300,000 kN and strokes to about 2 meters. In this market the hydraulic hammer was introduced and has become increasingly common. This machine has the additional advantage of being able to operating underwater. In this environment large cost savings can acrue to the use of underwater hammers. With an underwater hammer large pipe piles can be delivered to the site in full final length, flooded, lowered to the bottom and driven to final position. On-site welding is radically reduced and the construction time is reduced. Instrumentation to perform underwater dynamic monitoring is available and this is particularly important since easy visual observation is not possible.

During the late 70's and 80's a variety of hydraulic hammers became increasingly popular. Probably the earliest hydraulic hammer was developed and tested by the Raymond Company in the late 60's or early 70's. At the time, they argued that this was the hammer of the future. The machine that they built and tested was double-acting but its performance was not completely satisfactory, possibly due to the method of operation. An early hammer in Sweden used hydraulic actuators to push the ram up and then just retracted them faster than the velocity of fall. This simple system is particularly attractive since it is quite simple to control the stroke accurately, a necessary requirement when driving the commonly used concrete piles through the soft clay to recently glaciated granite, a condition that is common in Sweden.

The IHC hammer is particularly interesting in that it uses a double acting design that has the cylinder acting against compressed nitrogen on the up stroke and gaining velocity on the down stroke. Another important feature provides a measurement of ram velocity shortly before impact. This value is displayed and the hammer operation can be adjusted to give the desired value.

Today a number of manufacturers are producing hydraulic hammers of a variety of types. Impact velocities of almost 6 meters per second are available and they can usually be easily controlled.

III. Piles

Over the time period under discussion here, piles also developed dramatically. While the percentage usage of timber piles has decreased, design capacities have increased. Today the Nevada Department of Transportation uses working load capacities for treated Douglas Fir of 65 tonnes with a factor of safety of two, providing that the capacity is proven with a static load test. Acceptable driving stresses up to 3.0 ksi are generally used in well controlled situations.

In this time frame, steel H-piles have probably also lost market share. At the beginning of the period under consideration many designers used H-piles in all cases without careful reflection regarding the appropriateness of the design. As designers became more sophisticated they turned to other solutions and H-piles were often replaced. In the past few years, the cost of H-piles has dropped (due to a drop in the price of steel) so that they have become much more cost competitive. In addition, steels with yield points of 350 MPa (50 ksi) or even higher are available at little or no increased cost. When considered at appropriate sites they can be a competitive solution. In order to drive these piles using the higher yield stresses, the pile driving process must be understood and the appropriate equipment must be available.

For most ram-pile weight ratios, the maximum stress occurs at impact and is determined by the ram impact velocity and the helmet cushion system. If pile yield stress is increased, then the ram impact velocity must be increased to take advantage of the increased strength. Over the recent past, 90% of the yield strength has become accepted as a tolerable maximum for the maximum driving stress in steel piles. This translates into a stress of 250 MPa (32 ksi) for the most common steel type. If the increased yield strength is used, this will imply a peak driving stress of 310 MPa (45 ksi) for comparable conditions. The 45 ksi stress implies an impact velocity in excess of 25 feet per second and a free fall stroke of 10 feet. It would be unwise to use the 90% allowable driving stress in this application without gaining considerable experience with gradually increased driving stresses in the high strength steel. Bending stresses may become more critical so care in driving system alignment will be important.

Steel pipe piles have remained a common solution during this entire time frame. In land use, the most common application has been of closed end pipes. In the central part of the country these pipes were often quite thin walled with wall thicknesses as light as 7 gage with diameter of 324 mm (12-3/4 inches). This pile type had to be driven with great care and in the process of driving these piles contractors developed considerable skill in operating at very high stress levels. After driving, these piles are usually filled with concrete. As pile capacities have increased, the frequency of use of very thin wall thicknesses has decreased. Piles of this type have frequently used pipe that was manufactured for some other purpose such as oil well pipe but failed to meet some aspect of the specification for that application. Usually such pipe is of very high strength and easily driven.

A special type of pipe is the Monotube pile that is manufactured from steel sheet in a special facility. Pipes are formed from sheet and then deformed into a fluted shape by rolling over a mandrel. The forming process raises the yield point and the fluted shape makes it possible drive the pile very hard without damage. The lower section of the pile is usually tapered. The tapered shape seems to offer advantages in generating load carrying capacity that has not been mechanistically explained. This pile is also filled with concrete.

Some pipe piles are driven open-ended to achieve greater penetration where scour may be a problem. With the increased interest in the scour problem for bridges, this solution is seen much more frequently than was it was a few years ago. In some such cases, capacity prediction by dynamic methods can seriously underpredict. In these cases, the failure mode during driving does not have a plug forming while it may form during a static test. Recently, a case was observed where static load tests were performed on two piles that were within a short distance of each other in a single footing. One of the piles failed apparently with a plug while the other probably did not plug and failed at a much lower capacity.

In the thirty year time frame covered here, there was a substantial increase in market share for concrete piles. In the United States and Japan, the prestressed concrete pile is dominant today while in other countries most concrete piles are made of reinforced concrete that is not prestressed. Fairly high strength concrete is commonly used with strengths of 42-55 MPa (6000-8000 psi) common.

Prestressed concrete pile usage in the United States has grown, both in the frequency of use and the size and capacity of the pile. In the typical land application, 600 mm precast piles have become quite common as are lengths up to 40 meters. These large cross section elements may have a circular void. Square piles of this type up to 1120 mm in size with void have been driven on a batter up to 45 degrees.

In some locations, prestressed concrete cylinder piles have been used very effectively. These piles are precast in fixed lengths by centrifugal casting, usually in a facility for manufacturing large diameter concrete pipe. After casting and curing they are connected together by post-tensioning and grouting. Any desired length can be manufactured and diameters from 900 mm to 1800 mm with wall thicknesses of about 120 mm are available. These piles are sometimes difficult to drive but the potential for cost saving is substantial in applications such as shallow tidal waters where de-watering and pile caps can be avoided. It is usually necessary to transport these piles by barge so water transportation is necessary.

In most of the world, reinforced rather than prestressed concrete piles are used. These piles are precast in standard, relatively short lengths. They have mechanical connectors of steel on the ends and can be cast and stockpiled for use when needed. The required pile length is achieved by splicing the available lengths as necessary. Tension stress problems are reduced since in the easy driving portion the pile is fairly short. However, these piles will often be cracked when driving is finished. Research conducted in Sweden indicates that the cracks will "heal" in time, probably due to further hydration of the cement. Usually they are of relatively small cross section and use high strength concrete. It is surprising that these systems have not found their way into North American practice.

The mechanical splices, developed for reinforced concrete piles, are often used on prestressed concrete piles when very long piles are required. Frequent problems have been encountered with damage in the vicinity of the splice. First, great care must be taken that the splice fitting is properly installed and that its end is square with the axis of the pile. A long embeddment length must be used for the reinforcing bars that are attached to the splice fitting. In addition, it is desirable to examine tension stresses in the splice region where the prestress force is developed. It may be desirable to avoid tension splices unless tension forces must be carried under service load conditions. In many cases, a simple non-tension splice will be adequate, less expensive, and less likely to generate damage during driving.

D. FUTURE

I. Engineering

It can be expected that the electronics and computer revolution will continue and that measurements will become easier and computation cheaper. This will make possible more difficult measurements, but probably these changes will be only incremental. However, on the computational side the availability of of greater computational speed at less cost can produce the use of pile type measurements for subsurface investigation purposes. All of the geophysical type measurements produce elastic soil constants since the deformations are very small. With the use of measurements on sounding rods and a CAPWAP-type analysis insitu finite element constitutive properties including non-linear components can be determined.

In the area of small strain testing, it seems that improvements in the reliability of the technique are necessary. Perhaps, tools like expert systems or neural nets need to be applied. It is necessary that more conclusive results be obtained.

Greater care must be applied in performing static load tests. As pile capacities have increased the difficulties in performing static load tests have also increased. Too often a static load test is considered to be a routine task and the results produced are of poor quality. Standards of performing the test must be tightened and the personnel must be better trained. It is even more important (since it would be easier) to further standardize the evaluation of static load tests. The arguement is frequently made that the evaluation of the static load test must be left to the judgement of the geotechnical engineer. Too often the geotechnical engineer has had very little experience and needs the guidance of a specified method. With the advent of LRFD design procedures, it becomes easy to vary the ϕ-factor as the margin of safety changes.

II. Hammers

It appears that the development of hydraulic hammers will continue. Also, this development will include more quality control tools on the hammer. It should soon be possible for the engineer to specify exactly the driving procedure that he wishes to use and to expect that the contractor can execute them.

Surely higher impact velocity hammers will be required to drive the high strength steel piles. Manufacturers and contractors must deal with the noise problem. It should be possible to reduce these problems to the point where they are not a consideration. Ground vibration is a similar problem. The magnitudes must be determined and quantified to the point where the problem can be approached in a rational manner. Surely this problem is not more serious than the effects of drilling near the foundations of a large building.

III. Piles

Timber piles face an environmental problem regarding the influences of creasote. The industry has made the point that this problem has been over-stated but the problem remains. In a well-engineered job with good quality control, it should be possible to increase the design loads on timber piles. However, if loads are increased contractors must expect that higher driving resistances will be required. The effect of taper is clearly beneficial but needs to be better quantified.

Steel piles can be driven to higher stresses, particularly the high strength steels. However, the higher impact velocities will increase the possibility of damage and it is essential that the driving system be held in good alignment, and that the blow be uniformly and concentricly applied. Changes in practice of this type should be made gradually.

Higher strength concretes should be tested and probably the real field conditions should be examined. It seems that these concretes reach their maximum stress at a lower strain level than ordinary concrete so higher impact velocity hammers may not be required. The very high strength concretes are more brittle and the influence of this property must be evaluated in field tests. Again strength increases should be investigated with gradual increases.

E. CONCLUSIONS

Thirty years of inovation and modification has greatly increased the efficiency of pile driving operations. The understanding of the mechanics of pile driving has advanced dramatically over this time period. The time has now come to capitalize with practical applications on these developments. The opportunities for cost saving advances are available to the creative foundation designer and constructor.

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PILE LOAD TESTING - NEW AND IMPROVED

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ABSTRACT

A recent series of pile load tests performed on tapered concrete piles incorporated telltales and embedded strain gages at selected depths within the pile. These instruments enabled the elastic deformation of the pile to be computed and the loads at the instrument locations to be estimated. This formed the basis for a more accurate interpretation of the ultimate capacity of the pile and provided a means of verifying that the concrete in the reduced cross-section at the pile base was not being overstressed at design load.

INTRODUCTION

Part of the cost benefit of using a tapered pile (such as a step-taper or monotube) comes from reducing the amount of concrete required. It is important to verify that these piles develop the skin friction assumed in design to ensure that the lower portions of the pile with reduced cross-section are not overstressed. The skin friction distribution can be determined by installing strain gages at points within the pile. In addition to providing information on pile stresses, the skin friction distribution data from the strain gages can be used to estimate the elastic deformation of the pile, which can be compared to the elastic deformation measured by telltales. These elastic deformation data can lead to a more accurate interpretation of the load test results.

In the past two years, Bechtel has used step-tapered piles on two major power plant projects in the United States. On both projects it was essential to verify that the pile could shed sufficient load through skin friction over its length so that the concrete in the lower section of the pile would not be overstressed. To accomplish this goal, the pile contractors were required to instrument all the test piles with strain gages and telltales at various depths. This paper presents, for one of these projects, the details of the pile testing and instrumentation, and discusses the results.

HISTORY OF PROJECT

The Indiantown Cogeneration Project site is located in south central Florida just north of the town of Indiantown and east of Lake Okeechobee. The plant, when completed, will be the largest coal burning unit in South Florida. The major plant structures include a turbine, boiler, concrete stack (495 feet high), and coal and ash handling systems.

A detailed subsurface investigation was conducted at the site in March, 1992. The investigation included standard penetration borings, cone penetrometer soundings and dilatometer testing. Figure 1 shows a subsurface profile through the major structures. The results of the investigation indicated that piles would be needed to support the major plant foundations. The primary reason for this decision was the loose to medium dense sand layer (Stratum III) between 10 and 25 feet below existing grade.

PILE DETAILS

Analysis of structure loading and pile spacing indicated that the optimum pile for the project would have a compression capacity of 95 tons, a tension capacity of 40 tons, and a lateral capacity of 8 tons. Pile calculations using various methods of static analysis were then used to determine the pile requirements (size and length) for various types of piles. The analyses included pipe, precast concrete, augercast, H-pile, and step-tapered piles. Typical computed length for displacement-type piles was about 80 feet, with the piles terminating in the Stratum IV sand.

The pile contract package was sent to four qualified pile contractors allowing each contractor to bid any type of pile provided the pile capacity requirements were met. The total number of piles estimated for the project was about 1250, or approximately 100,000 feet of piling. The contractor would be paid per pile, not per foot of pile as is the case in most pile contracts. The performance of the pile would be verified by load testing. The pile contract required that a minimum of four pile tests be performed for each load condition.

Based on the technical and commercial evaluation of the bids received from the four contractors, it was determined that the step-tapered piles offered by the Franki Northwest Company would be the most advantageous for the project. The Franki Northwest Company bid included two sizes of step-tapered piles, the first with a 10 3/8-inch tip section and the second with a 8 5/8-inch tip section. Both piles would consist of 12 foot sections, with each section increasing in diameter by about one inch. A sketch of each pile section is shown in Figure 2. Franki hoped that the overall cost of the larger diameter pile would be less since it would achieve the design capacities with a shorter length.

One concern about using the step-tapered pile was that the concrete in the lower sections could be overstressed if a large proportion of the applied load on the pile was transferred to the tip. The applicable building code (Southern Building Code) allowed only 25 percent of the 28-day strength of the concrete for the design of the pile. Using concrete with a 28-day strength of 4000 pounds per square inch, the maximum allowable load in the bottom section of the pile with the 8 5/8-inch tip would be about 30 tons. Thus, for the 95-ton design capacity, it would be necessary to have 65 tons of load taken up in friction before reaching the bottom step of the pile. The static analysis for the step-tapered pile indicated that about 80 percent of the design load would be carried in friction with the remaining 20 percent carried in end bearing. This translated to about 19 tons loading at the tip of the pile and about 29 tons loading at the top of the bottom section.

Although the estimates from the static analysis indicated that the bottom section of the pile would not be overstressed, the results were marginal, and the actual load distribution within the pile needed to be verified. (All static pile capacity computations make assumptions about the frictional and bearing capabilities of the soil based on values selected from a fairly wide range, and thus can be significantly in error.) To estimate load at critical points in the pile, the contractor was required to install strain gages in the pile during the load testing. The strain gages selected were Model VCE-4200 vibrating wire gages supplied by Geokon Incorporated.

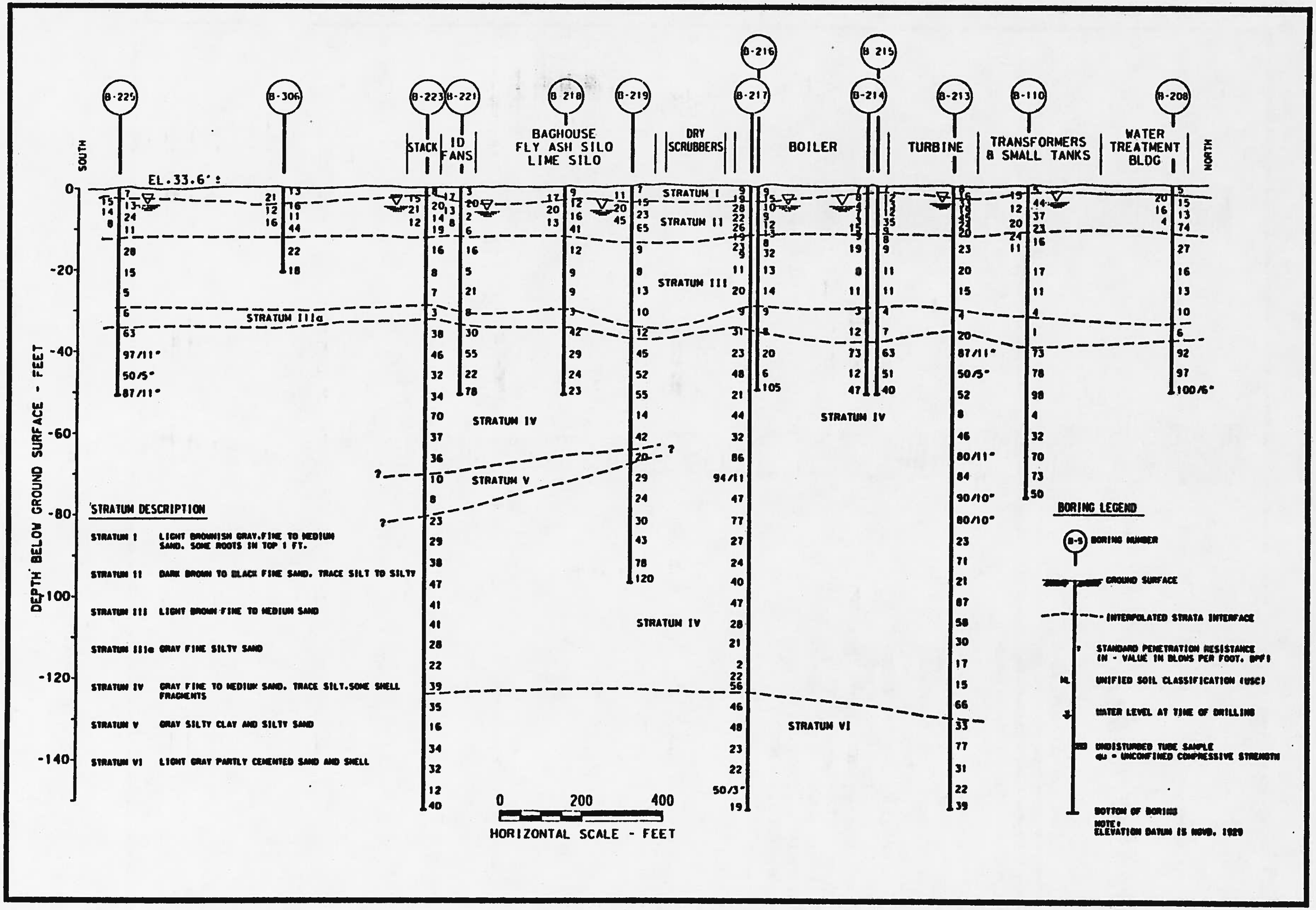

Figure 1. Subsurface profile beneath major structures

STEP-TAPER® PILE

Step-Taper Piles are steel encased concrete piles installed by driving the required length of steel shell with the aid of an internal steel mandrel; withdrawing the mandrel leaving the steel shell in place; inspecting the driven shell internally; and filling the shell with concrete. Numbered shell sections are made in 4, 8, 12 and 16 foot lengths. Longer lengths can be furnished for special conditions. Within practical limits shell sections of different lengths can be combined in a single pile. Shell sections are screw connected. The pile shell is closed at the bottom by a flat steel plate welded to the drive ring.

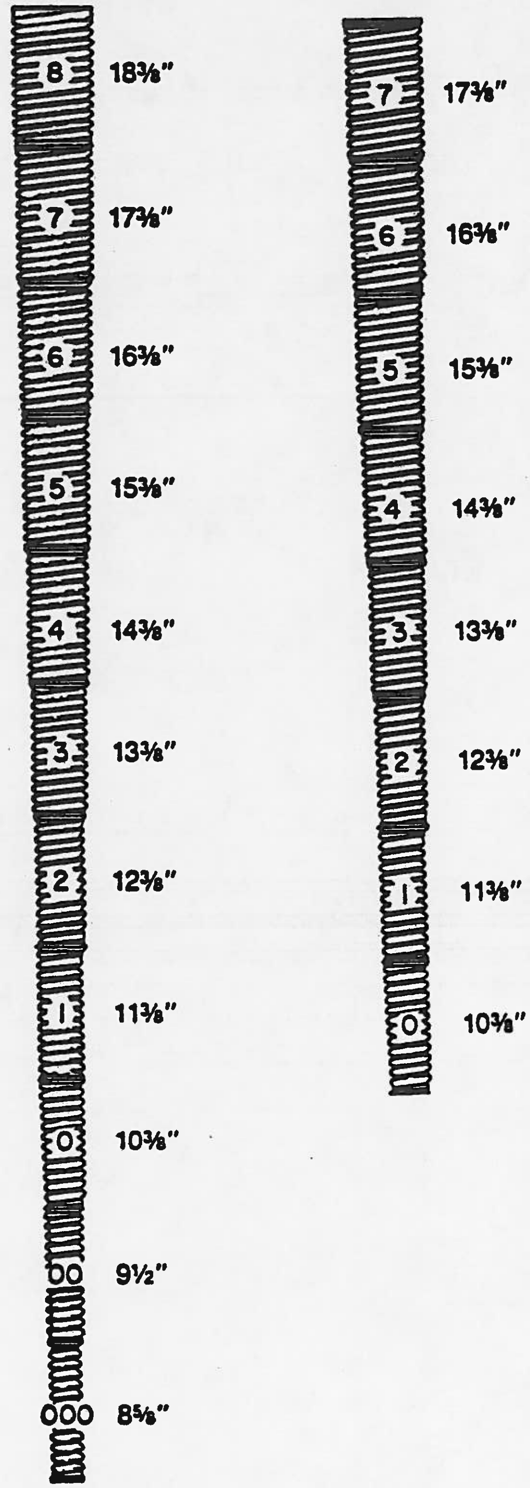

Figure 2. Dimensions of step-tapered piles

TEST PILE INSTALLATION

The test pile program required four test piles, one each at the location of the chimney, boiler building, turbine pedestal, and fly ash silo. Installation of the test piles began in November, 1992. The step-tapered piles were installed by inserting a thick-walled steel mandrel into the corrugated steel shell (the mandrel's stepped shape corresponded to that of the shell) and then driving the mandrel using a Raymond 80C hammer (24,500 foot-pounds rated energy). After the pile shell was driven, the mandrel was withdrawn, the required reinforcing was installed, and the corrugated steel shell was filled with concrete.

At the first two test locations, Franki elected to install and test both sizes of pile. At the first location (in the boiler building), the pile with the 8 3/8-inch tip (TP-3A) was driven to a depth of 62.5 feet below grade. The pile with the 10 3/8-inch tip (TP-3B) was driven to a depth of 49 feet below grade. The driving record of TP-3A along with the standard penetration resistance of the nearest soil boring are shown in Figure 3. Although the data from any of the test piles installed and tested at the Indiantown site could have been used to demonstrate how the strain gages and telltales behaved and how the results were interpreted, the TP-3A data were typical and are used for the remainder of the paper.

The reinforcing for the piles consisted of a single 1 3/8-inch diameter Dywidag bar for the full length of the pile and 8 No. 6 bars in the top 20 feet. The vibrating wire strain gages were attached to the Dywidag bar using flexible wire which allowed the strain gages to move independently of the bar. The strain gages were placed one foot below the top of the pile, at mid-depth (which corresponded approximately to the change from Stratum III sand to the denser Stratum IV sand) and either at the top of the bottom 12-foot pile section (at the location of maximum load in the smallest diameter section) or at the tip (for comparison with the telltale results). The purpose of the strain gage near the top of the pile was to obtain a measurement of the elastic modulus of the pile instead of assuming a modulus value based on the anticipated 28-day strength of the concrete. In addition to the strain gages, telltales were placed at mid-depth and at the bottom of each pile.

Pile TP-3A was cut off 1.5 feet above grade giving a total concreted pile length from top to tip of 64 feet. The strain gages were installed 1 foot, 33 feet and 63.5 feet below the top of the pile. The telltales were placed 33 and 63.5 feet below the top of the pile.

PILE LOAD TESTING OF TP-3A

The load testing of pile TP-3A was started 8 days after the pile was installed. The test was conducted in accordance with the ASTM D 1143 "Quick Load Test Method", with the loading applied in increments of 10 percent of the design load (i.e., 10 percent of 95 tons) to a total load of 200 percent of the design load (190 tons) and then backed off in increments of 50 percent of the design load. Each loading and unloading increment was held for a period of 15 minutes. The load tests were run by Consulting Foundation Engineers under subcontract to Franki Northwest Company.

The load test set-up used four reaction (anchor) piles and a reaction (test) beam as shown in Figure 4. A calibrated load cell was used between the jack and the reaction beam to validate the load readings from the pressure gage attached to the jack. The downward movement of the pile top during the test was measured by four dial gages evenly spaced around the pile and attached about 1 foot below the top. During the testing, readings were taken immediately after applying each incremental load and at 7 1/2 minutes and 15 minutes after load application. Readings were taken on each of the three strain gages, the two telltales, and the four dial gages at the top of the pile.

The three strain gages were connected to a switching box, which was then connected to a Geokon Model GK-401 Vibrating Wire Readout Box. The readout box gave direct readings of strain in microinches/inch. Prior to the start of testing, an initial reading was taken on each strain gage to provide a zero point for the gage.

INTERPRETATION OF TEST RESULTS FOR PILE TP-3A

As noted earlier, the purpose of the strain gages and the telltales was to help estimate (1) the load distribution along the pile, (2) the elastic modulus of the pile, and (3) the elastic deformation of the pile. It should be noted that the results discussed below are based on data from a "quick" load test. It is the authors' experience that data recorded from this type of test are usually similar to those from the standard ASTM load test where each load is held for a maximum of 2 hours and the final load is held for a maximum of 24 hours. However, neither the quick nor the standard test can provide information about the long-term distribution of loads and stresses in the loaded pile. The soil materials at the Indiantown site appeared to have no unusual characteristics, and no significant long-term distribution adjustments at working load from freeze, relaxation, etc., were anticipated.

Elastic Modulus

The procedure to compute the elastic modulus of the pile is straightforward. First the stress level at the strain gage at the top of the pile is calculated by dividing the load applied by the jack (and confirmed by the load cell) by the measured cross-sectional area of the pile. It is assumed that the full load being applied at the top of the pile is reaching the upper strain gage a foot below the top (and above ground), and that the load is fully and evenly distributed across the entire area of the pile at that depth. The elastic modulus is then calculated by dividing the computed stress by the measured strain.

Although the computation of elastic modulus from the strain gage readings at the top of the pile was straightforward, the choice of what value of elastic modulus to use to compute stresses and deformations at the other two strain gage locations was not. First, the elastic modulus estimated from the top strain gage readings was not constant, rising in pile TP-3A as the applied load was increased to about 50 percent of the design load and then falling consistently until the applied load reached twice the design load. A similar pattern of variation was observed in all the test piles. Presumably, the stress distribution within the pile head was not completely even as assumed, and was probably quite complex, being affected by the rebar throughout the concrete area and the steel shell surrounding the concrete.

The estimated elastic modulus of the concrete and steel in pile TP-3A was considerably higher than in the remaining test piles, averaging about 6,650 kips per square inch (ksi), while the other piles averaged from about 4,000 to 5,300 ksi. In pile TP-3A, the modulus computed from the strain gage reading at initial loading was about 6,440 ksi, rising to about 7,780 ksi at 47.5 tons, and then falling to about 5,475 ksi at 190 tons.

Since each step taper section had a different proportion of shell steel area to concrete area, and the No. 6 rebars were in only the top 20 feet, there were 7 sections with different pile elastic moduli in the 64-foot long pile. Taking the modulus computed from the top strain gage at 190 tons as 5,475 ksi, and using a steel modulus of elasticity of 29,000 ksi, a concrete modulus of about 3,780 ksi was obtained. With this concrete modulus, the computed pile moduli in the various sections based on the proportional areas of steel and concrete ranged from about 5,080 ksi to about 5,950 ksi.

