1169

TRANSPORTATION RESEARCH RECORD

Geotechnical Instrumentation

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1988

Transportation Research Record 1169
Price: $13.00
Production: Harlow A. Bickford

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Printed in the United States of America

Library of Congress Cataloging-in-Publication Data
National Research Council. Transportation Research Board.

Geotechnical instrumentation.
(Transportation research record, ISSN 0361-1981 ; 1169)
1. Soils—Testing—Instruments. 2. Inclinometer.
3. Piling (Civil engineering) I. National Research Council (U.S.). Transportation Research Board.
II. Series.
TE7.H5 no. 1169 622 s 88-34472
[TA710.5] [624.1'5136]
ISBN 0-309-04705-6

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Foreword

The first four papers in this Record are on developments in soil instrumentation. The next four papers are on construction control of pile driving, and the last three papers are on lateral loading of piles and piers and on design of pile groups.

The paper by Green and Mikkelsen provides guidance on inclinometer system design, installation, monitoring, data processing, and sources of errors and their resolution. The authors suggest maintaining good continuity in the monitoring process through cooperation and understanding among engineers, clients, and contractors.

Mayne and Frost present several case histories on usage of the flat dilatometer in various geologic formations to obtain reasonable interpretations of soil properties. They indicate that the dilatometer test is an expedient and cost-effective method of profiling subsurface conditions. Dowding and Jessen present the results of an investigation on the components of a ground vibration monitoring instrumentation and computer surveillance system.

Data on the manner in which load is transferred in both end-bearing and skin friction are needed to allow for a redesign of piles. Reese and Stokoe discuss the significance of selecting the proper instrumentation for determining the load transfer in skin friction and end bearing.

Ledbetter presents an outline of the development of wave equation analysis to control pile driving. He presents three case histories to demonstrate the application of the analysis.

Bailey and Sweeney describe the dynamic testing program used by the New York State Department of Transportation to determine final pile-driving criteria. They present examples of field tests to illustrate how the test program was used to check on concerns for pile capacity, pile length, pile driving stress, and hammer operation.

Engel presents a case history illustrating how the Ohio Department of Transportation uses static analysis, the *Engineering News* driving formula, static load tests, dynamic load tests, a wave equation program, and the Case Pile Wave Analysis Program to determine pile capacity.

In the past 20 years, extensive research and implementation efforts have been directed at improved construction control of driven piles. According to Cheney, modernization of current specifications for construction control of driven piles requires that areas specifying ordered pile lengths be addressed in detail, using ultimate pile capacity, approved driving equipment, field verification of pile capacity, and pile payment method.

Analysis of the response of pile foundations to lateral loads requires accurate characterization of the behavior of the pile and the soil surrounding the pile. Kramer and Heavey present a model for representing nonlinear pile bending behavior in the analysis of laterally loaded piles.

Fellenius presents a unified design of piles and pile groups in which capacity, residual compression, negative skin friction, and settlement are related. Finally, Gabr and Borden present a model for the analysis of laterally loaded piers. The computer code LTBASE, which implements the model, is also described.

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Contents

Deformation Measurements with Inclinometers

GORDON E. GREEN AND P. ERIK MIKKELSEN

Inclinometers have steadily gained widespread use for measuring deformations in landslides, natural slope creep, temporary excavations, earth and rock embankments, slurry walls, shafts, tunnels, lateral pile movements, and settlements beneath tanks, fills, and foundations. Inclinometer data can be used to determine displacement magnitude and rate and the location of the zone of displacement, as well as the absolute position of the inclinometer casing in the ground. High-precision inclinometers today utilize servo-accelerometer sensors in a uniaxial or biaxial configuration, mounted inside a waterproof wheel carriage. The most common is the traversing borehole type, where the wheels are oriented and guided by specially made grooved casing. Fixed-in-place inclinometers employ the same guide casing but are used infrequently due to high cost. Inclinometer monitoring is one of the most labor- and data-intensive geotechnical measuring activities. In addition to straightforward data reduction, many errors have to be identified and dealt with before results can be presented and correctly interpreted. In the authors' experience, many inclinometer monitoring programs have failed to yield good results because errors were not recognized. In this paper, guidance is given to the reader regarding inclinometer system design, installation, monitoring, data processing, and sources of errors and their resolution. Accuracy of the inclinometer is highly dependent on the equipment selected, the method by which it is installed in the ground, monitoring techniques, correct scrutiny of the data, and ability to correct instrument errors. A servo-accelerometer-type system is capable of a precision of ±0.05 to 0.25 in. over 100 ft when the casing is vertical or horizontal. The best results are achieved when cooperation and understanding between engineers, clients, and contractors provide continuity in the total monitoring process.