The variation of pile modulus in the different pile sections made the analysis more time consuming, but was not considered a problem. The relatively large variation of computed modulus with load did pose a problem since both steel and concrete are both nearly linearly elastic in the stress and strain ranges in the load

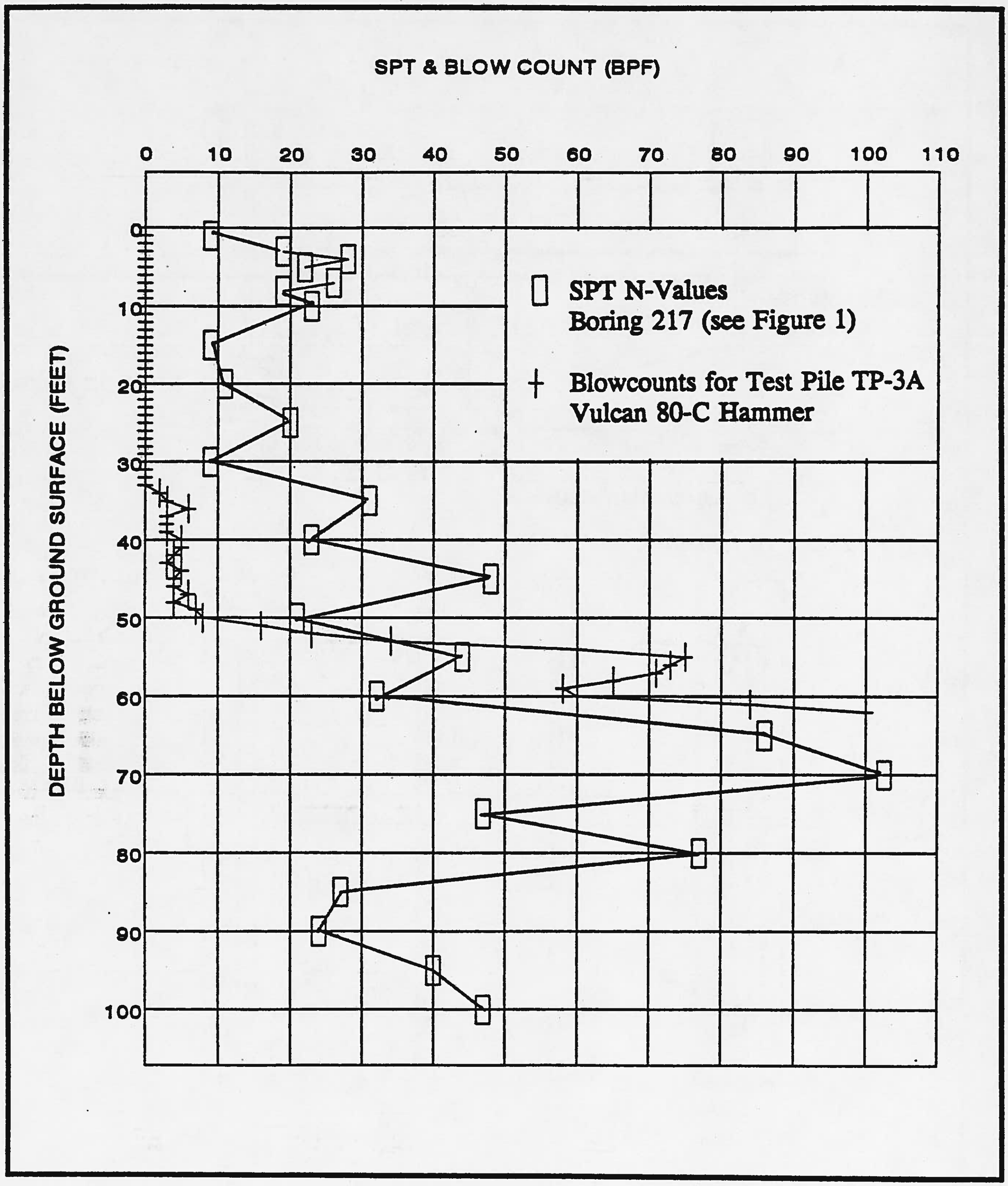

Figure 3. Driving record of test pile TP-3A and SPT N-values of adjacent boring

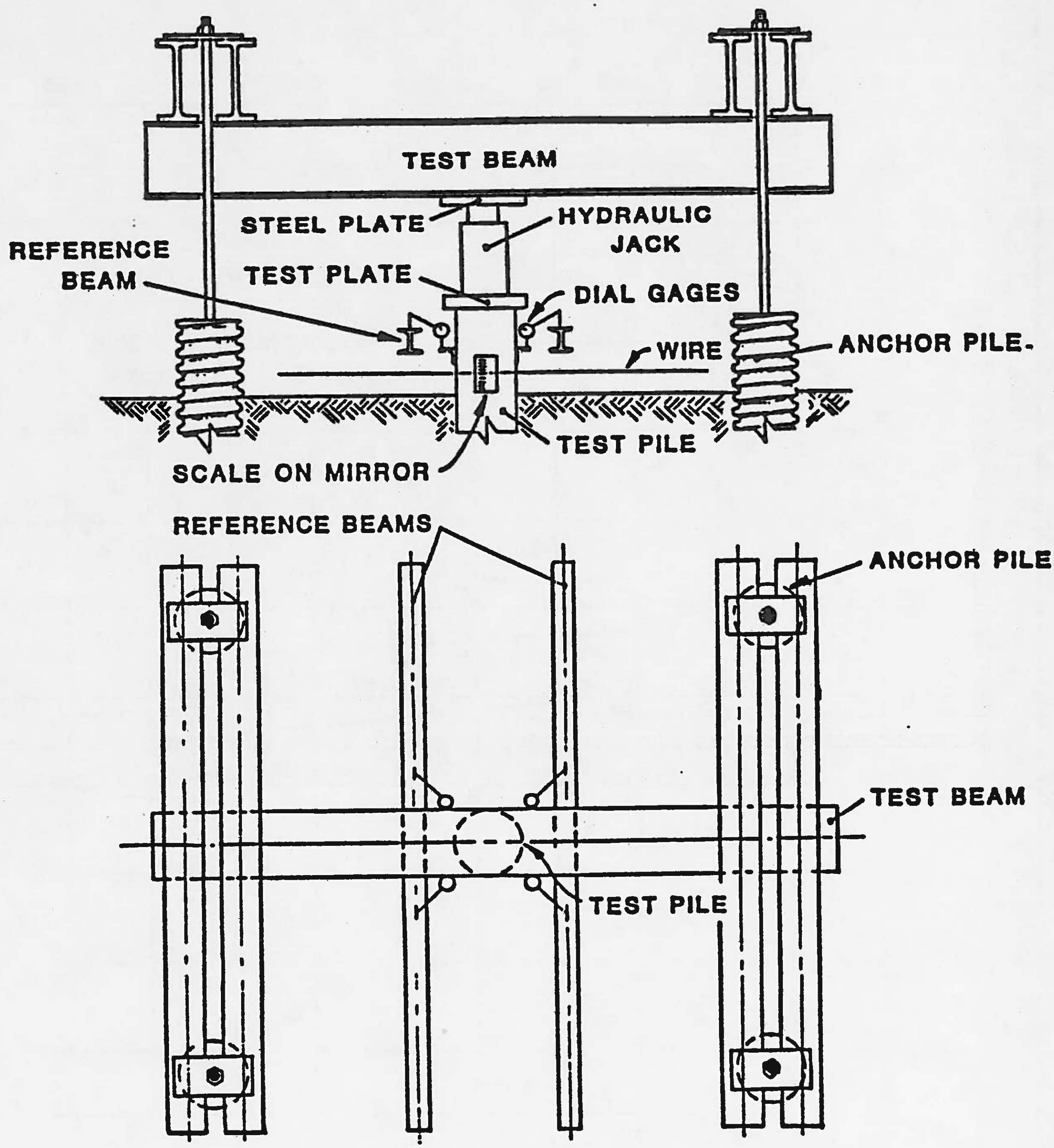

Figure 4. Pile load test setup

test, and thus should have only one composite modulus value for any given combination of concrete and steel areas. For computation purposes in pile TP-3A, the lowest measured pile elastic modulus was chosen since it provided a concrete modulus value fairly close to that which would have been expected from the concrete, which was a high early strength mix.

Given the apparent difficulty of measuring the pile modulus directly, it may have been better to have measured or computed the elastic modulus of the concrete and then computed the elastic modulus of the pile knowing the areas of steel and concrete in the pile and the elastic modulus of the steel. The elastic modulus of the concrete could have been estimated by averaging the results of a series of concrete breaks performed on cylinders of concrete taken from the test pile pour and tested at the same time as the load test. This modulus could be derived directly from these tests if the deformation at failure was recorded, or it could be derived indirectly from the compressive strength.

Stresses and Loads in the Pile

Once the elastic modulus of the top of the pile is estimated from the top strain gage readings, the stress and loading in the pile at each of the other two strain gages can be determined by simply reversing the process. First, the elastic modulus of the pile at each of the lower strain gages is computed using the derived elastic modulus of concrete and the areas of steel and concrete at the strain gage location. The stress in the pile is then calculated by multiplying the elastic modulus by the measured strain, and the load in the pile is calculated by multiplying the calculated stress by the cross sectional area of the pile at the depth of the gage. The results of this evaluation for TP-3A are presented in Figure 5.

Figure 5 shows that, at the design loading of 95 tons, just over 6 tons was reaching the tip of the pile. Although the load reaching the top of the bottom section would have been greater than 6 tons, it was reasonably assumed that it would be well below the allowable limit of 30 tons. This was borne out by strain measurements taken in the top of the bottom section in other test piles, and confirmed that the pile stresses developed were acceptable for the project.

Figure 5 also shows that, as loading increased beyond the design load, the tip load started to increase considerably, with most of the loading between 95 and 190 tons being taken on the tip. In addition, between the top and mid-point of the pile, the load carried in friction started to decrease when the loading increased beyond about 120 tons, indicating that once the ultimate frictional capacity at the soil/pile interface was reached, it decreased at greater strain. This stress-strain behavior pattern, i.e., build up to a maximum stress and then decrease to a residual stress beyond a certain strain level, is frequently observed in soils, and is incorporated into some of the more sophisticated types of axial pile analyses. As can be seen in Figure 5, the load being carried in the upper portion of the pile reached a maximum of about 23 tons at around 120 tons applied load and decreased gradually to about 17 tons at around 160 tons applied load. The decrease down to only about 5 tons frictional load at 190 tons final loading was more than would normally be expected, and the performance of the middle strain gage at the highest loading levels was considered questionable.

The friction in the lower section of the pile appears to behave somewhat differently with strain. The friction maximized at about 90 tons with a total load on the pile of 140 tons, and then remained nearly constant through the remaining portion of the testing. In fact, although the total frictional load remained essentially constant on the lower half of the pile during the latter stages of the load test, the distribution of the load probably changed, i.e., at around 140 tons, the frictional load near the middle of the pile may have been at or close to maximum

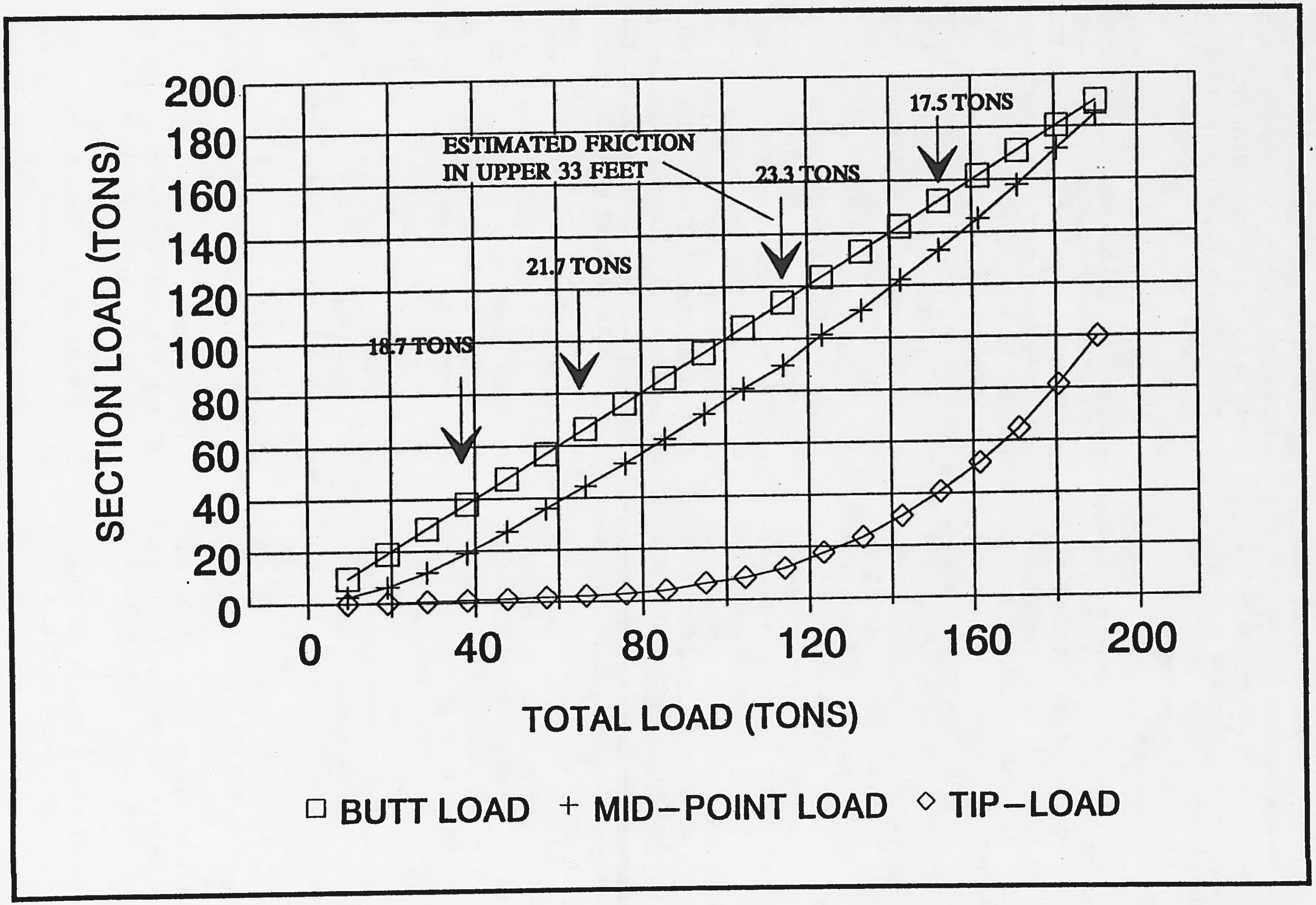

Figure 5. Computed load at strain gage locations versus applied load

while frictional load near the bottom of the pile was low, corresponding to low strain levels. As the applied loading increased, frictional loading near the middle started to fall as strain levels progressed beyond maximum stress levels, while frictional loading increased at the bottom as strain levels built up. The actual stress distribution pattern that occurred with increasing applied load could have been better demonstrated if additional strain gages had been placed at intermediate points on the pile.

Ultimate Soil Skin Friction

The strain gage data can also be used to back-calculate the ultimate frictional capacity of the soil, which in turn can be used to check the initial static calculations and provide better soil parameter values for use on future projects in similar soil conditions. The maximum measured value of skin friction in the upper 33 feet was about 23 tons, which corresponded to an average ultimate skin friction in the Strata I, II and III sands of about 0.24 tons per square foot (tsf). In the lower 30 feet (Stratum IV) the maximum measured value of skin friction was about 90 tons, which corresponded to an average ultimate skin friction in the Stratum IV sand of about 1.2 tsf.

The results from the other three load tests on the 8 5/8-inch tip piles gave very similar average ultimate skin friction results to pile TP-3A. This is not surprising since, although there were local variations in stratum thickness and sand density throughout the test pile area, these variations tended to be averaged out over the length of each pile. This is why the factor of safety applied to the skin friction component of pile capacity is sometimes chosen as less than the safety factor on end bearing, since the end bearing relies heavily on the strength of a fairly small zone of soil directly below the pile tip.

Elastic Compression of the Pile

The difference between the total downward movements of the top and the tip of the pile is the elastic compression of the pile. This movement can be measured directly if a telltale is installed at the pile tip. At 190 tons applied load in pile TP-3A, the elastic deformation measured by subtracting the tip telltale movement from the average movement of the four dial gages at the top of the pile was about 0.42 inches. Most load test criteria that assess the ultimate capacity of the pile use the elastic deformation of the pile in their interpretation. For example, the BOCA National Building Code (1993) defines the ultimate load in the pile to be that which causes a butt movement of 3/4 inch more than the elastic compression of the pile. Davisson (1973) proposed the ultimate load as that which caused a butt movement that was the elastic deformation plus 0.15 plus the pile diameter divided by 120 (all in inches). Using the BOCA criterion, the ultimate capacity of pile TP-3A was almost 200 tons. The ultimate capacity using the stricter Davisson criterion was about 150 tons.

Where a tip telltale is not used, the elastic deformation of the pile can be estimated using the well-known relationship that equates deformation to PL/AE, where P is the average load, L is the length, A is the cross-sectional area, and E is the elastic modulus of the section of pile under consideration. If there are no strain gages, then the load distribution must be estimated. The distribution assumed can make a large difference in the computed elastic compression. For example, if it is assumed that all the applied load is transferred to the tip, the computed elastic deformation will be twice that if the load is assumed to be shed evenly in friction throughout the length of the pile, and more than twice if the friction is greater at the top of the pile.

For pile TP-3A at 190 tons load, the estimated load at the middle strain gage was about 185 tons, while the load at the tip strain gage was about 99 tons. Assuming a linear distribution of load in the pile sections between the strain gages and using the computed elastic modulus value for each of the pile sections, the estimated elastic deformation was about 0.53 inches.

As noted earlier, the final readings of the middle strain gage appear to be questionable. If the pile load estimated from the middle gage is neglected and a linear distribution of load is assumed between the top and tip of the pile, the estimated elastic deformation is about 0.46 inches, relatively close to the 0.42 inches measured from the telltale.

TELLTALES AND STRAIN GAGES

The previous section presented the results of the load test interpreted from the strain gage and telltale data. In this section, the advantages and disadvantages of these instruments are discussed, both from the standpoint of the different types of data each instrument produces, and from considerations of installation, maintenance and reliability.

Interpretation

With telltales, the deformations are read directly and the difference in load in the pile between the telltales can be computed from PL/AE. The loads at each telltale level can then be calculated by subtraction, and stresses computed from the loads. Strain gages measure strain directly at the strain gage location and stress and load can be computed at that location from the strain and the elastic modulus. Deformations are then computed from PL/AE. It might be concluded that the two types of instrument similarly located in a pile would provide similar results. This will only be the case when the distribution of skin friction along the pile (and hence the distribution of load, stress, strain and deformation) assumed in the PL/AE equation is the same as the actual distribution along the pile. The distribution between instruments is typically assumed to be linear unless there are apparent reasons for non-linearity. Thus, if the actual distribution is linear, the computed loads and deformations in the pile will be the same using telltales and strain gages. If the actual skin friction distribution is non-linear, then the telltales and strain gages will give somewhat different interpretations of load and deformations if a linear distribution is assumed. In such a situation, the telltale will read the correct deformation but this will lead to an incorrect computation of load. The strain gage will read the correct strain and hence stress and load, but this will lead to an incorrect computation of deformation. Whether the incorrect computation is lower or higher than the actual value will depend on the distribution pattern. For example, if, in the length of pile between two instruments, there is a greater amount of skin friction towards the bottom, then the actual amount of elastic compression measured from the two telltales will be greater than that computed from the strain gages assuming a linear distribution. Similarly, the actual load at the lower strain gage (computed from strain) will be less than that computed from the telltales. The amount of error in the computed value will decrease as the spacing of the instruments along the pile decreases.

In test pile TP-3A at the 95 ton design load, elastic deformation of the pile measured between the dial gages on the butt and the telltale at the mid-point was 0.074 inches compared to 0.112 inches computed from the strain gage readings assuming a linear distribution of skin friction. The corresponding readings between the middle and the bottom of the pile were 0.045 inches for the telltales and 0.072 inches for the strain gage. These results indicate that there is proportionally more skin friction occurring on the upper portion of both the upper and lower sections of the pile. The profile in Figure 1 tends to support this supposition for the upper half of the pile since the soils in Strata I and II are generally denser than those in Stratum III. In the lower half of the pile, the strains towards the bottom of the pile were very low at design load, and would probably not produce as much skin friction as higher in the pile. It should be noted, however, that with only two strain gage locations, a relatively small error in one or both gages can have a fairly significant effect on the computed elastic deformation. In order to make a conclusive determination of skin friction distribution, several more strain gages would have been needed.

Installation

Telltales and strain gages both have advantages and disadvantages when it comes to installation, reliability, and operation. For piles such as driven closed-ended pipes, step-tapered piles and monotubes, telltales and strain gages can be fairly easily installed before concreting. When these piles have relatively small diameters, the space occupied by a series of telltales can be prohibitive, particularly if there is reinforcing to contend with. In general, strain gages are preferable when a large number of instruments are needed in one pile.

For driven precast concrete piles and H-piles, the strain gages need to be cast into or attached to the pile before driving. Both the strain gage and the attachment system need to be able to withstand the driving forces. (On a Bechtel project after Indiantown that used driven precast prestressed concrete piles, strain gages were embedded in the pile during the casting operation. During the load testing, all of the gages worked satisfactorily.) For telltales, only the sleeve needs to be cast into or attached to the pile before driving. The telltale rod is installed in the driven pile.

Strain gages are read electronically, and the data can be stored on computer disk and reduced directly using appropriate software. Telltales have traditionally been attached to dial gages and read and recorded manually during the load test. However, the dial gages could be replaced by electronic instruments such as LVDTs and the data handled in the same way as strain gage data.

A frequent problem with telltales is the telltale rod binding or jamming in the sleeve, particularly with long piles. On the other hand, strain gages are not without problems. Although the strain gages at the Indiantown project proved to be very reliable, the results at the other power project mentioned earlier were not as good. As with all instrumentation, good quality equipment, installation and maintenance by experienced personnel, and knowledgeable operation is the key to success.

PILE DRIVING ANALYZER

The ultimate load for each of the four load tests conducted on the 8 5/8-inch tip piles was computed using Davisson's criterion. These ultimate loads were approximately proportional to the pile length, which ranged from 62.5 feet to 82 feet below grade. Based on these results, a minimum embedment of 71 feet along with a specified minimum final blowcount (based on the test pile results and wave equation analyses) were specified for the production pile driving criteria.

To confirm that the selected driving criteria for production driving based on the load test results were adequate in the area where piles shorter than 71 feet had been tested, an additional pile was tested using a Pile Dynamic Analyzer (PDA). The PDA work was performed by Goble Rausche Likins and Associates, Inc. under subcontract to Franki Northwest Company. The PDA testing was conducted on a pile driven to 71 feet at the specified final blowcount. The results of the PDA testing and the CAPWAP analysis indicated that the pile had an ultimate capacity of 201 tons with 191 tons carried in friction and 10 tons carried in end bearing. Although the total pile capacity from these analyses was in good agreement with the test pile driven to 71 feet, the proportion of skin friction to end bearing was different.

SUMMARY AND CONCLUSIONS

The use of telltales and embedded strain gages in a pile during the load test provides (1) a more accurate interpretation of the ultimate load in terms of elastic deformation, (2) a better understanding of how the loading is transferred to the soil, (3) an indication of the average skin friction, and (4) verification that the pile is not being overstressed.

The telltale reading at the pile tip measures the pile tip movement directly and, when subtracted from the movement of the top of the pile, provides a measure of the elastic deformation of the pile. To obtain estimates of stresses and loads in the pile at the telltale location, the elastic modulus of the pile and the skin friction distribution have to be assumed. The strain gage measures strain at the gage location and provides an estimate of the load at that location assuming an elastic modulus for the pile. The skin friction distribution needs to be assumed to compute elastic deformation of the pile.

The instrumented load test on the concreted step-tapered pile discussed in the paper used a strain gage at the top of the pile to measure the elastic modulus of the pile. The computed modulus showed a significant variation with applied load, presumably due to a complex stress and strain pattern within the pile resulting from the combination of concrete and steel rebar and shell. A more accurate pile modulus might have been derived by measuring or estimating the elastic modulus of concrete and then computing the pile modulus based on the areas of concrete and steel.

Based on the results and discussions presented here, a large amount of useful information can be obtained about pile elastic deformation, loads in the pile, and skin friction distribution along the pile by installing a telltale at the tip and several strain gages in the pile. The spacing of the strain gages will depend on the soil stratification; 10 to 20-foot spacing will give a lot of data for the dollar. The relatively large strain gage spacing on the test pile presented in the paper allowed only tentative conclusions to be drawn about the skin friction distribution. However, the instrumentation did serve its intended purpose, i.e., to provide sufficient data to enable the load test acceptance criteria to be applied accurately, and to show that the concrete in the pile does not exceed the code limits at design load.

The additional cost of adding the three strain gages to the pile was about five hundred dollars per pile, plus about two thousand dollars for the purchase of the readout box. The readout box can also be rented on a monthly basis for under two hundred dollars. In total, six piles were tested using 18 strain gages. During the testing, only one gage failed.

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ACKNOWLEDGEMENTS

The authors would like to thank Bechtel Corporation, Bechtel Power Corporation and U.S. Generating Company for their support in preparing this paper.

The following paper is a reprint of Design of Bridge Pier Pile Foundations for Ship Impact by Bogdan O. Kuzmanovic and Manuel R. Sanchez. It was originally published in the **Journal of Structural Engineering**, Vol. 118, No. 8, August, 1992. ©ASCE, ISSN 0733-9445/92/0008-2151. Paper No. 1883. Permission to reprint is granted by the American Society of Civil Engineers.

Design of Bridge Pier Pile Foundations for Ship Impact

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Abstract: Procedures to assess the equivalent static force acting on a bridge pier due to a vessel impact and various pier protective systems are briefly discussed. Equations and closed-form expressions are developed to obtain transverse and longitudinal components of ship impact, redistributed between pier and superstructure according to their relative stiffnesses. Plumb- and batter-pile combinations are compared. The necessity of investigating batter-pile combinations in several iteration cycles is stressed. Zero forces in "pulled-out" piles are assumed in each next consecutive iteration cycle until either stability or failure is reached. Battered piles increase axial forces but decrease bending moments. This reduction in flexure is important because it reduces significantly the required number of prestressing strands in the pile. As an illustration, a comparative plumb and batter pile foundation design of the channel pier of Howard Frankland bridge, in Florida, is given.

Introduction

Since the Maracaibo bridge disaster in May 1964 and a number of other major collision events, there are now many examples of the dire consequences of a vessel striking a bridge pier over navigational waters that justify making the protection of the piers a basic design requirement. This protection is achieved either by designing the pier and superstructure to withstand alone the ship impact or using external protective systems to prevent (or reduce) the collision or both. This choice should be based on a complex economic analysis involving threat analysis and cost of the economic risk due to the passage of vessels under a bridge as compared to the cost of its protection. The difficulty in the analysis is knowing how safe is safe enough and what an acceptable "causation probability" is. The Louisiana manual (*Criteria* 1985) accepts an annual risk of bridge interruption of 0.0001. As with any form of statistical analysis, the accuracy of results is highly dependent on the extent of knowledge and research utilized for the formulation of important input parameters for the model. Unfortunately, the experience with actual collisions shows that the risk involves not only the piers adjacent to navigation spans, but, rather, all bridge piers in sufficiently deep waters, because off-course vessels may hit anywhere. If it would be possible for a vessel to approach a pier then that pier should be protected against such a risk however small the statistical probability of an accident might be.

The type of protection adopted depends upon the size, speed, and frequency of the ships and barges passing the bridge site, the profile of the river- or seabed, the pier alignment within the water, and the cost of pier protection system in relation to the cost of the bridge. In Sweden (Olnhausen

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Note. Discussion open until January 1, 1993. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on May 5, 1991. This paper is part of the *Journal of Structural Engineering*, Vol. 118, No. 8, August, 1992. ©ASCE, ISSN 0733-9445/92/0008-2151/$1.00 + $.15 per page. Paper No. 1883.

1983) for instance, a cost increase between 5% and 20% of the whole bridge cost is accepted depending on the importance of the roadway. The piers, if placed in deep water, are designed to be rigid, i.e., the total collision energy must by absorbed by movement or deformation (damage) of the ship.

It is true that collision forces of a ship can reach 34,000–68,000 tons (300–600 MN), in which case the reasonable design of a rigid pier is impossible, but a high proportion of the actual collisions with a proposed bridge would involve smaller vessels and lower impact velocities. Small vessels up to 5,000 tons displacement, especially when traveling light and without ballast, are very often involved in pier collisions. For instance, three ships and seven barges of this kind collided with the Sunshine Skyway Bridge, in Florida. Therefore, a requirement that all outbound light vessels meet some minimum ballast criteria has been suggested in Florida. For such types of collisions, providing an adequate protection is practicable, even for main piers. Lower piers in bridge approaches of normal design often can withstand the probable impact forces. This is true especially when the dead load of the superstructure is considerable.

To reduce the length of the present paper, its main purpose is the actual, practical design of piers for ship impact. Therefore, the pier protective devices, the determination of the design ship, and the equivalent statical load are only briefly discussed.

Pier Protection Systems

An economic examination of using pier protection devices in lieu of designing the substructure for impact forces should be done with an analysis based on realistic data, otherwise the conclusions will be wrong. One should always remember that more data do not necessarily mean more accuracy.

In choosing the best-suited pier protection devices, a well-organized decision process must be followed. This process is based on current and expected vessel traffic, risk and cost analysis, safety, constructability of the protective system, and its environmental impact. Protective systems in general may be separated from the pier they protect, either being connected to river- or seabed, like independent fender systems, dolphins cofferdam cells, or man-made islands, or floating around the pier anchored by prestressing and interconnecting tendons or chains, producing a shock-absorber screen to intercept off-course vessels. Another kind of protective system is connected directly to the pier as a multiframed buffer structure of various configurations.

All protective systems are means to reduce the consequences of collisions. By their energy-absorbing capability they help to reduce the energy absorbed by the ship (actually, the damage) and eventually, if the ship is not completely stopped, that part of the impact that the pier itself has to absorb. Fenders and dolphins will in many cases be found completely out of scale with the energies to be handled. Fender systems function more by guiding the vessel than directly protecting the pier by its energy absorption. Analytical techniques to design fender systems and dolphins can be found in papers by Jiang and Jamava (1983) and Heins (1983) and in the book by Derucher and Heins (1978).

Cofferdam cells consisting of circular sheet piling filled with gravel and braced by a top slab may form efficient and relatively inexpensive protection, provided a firm bottom is available at reasonable depth.

For many shallow-water bridge piers man-made islands around the pier are an economical solution. Protection against wave damage is usually required, and may be provided by rock or prefabricated armoring units [for the design of islands see Saul and Svenson (1981) and Denver (1983)]. The dimensions and costs of islands increase rapidly with the water depth, and the subsequent reduction in water section may make them an unacceptable solution. In such cases a floating protection system may be a solution, although further development is required to render such systems fully reliable.

Buffer structures attached to a water pier are usually box-type steel structures with stiffeners, capable of absorbing the collision energy by crushing itself, thus reducing the damage to the ship and protecting the pier. The load-deformation relation and the energy-absorbing capacity of the structure is developed by means of inelastic large-deformation analysis (Namita and Nakanishi 1983).

Ship Impact Criteria

If piers have to be designed for ship impact, either for static or dynamic analysis, the design ship and design criteria have to be determined. From the records of the existing and projected boat traffic under a bridge and the regulations for the operation of merchant vessels, the most frequent, largest type of the vessel has to be found. The main vessel characteristics are length L, width (or beam) B, draught D, and load carrying capacity dwt, in long tons. Besides this carrying capacity (cargo weight) in deadweight tons (1 dwt = 2.24 kip = 9.96 kN), sometimes two other ship characteristic values are given for passenger ships, its volume in gross registered tons (1 GRT = 100 cu ft = 2.83 m^3) and the water displacement in long tons (1 metric ton = 1 m^3 water = 2.240 kips). Fig. 1 is the correlation diagram of dwt, GRT, and metric tons [adapted from Saul and Svenson (1982) page 36].

Based on tidal flow and/or current velocities, ship operating characteristics, and information from local bay or river pilots, the most probable

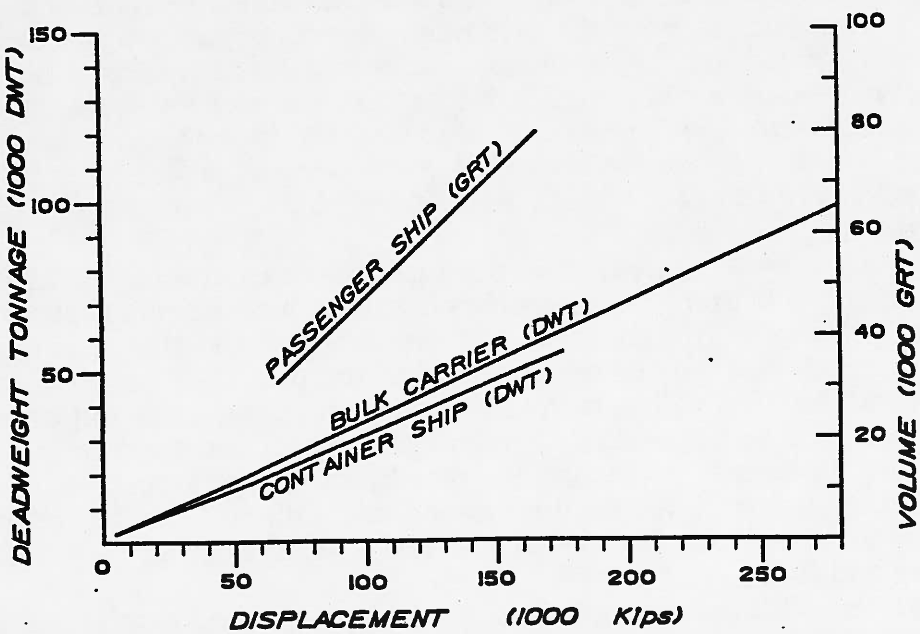


FIG. 1. Relation between Displacement (kips), Load-Carrying Capacity (*dwt*) and Volume (Passenger Ships) (GRT) for Various Types of Ships

speed of the vessel is estimated, from which the design ship and design criteria are obtained.

Several methods and procedures are available to calculate the equivalent impact force: The Louisiana manual (*Criteria* 1985), *Interim Proposal* (1988) by the Special Task Force of the AASHTO Bridge Committee (1988), Saul and Svensson (1982), and Woisin and Gerlach (1970), among others. The Louisiana manual is the most extensive developed to date, and is the best suited for Florida of the aforementioned publications.

A pier designer assumes that the kinetic energy of the ship impact and the hydrodynamic supplementary mass (in ship's longitudinal direction about 5%, and in lateral about 40–50% of the ship displacement) must be absorbed by crushing of the ship hull and hydrodynamic damping. The impact force developed in a ship's collision depends upon the deformation resistance of the bow and lateral ship parts. It depends also on the degree of ballast water filling the fore peak. A water filling, due to its incompressibility, stiffens the bow and therefore shortens the length of the damaged ship's hull. This stiffness increases the average impact force up to 50%. This event is actually responsible for the scatter in force magnitude. From collision tests it is known that after 0.1–0.2 s the impact force reaches its maximum value, equal to about twice the average, medium force P_m. This medium force is approximately constant during the collision, and there is not rebounding of the ship; i.e., ship and pier are in contact all the time (about 1 s). Fig. 2 shows such a force-time diagram, adapted from the results of some model tests by Woisin and Gerlach (1970).

Saul and Svensson (1982) summarized the theory of ship collision against bridge piers. For the maximum force of a right-angle impact of bulk carriers against a stiff pier they gave the following expression:

$$P_{max} = 0.88\,(\text{dwt})^{1/2} \pm 50\% \qquad (1)$$

where weight of cargo dwt is in long tons, and the force is in MN (1 MN = 4,448.22 kips); Fig. 3 illustrates (1).

The Louisiana manual (*Criteria* 1985) gives collision forces for each class

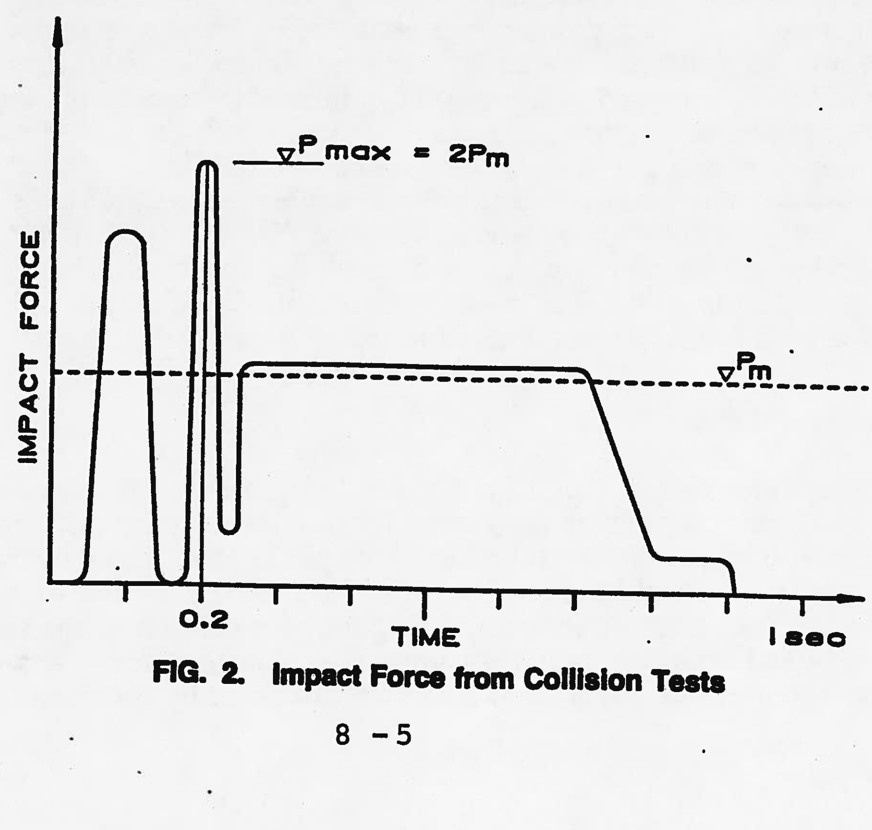


FIG. 2. Impact Force from Collision Tests

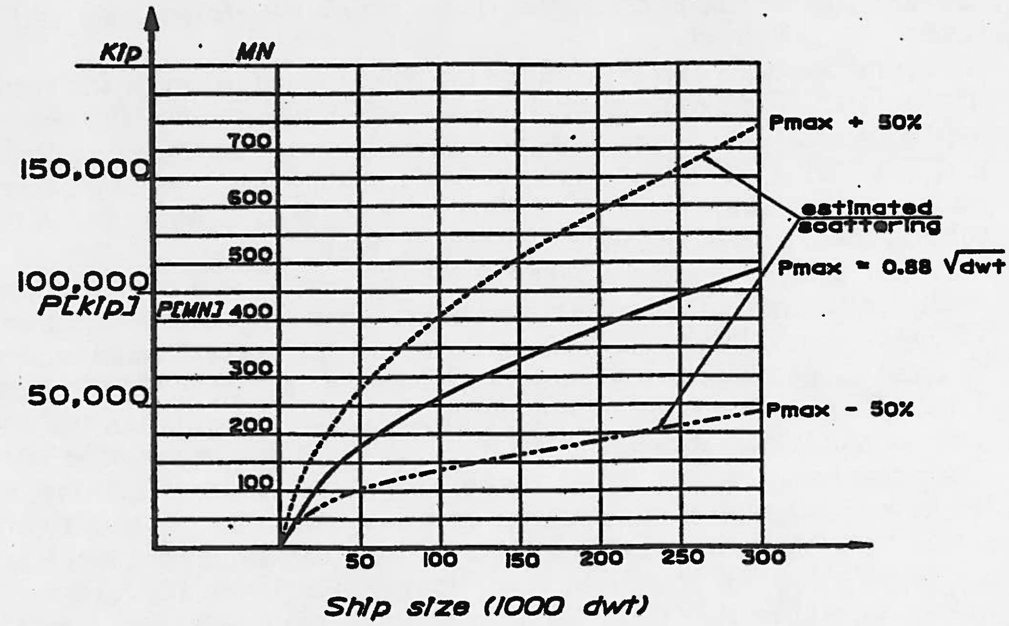


FIG. 3. Approximate Relation between Impact Force and Cargo Ship Capacity (*dwt*) for Bulk Carriers

of waterways as a function of the water depth at the location of the pier and, in the case of deep-draft waterways, as a function of the design ship selected and a typical average speed.

Once the force is determined, its distribution on piers with respect to their distance from the navigational channel has to be found. The basis for this variation in the force magnitude is the assumption of an acceptable risk. Both the Louisiana manual and the AASHTO proposal (*Interim* 1988) distribute the impact force in this way.

According to the force magnitude, the decision is made whether or not to use pier protection. The degree of protection must be a compromise between the acceptable risk and the cost of construction. The best procedure is to consider separate pier designs, with and without ship impact, in order to be able to find the increase in pier cost due to the ship impact. On the basis of this difference a comparison can be made between the approximate costs of protection systems that are compatible with the local navigational requirements, and the correct decision can be reached.

Normally, smaller equivalent ship impact forces, below about 3,000 kips (13.5 MN), cannot warrant economical use of a special protection system. In such instances, the piers have to be designed for the loading case of dead load and ship impact. This load group is considered as the ultimate load, and for both loads the multiplication factor is unity.

Pier under Impact

The components of a pier system (i.e., single or double column or shaft with cap beam, supported on a large footing founded on piles plus the soil and surrounding water) act together as the pier is accelerated by the colliding mass until it is brought to rest. By its dimensions, the footing should prevent a colliding ship from directly impacting the pier column or the shaft, which are designed only to transfer a part of the impact force shared by the superstructure, but never to withstand a direct hit by the vessel.

The depth of the footing should be sufficient to effectively prevent the rotation of the top ends of the piles and to supply the required development length (in case of prestressed concrete piles) such that the full ultimate-bending-moment capacity of piles is available at the bottom of the footing. This large mass of the footing block is also very helpful in reducing vibrations. Dual-shaft piers are usually connected either at the top by a common pier cap or at the waterline by the footing or a strut enabling the two columns or shafts to act as a frame.

For partial transfer of lateral force to the superstructure, i.e., of transverse shear force, "positive steel or concrete connections of superstructure to substructure" (*Structures* 1987) must be provided. This can be accomplished in a variety of ways, usually in the form of keys.

For partial transfer to the superstructure of the longitudinal component of the impact force, the shear capability of the bearings must be adequate. For the longitudinal component of dead load and ship impact, the shear forces and corresponding displacement of bearings should be checked and proven to be at or below the ultimate values, as shown later in this paper. If the ultimate capacity is exceeded, or the bridge bearings are not elastomeric pads and have little or no longitudinal shear capacity, then the pier must take the total longitudinal force and in this direction for the design the pier is treated as a cantilever.

Pile Load Capacities

The type, size, and number of piles depend on the loading involved and the foundation ground-soil profile. For Florida coastal bridges the soils are normally a cohesionless mixture of silt, sand, gravel, broken coral reefs, sand, or limestone, with their hard zones found in two or more layers. The top layer is usually of variable thickness along the bridge alignment. If a pile penetrates this layer before reaching the desired capacity (about 5–10 blows per inch penetration), much longer piles will be required, resulting in problems for pile handling and driving.

Most frequently in Florida, prestressed square concrete piles of 18 in. (46 cm), 24 in. (61 cm), and 30 in. (76 cm) are used. An economy analysis would determine which size to use. Ship impact can sometimes change this analysis, showing that heavier piles are more economical, particularly if only plumb piles in a footing are used. Sometimes drilled shafts are a viable option that should be explored, though the unit price for a drilled shaft constructed in the water is substantially higher than if constructed on the ground. In the first approximation, plumb piles always have an advantage in comparison with battered piles, provided that their lateral capacity is sufficient. The lateral stability of plumb piles is achieved through bending, with pile axial forces for the service dead load (DL) + ship impact being mostly compressive, i.e., the uplift and pile tensile connection capacity are not primarily important. This advantage is offset by about 20% larger bending moments. If battered piles are used to reduce these moments, some piles on the impacted side of footing will have very large tensile forces, usually far beyond their uplift capacity. If the analysis stops there, the obvious conclusion is that this configuration of battered piles is unsafe and that the plumb-pile configuration is preferable. Because this contradicts the good practice of using battered piles for better stability, it is worthwhile to investigate further the events that take place after this first phase. The following computerized approach gives fairly accurate, fast answers to this question.

The impact force from the collision is approximately 1 s in duration (Fig. 2). The force produces a mechanical disturbance through the pier system, which travels with the speed of the sound [12,000–15,000 ft/s (3,500–4,500 m/s)] through concrete. For typical dimensions of piers and footings this means an instantaneous action (1–2 ms). Piles under large tension forces are not pulled out from the ground, because the rigid footing prevents this. They only become useless to receive any further tension due to the already destroyed friction between piles and soil. They are not broken either, because the peak tensile forces, obtained after the first computer run, never materialize. When the pile force reaches the friction capacity it drops practically to zero. Therefore, their bending stiffness is still present for helping other piles, and such a pile is modeled for the next computer run as having a very small cross-sectional area but full moments of inertia.

This "pulling out" of some piles requires a new equilibrium distribution of forces among the remaining number of active piles in the second phase. If these new forces are again beyond the tensile capacity of some piles, an additional number of piles becomes eliminated. This process is then repeated until either the stability is reached, when no new failures in tension of piles occur, or the collapse of this configuration of piles takes place. Thanks to the high speed of pressure-wave transmission through the pier systems all these events of force redistribution among the piles have time to develop within the 1 sec or so of the collision, especially because all the deformations in piles are elastic, and, therefore, also instantaneous. Although plastic deformations require some time to be fully developed, the described scenario is certainly realistic, and it is very easy to model for computer applications considering the attempts to have plastic analysis applied to the impact phenomena.

If such computer applications are used in comparing plumb and battered piles, a configuration with a sufficient number of battered piles may prove to be more economical than a configuration of the same number of plumb piles, because the bending moments are reduced. In any case, both configurations must be studied and adequately treated, as shown in the example to follow. It is true that large tensile pile capacity in typical Florida soils is achieved with quite a large pile length; but a low-value capacity (equal to about 8–10% of the pile bearing capacity) is always achieved in piles of normal lengths of soil embedment, 30–40 ft (9–12 m). This low capacity is thus practically always available. Therefore, the described process can start and the actual tension forces in piles are very small: no cracks develop in the piles during force redistributions; as already said, the peak forces obtained by computer never materialize. This is quite similar to the high stresses far beyond a material's yield stress obtained when analytically treating notch effects that consider the material to be elastic without limits. The peak stresses are not "gradually" reduced as sometimes said, because they never occurred in the first place.

Transverse Impact Load Distribution

To find out how much of the transverse impact force is taken by the impacted pier with plumb or batter piles and how much is transferred to the superstructure, the stiffness of both must be known. For that purpose two analytical models are used. One is a three-dimensional (3-D) model of the complete pier under the consideration and the other is a two-dimensional (2-D) model of the superstructure projected on a horizontal plane. If the concrete strengths of the super- and substructures are different, one material is chosen for both models. Given the Young's moduli of the material of superstructure is E_s and the pier is E_p, properties (areas and moment of inertias) are adjusted; for example, to the material used for pier by multiplying A_s and I_s of superstructure with the ratio E_s/E_p. The same is valid for piles. For the 3-D model only one item needs discussion, i.e., how to find the fixity points of piles. For this purpose it is best to use the COM 624 computer program, provided that the required soil parameters, needed for the input into the program Reese (1984) are available from a complete approved geotechnical foundations report. The program also takes care of the group effect of piles by lumping all piles into one fictitious pile. If the detailed data are not available at the time of design, or in the case of preliminary design, the procedure of Davisson (1970) for partially embedded piles can be used. The value for the spring stiffness or subgrade modulus has to be an educated guess in that case, based either on previous experience or supported by the literature.

The 2-D model represents the whole superstructure in a horizontal plane as a beam, straight or curved depending on the actual bridge geometry,

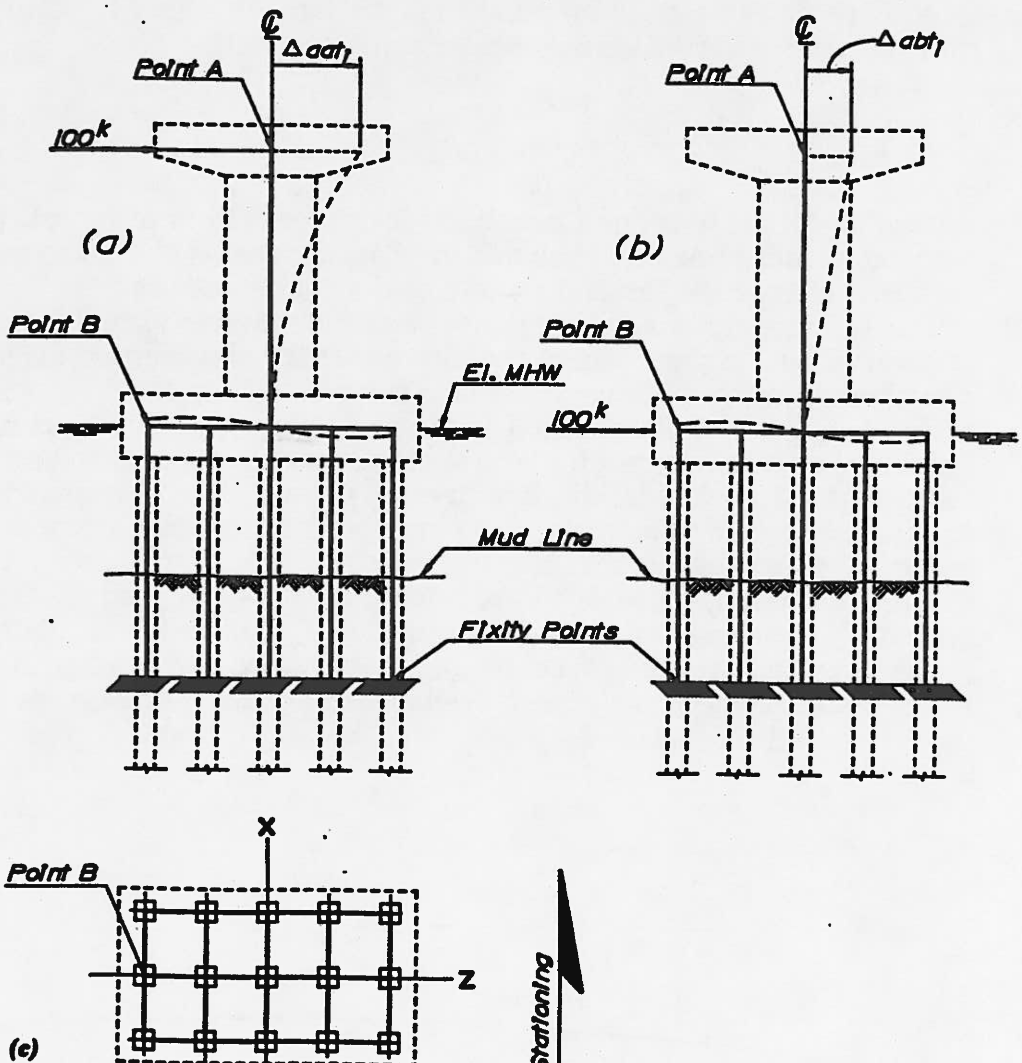


FIG. 4. 3-D Pier Model Shown in Front Elevation with Transverse Virtual Forces Applied: (a) At Point A; (b) At Point B

with the moment of inertia of the superstructure referred to a vertical axis on the center line of the roadway. If intermediate expansion joints are present in the roadway of the bridge superstructure, the whole length of the bridge can be considered for the 2-D model by introducing hinges in the horizontal beam at these locations. These hinges are easily modeled by introduction of fictitious structural members of very short length and small moment of inertia but very large cross-sectional area so that they cannot transmit any bending forces, only shearing forces. The beam is supported at pier locations by elastic horizontal fictitious columns (or springs) of arbitrary length but with a cross-sectional area such that the longitudinal flexibility of the column is equal to the transverse flexibility of each pier. The flexibility of all piers is obtained from the 3-D models of corresponding piers using virtual forces of substantial magnitude [100 kips (445 kN) or more] to avoid errors due to ratios of small numbers. Virtual forces are applied at points A and B (Fig. 4) successively, and the corresponding displacements of point A, Δaat_i, and Δabt_i of pier i are found. Flexibility coefficients (displacements per unit force at the point A) when forces are acting first at A and then at B are $\Delta aat_i/100$ and $\Delta abt_i/100$, respectively. From $\Delta aat_i/100$, the cross section of the fictitious column i, A_i is obtained using E of the same material chosen for the models, either the one for the substructure (as shown earlier) or the superstructure

$$A_i = \frac{100l}{E\Delta aat_i} \quad \text{(2)}$$

where l = arbitrary column length, 50 ft or more (15-m or more). For the moment of inertia of that column a very small value is chosen so as not to artifically change the flexural rigidity of the superstructure.

The 2-D model is loaded only once, with a transverse virtual force acting at point A, at the place where the pier under the consideration is located. The column in the model at that place is omitted (Fig. 5). A displacement of point A, Δst_i is thus obtained, and the flexibility coefficient is $\Delta st_i/100$. Point A is a common point for both substructure and superstructure. Therefore, the total horizontal displacement of point A due to impact force H obtained from the substructure is equal to the true displacement of the superstructure due to the transmitted part of force H.

If the actual transverse ship impact force is equal to H, and if it is applied at point B, then a part of it ΔH_{ti} is transmitted by pier i to the superstructure. In other words, the superstructure will restrain the pier at point A by this force. By equating the total true displacement of the pier at point A from the 3-D model (i.e., $H \cdot \Delta abt_i/100 - \Delta H_{ti} \cdot \Delta aat_i/100$) to the displacement of

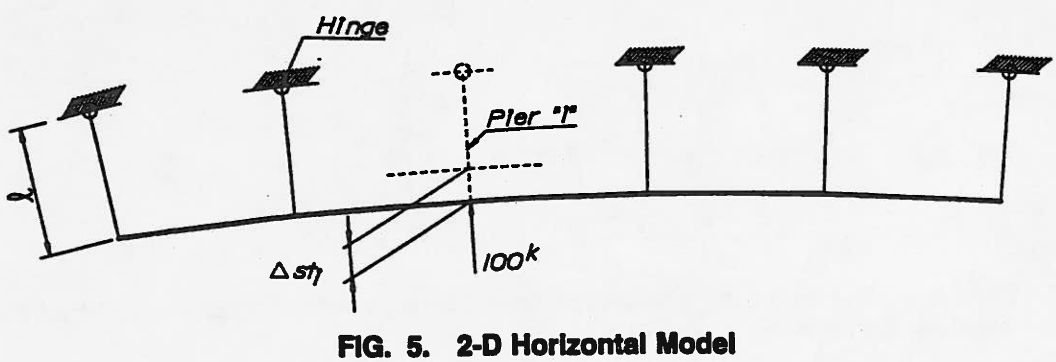


FIG. 5. 2-D Horizontal Model

the superstructure $\Delta H_{ti} \cdot \Delta st_i/100$, the magnitude of the force transmitted to the superstructure ΔH_{ti} is obtained as

$$\Delta H_{ti} = \frac{\Delta abt_i}{\Delta aat_i + \Delta st_i} H \qquad (3)$$

This process is repeated for all piers over water. This transmitted force acts at point A in the 3-D model in the direction opposite the ship impact because it represents the superstructure's support of the pier. To get the forces and moments in any pier structural member due to DL and transverse ship impact H, both forces, H and ΔH_{ti}; besides DL, are simultaneously applied at points B and A in the 3-D model.

Longitudinal Impact Load Distribution

The longitudinal impact load distribution is shown under the assumption that elastomeric neoprene pads (of various thicknesses t_i but of the same nominal hardness of neoprene) are used as bridge bearings. Otherwise, bearing that are free to move are used, due to their very small friction no participation of the superstructure can be assumed in longitudinal forces. Point A, i.e., the contact point at the center line between the super- and substructures is now at the level of the neoprene pad's top. Therefore, the total virtual displacement in longitudinal direction of point A, if first considering the substructure of pier i, is composed of the virtual displacement of the pad, Δpad_i, and of the pier, Δaal_i. Fig. 6 shows the pad displacement. If the virtual now longitudinal force applied at A is again 100 kips (445 kN) and there are n pads per pier, then the virtual shearing force per pad is $100/n$. If G is shear modulus and γ is shear strain of the neoprene, then the virtual pad displacement is equal to

$$\Delta \text{pad}_i = t_i \gamma = \frac{100 t_i}{n G l_i w_i} \qquad (4)$$

where l_i and w_i = pad's length and width, respectively, at pier i. Fig. 7 shows the virtual-displacement component of the pier, Δaal_i [Fig. 7(*a*)] and Δabl_i [Fig. 7(*b*)]. Therefore, the total virtual horizontal longitudinal deflection of point A of pier i, $\Delta' aal_i$, is obtained by summing both displacements

$$\Delta' aal_i = \frac{100 t_i}{n G l_i w_i} + \Delta aal_i \qquad (5)$$

This deflection is found for each pier of the whole bridge, irrespective of

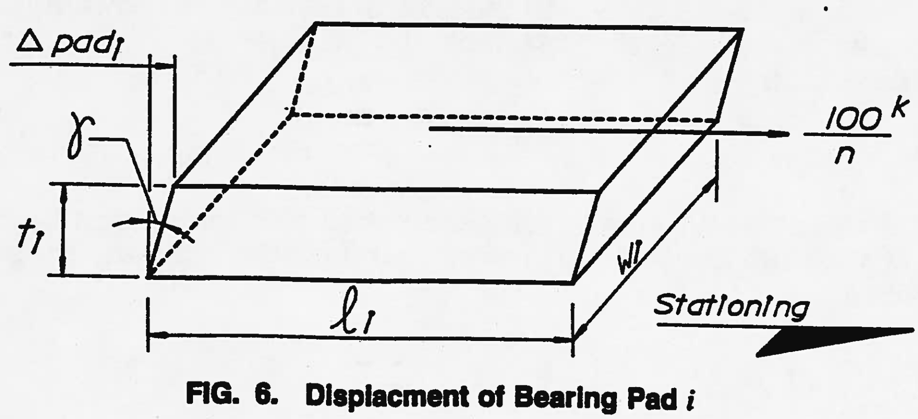


FIG. 6. Displacment of Bearing Pad *i*

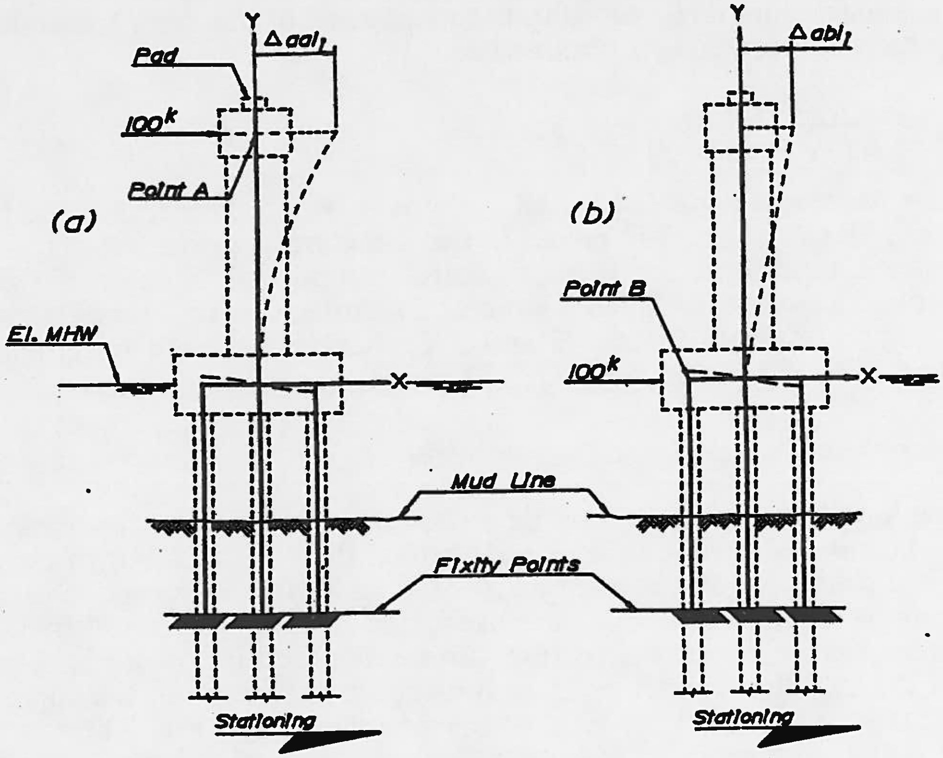


FIG. 7. 3-D Pier Model Shown in End Elevation with Longitudinal Virtual Forces Applied: (a) At Point A; (b) At Point B

the pier location, whether over ground or water. The flexibility coefficient in longitudinal direction of any pier f_i is then

$$f_i = \frac{\Delta' aal_i}{100} \quad \text{.......} \quad (6)$$

Now, consider the superstructure. When a virtual longitudinal force of 100 kip (445 kN) is applied on the superstructure at the location of an impacted pier, the corresponding virtual displacements of all piers must be the same and equal to Δsl, because the superstructure can be assumed as absolutely rigid in the longitudinal direction. Because the pier flexibility coefficients f_i for each pier are different, depending upon the different pad sizes and pier longitudinal flexibilities the forces exerted by the superstructure on the pads and piers must also be different, varying from pier to pier. If these virtual forces are in general hl_i, then the corresponding displacements $f_i hl_i$ must be all constant and equal to each other. Therefore, any such force hl_i can be expressed in terms of only one unknown force acting at any particular pier. If for example the first pier ($i = 1$) is chosen, then it follows that:

$$hl_i = \frac{f_1 hl_1}{f_i} \quad \text{.......} \quad (7)$$

For equilibrium, the sum of all these virtual pier forces must be equal to the applied 100 kip (445 kN) force. Therefore the following expression is obtained:

$$100 = \sum_{i=1}^{m} hl_i = f_1 hl_1 \sum_{i=1}^{m} \left(\frac{1}{f_i}\right) \quad \text{.......} \quad (8)$$

where m = total number of piers, either in the bridge or the unit. From there, the chosen unknown virtual force hl_1 is equal to

$$hl_1 = \frac{100}{f_1 \sum_{i=1}^{m} \left(\frac{1}{f_i}\right)} \quad \text{(9)}$$

Therefore, knowing hl_i, the constant virtual longitudinal displacement of the superstructure Δsl using (6) is

$$\Delta sl = \frac{hl_i \Delta' aal_i}{100} = \text{constant} \quad \text{(10)}$$

Thanks to this constant displacement, there is no need to put 100 kips (445 kN) longitudinal force on the 2-D model and obtain the displacement using a computer.

If the longitudinal component of the ship impact is H_{li} (one-half of the full ship impact force H) (Fig. 8), one part of it, ΔH_{li}, is taken by the superstructure; i.e., it restrains pier i in the longitudinal direction by a force $-\Delta H_{li}$. To get this force we write again that displacements of the top of the pad and the superstructure are the same. Total displacement of the top of the pad under the action of these two forces, H_l and ΔH_{li}, at any pier i is

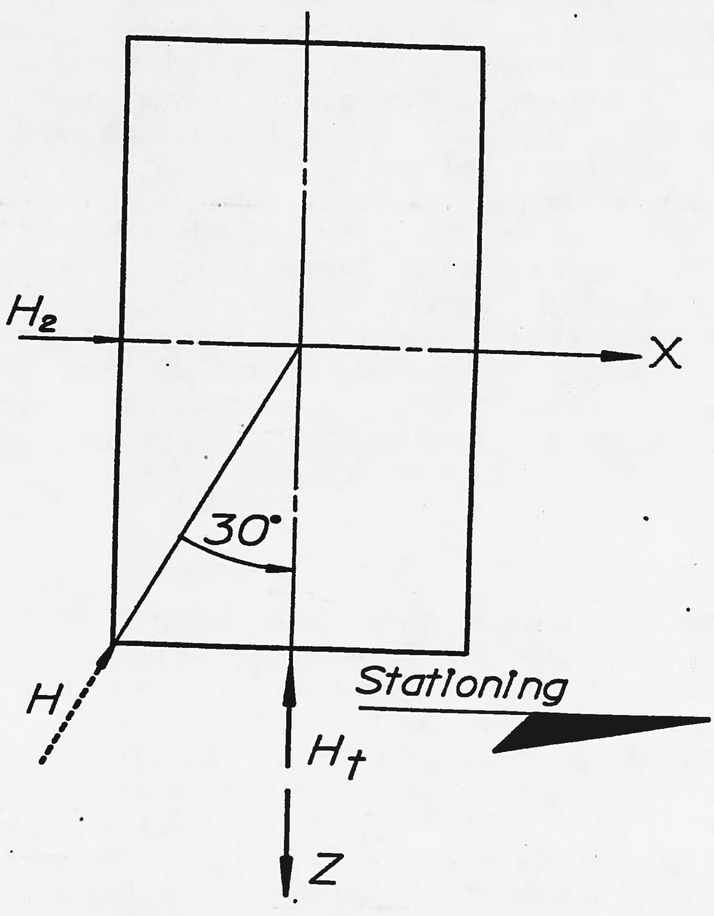


FIG. 8. Plan View of Pier Footing under Skew 30° Ship Impact

$$H_t \cdot \frac{\Delta' abl_i}{100} - \Delta H_{li} \cdot \frac{\Delta' aal_i}{100} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

The corresponding longitudinal superstructure displacement, using (10), is

$$\frac{\Delta H_{li} \cdot \Delta sl}{100} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

Equating expressions (11) and (12) and solving for the unknown force ΔH_{li}, we have

$$\Delta H_{li} = \frac{\Delta' abl_i}{\Delta' aal_i + \Delta sl} \cdot H_l \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (13)$$

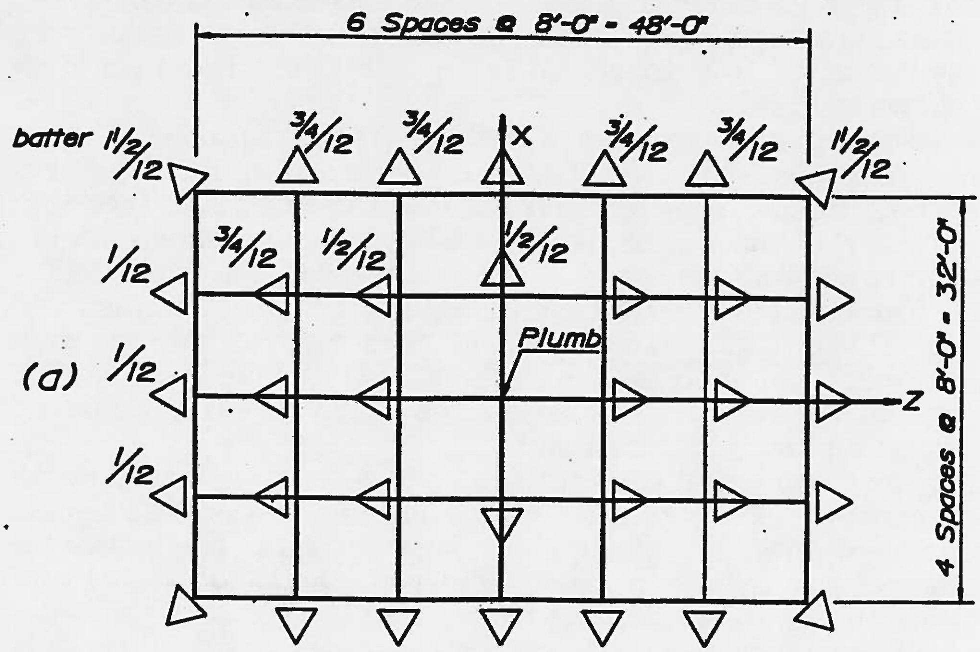
This is repeated for all water piers.