The measurement of ground movements is an essential part of many civil and mining engineering operations. Monitoring subsurface movements with inclinometers can be considered a direct extension of normal surface survey techniques. Inclinometer systems are used extensively to monitor the displacement magnitude, rate of displacement (i.e., accelerating or decelerating), and location of the movement zone. In many situations, where, how much, and whether or not the movement is increasing or decreasing are key issues. Inclinometer systems can be used to monitor displacements in landslides, natural slope creep, temporary excavations, earth and rockfill dams, slurry walls, shafts, tunnels, lateral pile movements, and settlement under tanks, fills, and foundations. Inclinometers are also used to determine the absolute position of boreholes prior to installing other instruments, for example, geophones or tieback tendons. They may also be used indirectly to measure moments and stresses in structures and can give good results with flexible steel structures where the deflection is large and the section modulus is known. Their value in composite concrete structures where the moment of inertia is uncertain is more limited.

The inclinometer commonly used today developed from a device built in 1952 by S. D. Wilson at Harvard University. It first became available commercially in the late 1950s from the Slope Indicator Company. Such an inclinometer consists of a probe, containing a gravity-actuated transducer, which is fitted with wheels and lowered on an electrical cable down a grooved casing to control orientation (*1*, *2*) (see Figure 1). The cable is connected to a readout unit, and data can be recorded manually or automatically. The inclination of the casing with respect to gravity is measured at incremental depths, and the entire casing profile is obtained by numerical integration. Sets of readings taken periodically enable both the magnitude and rate of lateral casing movement to be calculated. The casing is normally set in vertical drillholes or attached to a structure in a vertical position to measure horizontal movements (Figure 2). It can also be set horizontally to measure heave or settlement (Figures 3–5) but will not measure movements in a horizontal plane. Movements of casings inclined up to 45 degrees may also be monitored but with considerably less accuracy (*7*).

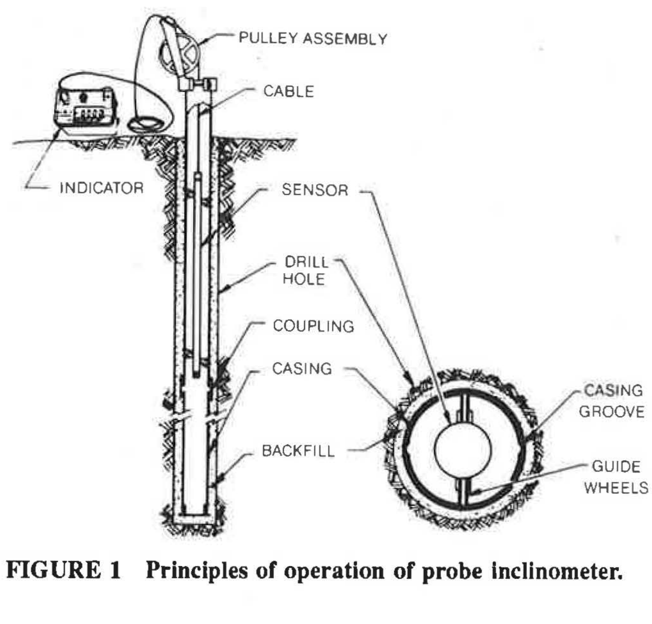


FIGURE 1 Principles of operation of probe inclinometer.

G. E. Green, 9835 41st Avenue, N.E., Seattle, Wash. 98115. P. E. Mikkelsen, Slope Indicator Company, 3668 Albion Place North, Seattle, Wash. 98103.

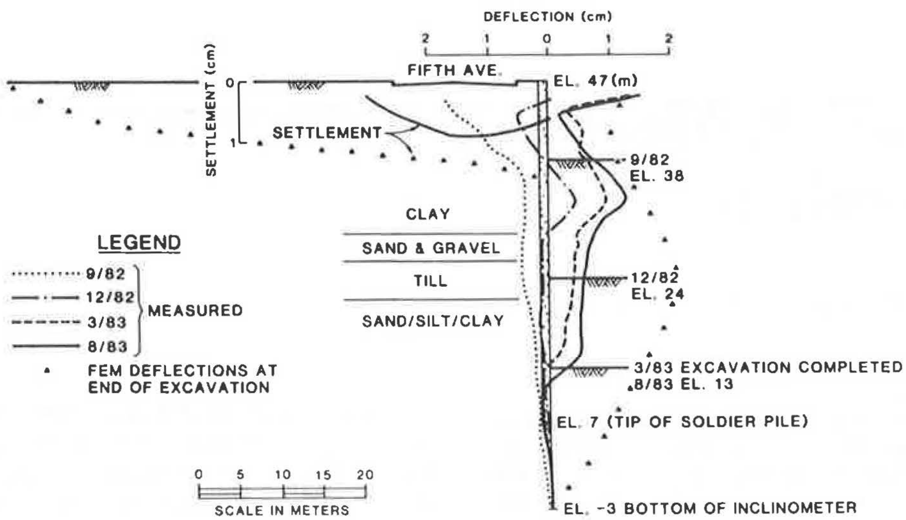


FIGURE 2 Shoring wall movements on the 34-m-deep Columbia Center excavation in Seattle, Washington (3).

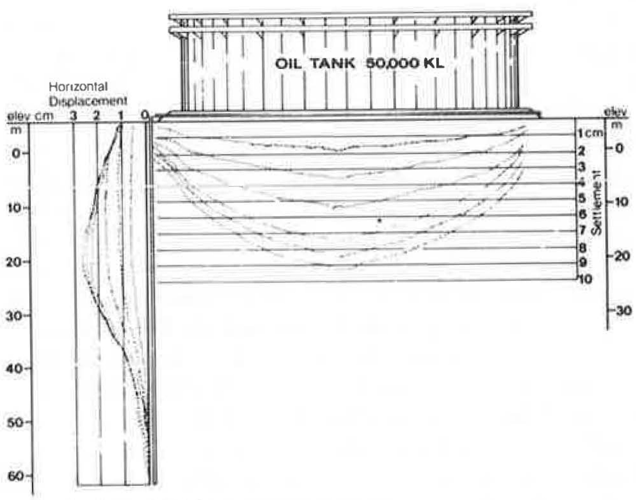


FIGURE 3 Vertical and horizontal monitoring of an oil tank foundation during water test loading (4).

Fixed-in-place inclinometers are available. They consist of a series of sensors with guide wheels, each containing a uniaxial or biaxial gravity-operated transducer that generates electrical signals. The sensors are joined by articulated rods and are suspended permanently in vertical grooved casing. Continuous or remote monitoring or both, by direct wire link, radio, or telephone modem, permit real-time data to be obtained in critical situations, in contrast to the manually read probe inclinometer system.

Other types of drillhole survey tools are also available. The oil well and geophysical logging industry use a variety of photographic, gyroscopic, and gravity-based electronic sensors that operate in unlined vertical drillholes and do not require casing with oriented grooves. Attempts have been made to develop versions of these tools (*8*) that are suitable for civil and mining use in boreholes 50- to 300-ft deep as an alternative to the probe inclinometer used inside grooved casings, but these efforts have been relatively unsuccessful to date due to cost, convenience, size, and accuracy. The rate gyro–based tools that have recently become available appear to be more promising (*9*).

Inclinometers may also be combined with probe extensometer systems that measure extension or compression along the axis of the inclinometer casing, a mechanical probe that locates casing joints, or an induction coil or magnet/reed switch sensor. When these devices are installed on vertical casings, they permit settlement or heave monitoring so that subsurface three-dimensional movements are monitored (*10*).