3-D Iteration Procedure

In the case of battered piles, when the restraining forces exerted by the superstructure on pier top ΔH_{ti} [(3)] and ΔH_{li} [(13)] for the pier i under consideration are known, full areas and moments of inertia of all piles are input in the first iteration cycle for (DL + ship impact) loading. After this first run, some of piles will fail in tension; i.e., their axial tensile force is larger than twice the given service load pile friction capacity in tension. Areas of such piles for the second cycle are then reduced to a small value (but different from zero!), about 0.01 sq ft (0.001 m^2) for the second cycle and input. This is repeated in the same way for each subsequent cycle until either there are no more pile failures or the collapse of the system is reached. In the first case, the compression axial forces and moments increased by the $P \cdot \Delta$ effect in the remaining piles must be checked. An interaction P-M diagram of the pile used in the design should be constructed and extreme pairs of values (for forces and corresponding moments) checked. If acceptable and none of the pile compressive forces is larger than twice their service load-bearing capacity, the most loaded pile is checked for the buckling stability. The pile is treated as a beam-column with the boundary conditions corresponding to the realistic design assumptions [see Davisson (1965) and Siva (1970)]. Through the iteration cycles the compression in remaining piles actually diminishes, because large tensile pile forces require corresponding large compressive pile forces for the sake of equilibrium. When these tensions are shaken down the compression in remaining piles is, of course, reduced.

Example

As an illustration of the comparative piling design for dead load and ship impact, the design of pier No. 56E (channel pier) of Howard Frankland bridge, Hillsborough and Pinellas County, Fla., is shown in Fig. 9. Both pile configurations, i.e., battered and plumb piles have 35–30 in. (76 cm) square concrete piles prestressed with 32 × 9/16 in. (14 mm) diameter Lo-Lax strands. The ship-impact static equivalent force acting at the mean high-water level (MHW) load is 2,000 kips (8.9 MN). A ship impact acting at a skew angle of 30° controls the design. The superstructure restraining force at pier No. 56E in the direction transverse to the bridge is 27% of the transverse ship impact component of 1,732 kips (7.70 MN), or 468 kips (2.08



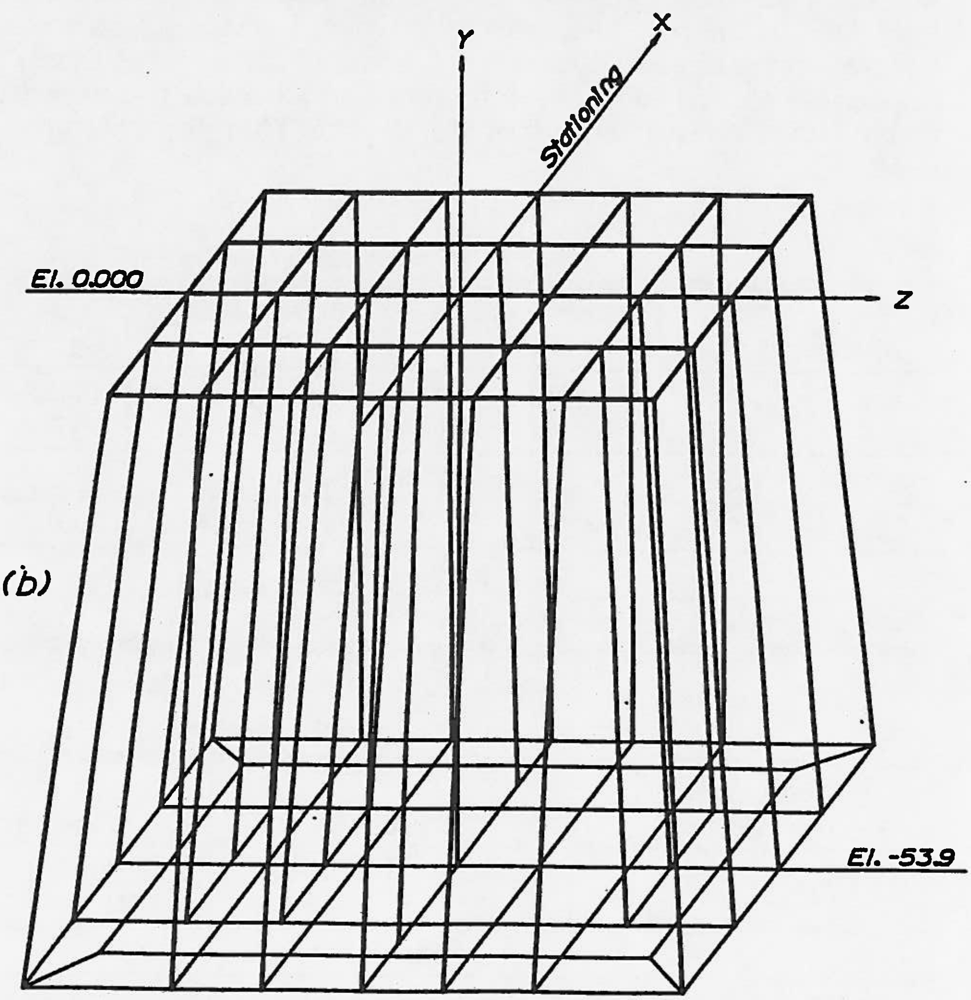


FIG. 9. Pier No. 56E: (a) Pile Locations for Battered Piles at El. 0.00; (b) Piling Shown in Elevation

MN) and 0% of the component in the longitudinal direction; i.e., the pier has to take total H_l or 1,000 kips (4.45 MN). Pile locations for the battered system at El. 0.00 are shown in Fig. 9(*a*); the pile system in elevation is shown in Fig. 9(*b*).

Plumb piles are spaced at 8.00 ft (2.44) center to center. The pile ultimate bearing capacity is 1,200 kips (5.34 MN) and in uplift −80 kips (−0.36 MN). The equivalent length of partially embedded piles (actually the depth to fixity) obtained from the COM 624 computer program was in very good agreement with the results of the procedure of Davisson (1973).

The battered-pile system required five cycles of iteration, with 15 piles (or 42.9%) failing in uplift. Plumb piles required only one cycle because five piles had uplift but below the ultimate tensile capacity. The battered system had a larger pile load and the plumb system had a larger moment, as shown in next Tables 1 and 2.

The compression pile forces in both systems are far below the ultimate capacity [1,200 kip (5,338 KN)]. Therefore, it is of little significance that battered piles have about 82.6% greater forces. The increase in bending moments of the plumb piles, although only 21.6%, was of concern. The ultimate flexural capacity of the piles at the bottom of the footing with 32 Lo-Lax strands of 9/16 in. (14 mm) diameter was only 934 ft·kip. (1,266 kN·m), i.e. just slightly over the required capacity for battered piles, but only 92% of the required capacity for plumb piles. Strand spacing, with a concrete cover of 4 in. (10 cm) is at 2 3/4 in. (7 cm) average. It was practically impossible to add more strands, required to reach the capacity of 1,133 ft·kip (1,536 kN·m) needed for plumb piles. Therefore, battered piles were used.

TABLE 1. Comparative Results

Maximum Force		Corresponding Moment		Corresponding Force		Maximum Moment	
(kip) (1)	(kN) (2)	(ft-kip) (3)	(m·kN) (4)	(kip) (5)	(kN) (6)	(ft-kip) (7)	(m·kN) (8)
(*a*) Plumb Piles							
260.0	1,156.5	1,064.72	1,443.59	260.0	1,156.5	1,064.72	1,443.59
(*b*) Battered Piles							
474.8	2,111.8	880.84	1,194.28	0	0	931.89	1,263.49

TABLE 2. Comparative Results

Corresponding Displacement		Effect $P \cdot \Delta_z$		Final Moment	
(ft) (1)	(m) (2)	(ft-kip) (3)	(m·kN) (4)	(ft-kip) (5)	(m·kN) (6)
(*a*) Plumb Piles					
0.262	0.080	68.12	92.36	1,132.84	1,535.95
(*b*) Battered Piles					
0.209	0.064	0	0	931.89	1,263.49

Conclusions

As a summary of the described design process, the following points are emphasized.

Navigational records of ship sizes, speeds, and frequency of passages under a bridge and the profile of the water bed, currents, and environmental conditions in the vicinity of a bridge site must be studied to get a realistic assessment of the magnitude of the design ship-impact force.

The ship-impact force should be determined by trying several available procedures. The final choice of the best-suited value is made by engineering judgment.

The distribution of the calculated impact force on piers along the pier alignment should be made.

Before starting actual bridge design a decision has to be made on whether to use some pier protective system or to let the substructure receive the collision impact. A comparative cost analysis of the best-suited protective system and the additional cost of bridge substructure if the system is not used should be made.

Based on the relative stiffnesses of a pier and the superstructure, the sharing of ship-impact forces between them should be found for each pier of concern.

Plumb piles are often a more economical foundation solution than a system of batter piles, but the pile flexural capacity should be checked at the bottom of a footing and compared to the required flexural capacity.

In case of large moments, battered piles could be a better solution.

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Appendix II. Notation

The following symbols are used in this paper:

A_l = cross-sectional area of fictitious horizontal column at pier i;
A_s = cross-sectional area of superstructure;
dwt = dead weight tonnage, cargo weight capacity of ship in long tons;
E_p = Young's modulus of pier material;
E_s = Young's modulus of superstructure material;
f_i = flexibility of top of pad in longitudinal direction (displacement due to unit force);
H = ship-impact force, H_{li}, H_{ti} = in longitudinal and transverse directions, respectively, acting on pier i;
hl_i = longitudinal virtual force at top of all pads on pier i due to constant superstructure displacement;
I_s = moment of inertia of superstructure referred to vertical axis at its center line;
l = constant length of fictitious horizontal column;
l_i = length of neoprene pad (in direction of stationing);
n = number of neoprene pads on one pier;
T, t = long ton;
t_i = thickness of neoprene pad;
w_i = width of neoprene pad;
X, Y, Z = global coordinate axes;
γ = shear strain;
Δaa_i = virtual displacement of pier top of pier i, $\Delta aali$ = in longitudinal direction, Δaat_i = in transverse direction when force is acting at pier top;
$\delta \mathrm{pad}_i$ = virtual displacement in longitudinal direction of top of neoprene pads on pier i due to force action applied at top of neoprene pad;
Δsl = constant virtual longitudinal displacement of superstructure at any pier;
Δst_i = transverse virtual displacement of superstructure at pier i; and
Δaal_i = total virtual longitudinal displacement of top of neoprene pad on pier i.

The following are additional figures from B. O. Kuzmanovic's presentation that were not included in the original paper.

PLASTIC DESIGN OF BRIDGE PIER FOUNDATIONS FOR SHIP IMPACT

by
B. O. Kuzmanović

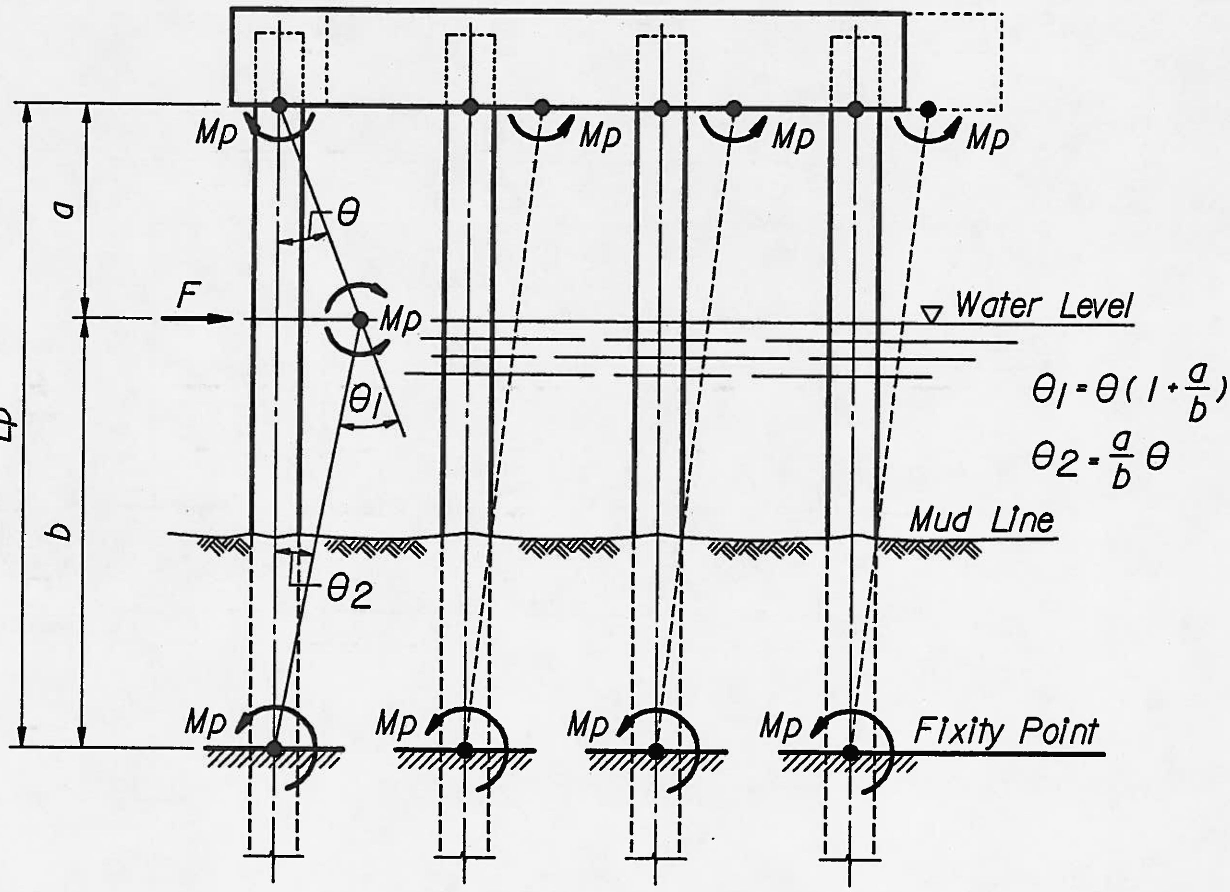


FIG. 1 - PILE BENT COLLAPSE

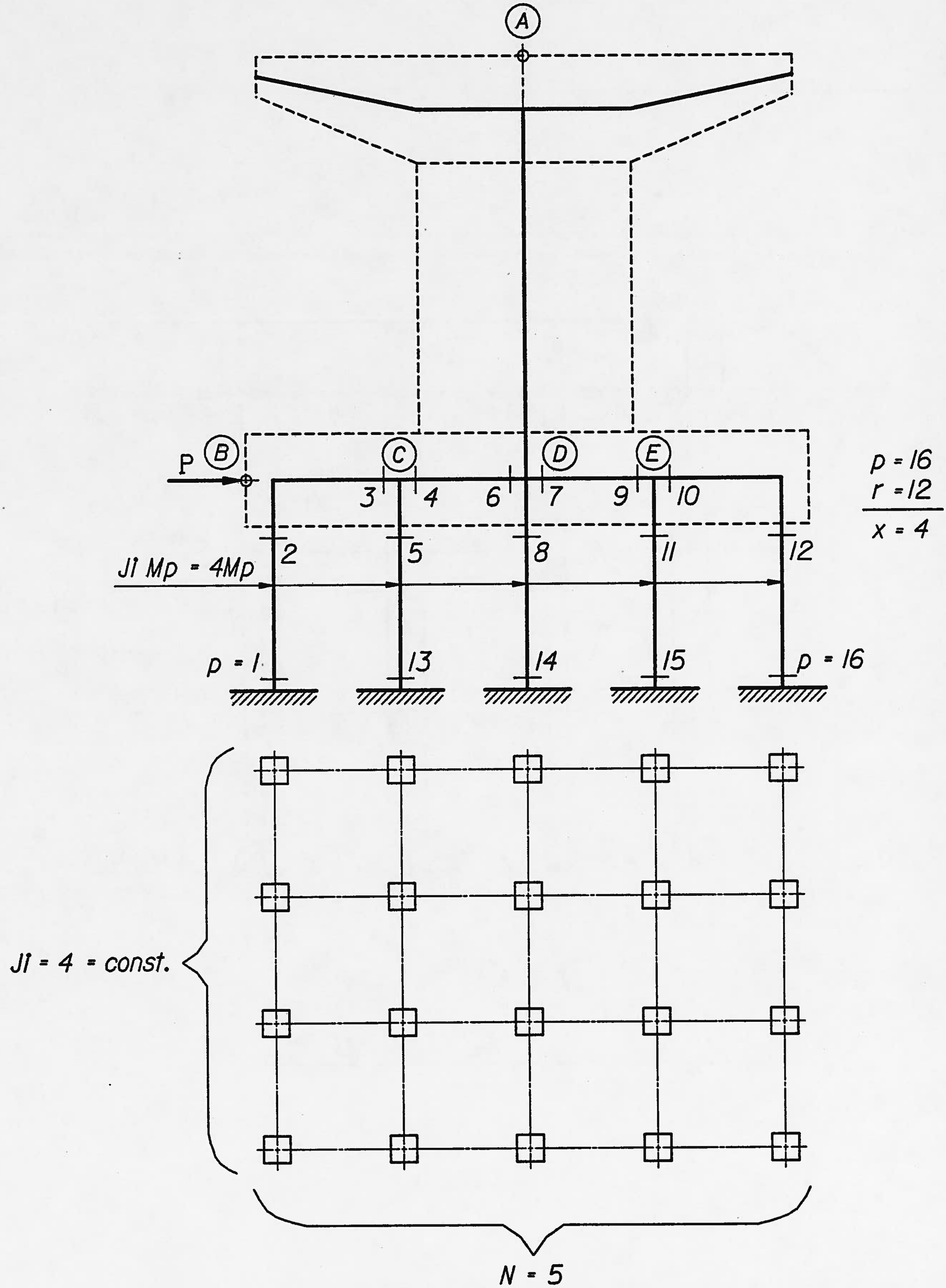


FIG. 2 - POSSIBLE SECTIONS FOR HINGE FORMATION IN THE EQUIVALENT FRAME

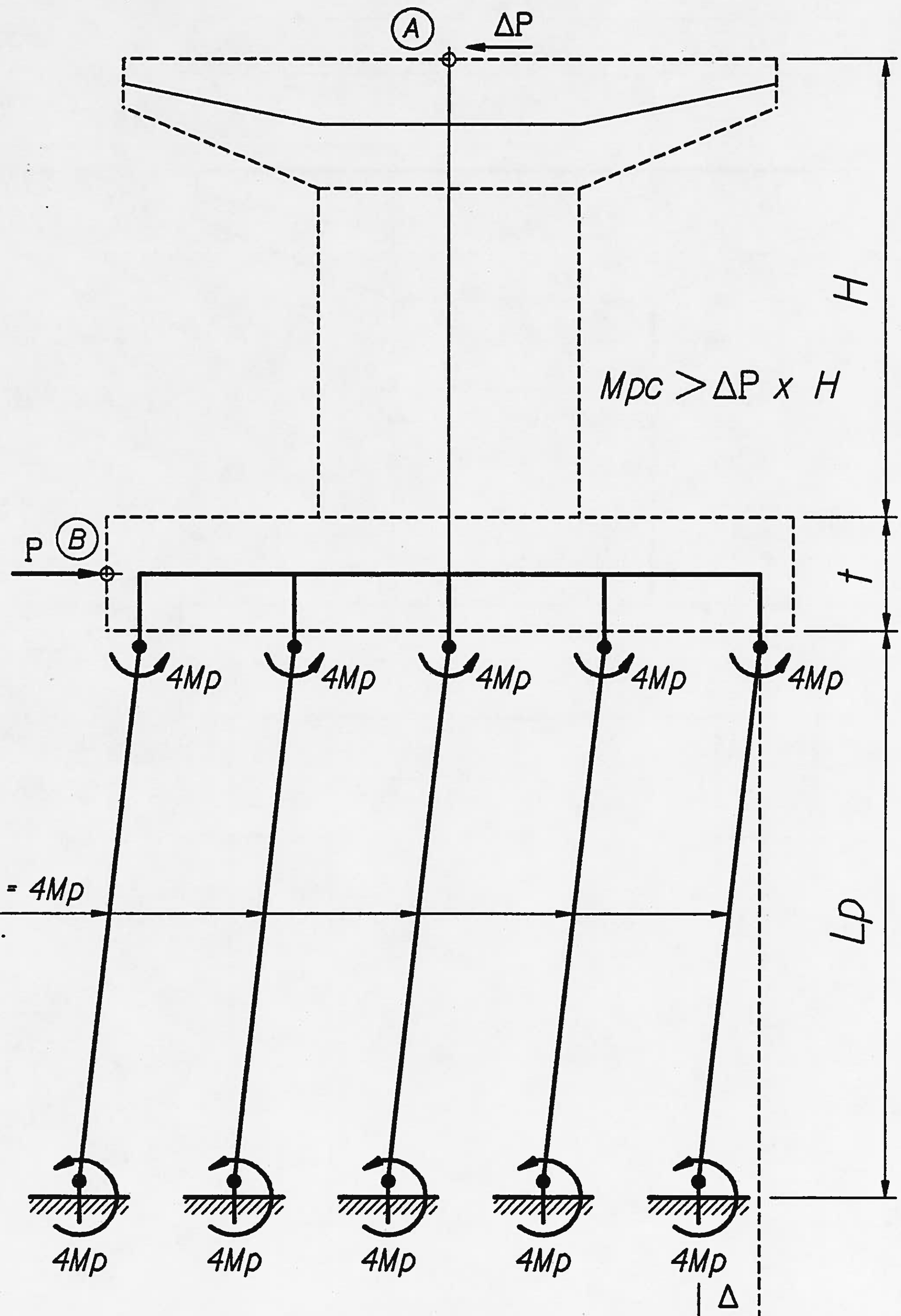
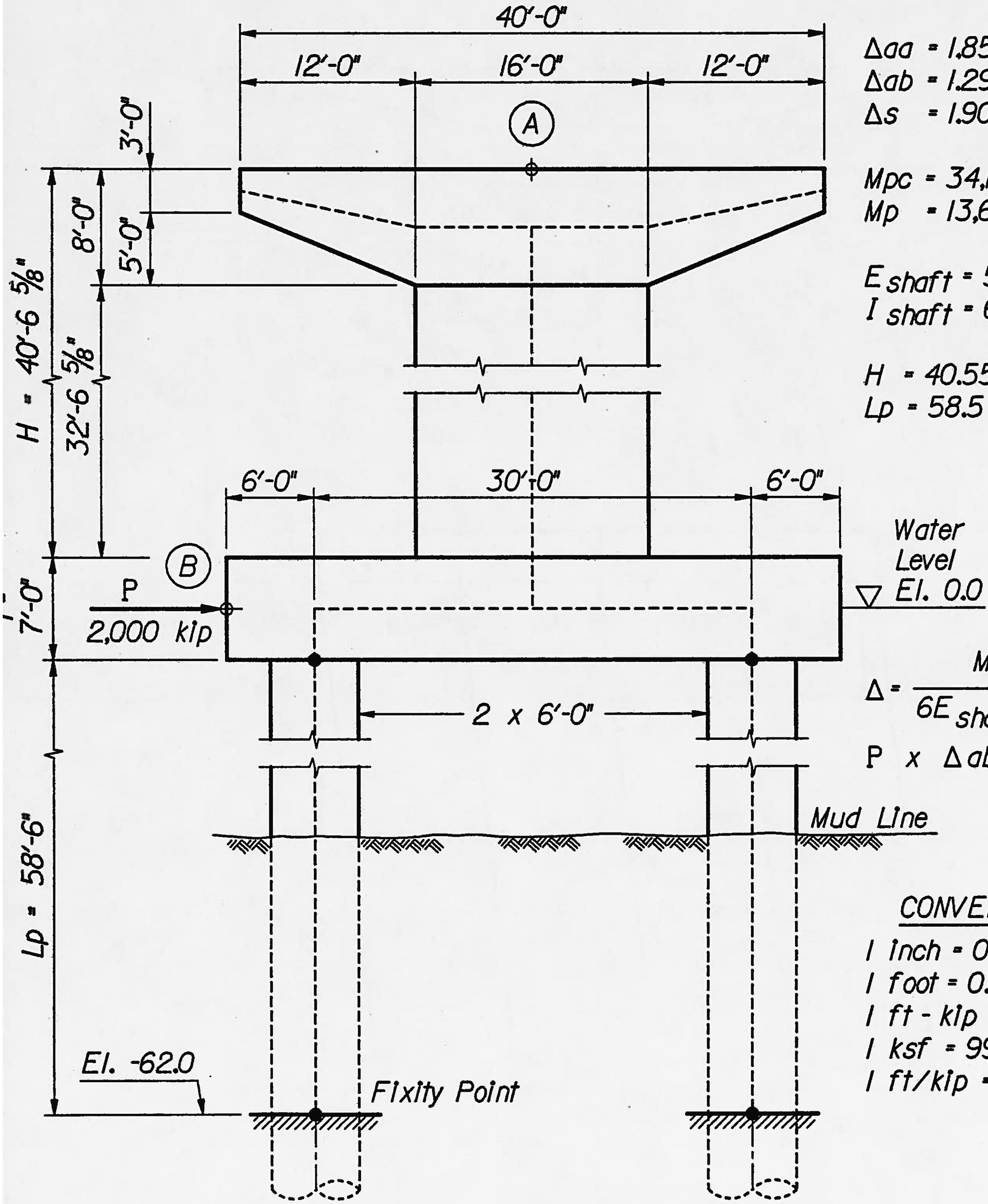


FIG. 3 - SWAY COLLAPSE MECHANISM



$\Delta aa = 1.8559 \times 10^{-4}$ ft/1 kip
$\Delta ab = 1.2906 \times 10^{-4}$
$\Delta s = 1.908427 \times 10^{-4}$

M_{pc} = 34,180 ft - kip
M_p = 13,624 ft - kip

$E_{shaft} = 525 \times 10^3$ ksf
$I_{shaft} = 63.617$ ft^4

H = 40.55 ft
Lp = 58.5 ft

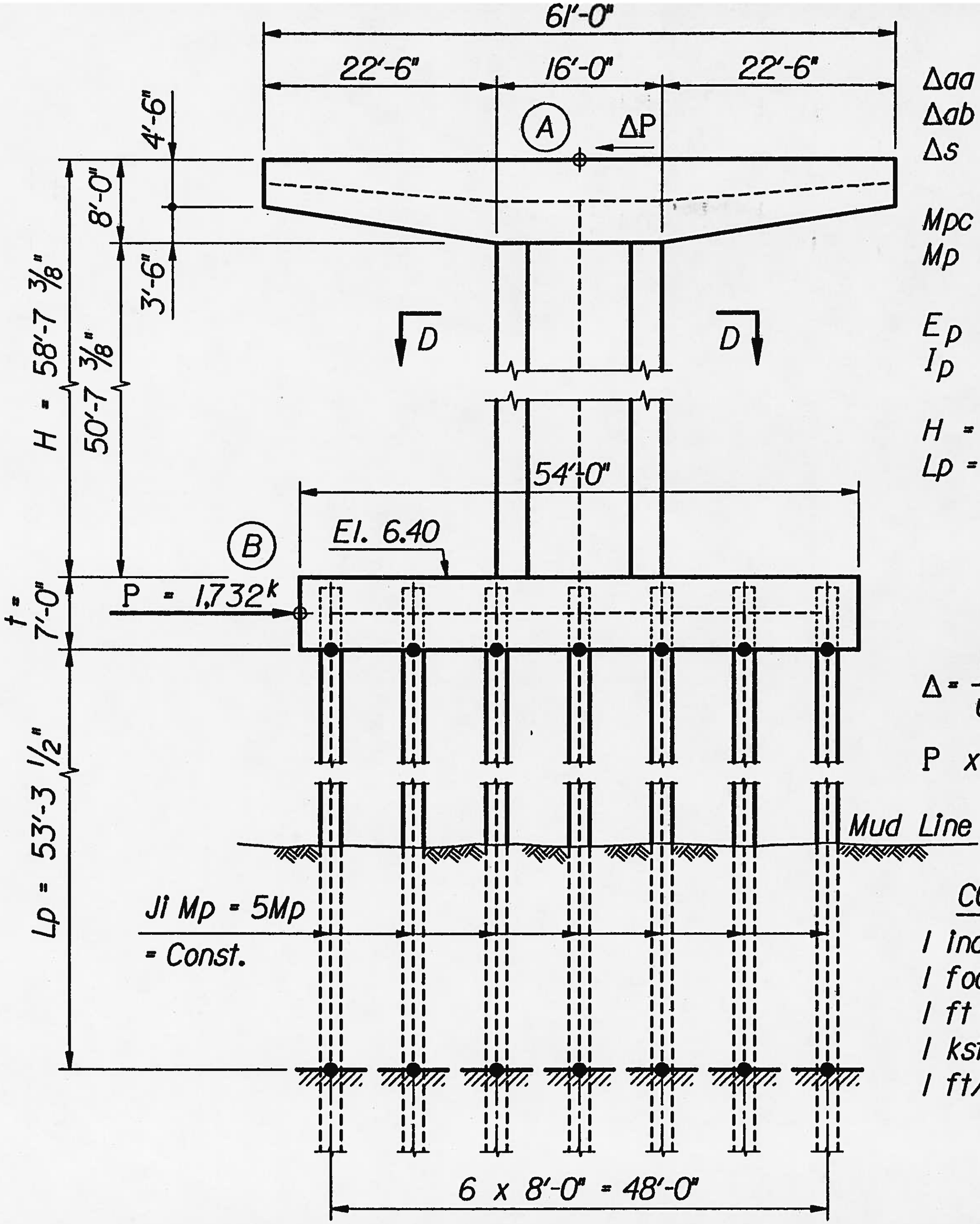
$$\Delta = \frac{M_p L_p^2}{6E_{shaft} I_{shaft}} = 0.2327 \text{ f}$$

$P \times \Delta ab = 0.2581$ ft

CONVERSION FACTORS

1 inch = 0.0254 m
1 foot = 0.3048 m
1 ft - kip = 1.3558 kN·m
1 ksf = 992.8512 N/mm^2
1 ft/kip = 6.71953×10^{-4} m/

FIG. 4 - PIER NO. 71 CROSS-SECTION AND DATA
GANDY BRIDGE

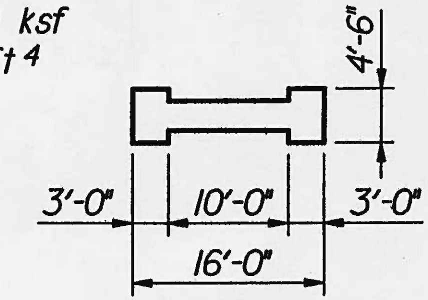


$\Delta aa = 1.49987 \times 10^{-4}$ ft/1 kip
$\Delta ab = 1.134852 \times 10^{-4}$
$\Delta s = 2.724605 \times 10^{-4}$

$Mpc = 42{,}700$ ft - kip
$Mp = 1{,}075$ ft - kip

$E_p = 576{,}000$ ksf
$I_p = 3{,}255$ ft^4

$H = 42.5$ ft
$Lp = 50.4$ ft



SECTION D-D

$$\Delta = \frac{M_p L_p^2}{6E_p I_p} = 0.2427 \text{ ft}$$

$P \times \Delta ab = 0.1966$ ft

CONVERSION FACTORS

1 inch = 0.0254 m
1 foot = 0.3048 m
1 ft - kip = 1.3558 kN·m
1 ksf = 992.8512 N/mm^2
1 ft/kip = 6.71953×10^{-4} m/N

FIG. 5 - PIER NO. 56
H. FRANKLAND BRIDGE

TEST PILE PROGRAM FOR A CABLE-STAYED ARCH BRIDGE

by

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ABSTRACT

As part of a federally-funded demonstration project, a test pile program was recently completed in Columbus, Indiana, for a cable-stayed arch bridge. The program included the driving of two-battered steel H-piles (i.e., HP 14 x 102) within the northwest quadrant of the intersection of Interstate 65 and State Route 46. The piles were driven with a Vulcan® 512 hammer at a one horizontal to one vertical batter. During driving, each pile was dynamically tested using a Pile Driving Analyzer™. Each pile was also equipped with a metal "sleeve" located at one corner of the web and flange. The sleeve was welded to and along the length of the pile to provide protection for an inclinometer casing. Following driving, a portable Digitilt® inclinometer probe was lowered within the casing to observe deflections in the pile. Once the installation activities were completed, the piles were then extracted for observation, and a CAPWAP® analysis was performed to check the field Case Method estimates of pile capacity. Results of the test pile program are discussed.

INTRODUCTION

The City of Columbus, Indiana, has long been noted for its tradition of promoting outstanding architecture. Internationally acclaimed architects including I.M. Pei, Robert Venturi, and Eliel and Eero Saarien have designed many of Columbus's famous buildings. As part of this architectural tradition, a new "Gateway Arch Bridge" on Interstate 65 (I-65) near Columbus will be constructed beginning in late 1994. This innovative bridge will serve as a landmark for entrance to the City of Columbus.

This paper discusses the results of a test pile program for a proposed foundation scheme for the new bridge. The purpose of the test pile program was to assess the feasibility of the proposed scheme using significantly battered piles (i.e., one horizontal to one vertical) and to gain insight on such items as the performance of the hammer and driving system, pile driving stresses and structural integrity of the piles, and provide an estimate of the pile's capacity under impact loads. Prior to performing the test pile program, a wave equation analysis was completed to predict the driving behavior of the piles and aid in the selection of an optimal driving system. During the program, each pile was dynamically tested using a Pile Driving Analyzer™ (PDA). Records of pile force and velocity data were obtained in the field via the PDA. Once the installation activities were completed, the piles were then extracted for observation, and a CAPWAP® (GRL and Associates, Inc., 1993) analysis was performed to check the field Case Method (Goble et al., 1980) estimates of pile capacity. The latter analysis was performed as a follow-up to the test pile program; in which, real time data were used to evaluate the preceding objectives. Discussion regarding the project, subsurface conditions, and test pile program and results are provided.

PROJECT DESCRIPTION

As part of the overall project, the new

bridge will be constructed on I-65 over State Route 46 (SR 46) to accommodate the interchange's increased capacity while minimizing the height of the overpass. The intersection will include a new single-point urban interchange which will bring the exit and entrance ramps of I-65 to one controlled intersection under the bridge. As a result, the interchange will require significantly less right-of-way and be less expensive than a conventional coverleaf. Additionally, the interchange will require a long clear span to assist traffic movements along SR 46.

To facilitate the span, a four-lane prestressed composite-steel arched bridge will be constructed. The structure will be 292 ft (89 m) in length and, due to the divided roadway, will be 137 ft (41.8 m) in width. Two inclined arches/ribs between the roadway lanes will be constructed to suspend a composite bridge deck system from cable stays. As a result of the batter [i.e., one horizontal to one vertical (1H:1V)], the arch will primarily be a compression member with a thrust of about 1,800 kips (8,000 kN) at each rib foundation.

To support the structure, the bridge foundation system will consist of a sixteen-pile group at each "leg" of the rib with individual piles driven at a 1H:1V batter. A batter of this nature will be necessary to minimize eccentric effects at the pile cap.

SUBSURFACE CONDITIONS

Subsurface conditions in the vicinity of the bridge were investigated by performing eight exploratory test borings to depths of 75 ft (23 m) to 88 ft (27 m) below the ground surface. Conventional hollow stem augering techniques were used to advance the boreholes, and split-spoon samples using Standard Penetration Test (SPT) procedures (ASTM D 1586) were obtained at selected intervals. The sampling of cohesive strata was also supplemented with relatively undisturbed, 3-in. (75-mm) diameter Shelby tube samples (ASTM D 1587).

Within the depths explored, the subsurface profile at the boring locations generally consisted of (in descending order): 1) embankment fill (for the existing I-65 embankment); 2) naturally-occurring cohesive and granular soils; and 3) shale bedrock. The fill for the I-65 embankment extended to a depth of about 27 ft (8 m) and generally consisted of layers of cohesive (i.e., silty clay and clay) and granular (i.e., sand/gravel and silty sand) soils.

The embankment soils were underlain by layers of naturally-occurring cohesive and granular soils. Typically, a 4-ft (1.2-m) to 11-ft (3.4-m) thick layer of silty clay was encountered directly beneath the embankment. The silty clay layer (Stratum 1) extended to depths of 27 ft (8 m) to 36 ft (11 m) below the ground surface. The consistency of the silty clay was generally very soft to very stiff. Results of triaxial compression tests indicated undrained shear strengths ranging from 0.6 kips/sq ft (ksf) (29 kPa) to 3.1 ksf (148 kPa). The higher strength values were noted near the fill/natural soil interface and were likely a result of desiccation prior to embankment construction.

Stratum 1 was underlain by a 9-ft (2.7-m) to 30-ft (9.1-m) thick layer of fine to medium sand and gravelly sand (Stratum 2) which extended to depths of 47 ft (14.3 m) to 64 ft (19.5 m). In general, the granular soils extended to greater depths beneath the south embankment in comparison to the north. SPT N-values ranged from 4 to 33 blows/ft (bpf) averaging about 16 bpf (i.e., signifying a relative density of medium dense).

Stratum 2 was underlain by a 4½-ft (1.4-m) to 32-ft (9.8-m) thick layer of silty clay (Stratum 3). In general, Stratum 3 extended to greater depths beneath the north embankment in comparison to the south. Stratum 3 was encountered to depths of 63½ ft (19.4 m) to 74½ ft (22.7 m), and the consistency of the silty clay was generally soft to medium. Results of triaxial compression tests indicated undrained shear strengths ranging from 0.88 ksf (42 kPa) to 1.23 ksf (59 kPa).

Stratum 3 was underlain by shale bedrock (Stratum 4) which included a 1-ft (0.3-m) to 10-ft (3.1-m) thick layer of weathered shale atop of more competent bedrock. The competent bedrock was determined to be 73 ft (22 m) to 78 ft (24 m) below the ground surface in the vicinity of the I-65 embankment. Rock Quality Designation values for the competent bedrock ranged from 25 to 73 percent. A generalized subsurface profile, including information regarding soil properties, is depicted in Table 1.

TEST PILE PROGRAM

Rationale

Given the adverse batter of the proposed foundation scheme and relatively large axial thrust, a test pile program was performed to evaluate the feasibility of the foundation scheme. Insight on such items as the performance of the hammer and driving system, pile driving stresses, structural integrity of the pile, and an estimate of the pile's capacity under impact loads would also be gained. Concerns regarding the feasibility of the scheme included deflecting of the pile tip upon reaching the surface of the bedrock, equipment setup (i.e., performance of the hammer and driving system), and pile capacity.

Procedures

The program included the driving of two-battered steel H-piles [i.e., HP 14 x 102 (356 mm x 454 N)] within the northwest quadrant of the intersection of I-65 and SR 46. The pile type and section was selected on the basis of pile stiffness and impedance characteristics.

Hammer and Driving System. Each test pile was driven with a Vulcan® 512 hammer at a 1H:1V batter. The Vulcan® 512 is a single-acting air/steam hammer with a maximum-rated energy of 60 ft-kip (81 m-kN) on the vertical. The hammer was selected on availability at the time of the test pile program. A wave equation analysis was performed in advance to predict the driving behavior of the piles and evaluate the driving system. The hammer cushion consisted of an 8-in. (203-mm) thick layer of Nycast Nylon (blue) beneath a ½-in (13-mm) thick aluminum striker plate. Each pile was oriented such that the strong axis was perpendicular to the plane of anticipated bending, and a swinging lead was used to maintain alignment of the pile. The pile tips were protected with a Hard-Bite™ point (i.e., HP-77750), and one splice was used for each pile. The total length of each pile was approximately 21 m (70 ft) (i.e., along the batter).

Instrumentation. During driving, each pile was dynamically tested using a PDA. Dynamic measurements of strain and acceleration were taken approximately 4 ft (1.2 m) below the head of the test piles. Two strain transducers and two piezoresistive accelerometers were bolted at opposite sides of each pile to monitor strain and acceleration and to minimize, by averaging, the effects of non-uniform hammer impacts. Strain and acceleration signals were then conditioned and converted to forces and velocities by the PDA. During driving, the PDA calculated values for maximum transferred hammer energy, maximum compression stress at the gage location; and an estimate of the pile capacity by the Case Method. In addition, each pile was equipped with a "sleeve" located at one corner of the web and flange. The sleeve consisted of a heavy-wall 2¼-in. (57-mm) I.D. metal pipe welded to and along the length of the pile to

Table 1. Generalized Subsurface Profile

Soil Property	Stratum 1	Stratum 2	Stratum 3	Stratum 4
USCS Symbol	CL	SP	CL	-
Depth (i.e., from I-65 embankment), ft	27 to 36	47 to 64	64 to 74	73 to 78
Total Unit Weight, lb/ft^3	130	_	125	-
Moisture Content, %	19 to 30	_	14 to 34	-
Liquid Limit, %	33	_	42	-
Plasticity Index	11	_	19	-
Shear Strength				
Undrained c_u, ksf	0.6 to 3.1	_	0.9 to 1.2	-

provide protection for a 1.9-in. (48-mm) O.D. inclinometer casing. Following driving, a portable Digitilt® inclinometer probe was installed and lowered within the casing to observe deflections in the pile. However, in one of the two test piles, the sleeve was damaged during the driving process; hence, deflections were not obtained.

Once the deflections were observed, the piles were extracted for observation, using an ICE® 812 vibratory extractor. Following the field activities, a CAPWAP® analysis (i.e., a signal matching analysis of PDA data) was performed to check the field Case Method estimates of pile capacity. The test program was also documented via video tape and photographs.

DISCUSSION OF ANALYSES AND RESULTS

Wave Equation Analysis

Prior to performing the test pile program, a wave equation analysis was completed to predict the driving behavior of the piles and aid in the selection of an optimal driving system. The analysis was performed by utilizing a personal computer and a commercially-available software program, titled GRLWEAP™ [GRL's Wave Equation Analysis of Pile driving, Version 1.00 (GRL, 1993)]. Theory and discussion regarding the soil-structure interaction model are described in GRL.

For this analysis, the critical factors included the hammer, driving system, pile type and dynamic soil resistance parameters. Considering the anticipated reductions in the driving system and friction losses for battered piling, a rated transfer efficiency of 38 percent (which is defined as the energy transferred to the PDA gage location near the pile head divided by the manufacturer's rated hammer energy) was used in the evaluation of the energy transferred (i.e., enthru) to the pile head. Energy transferred to the pile head was evaluated to be on the order of 20 ft-kip (27.6 m-kN) to 22 ft-kip (30.4 m-kN). Additionally, the compression stresses at the pile head, assuming end of driving conditions, were estimated to be on the order of 15 ksi (103 kPa) which is well below the normally acceptable maximum compression stress of steel (i.e., $0.9F_y$) during driving. The maximum pile compression stress was evaluated to be 21.3 ksi (147 kPa) at a depth of 47.5 ft (14.5 m). In the model, it was also assumed that 40 percent of the total ultimate resistance would be accounted for in shaft friction (based on a static analysis), and the shaft friction would be distributed in a triangular form over the lower 60 percent of the pile.

Based on the anticipated enthru and assuming representative dynamic soil resistance parameters, an estimate of the expected blow count (during driving) vs. ultimate soil resistance was computed. Table 2 summarizes cases analyzed for a parametric study of the dynamic soil resistance values. Results from the parametric study are illustrated in Figure 1 (via a bearing graph).

Dynamic Testing

Tables 3 and 4 summarize the results of the dynamic pile testing. Table 3 provides a summary of the dynamic testing results at selected pile penetration depths. This table includes average values of: the energy transferred to the pile head (EMX); the maximum calculated pile head compression stress (CSX); and the Case Method estimates of pile capacity (RMX and RAU). RMX is the maximum Case Method resistance using damping at any time during the hammer blow. RAU is the Case Method static resistance at time of zero damping for piles without shaft resistance. The RAU method is most appropriate for pure end bearing piles since it does not rely on an assumed soil damping factor to reduce the dynamic resistance to a static capacity.

The CAse Pile Wave Analysis Program (CAPWAP) computes soil resistance forces and their approximate distribution using the force and velocity data recorded in the field during the driving process. Results of the CAPWAP analyses are summarized in Table 4. Final CAPWAP results include an evaluation of the soil resistance distribution, soil quake and damping factors and a simulated static load-set graph. The static load-set graph is based on the CAPWAP calculated static resistance parameters and the elastic compression characteristics of the pile. CAPWAP analyses

Table 2. Summary of cases for parametric study of dynamic soil resistance values				
Case	Soil Quake along Shaft (in.)	Soil Quake at Toe (in.)	Soil Damping along Shaft (sec/ft)	Soil Damping at Toe (sec/ft)
1	0.10	0.20	0.05	0.15
2	0.10	0.20	0.20	0.15
3	0.10	0.20	0.05	0.05
4	0.10	0.20	0.20	0.05
5	0.10	0.20	0.10	0.07
6	0.10	0.50	0.10	0.07
7[1]	0.10	0.35	0.10	0.07
Notes: 1. "Best estimate".				

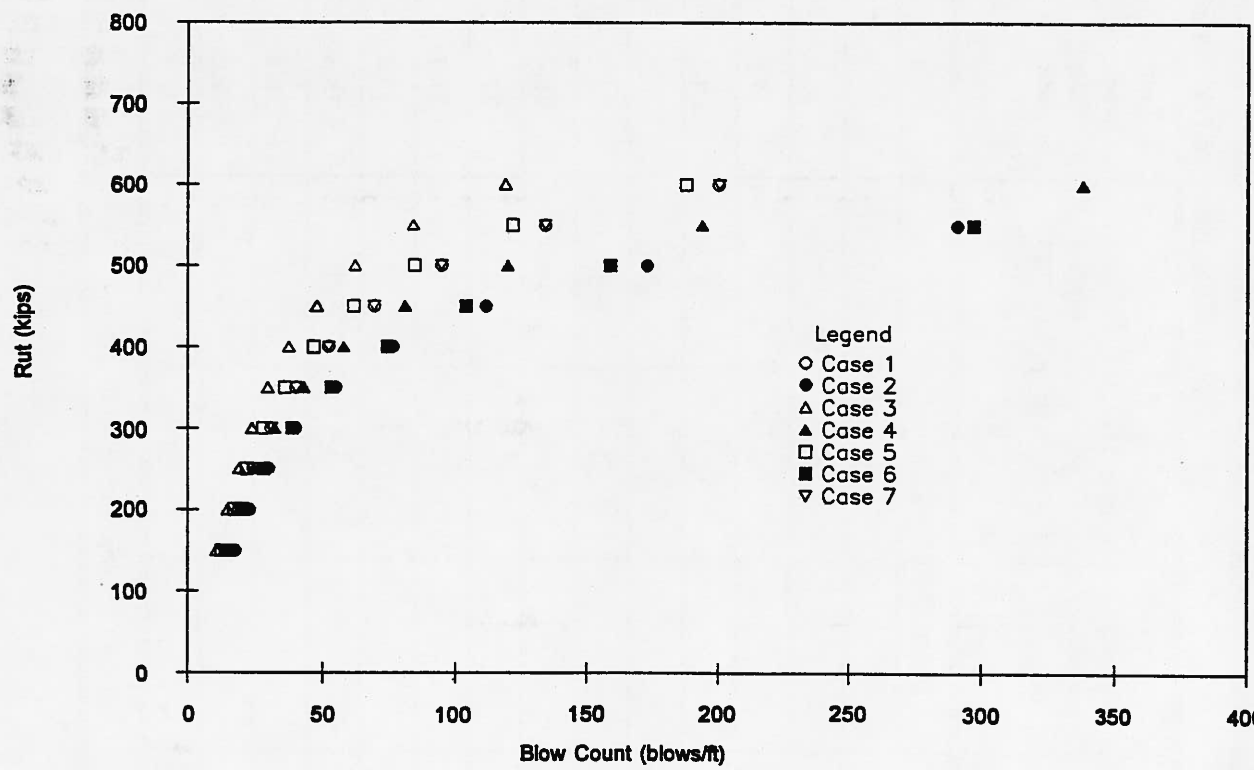

Figure 1. Bearing graph

Table 3. Summary of pile dynamic testing results

Pile Designation	Driving Status	Approx. Pile Penetration (ft)	Reported Driving Resistance (blows/ft)	EMX, Average Energy Transfer To Pile Head (kips-ft)	CSX, Average Pile Head Compression Stress (ksi)	Average Case Method Capacity Estimate RMX, J=.70 (kips)	Average Case Method Capacity Estimate RAU (kips)	CAPWAP Capacity Estimate (kips)
Test Pile 1	EOID[1]	45	11	26.9	21.72	139	135	---
		50	11	26.4	20.28	120	106	---
		55	7	26.8	20.47	119	93	---
		60	8	24.5	22.13	124	114	---
		61	11	24.7	22.34	159	120	---
		62	34	24.4	22.08	383	322	---
		63	87	24.0	21.81	538	476	---
		63.3	8/in.	25.4	19.23	574	511	576
Test Pile 2	EOID[1]	50	7	21.4	19.13	94	56	---
		55	8	27.1	21.14	106	76	---
		60	6	24.4	20.54	104	90	---
		65	8	25.4	20.33	100	51	---
		68	9	26.5	20.76	112	71	---
		69	35	22.6	19.04	382	352	---
		69.1	13/in.	17.3	14.20	538	495	497

Notes: 1. EOID implies end of initial driving

Table 4. Summary of CAPWAP results

Pile Designation	Driving Status	Approx. Pile Penetration (ft)	Reported Driving Resistance (blows/in.)	CAPWAP Static Capacity Estimate Shaft (kips)	Toe (kips)	Total (kips)	Smith Damping Shaft (sec/ft)	Toe (sec/ft)	Soil Quake Shaft (in.)	Toe (in.)
Test Pile 1	EOID[1]	63.3	8	207	367	574	0.095	0.049	0.09	0.32
Test Pile 2	EOID[1]	69.1	13	199	296	495	0.06	0.05	0.10	0.34

Notes: 1. EOID implies end of initial driving were performed on dynamic test data obtained near the end of initial driving for each test pile. Figures 2 and 3 illustrate an actual wave trace and skin resistance distribution, and simulated load-set, respectively, for Test Pile 1. Although a static pile load test was not performed for this program, others (Rausche et al., 1994) have obtained relatively good agreement between simulated load-set and actual test data, for a limited number of cases.

Hammer and Driving System Performance. The performance of a hammer and driving system can be evaluated from a driving system's rated transfer efficiency. The rated transfer efficiency is defined as the energy transferred to the PDA gage location near the pile head (EMX), divided by the manufacturer's rated hammer energy. Near final driving, the average energy transferred to the gage location ranged from 17.3 ft-kip (23.4 m-kN) for Test Pile 2 to 25.4 ft-kip (34.4 m-kN) for Test Pile 1. This corresponds to a rated transfer efficiency of 29 to 42 percent based on a rated energy of 60 ft-kip (81.3 m-kN). However, due to the batter, the actual stroke of the hammer is only about 3½ ft (1.1 m) instead of 5 ft (1.5 m). Therefore, the maximum energy that can be delivered on a 1H:1V batter is about 42 ft-kip (57 m-kN). The energy transfer compared to the maximum energy that can be delivered ranged from 41 to 60 percent which is what might normally be expected for a pile being driven vertically (Rausche et al., 1985).

Driving Stresses and Pile Integrity. For each hammer blow, the maximum pile head compression stress (CSX) was calculated by the PDA using the average signal from the two strain transducers. The maximum pile head compression stresses averaged over the final pile penetration increment ranged from 14.2 ksi to 19.2 ksi (97.9 to 132.4 MPa). The maximum pile head compression stress calculated during driving of either test pile was 22.9 ksi (157.8 MPa).

The PDA calculates the maximum compression stress at the gage location near the pile head. However, the maximum compression stress at other locations in the pile may be greater depending upon the soil resistance distribution along the pile. The maximum compression stress at other locations in the pile can be evaluated by the CAPWAP analysis. For the hammer blows analyzed, the CAPWAP analysis indicated the maximum compression stress at other locations in the test pile ranged from 1.01 to 1.03 times the compression stress calculated at the pile head by the PDA. The maximum pile compression stress calculated in the CAPWAP analysis was 20.4 ksi (140.6 MPa) and occurred 47 ft (14.3 m) below the head of the pile, near the end of initial driving of Test Pile 1.

During data acquisition, force and velocity records were evaluated for indications of pile damage below the gage location. Observable signs of pile damage were not indicated in the field assessment of the dynamic records. Although no signs of pile damage were observed via dynamic records and following extraction of the piles, an elastic deflection of Test Pile 2 was observed. Based on the results of inclinometer readings, Test Pile 2 had deflected approximately 4.8 ft (1.5 m) and 4.2 ft (1.3 m) into the planes of the strong and weak axis of the pile, respectively. Information from Test Pile 1 was not obtained because the metal "sleeve" was damaged. The direction of the observed deflection was toward the side/corner of the pile where the sleeve was located, indicating an imbalance of the shaft forces, possibly causing the pile to "corkscrew".

Pile Capacity. Two Case Method equations including the maximum Case Method (RMX) and the automatic Case Method (RAU) were used to obtain estimates of pile capacity during initial driving. A Case damping factor of 0.70 was used with RMX. Case Method estimates of mobilized pile capacity were averaged over the final pile penetration increment. The average Case Method capacity estimates for Test Pile 1 ranged from 511 kips (2.3 MN), using the RAU equation, to 576 kips (2.6 MN), using the RMX equation. For Test Pile 2, average Case Method capacity estimates ranged from 497 kips (2.2 MN), using the RAU equation, to 538 kips (2.4 MN), using the RMX equation.

CAPWAP analyses were also performed on dynamic test data obtained for one hammer blow near the end of initial driving of each test

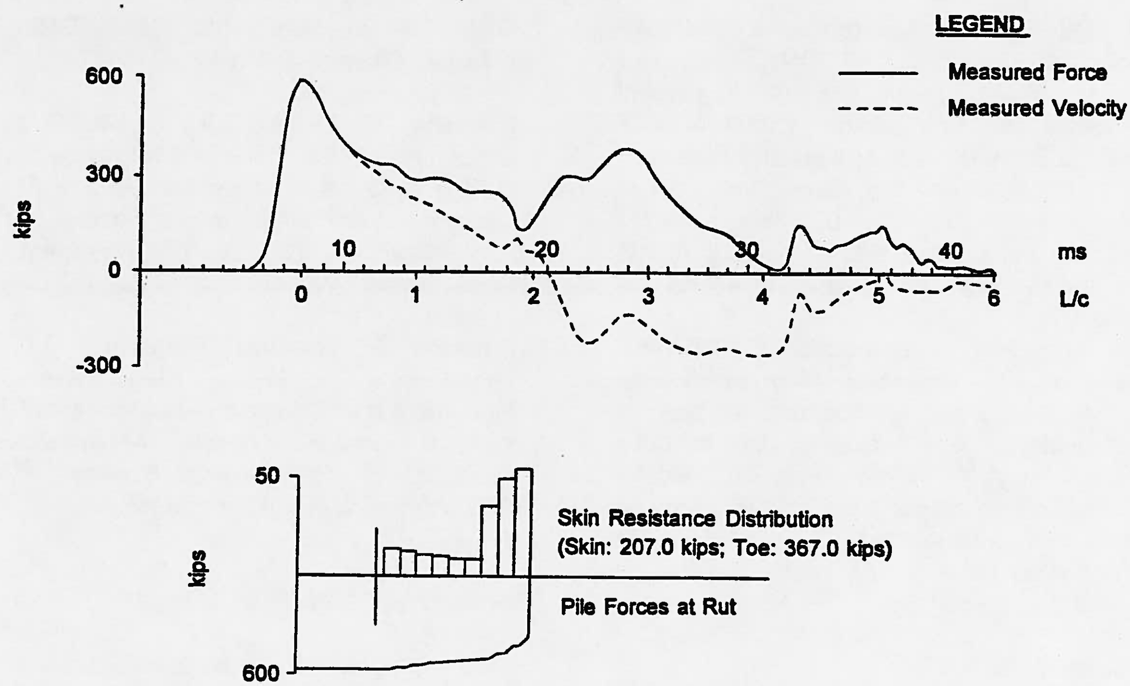

Figure 2. Measured wave trace and skin resistance distribution for Test Pile 1

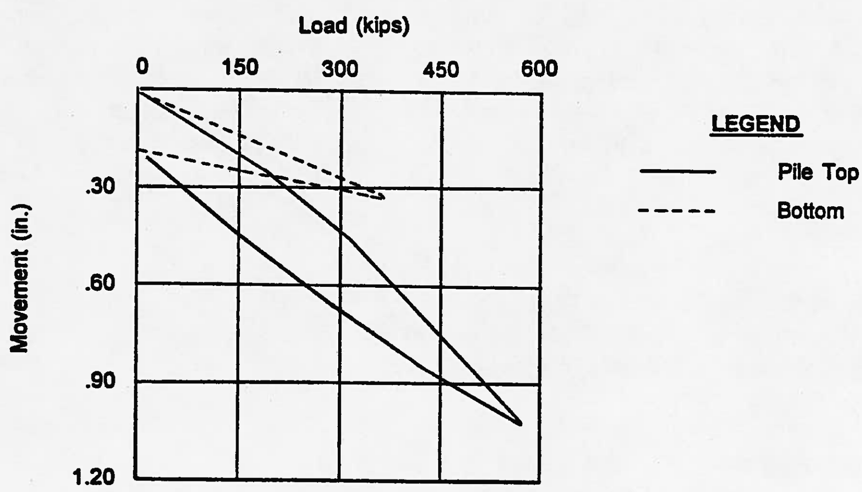

Figure 3. Simulated load-set graph for Test Pile 1

pile. Test Pile 1 had a mobilized CAPWAP capacity of 574 kips (2.6 MN) which was distributed as 64 percent toe and 36 percent shaft resistance. Soil quakes of 0.09 in. (2.29 mm) and 0.32 in. (8.13 mm) were matched (i.e., fitted) for the shaft and toe, respectively. Smith damping parameters of 0.095 sec/ft (0.311 sec/m) for the shaft and 0.049 sec/ft (0.161 sec/m) for the toe were also matched for the measured data.

Test Pile 2 mobilized a CAPWAP capacity of 495 kips (2.2 MN) which was distributed as 60 percent toe and 40 percent shaft resistance. Soil quakes of 0.10 in. (2.54 mm) and 0.34 in. (8.64 mm) were matched for the shaft and toe, respectively. Smith damping parameters of 0.06 sec/ft (0.197 sec/m) for the shaft and 0.05 sec/ft (0.164 sec/m) for the toe were also matched.

CONCLUSIONS

This paper has presented the results of a test pile program for an innovative bridge design. Results indicated that the proposed foundation scheme can adequately be supported on pile foundations, as planned. Prior to performing the test pile program, a wave equation analysis was performed to predict the driving behavior of the piles and aid in the selection of an optimal driving system. Comparison of results obtained from the wave equation and CAPWAP analyses indicated reasonably good agreement at end of driving conditions.

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ACKNOWLEDGEMENTS

The author would like to acknowledge the contribution of: Woolpert Consultants of Indianapolis, Indiana, as the design consultant; J. Muller International of Chicago, Illinois, as the bridge consultant; Milestone Contractors of Columbus, Indiana, as the test piling contractor; GRL and Associates of Chicago, Illinois, for performing the dynamic testing; Purdue University, for the use of the Digitilt® inclinometer probe, interpretation of the inclinometer data, and review of this manuscript by Dr. Richard J. Deschamps; and the staff of Earth Exploration, Inc. for preparation of this manuscript.

RECENT RESEARCHES INTO THE BEHAVIOR OF HIGH CAPACITY PIN PILES^{SM}

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ABSTRACT

Minipiles are known by many names, but are generically small diameter, cast-in-place bored piles. Although they have been used in Europe for over 40 years, in the United States, where they are commonly referred to as Pin Piles^{sm}, they have become a popular choice for underpinning only during the last 10 years. The paper describes fundamental laboratory and field researches recently conducted to better understand load transfer mechanisms. This work has led to the development of the Elastic Ratio concept which is now proving extremely useful in analyzing and predicting pile performance, and in particular the phenomenon of progressive debonding with increasing load. Pin Piles are becoming more popular in applications for seismic rehabilitation of bridges, and this paper focuses on this aspect via recent case histories.