Inclinometer monitoring is one of the most labor-intensive geotechnical measurement activities. It generates large volumes of numerical data that must be correctly recorded, processed, scrutinized for errors, plotted, and interpreted. Experience indicates that many inclinometer measurement projects fail to achieve their intended aim because of a lack of appreciation of the many factors, both human and instrument, that need to be correctly implemented. Thus, in many cases, thick files of unplotted data are generated, or depth/displacement profiles wander mysteriously back and forth across a graph, or large movements are believed to be occurring when there may actually be little or none. More information concerning inclinometer systems and their use may be found elsewhere (*10–16*).

INCLINOMETER EQUIPMENT

The accuracy and reliability of the measured position or displacement profile is dependent on the quality of the casing, probe, cable, readout, and accessories selected. A poorly engineered probe, stretchy cable, faulty readout, or inferior casing will result in poor quality data at best, and an unhappy user.

Inclinometer casings may be plastic, aluminum alloy, or steel, with rigid or telescopic couplings (see Figure 6). Plastic casings may be ABS, PVC, or fiberglass and the grooves may be formed by broaching, extrusion, or moulding. ABS is flexible and easily cemented, PVC less so. The casing groove spiral should be less than 1 degree per 10 ft, and for plastics, a broached tube appears to be better than an extruded casing. Even so, hot sun and improper storage can warp and twist an initially straight casing. Casing diameters range from 1.9 to 3.8

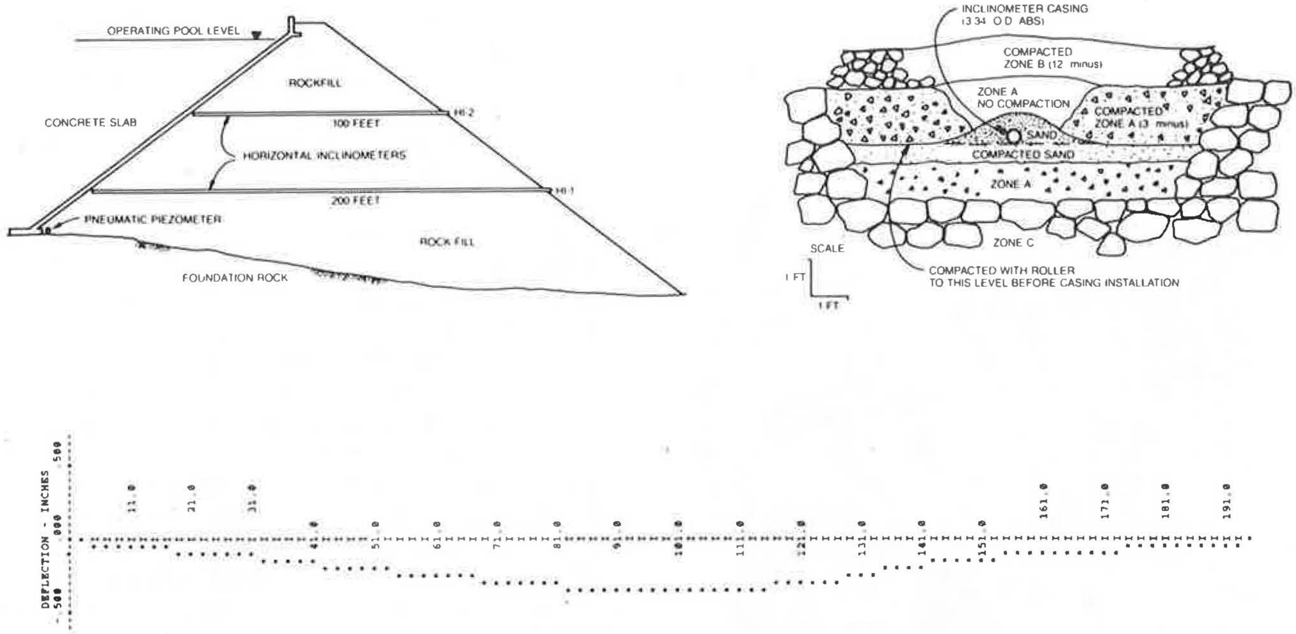


FIGURE 4 Horizontal inclinometer monitoring of a rockfill dam (*5*).