INTRODUCTION

The last decade has seen a significant growth in the use of Pin Piles^{sm} in the United States. Generically, these piles may be classified as small diameter, bored, cast-in-place elements, and they owe their origins to developments by specialty contractors in Italy over 40 years ago. As a result of the kind of research and development activities described below, their allowable load range has been extended from 50-100 kips to up to 300 kips while special test piles have yielded capacities of over 1000 kips in favorable conditions.

Initially, these advances were made as a result of the careful execution and analysis of full scale field test programs, and such experiences have been widely published (References 1-8). However, within the last few years it has become apparent that extra dimensions of research efforts were necessary to explore and understand fundamental aspects of Pin Pile behavior, and especially those related to the performance of the component materials in resisting and transferring high axial loads.

This research was funded by Nicholson Construction Company and conducted jointly with the University of Pittsburgh. The laboratory work comprised three major phases:

Phase 1, where single, grout-filled steel casings, simulating the upper (free) section of a typical high capacity Pin Pile, were compressed to failure, to establish their composite strength and elasticity characteristics.

Phase 2, as Phase 1 but using connected casing sections with threaded ends.

Phase 3, where similar tests were conducted on internally reinforced grout columns simulating the lower (bonded) section.

In parallel, the opportunity was taken to run full scale field tests at two major contemporary underpinning projects, one at a petrochemical facility near Mobile, AL, (Reference 7), the other at a grain silo complex near Port Vancouver, WA (Reference 9). The latter case history is summarized in this paper.

CONSTRUCTION AND CLASSIFICATION

Pin Piles are most commonly used to underpin existing structures settling, or liable to settle, as a result of changes in loading or foundation conditions. Construction methods have therefore been developed to accommodate the gamut of ground and structure types, while causing the minimum of damage to either, or the environment. Also Pin Piles operate principally in side shear and so these techniques have been honed to enhance bond capacity at the grout/soil interface.

The successive steps of pile construction are well known and documented (e.g. References 1-9). They comprise drilling and casing, placing of reinforcement and tremie grouting, and, typically, pressure grouting of the bond zone.

In most countries, the temporary casing is fully extracted (as any auger must always be) during the pressure grouting process. However, in the United States, it has been proved that by leaving the casing in place through the zones above the bond length, the Pin Pile performance is greatly enhanced, both vertically and laterally. This option also prevents wasteful travel of grout into often permeable upper horizons, and provides excellent corrosion protection to the interior of the pile in what is usually the most vulnerable zone. A useful subclassification of Nicholson Pin Pile types, based on the geology of the founding zone, and the internal composition of the pile is provided in Figure 1:

- Type S1 - A steel casing is rotated into the soil using water to externally flush the cuttings up around the outside of the pipe. A cement grout is tremied from the bottom of the hole to displace the water. The reinforcing element is then placed to the bottom of the hole. As the casing is withdrawn over the length of the bond zone, additional grout is pumped under excess pressure. The casing is then seated back into the grouted bond zone. In granular soils, a certain amount of permeation and displacement of loosened soils takes place. In cohesive soils, some lateral displacement or localized improvement of the soil around the bond zone is accomplished with the pressure grouting. Postgrouting may be used later to further enhance soil/grout bond (References 10 and 11).

- Type S2 - The pile is installed in the same fashion as the S1 except that:
  - the centralized reinforcing element is not needed;
  - the steel casing is installed to the full length of the bond zone after pressure grouting is completed.
  - post-grouting is not feasible with this type of installation.

- Type R1 - This pile uses the same technique for advancing the steel casing as Type S1, except that the depth of penetration is limited to the top of rock. Once the pipe is seated into rock, a smaller diameter drill string is advanced through its center to drill the rock bond zone. Neat cement grout is then tremied from the bottom, and a reinforcing element is placed in the rock bond zone to complete the installation. A minimum transfer length is required for the reinforcement to develop bond inside the casing (typically 5 to 10 feet).

- Type R2 - This type differs from the R1 pile in that it uses a full length steel casing. The possible need for centralized reinforcement is dictated by internal pile capacity. In order to advance through both the overburden and the rock, a permanent drill bit is used on the end of the casing with

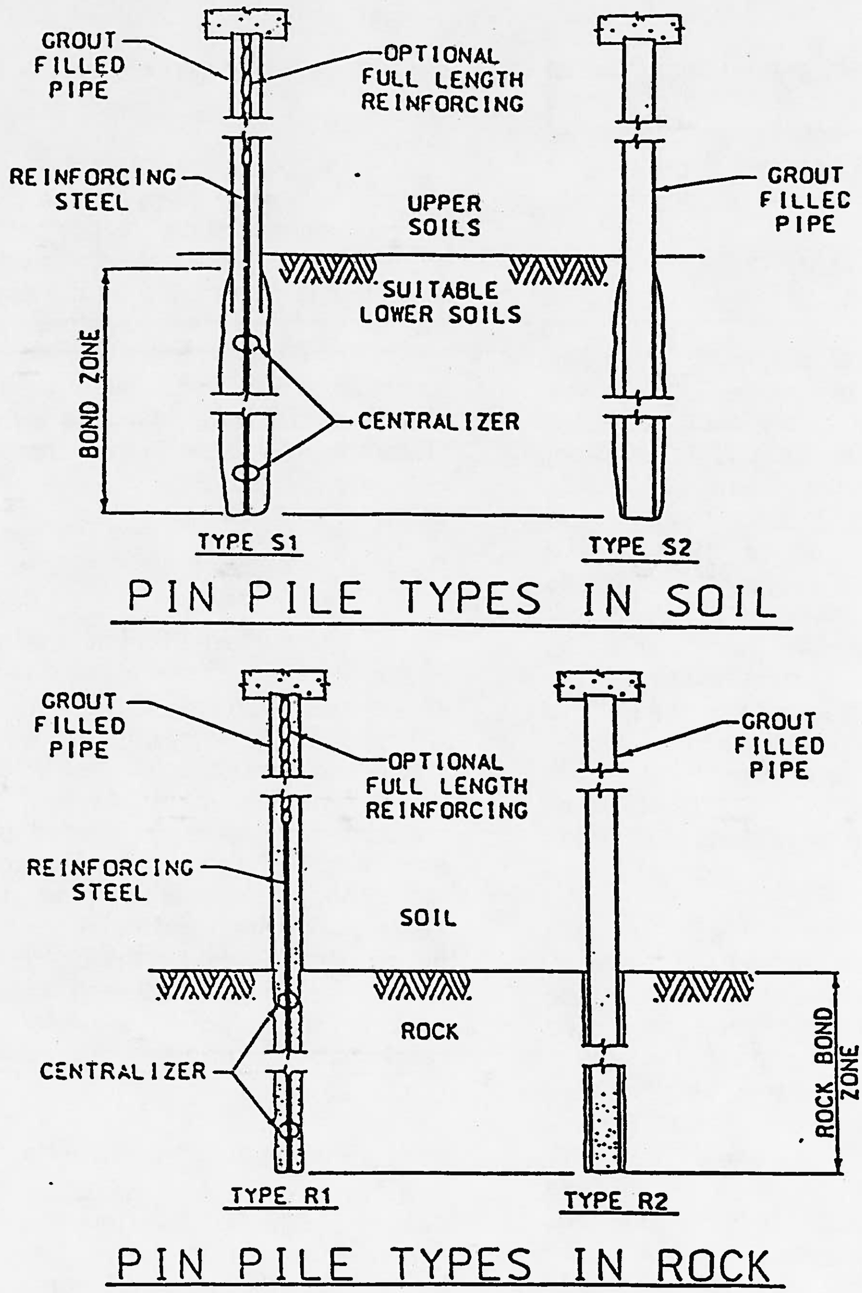

Figure 1 Generic Pin Pile Configurations in Soil and Rock

a diameter somewhat greater than that of the casing. At the desired final depth, grout is tremied from the bottom, and additional grout is pumped to ensure full grouting of the rock bond zone.

LABORATORY RESEARCH

Phase 1

Testing of composite members has been conducted for decades, worldwide, and the results of 68 tests of axially loaded concrete filled tubes were addressed in a Steel Structures Research Council (SSRC) report. (Reference 12). Actual steel yield stress varied from 38 to 88 ksi, and concrete compressive strengths from 2.9 to 9.6 ksi. A table of data comparing these test loads with the theoretical allowable loads, based on the proposed modifications to the AISC allowable stress equations, was prepared to give an indication of actual safety factors. These ratios varied from 1.28 to 3.68, average 2.26, standard deviation 0.45 and a coefficient of variation of 20%.

The tests in Phase 1 were run with composite tubular members of uniform, high strength steel (nominal 80 ksi) and grout (minimum 4 ksi), such as would comprise certain sections of Pin Piles. (Reference 13). Specimen lengths were selected to provide a set of slenderness ratios that would be consistent with previous experiments. Details are summarized in Table 1. These data proved consistent with the earlier tabulated work of SSRC.

Separate tests on material properties confirmed the specified minimum yield strengths to be 86 ksi for the 7 inch dia. casing, and 99 ksi for the 5.5 inch dia. casing. Grout strengths (28 day compressive) averaged about 5.6 ksi. Each column responded similarly throughout the loading range - initial local yielding at the ends followed by gradual bending. No evidence of buckling was observed. The shorter specimens exhibited a linear load/deflection relationship to about 75% maximum load, while the longer casings were linear almost to maximum load.

Of particular interest in Table 1 is the newly coined term Elastic Ratio (ER) for each configuration. ER is calculated by dividing the resultant displacement (in thousandths of an inch) by the applied load (in kips), and is therefore a simple indicator of the effective composite elastic modulus of the grout filled casing. This directly determined value can then be used to ascertain the seat of load transfer during the cyclic loading of Pin Piles, as demonstrated below.

Phase 2

The typical Pin Pile casing joint is formed by mating the male and female ends of successive lengths of casing. This joint is typically flush, with little or no resulting space between sections, and its strength is dependent on many factors including material yield strength, thread pitch, root size, length of splice, shoulder contact and the confining effects of pipe and grout. Tests were conducted (in tension also, but not detailed herein) on the typical Nicholson casing thread, with 36 inch long samples with and without external banding reinforcement around the female end. (Table 2).

A comparison of the data of Table 1 (single casing) and Table 2 (coupled casings) shows that for stub columns (simulating the fully braced pile configuration), no significant difference exists in the magnitudes of the ultimate loads or the ultimate failure modes. However, the ER values recorded for the Phase 2 tests were higher for two main reasons:

a) "Slop" in joints, creating higher total displacements, coupled with
b) the relatively short test lengths being more sensitive to these displacements.

The tension tests showed the joints to have about 60% less capacity than in

Spec #	Dia. (in)	Length (in)	Wall (in)	Max Load (kips)	Elastic Ratio*
1	7	36	0.502	1181	(Equivalent to 0.30 for 10' length)
2	7	36	0.502	1242	(Equivalent to 0.30 for 10' length)
3	7	120	0.502	969	0.32
4	5.5	36	0.363	685	(Equivalent to 0.70 for 10' length)
5	5.5	36	0.363	584	(Equivalent to 0.54 for 10' length)
6	5.5	120	0.363	450	0.53

Table 1 Summary of Phase I Laboratory Tests (Single Casing Lengths)

*Calculated for each specimen as compression (in thousandths of an inch) divided by load (in kips), in the elastic response field.

Spec #	Dia. (in)		Length (in)	Wall (in)	Max Load (kips)	Elastic Ratio
1	7	B	36	0.502	1300	0.49
2	7	U	36	0.502	1160	0.44
3	5.5	B	36	0.363	630	1.52
4	5.5	U	36	0.363	545	1.06

Table 2 Summary of Phase 2 Laboratory Tests (Coupled Casing Lengths)

B = Banded
U = Unbanded
10 ft. equivalent length

compression. Also, the failure mode in tension was explosive (i.e. the thread experienced sudden failure).

Phase 3

Clearly the structural capacity of Pin Piles with full length casing (S2, R2) can be designed conservatively by using the composite strength of the grout filled casing, ignoring the confining contribution of the annular grout. However, if the grout is neglected in the design of an internally reinforced bond zone (S1, R1), the resultant design would be significantly over-conservative. A series of tests was therefore run on such simulated bond zone configurations, as detailed in Table 3.

Each specimen was cast and tested in a tubular plastic mould, the lateral confining properties of which approximated that of a medium dense sand. The specimens were all 36 inches long and 10 3/4 inches in diameter. The length was selected to create a stub column, so allowing slenderness effects to be ignored, while the diameter reflected a typical effective pressure grouted bond zone diameter in situ.

Linear behavior was noted over an average of 84% of the ultimate capacity, and failure was characterized by axial crushing. The relationships between ultimate load, and ER, and the cross sectional area of steel in the sample are shown in Figures 2 and 3 respectively. The benefit of the simulated spiral reinforcement is clearly demonstrated - a 37% improvement in capacity over the comparable specimen without a confining cage.

FIELD RESEARCH - UNITED GRAIN SITE

Concurrent with this laboratory research, Nicholson Construction was the design-build contractor on two significant Pin Pile projects. The larger was at an operational grain export facility on the Columbia River at Vancouver, Washington, where certain major structures were threatened by settlement as a result of deterioration of the original 4050 timber piles driven in 1934-1939. (Reference 14). Prior to installing the 840 replacement Pin Piles of 300 kip service load, about half of which were to be located in the cramped basements of the three silo structures, an extensive test program was conducted, involving six full-scale special test piles.

The design foresaw each pile to be drilled with 7 inch casing to a depth of approximately 70 feet from grade and so a minimum of 30 feet into a very dense gravelly and cobbly bed. The upper portion was to be reinforced by the casing, with the lower pressure grouted portion reinforced by a central reinforcing bar, in a standard S1 type configuration. (Figure 1). The Specification called for an underpinning system to ensure additional differential settlements of less than 1.5 inches in 100 feet and additional uniform settlements of less than 6 inches.

The test program required the successful loading of three piles to 200% service load, held for a minimum 12 hour period. The service load was considered to be 300 kips for the test program, although final calculated individual pile loads were slightly lower, depending on location. These initial three piles (TP1-3) all reached the 600 kips maximum but all exhibited what appeared to be explosive internal (structural) failure prior to the end of the hold period. As a result, and after structural adjustments, a second group of three piles successfully passed the test, and subsequently attained ultimate loads of up to 750 kips. These piles established the production pile structural detailing and criteria for minimum embedments into the bearing stratum. Test loads were applied in cycles of increasing load, so permitting the partition of total pile settlements into elastic and permanent displacements at each maximum load attained. The elastic component therefore permitted the calculation of the Elastic Ratio for each load cycle maximum.

Spec #	Reinforcement Configuration	Cross Section Steel (in^2)	Max Load (kips)		Equivalent 10 ft. Elastic Ratio
1A	None - Plain Grout	zero	209	212	2.20
1B	None - Plain Grout	zero	214		
2A	1 # 10	1·23	315	308	1.66
2B	1 # 10	1·23	300		
3A	1 # 14	2·40	390	365	1.53
3B	1 # 14	2·40	340		
4A	1 # 18	3·97	490	470	1.45
4B	1 # 18	3·97	450		
5A	2 # 10	2·45	367	393	1.68
5C	2 # 10	2·45	418		
6A	2 # 14	4·81	563	572	0.71
6B	2 # 14	4·81	580		
7A	1 # 14 + Simulated Spiral cage	2·40+	500	500	1.53
7B	1 # 14 + Simulated Spiral cage	2·40+	500		
8A	11 ea. 0.6" strand*	2·39	394	355	2.20**
8B	11 ea. 0.6" strand*	2·39	315		

Table 3 Summary of Phase 3 Laboratory Tests (Bond Zones)

* Strand fy = 270 ksi (1860 MPa); all other rebar 60 ksi (410 MPa).
** Strand columns exhibited similar stiffness as plain grout columns. Strand stiffness in compression is suspect in these tests.

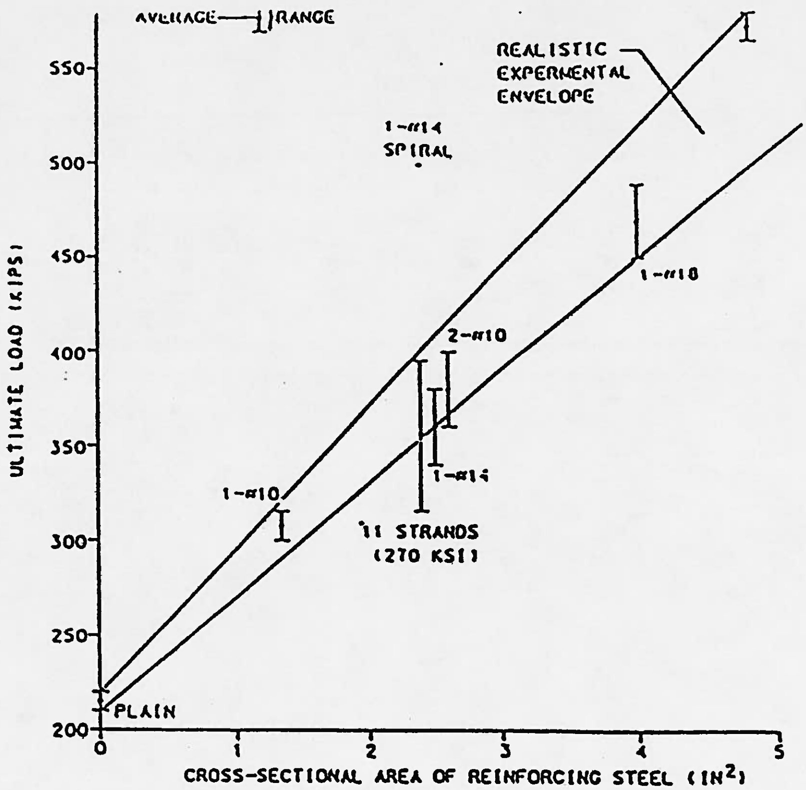

Figure 2 Ultimate Compressive Load vs. Reinforcing Steel Area, Phase 3 Tests. All steel = 60 ksi except for strand.

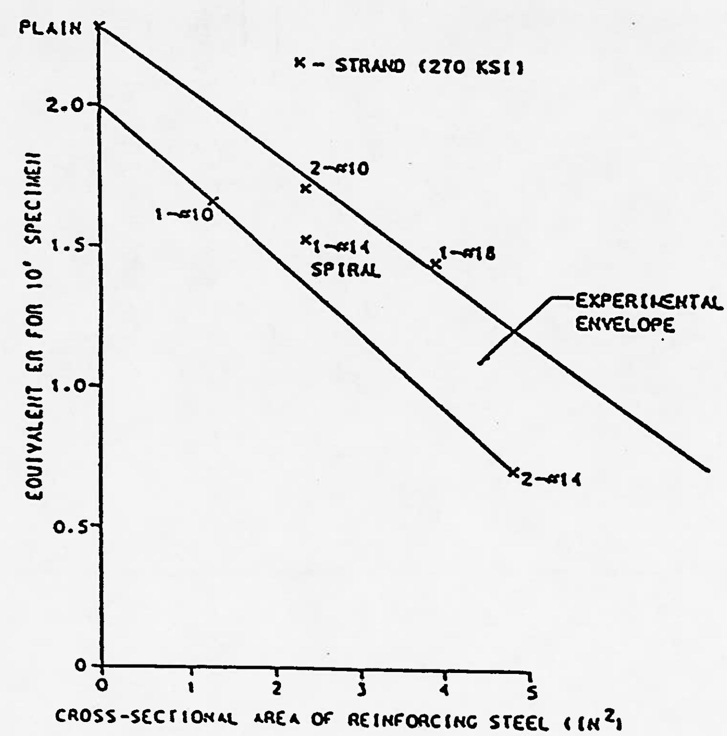

Figure 3 Equivalent ER vs. Reinforcing Steel Area, Phase 3 Tests.

Summaries of the soil strata encountered, and the individual pile installation details are provided in Tables 4 and 5 respectively. Table 6 summarizes the creep and failure behavior.

Table 7 shows the progressive increases in ER with increasing load for each pile. It was always recognized that this progressive increase was indicative of progressive debonding: if debonding were not occurring (i.e. if the pile were acting as a strut with fixed ends) then the ER would be constant, since deflection would be directly proportional to load. However, the question was the relation between ER and effective pile length, and until the Phase 1 laboratory tests this had not been satisfactorily resolved. These tests showed that the ER for a 10 foot length of grout filled 7 inch casing was approximately 0.32. Thus, for a recorded pile ER of, say 2.0, it can be calculated that the effective elastic pile length would be (2.0/0.32) x 10 feet = 63 feet.

The calculated free lengths in Table 7 are generous: no allowance is made for the decreased ER value of the pile in the casing/rebar overlap section. It is clear that explosive, structural failure occurred when the load had been fully shed to within a few feet of the bottom of the cased length. (It also is apparent that at a load of 300 kips, the casing had debonded only about 30 feet below the ground surface, highlighting the surprisingly high load holding capability of the poorer upper soils). The table also shows that the change from a #14 bar (TP1-3) to a #18 bar (TP 4-6) gave the (centrally reinforced) bond zone an extra 70 - 150 kips capacity to resist explosive bursting failure. This compares with the same range identified in the Phase 3 Laboratory Tests. (Table 3).

OVERVIEW OF LABORATORY AND UNITED GRAIN SITE TESTS

The laboratory tests have provided the key to determining effective debonding lengths in high capacity Pin Piles: the breakthrough is the Elastic Ratio concept. The Phase 3 Tests, on simulated bond zones, clearly demonstrated the range of capacities which can be expected, and this range was confirmed in field test programs. Both these field programs also highlighted the fact that large proportions of load may be shed in the upper reaches of Pin Piles, into soil which is usually neglected as having load transfer potential at the design phase. This explains neatly the surprising stiffness of Pin Pile systems especially in their lower range of capacity.

The field tests also remind us that in certain favorable geotechnical conditions, the grout-soil bond which contemporary drilling and grouting methods promote can be so large that it is the internal load carrying capacity of the pile, i.e. its structural strength, which is the determinant of ultimate pile capacity. The ER approach to field analysis of Pin Pile testing offers a precise analytical and predictive tool, especially when combined with creep data: when the extent of apparent casing debonding reaches to within a few feet of the end of the casing, explosive failure may be expected shortly. At such times, the creep monitored may be more a result of grout/steel interfacial phenomena rather than grout/soil conditions as conventionally assumed.

This analytical method opens the door to Pin Pile acceptance criteria similar to those used for prestressed ground anchors where elastic performance and creep patterns are used. This would be more rigorous than current "geometrical construction" type methods.

Two related questions remain to be addressed, namely the puzzle of why a failure load can be recorded in a pile, lower than a load safely reached in a previous cycle, and why failure can occur during a creep test at constant load. The first case is simply explained by reverting to the concept of non-recoverable bond: once the virgin interface around the casing has been disrupted, it cannot sustain the same level of bond stresses. Therefore, when reloaded, the load must pass below the point to be

Test Pile #	Upper Length Soil Strata: Sand Fill (ft)	Silt (ft)	Medium Dense Sand (ft)	Bond Length Soil Strata: Medium Dense Sand (ft)	Very Dense Sand (ft)	Very Dense Gravels (ft)
TP-1	22	16	5	13	12	5
TP-2	20	15	5	7	9	14
TP-3	10	21	8	4	14	12
TP-4	10	21	6	6	14	5
TP-5	10	21	6	6	14	5
TP-6	10	21	6	6	14	5

Table 4 - Soil Strata Thickness Encountered, United Grain Project, WA

Test Pile #	Installation Order	Total Pile Depth (ft)	Bond Length (ft)	Casing Insertion Into Bond Zone (ft)	Casing Length From Grade (ft)	Rebar Size	Rebar Length (ft)
TP-1	2	73	30	5	48	#14	30
TP-2	1	70	30	5	45	#14	30
TP-3	3	69	30	10	49	#14	32
TP-4	4	62	25	10	47	#18	27
TP-5	5	62	25	10	47	#18	27
TP-6	6	62	25	10	47	#18	27

Table 5 - Pile Installation Details, United Grain Project, WA

Pile #	Creep @ 525 kip 0-10 min (mm)	Creep @ 525 kip 10-30 min (mm)	Creep @ 600 kip 0-10 min (mm)	Creep @ 600 kip 10-100 min (mm)	Creep @ 600 kip 100-240 min (mm)	Creep @ 600 kip 240-720 min (mm)	Creep @ 675 kip 0-10 min (mm)	Creep @ 675 kip 10-100 min (mm)	Max Test Load Attained before failure	Hold Duration @ Max. Load before failure	Failure Description
TP-1	.024	.018	.031	.048	.022	---	---	---	600 kips	270 min	Explosive Drop to 300 kips
TP-2	.022	.016	---	---	---	---	---	---	600 kips	3 min	Explosive Drop to 355 kips
TP-3	.028	.012	.028	.031	---	---	---	---	600 kips	45 min	Explosive Drop to 417 kips
TP-4	.024	.010	.022	.032	.012	.037	.025	.059	750 kips	4 min	Explosive Drop to 372 kips
TP-5	.022	.013	.040	.071	.026	.027	.054	.070	750 kips	---	Plunging Drop to 534 kips
TP-6	.021	.013	.032	.036	.010	.038	.035	.061	675 kips	25 min	Explosive Drop to 150 kips

<u>Table 6</u> Test Pile Displacement Creep and Failure Behavior, United Grain, Vancouver, WA

Pile #	Casing Length (ft)	Total Pile Length (ft)	300 kip load Elastic Ratio	300 kip load Elastic Length (ft)	450 kip load Elastic Ratio	450 kip load Elastic Length (ft)	525 kip load Elastic Ratio	525 kip load Elastic Length (ft)	600 kip load Elastic Ratio	600 kip load Elastic Length (ft)	Failure Load (kips)	Failure Load Elastic Ratio	Failure Load Elastic Length (ft)
TP-1	50	75	0.80	25.0	1.29	40.3	1.42	44.4	1.68	52.5	600	1.68	52.5
TP-2	47	72	0.76	23.8	1.07	33.4	1.22	38.1	1.37	42.8	600	1.37	42.8
TP-3	51	71	0.90	28.1	1.21	37.8	1.36	42.5	1.51	47.2			
TP-3	51	71	1.20	37.5	1.47	45.9			1.74	54.4	600	1.74	54.4
TP-4	49	64	0.91	28.4	1.17	36.6	1.42	44.4	1.51	47.2	750	1.80	56.3
TP-5	49	64	0.99	30.9	1.41	44.1	1.54	48.1	1.65	51.6	750	1.80	56.3
TP-6	49	64	0.86	26.9	1.05	32.8	1.16	36.3	1.30	40.6	675	1.44	45.0

<u>Table 7</u> Test Pile Elastic Ratio and Length, United Grain, Vancouver, WA

resisted (Figure 4). This means that progressively less of the casing is capable of resisting load, and so progressively higher proportions of the applied load must be resisted in the bond zone. This bond zone has a finite capacity (internal and external), and when this is exceeded, failure results.

The second riddle has a similar explanation. As less of the casing surface area becomes capable of resisting load as a result of progressive debonding, the average bond stresses increase over the surface area remaining in virgin conditions. This increase accelerates the rate of interfacial creep, which reflects a continuing, accelerating progressive debonding. At lower loads, this creep tendency is low, and is soon stabilized: at higher loads, this creep rate will be higher and may reflect a rate of debonding so relatively fast that the underlying bond zone is being required to accept a substantially and progressively higher proportion of the load over a time interval within the period of the creep test. So, when the critical amount of load is transferred to the bond zone, a sudden explosive failure will occur. This time of transfer may vary from almost instantaneous to many minutes.

SEISMIC RETROFIT APPLICATIONS - GENERAL INTRODUCTION

The California Department of Transportation (CALTRANS) recorded numerous bridge failures in the 1971 San Fernando Earthquake. The failures were linked to separations at bridge deck expansion joints and a lack of ductility in the supporting columns. As a consequence, CALTRANS retrofitted 1250 state bridges to provide deck continuity, from 1971 to 1989, although column ductility retrofitting was delayed until 1986 due to budget constraints. Column ductility improvements of course result in an increase of load demand on both superstructures and foundations.

Investigations into various bridge foundation repairs have been intensified in recent years as a result of the increased availability of funds following the 1989 Loma Prieta Earthquake. (15). One of the most common measures is to add tension/compression piles around the perimeter of an existing footing. Driven precast concrete and steel piles are typically used for foundation support of bridges. However, due to constraints including noise and vibration level limitations, installation difficulties presented by low overhead conditions, difficult drilling and driving conditions due to ground obstructions or high water tables, limited right-of-way access, the inability to extend the footings and higher tension capacity requirements, alternates to standard driven piles are becoming more desirable. Since varying project conditions may be more practically and economically favorable to certain construction techniques, there is no one single "best" solution.

CALTRANS TENSION PILE TEST PROGRAM - SAN FRANCISCO, CALIFORNIA

In late 1991, CALTRANS initiated a full scale pile testing program as part of its seismic structural retrofit program. Foundation retrofits are the most costly element in the seismic retrofit program, fully justifying research into alternate construction techniques. This testing program was proposed as a joint effort between CALTRANS, the Federal Highway Administration, and contractors who could offer proprietary piling systems. CALTRANS tested traditional piling systems such as drilled shafts and driven steel H piles or pipe piles: proprietary systems offered alternatives. As part of this program, Nicholson Construction installed six Pin Piles of three different types at the test site in San Francisco. (16).

A simplified stratigraphy of the test site was:

0 to 20 feet Fill
20 to 110 feet Clay (Soft Bay Mud deposits)
110 to 160 feet Sand

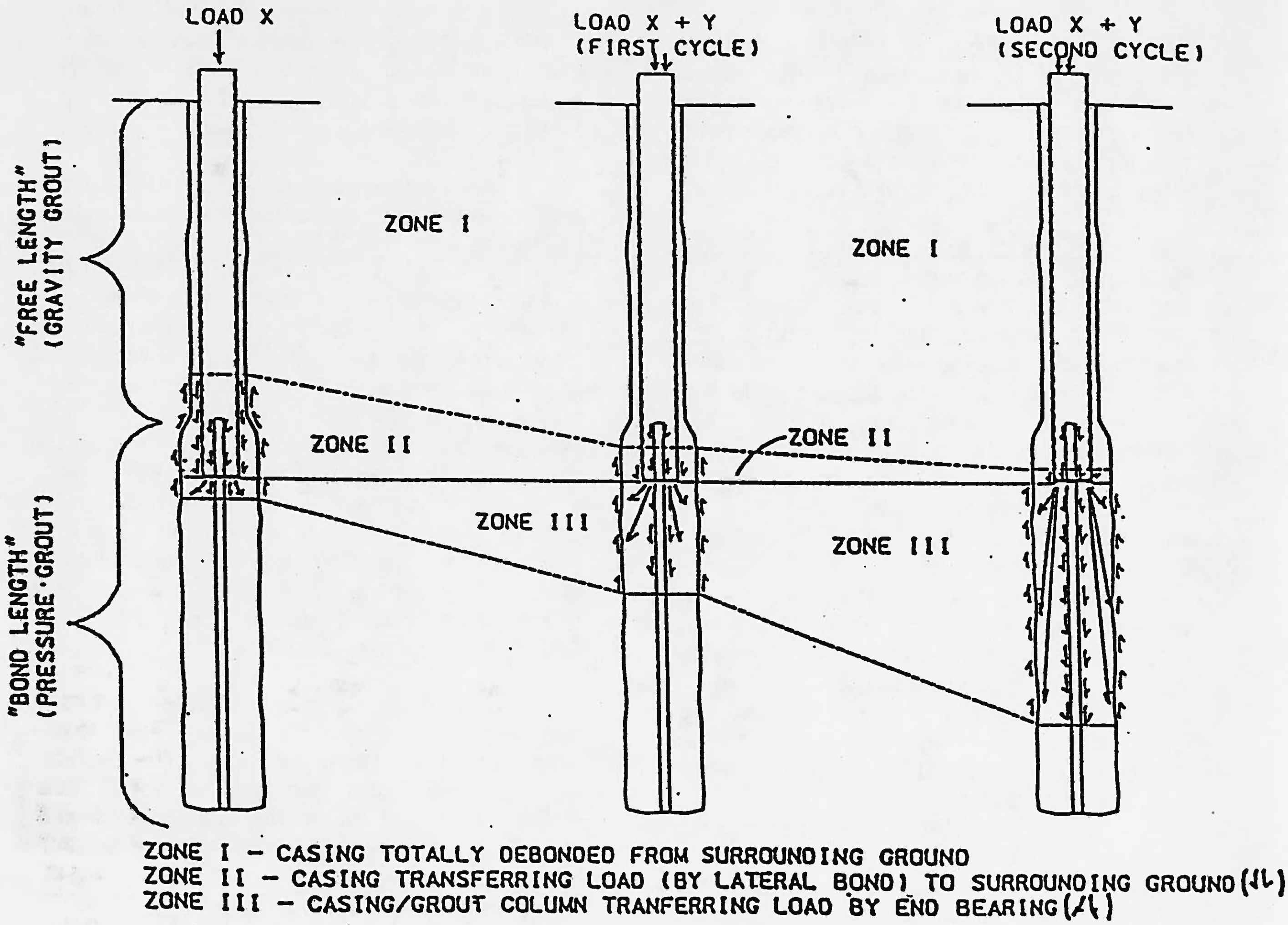

Figure 4 Conceptual illustration of load transfer mechanisms at increasing loads, and at repeated load.

During installation, actual pile lengths were varied in response to the actual conditions encountered. However, to limit test variables, many of the pile components and dimensions were held constant. The three types of piles installed were:

NCA-Pile. A high capacity multistrand tendon was installed within and below the 7 inch dia. steel casing. This tendon was stressed and locked off against the top of the casing prior to the pile test. A 35 foot long pressure grouted bulb was formed in the sand, which included a 10 foot embedment of the casing, a 5 foot buffer zone, and a 20 foot bond length for the tendon. This pile was similar to a type S-1 except that the prestressing tendon replaced the reinforcing bar.

Pin Pile. A 60 ksi reinforcing bar was installed within and below the 7 inch dia. casing (Type S-1). No prestress loading was applied to this pile. The pile had a pressure grouted bulb 30 feet long in the sand which included a 10 foot embedment of the pipe, and 20 feet of reinforcement extending below.

NFC-Pile. The 7 inch diameter steel casing was drilled full length into the lower sands and a 60 ksi steel bar placed full length. (Type S-2). For these piles, the grout was introduced as drilling in the sand progressed.

Two examples of each pile type were installed at the site: one deep pile founded in the sand and one shallow pile founded in the Bay Mud. Deep pile lengths varied from about 140 to 155 feet. Shallow piles were installed to about 105 feet. The piles were tested to a 200 kip design load and then loaded to failure. The Nicholson pile load test results are shown in Table 8.

FIELD RESEARCH - CALTRANS NORTH CONNECTOR OVERCROSSING - I-110 SITE, LOS ANGELES, CALIFORNIA

CALTRANS awarded a construction contract for the North Connector Overcrossing in Los Angeles in 1991. The original design involved retrofitting bents 2, 3, 5 and 6 by strengthening the existing footings. The design used sixteen 24 inch diameter cast-in-drilled-hole (CIDH) concrete piles placed around the existing footing at each single column bent.

An experienced and qualified drilled-shaft subcontractor attempted to install the specified piles. However, due to difficult drilling conditions, including concrete obstructions and water bearing (flowing) sand, and the installation difficulties caused by low overhead conditions, they were unable to complete the installation of any CIDH piles. CALTRANS was aware of the Nicholson Pin Pile through the San Francisco Test Program and subsequently, the general contractor engaged Nicholson to install 64 Pin Piles in lieu of the 64 specified CIDH concrete piles. Detailed plans and calculations were prepared, submitted and approved by the CALTRANS Office of Structures.

The project site was located in Los Angeles near Figueroa Street and the southbound on-ramp to I-5. The soils underlying the site consisted of loose to slightly compact fill in the upper 25 feet and dense to very dense sands and gravels below. The ground-water table was approximately 25 feet below grade.

The project site had been a dump location for a ready-mix concrete plant, and the upper fill zone contained large chunks of concrete and rubble. Three of the retrofitted footings were located adjacent to the Aroyo Seco drainage channel, and were accessible only by a graded road or from the edge of the Pasadena Freeway. The fourth footing was located in the middle of the Pasadena Freeway, creating very difficult access conditions. Overhead clearance under the freeway superstructure was approximately 20 feet.

The Type S-1 Piles for this project were required to support an ultimate compressive load of 500 kip with a maximum pile head total deflection of less than 0.60 inches. Each pile comprised:

- An upper pile length extending to 30 feet below the bottom of the existing footing, consisting of a 7 inch dia.,1/2" wall thickness steel casing, reinforced full length with two 1-3/8 inch, diameter grade 150 threadbars and filled with neat cement grout.

- A pile bond length extending from 30 feet to 60 feet below the bottom of the existing footing, consisting of a pressure grouted bond zone, reinforced with the two 1 3/8 inch diameter threadbars, extending to the pile tip, and the 7 inch diameter steel pipe, extending 5 feet into the top of the bond length.

- A specially designed connection between the pile and the cast-in-place extension to the structure footing.

The production test pile (Bent No. 3, Pile No. 3, selected by CALTRANS) was drilled with a high-torque, low-headroom drill rig. It was installed from existing grade to a depth of approximately 66.5 feet, allowing testing to be performed before footing excavation. The casing was placed in 10 feet lengths, and the threadbars were placed in 10 feet and 20 feet coupled lengths, centralized in the pile with plastic spacers. Maximum grout pressure attained during grouting of the pile bond length ranged from 100 to 140 psi measured at the drill rig.

The pile test was conducted by representatives from the CALTRANS Office of Structures. The tension test was completed to the required 300 kip load and the compression test to the required 500 kips. The pile was loaded in 100 kip cycles, with the load applied in 20 kip increments, and reduced in 20 to 100 kip increments. Each increasing load increment was held for 5 minutes the first time at that load, and for 2 minutes thereafter. Each decreasing load was held for one minute.

PILE NO.	DESCR.	PILE CAPACITY (KIPS)		ACTUAL ELASTIC DEFL. @ 200 KIPS		ACTUAL TOTAL DEFL. @ 200 KIPS	
		Tension	Compression	Tension	Compression	Tension	Compression
10,A	NCA-Deep	407	*	.461"	- - -	.503"	- - -
11,B	NCA-Shallow	243	160	.329"	N/A	.370"	N/A
12,E	NFC-Deep	500	>400	.302"	-.289"	.310"	-.348"
13,F	NFC-Shallow	195	220	.302"	-.270"	.414"	-.420"
73,A	Pin-Deep	455	>385	.530"	-.516"	.631"	-.581"
74,B	Pin-Shallow	189	373	N/A	.351"	N/A	.378"

Table 8 Summary of Test Results for Nicholson Piles in CALTRANS Test Program

* Pile damaged during tension test loading. No Compression Test Results

N/A Not Applicable

Figure 5 summarizes the load test data. The pile successfully resisted the required maximum tensile load of 300 kips with a total displacement at maximum load of 0.304 inches and a permanent displacement of 0.050 inches at zero load after loading. Creep movement during the 5 minute hold at 300 kips was 0.006 inches. The pin pile then successfully supported the required maximum compressive load of 500 kips with a total displacement at maximum load of 0.392 inches and a permanent displacement of 0.068 inches at zero load after loading. Creep during the 5 minute hold at 500 kips was 0.007 inches .

OVERVIEW OF CALTRANS TESTS

The Nicholson Pin Pile proved to be an excellent system for meeting the design load capacity and displacement requirements. The Pin Piles were also installed with relative ease at difficult access sites and ground conditions which prohibited the installation of conventional pile types. Pressure grouting techniques in the dense sands and gravels resulted in very high grout/soil bonds and small displacements. Even in the soft Bay Muds, surprisingly high skin friction values were mobilized. The response of the Pin Pile to test loads was essentially elastic, with very small permanent displacements. These observations offer real hope that the special demands imposed on pile performance by the particular demands of California can be adequately met by the appropriate proprietary option.

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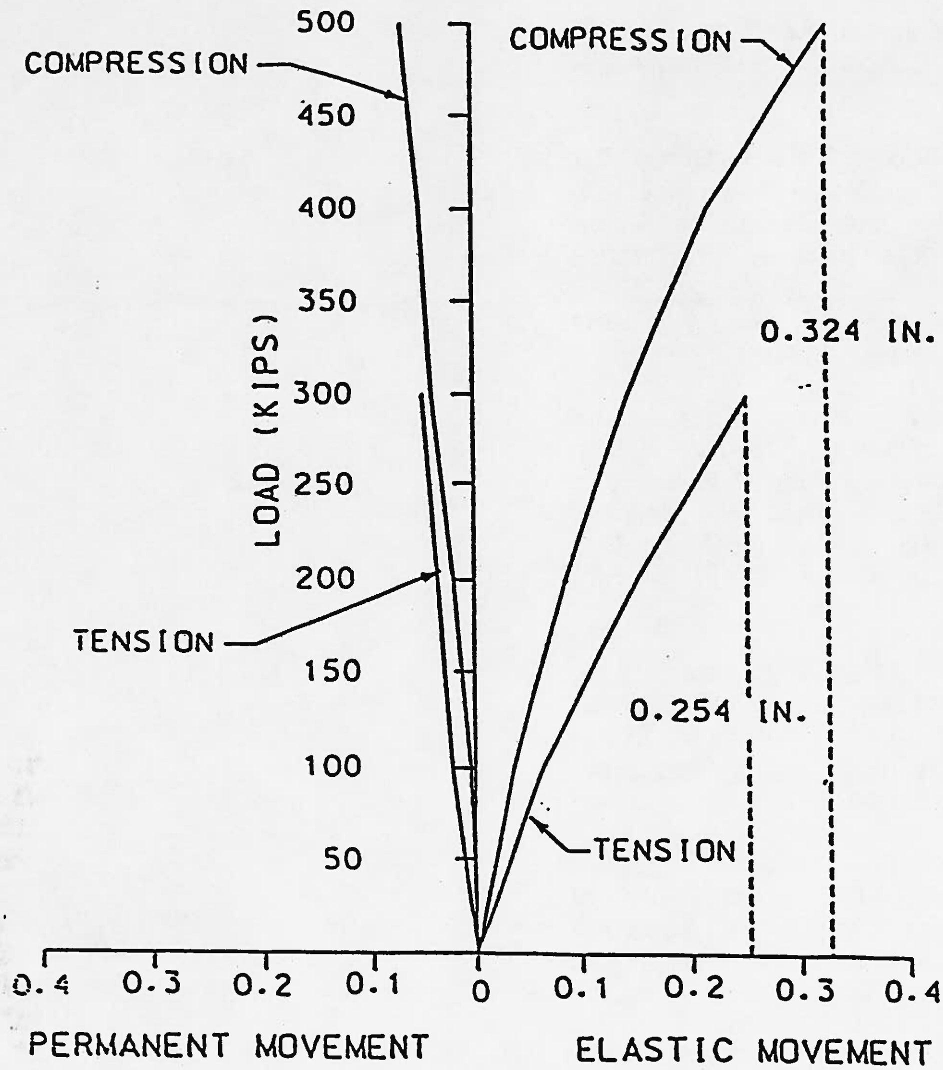

Figure 5 Permanent and Elastic Displacement Analysis North Connector Overcrossing I-110, Los Angeles, CA

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DATABASES FOR DEEP FOUNDATIONS

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ABSTRACT

In recent years, several collections of load test information have been developed and implemented into database form. This paper presents some of the recent databases developed. Current design procedures for deep foundations are influenced significantly by results of axial load tests. New theories, new experimental procedures, or new field tests for determining axial behavior of drilled shafts are adopted or dismissed based on the ability to predict behavior better, or more economically, than existing procedures. Uncertainties associated with predictive methods are also illuminated by comparing predicted and measured behavior for a large number of tests. Ideally, a convenient, and readily accessible collection of carefully documented load tests would provide the researcher, design engineer, or licensing agency a convenient tool for evaluating new methods. Benefits of employing database technology include the following: 1) the database allows the focus to be directed toward interpretation of data and frees the investigator from the laborious task of collecting, sorting, and manipulating data by hand, 2) the database enhances the development of new methods of analysis, 3) the database helps identify variables and topics requiring further study, 4) the database can be updated easily, and 5) the database can be modified easily to include additional data or can be extended to include other foundation types.

INTRODUCTION

A database is defined as a collection of related information arranged specifically for retrieval. Databases containing information on axial load tests on deep foundations are referred to as load test databases. The load test database can provide a convenient means to provide a variety of information to engineers, licensing agencies, and researchers.

A database in which the data are stored in electronic form suitable for retrieval is termed an electronic database. Electronic databases can manipulate large amounts of data quickly and accurately, and therefore, provide the user with an effective tool for collecting, sorting, filtering, and analyzing information. Similar manipulative efforts on the database attempted without the use of an electronic database would be too tedious and labor intensive to justify.

Databases on axial load tests are discussed herein, with particular emphasis on four electronic databases that have been developed over the last 15 years. A brief historical perspective of database efforts is provided first, followed by a discussion of two important database features: the quality and quantity data in the database, and the ability of the database to manipulate, analyze, and compare the data.

USEFULNESS OF LOAD TEST DATABASES

Before delving into details of load test databases, the question "why collect load test results" should be addressed. In discussions with anyone involved with developing a useful database on deep foundations, it is generally agreed that the effort required is immense, tedious, hard, and time consuming. Thus, the investment required to develop a database must be compared to the benefit expected from possession of the collection of load tests, and from analyses of the data.

The advantages of a load test database depend on the information required by the user. Four main important advantages of a database are 1) the data and results are organized for easy retrieval, 2) the collection allows a user to establish precedent, and identify what has been done previously, 3) the database can be used to support efforts to assess and improve current methods for predicting axial behavior, and 4) the database can be used to support efforts to assess and improve new methods for predicting axial behavior.

Three examples are provided to illustrate the variety of demands a user may impose on a database. Example 1 - a contractor requires selected tests that document the use of a specific foundation to convince the client that such a foundation has been used before under similar circumstances, and is therefore a viable foundation option. Example 2 - a government agency may be interested in reviewing their design procedures. Thus, the database must provide the user with information necessary for the design method to predict a capacity. Then predicted capacities can be compared with measured capacities from the database of load tests. Example 3 - a new design method has been developed, and the database is used to evaluate the new method's accuracy.

Undoubtedly, additional examples could illustrate other demands upon the database; however, the examples provided emphasize the need for the database to allow the user access to data in a variety of different forms. Therefore a database must provide the user a convenient means to extract a variety of data, or the usefulness of the database is jeopardized.

BRIEF HISTORY OF LOAD TEST DATABASES

A brief review of some past efforts to collect axial load test information allows current efforts to be viewed with some historical perspective; however, it is emphasized that this review presents only a small portion of all the collection effort that has been undertaken. A thorough review is beyond the scope of this paper.

Load test databases have been collected since axial load tests have been performed. Bergfelt (1957) reported results of axial load tests on timber piles in soft clay that were tested in the late 1800's. One of the earliest axial load tests on a large hand-dug pier was conducted in 1921 and reported by D'Esposito (1924). An axial load of 1000 tons was applied to a test pier for Union Station in Chicago. Other efforts include those by Tomlinson (1957, 1970), Peck (1958), Woodward, et al (1961), Norlund (1963), Whitaker and Cook (1966), Vijayvergia and Focht (1972), Meyerhof (1976), Blendy (1980), Coyle and Costello (1981), Dennis and Olson (1983), Kulhawy, et al. (1983), Rausche et al. (1985), Briaud, et al. (1986), Tucker (1986), Richman and Speer (1989), Reese and O'Neill (1988), Long and Shimel (1989), and Davidson and Townsend (1993). Although all these collection efforts are noteworthy only four of these references refer to a collection of load tests entered and interpreted using an electronic database. The discussions for the electronic databases are referred to as Olson, Briaud, Long, and Davidson and Townsend. Discussion of Peck's effort is described below, while additional discussion of the other collection efforts is provided later.

Peck(1958) presented results of an effort to collect and interpret the results of a large number of axial load tests as part of a study initiated by the Bridge Design Committee of the Highway Research Board. The study was directed toward identifying the axial capacity for driven piles. A questionnaire was sent to the bridge department of all state highway departments, to American and Canadian railroad companies, to major consulting firms, and to several other state and governmental agencies. Well over 1000 load tests were reviewed, but only 412 were deemed informative, and only 190 contained enough soil information to be useful (three years later the number of useful tests was reported as 117). In addition to collecting axial load test results, results were analyzed to develop methods to assess axial capacity. In 1961, a report summarizing 412 axial load tests was provided by Peck (1961). The report summarized each axial load test results with a one page graphical summary detailing the load test results, the soil profile, a driving history, and other information. An example of Peck's summary sheet is shown in Fig. 1.

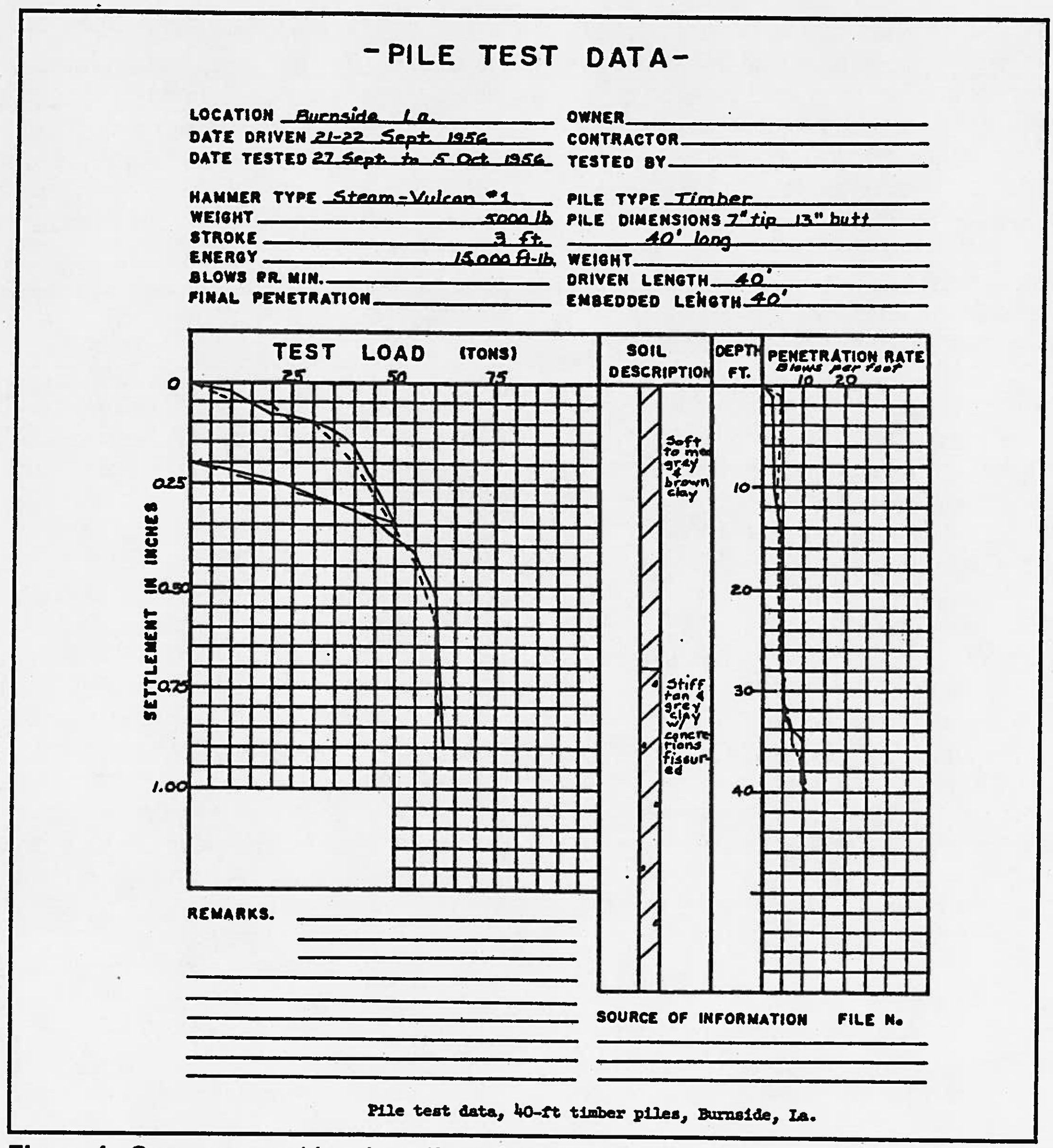


Figure 1. Summary graphics sheet (from Peck, 1958)

DATA COLLECTED IN DATABASE EFFORTS

The details recorded in a database of axial load tests help identify important characteristics that may assist in identifying specific behavior in later analyses. Therefore, as much detail as practical is collected for each load test. Important database details may be categorized into five categories. These categories are:

- General Information
- Pile Properties
- Subsurface Properties
- Construction Details
- Load Test Results and Details

Extensive details for each of these categories can be managed only with the assistance of an electronic database. The quantity of data entered in each of the categories depends on if the information is believed to be important. Some databases, in an attempt to be as comprehensive as possible, attempt to record very detailed information for each category. Other database efforts record only information that appears directly relevant to the current efforts. While these two approaches appear quite different, the lack of data in a typical load test report may result in the same data record for both databases. However, the more detailed database allows greater detail to be recorded for each load test, when the data are available.

General Information

General information for a load test refers to the data that identifies non-technical data important for the load test. An incomplete list of general information includes: a load test number, location, date of test, reference, owner, engineering firm, engineer, drilled shaft contractor, load test contractor, soil testing firm, and any additional comments that may be relevant to identifying the general details of the load test.

Pile or Shaft Properties

Information that identifies the specific geometry and material properties of a pile or drilled shaft is important to record. An incomplete list for identifying the properties of a deep foundation includes: its geometry (diameter vs. depth, total length, embedded length), its structural properties (axial stiffness, compressive strength of concrete, amount of reinforcement, detail of casing), and any additional comments that are necessary to further identify the properties.

Subsurface Properties

Information on soil properties and general subsurface conditions is very important to identify. Details are recorded in this category since soil properties play an important role in the development of axial capacity. Furthermore, the strength of soil may be estimated from a number of different tests, and may be affected by the sampling procedure. Therefore, it is important to identify which technique was used for identifying soil strength. An incomplete list of information for identifying soil properties falls into four subcategories which are general information, lab strengths and simple strength test results, field measurements for estimating strength, and consolidation properties. The incomplete list of items includes: general (position of GWT, sampling method, layer number, soil type {USCS}, layer thickness, γ, σ, water content, LL, PL, geologic origin, gradation, etc.), strength test results (PP, TV, LV, unconfined compression, triaxial {UU, CD, CU}, direct shear, interface shear, other), field results (Std Pen test, cone, vane, pressuremeter, other), consolidation properties (compression index, recompression index, swelling index, maximum previous pressure, over-consolidation ratio), and finally, any additional comments on soil properties that might identify the properties more precisely.

Construction

The effects of construction and installation on the capacity of deep foundations have been a topic of particular interest for drilled shafts, and are also relevant to driven piles. Accordingly, details of the construction procedure are recorded. Construction details for drilled shafts include: drilling equipment, bottom cleanout, method of drilling, slurry characteristics, concreting method, concrete slump, total construction time required, and any additional comments that will provide the user with a better idea of the construction details. Construction details for driven piles would be similar to the items listed for drilled shafts, but, as a minimum, should include items to identify the type of equipment used to drive the piling, and a record of the driving resistance with depth. Additional information may include dynamic measurements during pile driving.

Load Test Results and Details

A partial list of items that identify axial load test results includes: load direction, test procedure, primary and secondary methods to measure load and displacement, instrumentation, axial load versus axial displacement at top, axial load versus axial displacement at base, time between installation and testing, and any additional comments that can better describe details of the load test results. In addition, reference loads at deflections of 0.5" + PL/AE, 2.5%D + PL/AE, 5%D + PL/AE, are recorded as well as total load, load carried at tip, and load carried along the side.

Number Quality Factor

The data in the database have no mechanism for identifying the confidence associated with the number. For example, a high quality load test to failure may be performed that is worthy of entering into the database; however, the nearest soil boring is 300 yards away. While the load test results are good, the confidence in soil properties is not high. Furthermore, a standard

Table 1. Definitions for Number Quality Factor (NQF)

NQF	GENERAL DESCRIPTION	CONFIDENCE LEVEL	CONSTRUCTION PARAMETERS	SOIL PARAMETERS	LOAD TEST PARAMETERS
0	The specific parameter was not reported and an educated was not possible.	None	Unknown	No soil information	No load test information
1	The specific parameter was not reported and was determined by an educated guess	Very Little		1 test performed	Load at failure
2	The specific parameter was not reported directly, but was determined by indirect measurements or extrapolation of data from a drawing or other figure	Little		1 boring in near vicinity with less than 3 tests performed	Load and deflection at failure
3	The specific parameter was reported directly or was scaled with confidence from a drawing or other figure	Moderate	As-Designed	1 boring in near vicinity with 3 or more tests performed per layer	Load vs settlement plot
4	The specific parameter was reported directly and indirect data also confirms the data. The value may also be calculated indirectly from other parameters with high levels of confidence	High		2 borings near vicinity with 3 or more tests performed per layer which show comparable results	Load vs settlement plot and load distribution curves
5	The specific parameter was reported directly from extensive data that was carefully measured and clearly reported. There is complete confidence in this parameter as it was verified with in-field observations or measurements if applicable	Very High	As-Built	More than 2 borings in near vicinity with very comparable results seen in 3 or more tests	Very extensive load test data including load, settlement, and time

penetration test (blow counts) was performed, but due to the distance from the borehole to the pile load test location, the blow counts may not be representative of site conditions at the pile. The database should somehow reflect the low degree of confidence for the standard penetration test. However, at the same site, several cone tests were performed near the test pile and the results are therefore associated with a higher degree of confidence. Likewise the database should include a means to identify the cone results to be of high reliability.

There is no convenient and objective means to identify the level of confidence for a specific detail within the database. However, some database efforts have attempted to address the degree of reliability by assigning a "quality factor" to the database. For example, load test results may be assigned a "quality factor" to reflect the confidence in the load test results. Likewise, a soil quality factor may be assigned to represent the degree of confidence in the soil properties. A means to assess the confidence for a property is given in Table 1.

DATABASE FEATURES - METHODS FOR MANIPULATING DATA

Efficient handling of all the data described in the previous section requires that an electronic database be used to manipulate the data. An electronic database is designed precisely for the purpose of manipulating data; and therefore is well suited for database management for axial load test results. However, there are several approaches available for database management, and the usefulness of the approach depends on the specific needs of the user. The two main functions for a database would be data manipulation and data analysis. Four approaches are described that can be used to perform these database functions which are as follows: a flat-file database, a relational database, a spreadsheet, and a program written in computer code such as FORTRAN.

Database Management

Data manipulation. An effective database allows the user to manipulate information in ways that are useful. For example, the user may wish to see the database organized sequentially by load test number, or by date, or by location. This action is called *sorting*. If the user wishes to see only a portion of the data, for example, only piles between the length of 50 to 60 ft. in sand, then the database should provide a convenient means to allow the database to be *filtered*. Finally, the database should allow the user to *add* or *delete* any load test, or load test data.

Data analysis. Computational efforts to analyze the data may differ significantly depending on the requirements of the user. If the user needs the database mostly for reference purposes, the demands are primarily for data manipulation. However, if the user is developing and assessing current and new methods for analyzing deep foundations, more extensive computational effort is required. Analyses and comparisons include computing axial capacities by a method that incorporates soil properties, pile properties, and/or any other items that enter into the mathematical formulation of the bearing capacity formula. In addition, analyses may compare predicted and measured capacities by plotting results, and/or by determining specific statistics associated with the prediction method.

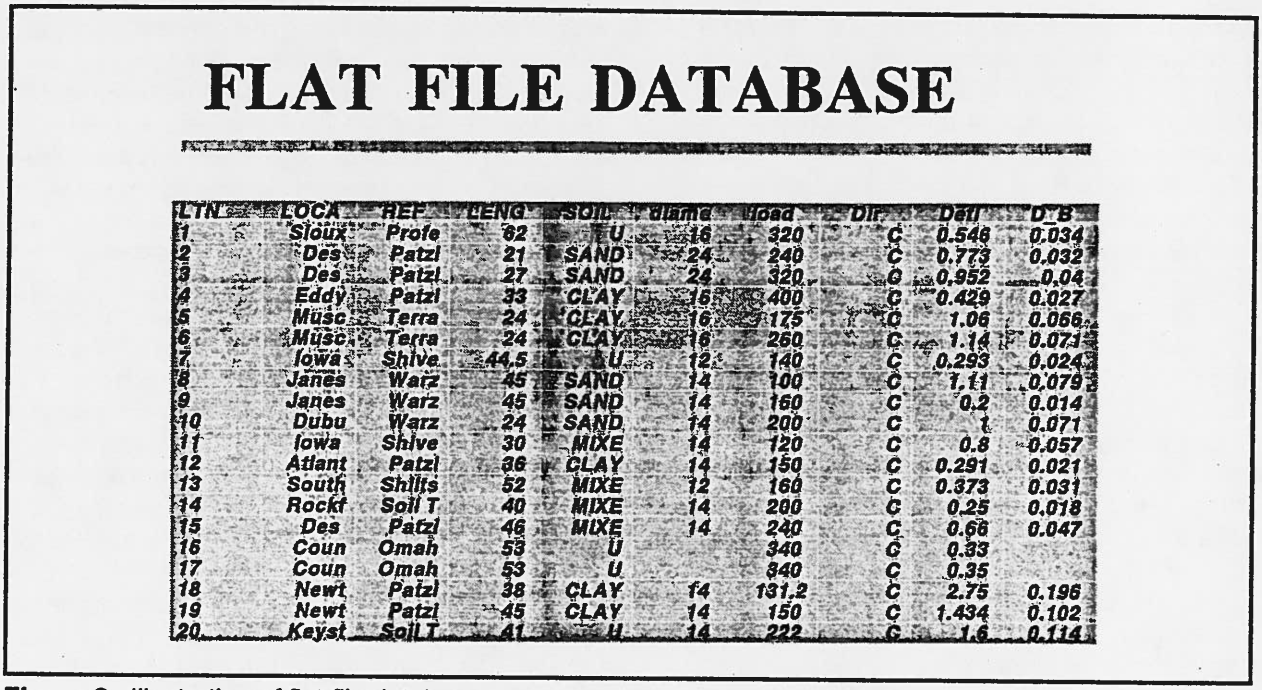
Four Approaches for Database Management

The four approaches for database management include a flat-file database, a relational database, a spreadsheet, and a computer program coded specifically for the database. Each of the approaches have significant advantages and disadvantages.

Flat-file database. A flat-file database is essentially a table of results. The table consists of rows and columns in which the columns represent a specific detail, such as strength, or diameter, etc. A flat-file database can accommodate a large number of columns. Each row represents a different load test. All information in the flat-file database fits in the table, and all the information for one load test exists along a single row. An example of a flat-file database is given in Fig. 2.

Flat-file databases are easy to use and learn. Little time is required to become a proficient user. Since the database is in the form of a table, the data can be accessed intuitively. In addition, many flat-file databases allow common forms of data manipulation to be conducted simply. Sorting, filtering, adding and deleting files are simple and fast. Columns containing results of analyses can be used for comparing predicted versus measured capacities. Many flat-file databases exhibit the capability to plot results of one column versus another. Thus, the flat-file database can be used for analyses of the database as well. For operations that require mostly data manipulation (sorting, filtering, adding, deleting, and plotting), the capabilities and simplicity of the flat-file database are generally superior to any other method. However, a flat-file database is not well suited for extensive and complicated calculations and cannot provide the user a convenient means to compute failure loads according to several different methods,

Relational Databases, and Relational Programmable Databases. A relational database offers a more powerful means to manipulate, organize, and analyze data. The relational database may consist of several flat-file tables linked together by special columns. For example, a relational database for axial load tests may have a table that identifies the general information for the load test, while another table identifies the soil characteristics, and another table identifies the pile properties, etc. The tables relate to one another by key columns as illustrated in Fig. 3. The "General " table contains three columns that identify the load test number, the location number of the load test,



FLAT FILE DATABASE

LTN	LOCA	REF	LENG	SOIL	diame	load	Dir	Defl	D_B
1	Sioux	Profe	62	U	16	320	C	0.546	0.034
2	Des	Patzl	21	SAND	24	240	C	0.773	0.032
3	Des	Patzl	27	SAND	24	320	C	0.952	0.04
4	Eddy	Patzl	33	CLAY	16	400	C	0.429	0.027
5	Musc	Terra	24	CLAY	16	175	C	1.06	0.066
6	Musc	Terra	24	CLAY	16	260	C	1.14	0.071
7	Iowa	Shive	44.5	U	12	140	C	0.293	0.024
8	Janes	Warz	45	SAND	14	100	C	1.11	0.079
9	Janes	Warz	45	SAND	14	160	C	0.2	0.014
10	Dubu	Warz	24	SAND	14	200	C	1	0.071
11	Iowa	Shive	30	MIXE	14	120	C	0.8	0.057
12	Atlant	Patzl	36	CLAY	14	150	C	0.291	0.021
13	South	Shilts	52	MIXE	12	160	C	0.373	0.031
14	Rockf	Soil T	40	MIXE	14	280	C	0.25	0.018
15	Des	Patzl	46	MIXE	14	240	C	0.66	0.047
16	Coun	Omah	53	U		340	C	0.33	
17	Coun	Omah	53	U		340	C	0.35	
18	Newt	Patzl	38	CLAY	14	131.2	C	2.75	0.196
19	Newt	Patzl	45	CLAY	14	150	C	1.434	0.102
20	Keyst	Soil T	41	U	14	222	C	1.6	0.114

Figure 2. Illustration of flat-file database

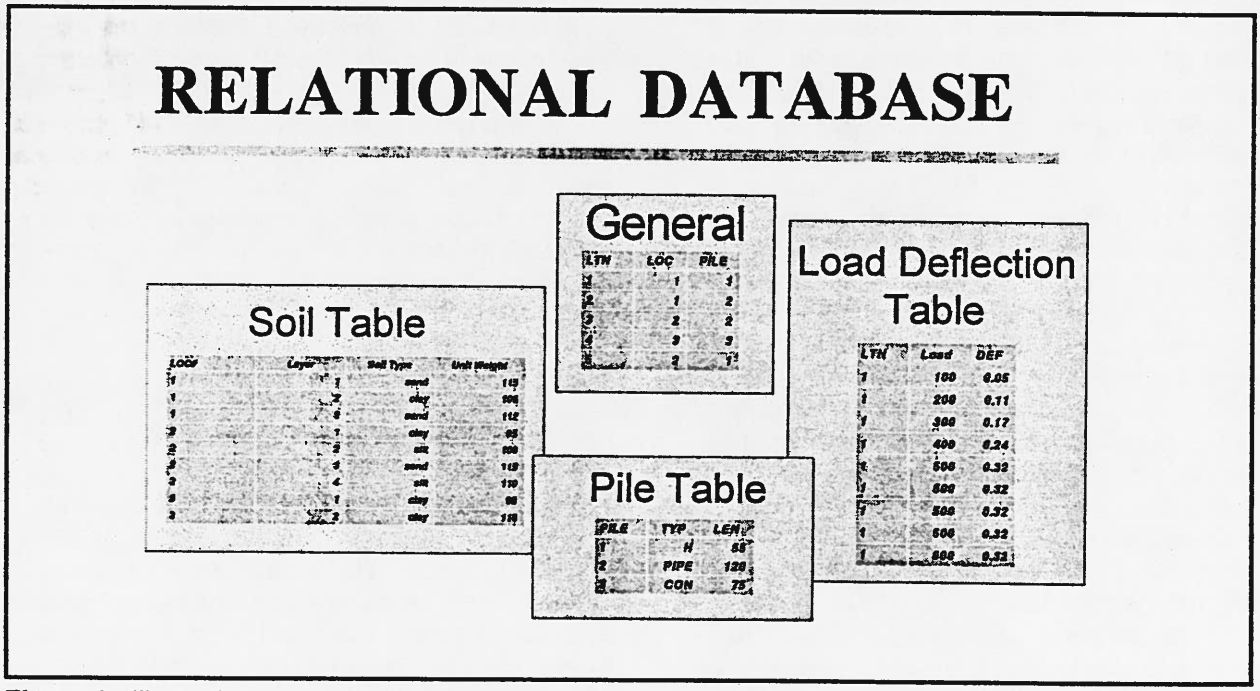


Figure 3. Illustration of relational database

and the pile number for each load test. The other tables identify additional information that *relates* to the pile number, or location number, or load test number. In turn, columns in other tables may *relate* to information in other tables.

An advantage of a relational database is that a complicated database can be divided into manageable pieces. Data may not have to be repeated (e.g. if several load tests were conducted at the same site with similar soil profiles, the soil information would only be entered once, and each of the load tests would refer to the one soil profile), thus, some economy and efficiency in data input and data storage is realized. New tables identifying new information can be added conveniently. Sorting, filtering, adding and deleing is possible and convenient to accomplish.

Programmable relational databases allow the developer of a database to include sophisticated operations that could include automatic calculation of axial capacity, automatic plotting of results, and automatic determination of the statistics used to assess the agreement between predicted and measured capacity. Programmable databases are versatile and powerful, but a more comprehensive discussion of their capabilities is beyond the scope of this paper. Some relational and programmable databases can control some executable programs written in a language such as BASIC, FORTRAN, PASCAL, or C. The combination of a relational database with programming languages offers the user a useful and convenient tool for data manipulation and analysis. The combination of a relational database with user-developed programs offers the most powerful option to the database user.

The end product of a programmable, relational database can be a convenient tool to the database user. The finished database can be user friendly with pop-up help screens, convenient plotting, analyses, etc. However, the database cannot be modified easily, so the user is restricted to follow the options identified by the database. The development of a programmable, relational, database is considerably more labor intensive than for the flat-file database.

Spreadsheet Approach. The third approach for data manipulation and analyses is to use a spreadsheet for storing, analyzing, plotting, and for data manipulation. The spreadsheet approach assumes the user has some knowledge of spreadsheets (which is often the case), but even if no knowledge exists, the user can become proficient with minimal effort. The spreadsheet approach is more complicated yet more versatile and powerful than the flat-file approach, but less complicated, versatile, and powerful than the relational, programmable database approach.

Embedded macro-commands can allow the spreadsheet to perform many of the same functions that one could achieve using a programmable, relational, database. Menus, automatic calculation and plotting, user friendly screens, can be designed with spreadsheets. Since all the commands and macros are part of the spreadsheet, and therefore, accessible to the user, a moderate knowledge of the spreadsheet macro commands allows the user to create their own macros and thus customize it to suit their needs.

ASCII Files and Custom Programs. All data for this approach are stored in an ASCII file (or files). Data manipulation (sorting, filtering) is conducted with an executable version of a program written specifically to analyze the database (plotting, computing capacities, etc.). This approach represents a method that is as powerful and versatile as any of the other methods discussed above. Furthermore, if the user has knowledge of a programming language, the user may choose to write a database analysis program for custom operations on the database.

Depending on the capabilities of the customized computer program that manipulates the data, this approach can offer the most advanced and powerful options, or the least advanced and least powerful options. In any case, the approach offers users an option to develop their own method for data analyses, or use and modify the existing program.

FOUR LOAD-TEST DATABASES

Database features and data characteristics for four electronic databases are described herein. The databases have been developed by Briaud and his co-workers at Texas A&M, Davidson and Townsend at the University of Florida, Long and his co-workers at the University of Illinois, and Olson at the University of Texas in Austin. Information is provided first on the quantity and character of data in the database, then the features of the database are described briefly.

Texas A&M Database

Briaud and Goparaju have developed a programmable, relational database called PILEHELP as a general purpose database to record load test data. The data in the database consist of 92 pile load tests and 6 load tests on drilled shafts. Using a definition that failure occurs at a settlement of D/10 + PL/AE, the number of piles loaded to failure reduces to 72, while the number of drilled shafts loaded to failure remains at 6. All the load tests are from Mississippi, and are described by Briaud, et al. (1986) and Briaud and Tucker (1988).

The database is comprehensive and allows the user to input very detailed data. The database can also record lateral load tests. Capabilities for analysis include 10 different methods for predicting axial capacity and comparative plots and statistics can be specified. The capability for sorting, filtering, adding and deleting load tests is conducted by moving through user friendly screens. Several pop-up help windows provide the user with guidance and helpful information.

University of Florida Database

Davidson and Townsend (1993) report on a collection of load test results and database developed for the Florida Department of Transportation. The database includes load test information on a total of 72 piles and 223 drilled shafts. Using the Davisson criteria for failure, 61 piles are identified as failed. Using the criteria for failure proposed by Reese (deflection at base of shaft equal to 5% of the base diameter) 42 drilled shafts are loaded to failure. The majority of pile load tests in the database are from Florida, and about 25% of the drilled shafts loaded to failure are from Florida.

The database developed by Davidson and Townsend really consists of two databases, one for piles and one for drilled shafts. The database is developed on LOTUS 123 spreadsheets. Several macros allow the user to access the data, analyze the data, filter the data, and plot and print the data by maneuvering through easy-to-follow, user friendly, screens. The spreadsheet macros allow the user to access the details of individual load tests. A moderate amount of data is used to identify the details associated with each load test.

University of Illinois Database

Long has directed database collection efforts specifically toward drilled shafts (Long and Shimel, 1989; Wysockey and Long, 1994), and therefore has no axial load tests on piles. The results of over 920 axial load tests on drilled shafts have been collected; however, only 280 tests exist with adequate load test information, and with enough deflection to satisfy the failure criteria specified by Reese. The load tests collected originate from the U.S. and from International sources.

Two databases are maintained. The first database is a flat-file database used to catalog all axial load tests conducted on drilled shafts. The second database contains detailed information on the load tests with enough information to be useful and enough deflection to meet Reese's criteria. The detailed database is maintained with a programmable, relational database. All sorting, filtering, and data manipulation is performed with the database. Analyses are conducted by writing the database information onto an ASCII file, and conducting all computations, comparisons, and plotting with FORTRAN programs. Summary plots of a load test is shown in Figs. 4a and 4b. The quantity of data recorded for each load test is extensive to very extensive.

University of Texas Database

Database efforts by Olson and his co-workers (Olson and Dennis, 1983; Dennis and Olson, 1983) have concentrated on axial load test carried to failure on piles. Drilled shafts are not included in his database. The collection of axial load tests to failure (Division criteria) currently exceeds 1100 piles, and thus, to the author's knowledge, represents the largest collection of axial load tests in existence. Olson and his co-workers have reviewed a total of over 7000 load tests.

Extensive data recorded for each load test is stored in an ASCII file and edited with a DOS editor. Filtering, sorting, analysis, and plotting of the data are conducted with a FORTRAN program. Sample output of computed versus measured capacity from the program is shown in Fig. 5.

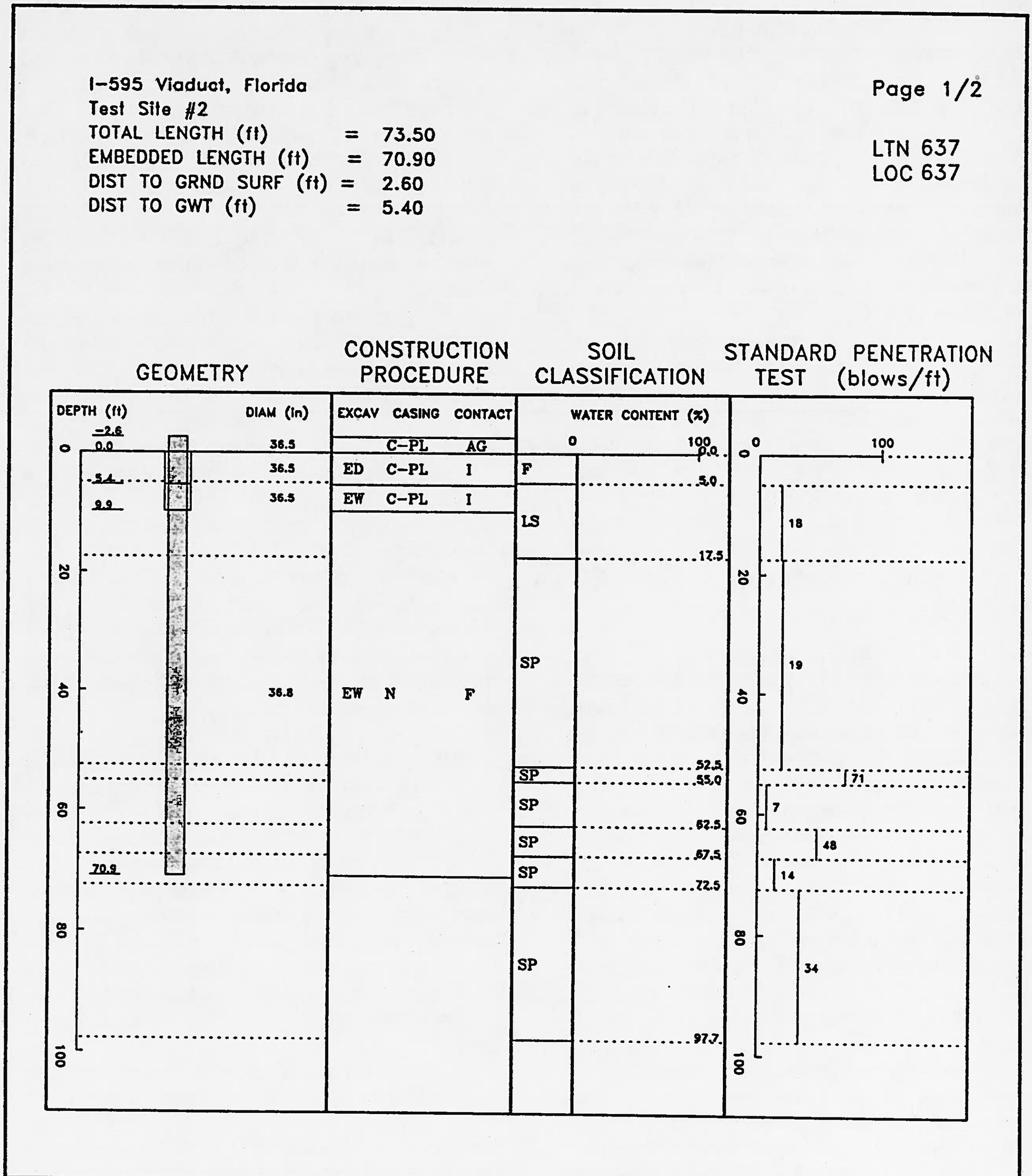


Figure 4a. First page of graphical summary in University of Illinois database

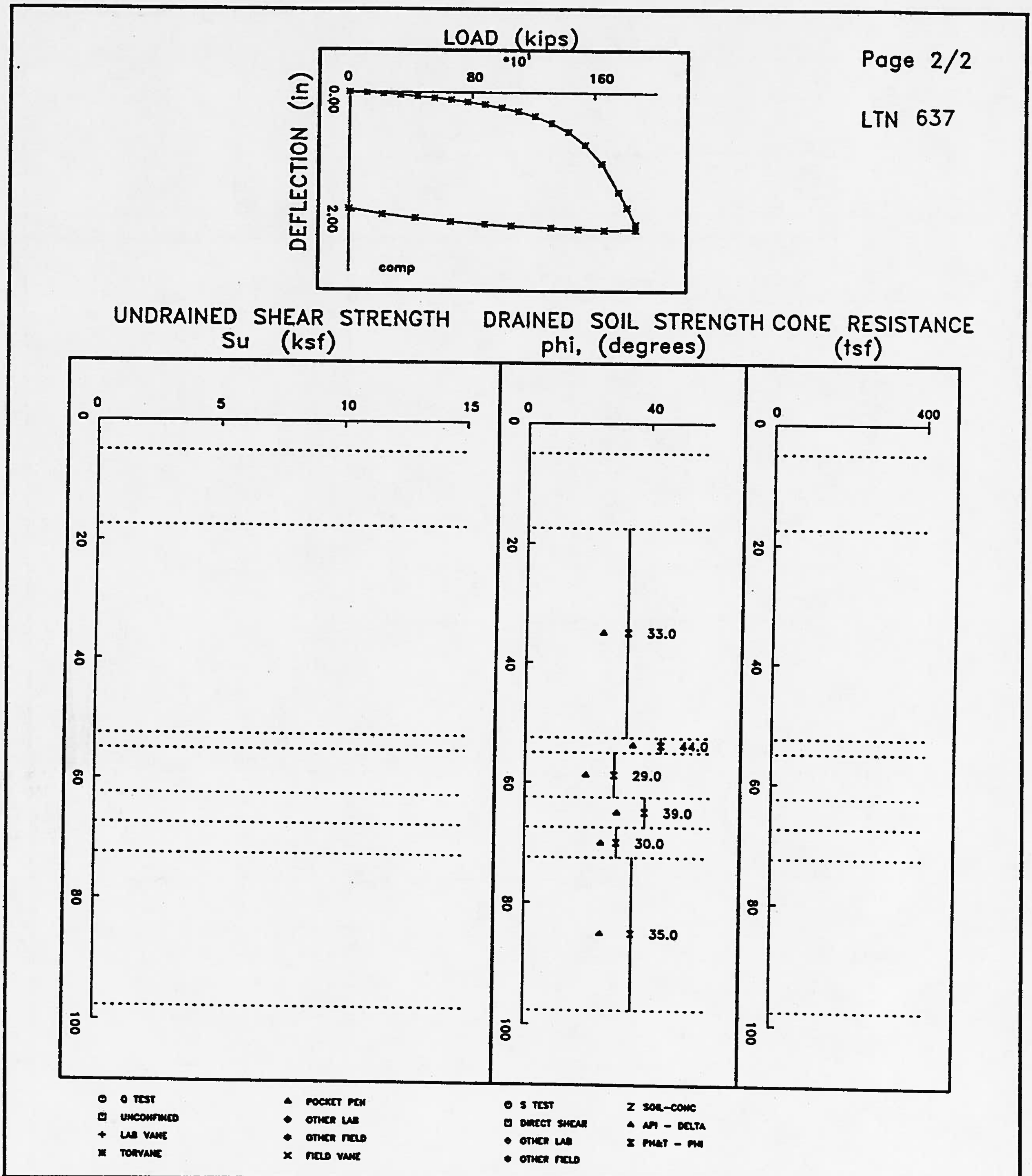


Figure 4b. Second page of graphical summary of University of Illinois database

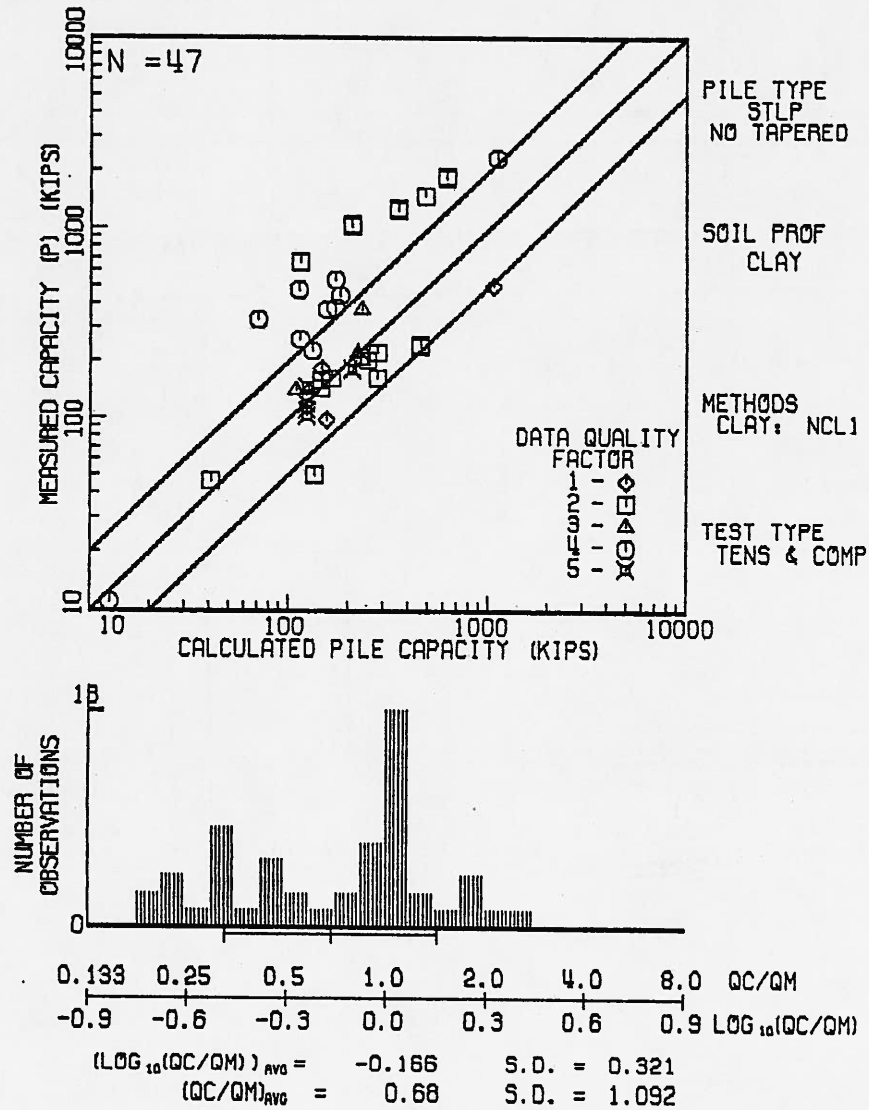


Figure 5. Graphical output summary from University of Texas database

SUMMARY - DESIRABLE FEATURES OF A DATABASE PROGRAM

A desirable database would contain a large number of load tests and be capable of computing axial capacities according to a number of different methods; however, some, non-technical features for a database are also important. Discussed herein are 7 features that would be useful to a large number of users, and may allow database users to be more efficient with their time.

A Summary Listing Containing Selected Data for Each Load Test. A single table that includes load test number, pile type, maximum axial load, maximum axial deflection, diameter, length, soil type, etc. could be used effectively for many of the less complicated database operations.

User Specified Table. The user should be able to specify and get a tabular listing of information with simple intuitive requests.

Summary Table For Each Load Test. Every database should include a notebook with an organized tabular summary sheet that provides all the information entered for each load test.

Summary Graph For Each Load Test. Every database should include a notebook with a graphical summary sheet (or sheets) that properly scales the geometry of the deep foundation in the soil profile, and provide the user with a quick, graphical summary of the soil profile, strength characteristics, and other information deemed important to identify the load test.

Method to Identify Data Quality. Details of the pile, soil, and load test that are subject to uncertainties should have a means to quantify the confidence with which the values were specified.

Ample Room for Comments. Every load test should be accompanied by a summary text that identifies important features of the pile, soil, and load test that may help the user interpret the validity of the load test results.

Accessible Original Documentation. In database studies, it may be necessary to return to the original reference from which the data were transcribed. Reasons may range from investigating effects that were not entered into the original database, to suspected errors in the transcription of data, or errors associated with the original reference. Access to the original load test documents is essential for a viable database.

The features suggested above could be incorporated into any of the four electronic databases, discussed previously. Indeed, many of the databases address most of these features to some extent. Finally, databases will continue to be developed, and new techniques for manipulating and visualizing data will undoubtedly become available; however, the importance of good load test data will remain permanent.

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APPENDIX

Past Proceedings of
OHIO RIVER VALLEY SOILS SEMINARS

ORVSS I	*BUILDING FOUNDATION DESIGN AND CONSTRUCTION* October 16, 1970, Lexington, Kentucky
ORVSS II	*EARTHWORK ENGINEERING, START TO FINISH* October 15, 1971, Louisville, Kentucky
ORVSS III	*LATERAL EARTH PRESSURES* October 27, 1972 Fort Mitchell, Kentucky
ORVSS IV	*GEOTECHNICS IN TRANSPORTATION ENGINEERING* October 5, 1973, Lexington, Kentucky
ORVSS V	*ROCK ENGINEERING* October 18, 1974, Clarksville, Indiana
ORVSS VI	*SLOPE STABILITY AND LANDSLIDES* October 17, 1975, Fort Mitchell, Kentucky
ORVSS VII	*SHALES AND MINE WASTES: GEOTECHNICAL PROPERTIES, DESIGN AND CONSTRUCTION* October 8, 1976, Lexington, Kentucky
ORVSS VIII	*EARTH DAMS AND EMBANKMENTS: DESIGN, CONSTRUCTION, AND PERFORMANCE* October 14, 1977, Louisville, Kentucky
ORVSS IX	*DEEP FOUNDATIONS* October 27, 1978, Fort Mitchell, Kentucky
ORVSS X	*GEOTECHNICS IN MINING* October 5, 1979, Lexington, Kentucky
ORVSS XI	*EARTH PRESSURES AND RETAINING STRUCTURES* October 10, 1980, Clarksville, Indiana
ORVSS XII	*GROUNDWATER: MONITORING, EVALUATION, AND CONTROL* October 9, 1981, Fort Mitchell, Kentucky
ORVSS XIII	*RECENT ADVANCES IN GEOTECHNICAL ENGINEERING PRACTICE* October 8, 1982, Lexington, Kentucky

ORVSS XIV	*FOUNDATION INSTRUMENTATION AND GEOPHYSICAL EXPLORATION* October 14, 1983, Clarksville, Indiana
ORVSS XV	*PRACTICAL APPLICATION OF DRAINAGE IN GEOTECHNICAL ENGINEERING* November 2, 1984, Fort Mitchell, Kentucky
ORVSS XVI	*APPLIED SOIL DYNAMICS* October 11, 1985, Lexington, Kentucky
ORVSS XVII	*NATURAL SLOPE STABILITY AND INSTRUMENTATION* October 17, 1986, Clarksville, Indiana
ORVSS XVIII	*LIABILITY ISSUES IN GEOTECHNICAL ENGINEERING AND CONSTRUCTION* November 6, 1987, Fort Mitchell, Kentucky
ORVSS XIX	*CHEMICAL AND MECHANICAL STABILIZATION OF SOIL SUBGRADES* October 21, Lexington, Kentucky
ORVSS XX	*CONSTRUCTION IN AND ON ROCK* October 27, 1989, Louisville, Kentucky
ORVSS XXI	*ENVIRONMENTAL ASPECTS OF GEOTECHNICAL ENGINEERING* October 26, 1990, Fort Mitchell, Kentucky
ORVSS XXII	*DESIGN AND CONSTRUCTION WITH GEOSYNTHETICS* October 18, 1991, Lexington, Kentucky
ORVSS XXIII	*IN-SITU SOIL MODIFICATION* October 16, 1992, Louisville, Kentucky
ORVSS XXIV	*GEOTECHNICAL ASPECTS OF INFRASTRUCTURE RE-CONSTRUCTION* October 19, 1993, Cincinnati, Ohio

PROGRAM

8:00 a.m.	**REGISTRATION**

Morning Session

Aubrey May, Principal - FMSM Engineers, Inc., presiding

9:00 a.m.	**WELCOME AND OPENING REMARKS** Calvin Grayson, Director, Kentucky Transportation Center Dr. Don Hancher, Chairman, Civil Engineering Department, University of Kentucky
9:15 a.m.	**"ORVSS - Twenty-Five Years of Excellence"**, Aubrey May, Principal, FMSM Engineers, Inc., Lexington, Kentucky
9:30 a.m.	**KEYNOTE ADDRESS - "Comparison of Deep Foundation Load Test Methods"**, Dr. F. C. Townsend, University of Florida, Gainesville, Florida
10:15 a.m.	**BREAK AND EXHIBITOR'S FAIR**
10:35 a.m.	**"Axial Response of Drilled Shafts in Intermediate Geomaterials in the Southeast"**, Dr. D. A. Brown, Auburn University, Alabama, and W. R. Thompson, Law Engineering, Birmingham, Alabama
11:05 a.m.	**"Drilled Shaft Experience - Owensboro, Kentucky Bridge"**, S. L. Murray, FMSM Engineers, Inc., Lexington, Kentucky, and D. J. Greer, Kentucky Transportation Cabinet, Frankfort, Kentucky
11: 35 a.m.	**"Quality Control for Jet Grouting"**, Debra Laefer, National Park Service, New York, New York (Author), and Silvana Toneatti, Booz Allen & Hamilton, Washington, D. C. (Presenter)
12:05 p.m.	**LUNCH AND EXHIBITOR'S FAIR**

Afternoon Session

Dr. Joseph Hagerty, University of Louisville, presiding

1:30 p.m.	**KEYNOTE ADDRESS - "Pile Driving - An International State of the Art"**, Dr. George G. Goble, Goble Rausche Likins and Associates, Boulder, Colorado
2:30 p.m.	**"Pile Load Testing - New and Improved"**, Dr.K. R. Bell and Dr.J. R. Davie, Bechtel Corporation, Gaithersburg, Maryland
3:00 p.m.	**BREAK AND EXHIBITOR'S FAIR**
3:20 p.m.	**"Design of Bridge Pier Pile Foundations for Ship Impact"**, Dr. B. O. Kuzmanovic and M. R. Sanchez, of Beiswenger, Hoch and Associates, North Miami Beach, Florida
3:50 p.m.	**"Test Pile Program for a Cable-Stayed Arch Bridge"**, S. J. Ludlow, Earth Exploration, Inc., Indianapolis, Indiana
4:20 p.m.	**"Pin Piles: An International Perspective[SM]"**, Dr. D. A. Bruce and Prof. I. Juran, Nicholson Construction Company, Pittsburgh, Pennsylvania
4:50 p.m.	**CLOSING AND ADJOURN**
5:00 p.m.	**SOCIAL HOUR AND EXHIBITOR'S FAIR**