in., and the larger sizes are preferable where movements are large, thin shear zones are present, and drilling costs permit. Rigid couplings are available, with self-aligning features that include flush couplings for use where borehole space is tight. Connections may be made with rivets, plastic cement, or a self aligning Westbay coupling that incorporates an O-ring and nylon shear key that is very convenient. The aluminum alloy casing is extruded with grooves, and a close-fitting, similar second extrusion is employed for rigid or telescopic joints. Joints are riveted and must be sealed with mastic and tape. This aluminum alloy casing is subject to corrosion in alkaline environments. Some records have shown that in the presence of steel, electrolysis has caused a total loss in a few months. Baked-on epoxy paint offers the best protection, but even so,

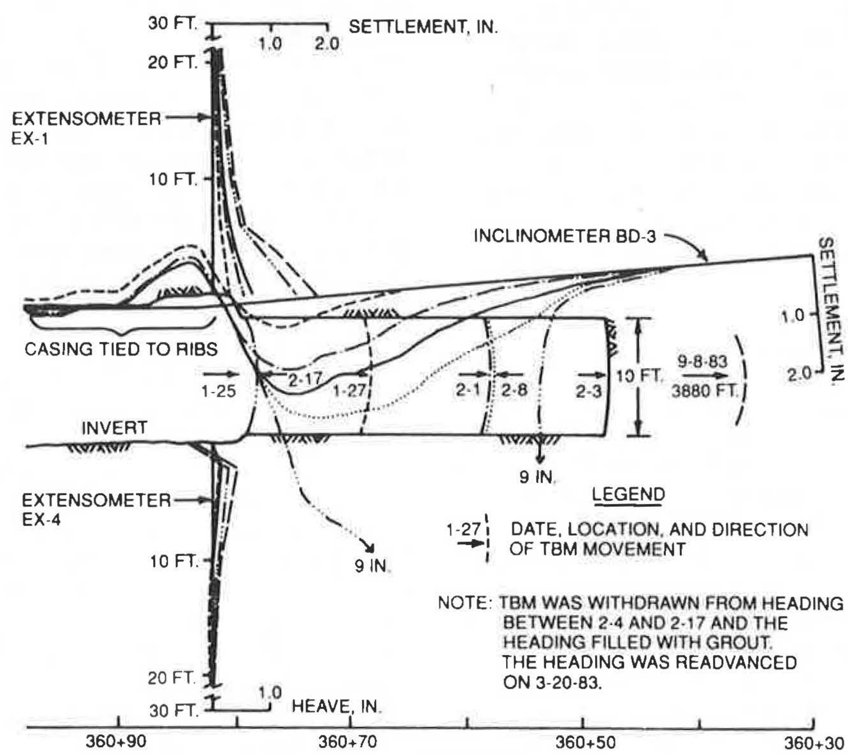


FIGURE 5 Vertical rock deformations in a tunnel in response to tunnel boring machine advance (*6*).

FIGURE 6 Digitilt inclinometer, manual readout, inclinometer casing, and couplings.

corrosion can occur, and ground conditions need to be carefully assessed before an aluminum alloy casing is selected. In benign environments, some users prefer aluminum casings over plastic (*17*). In some instances, seamless welded square steel tube can be used where great strength is required (e.g., attached to driven steel piles) or where the user wishes to attach strain gauges to the casing for axial strain measurements (e.g., in concrete piles). Otherwise, the steel tube is subject to corrosion and to loss of precision of probe location within the casing. Extruded steel tubing is usually twisted excessively and should not be used.

The gravity-actuated sensor may employ a rotary potentiometer, bonded resistance strain gauges, vibrating wire strain gauges, an electrolytic level, or a servo-accelerometer. The now discontinued Slope Indicator 200B employed a rotary potentiometer with a relay-operated contact to reduce friction. This sensor was stable and reliable but bulky. The Soil Instruments Mark II inclinometer, also discontinued, incorporated a strain gauge steel leaf spring with an oil-damped pendulum. The sensor was reasonably stable and reliable but subject to damage if it was not immobilized in transit. European manufacturers have incorporated the vibrating wire strain gauge on dual-axis pendulum devices, which are reliable but cumbersome. The electrolytic level has only been used in horizontal inclinometers because (a) it is currently too large to mount laterally in a vertical probe, (b) it has a more limited angular range, and (c) it is relatively sensitive to temperature. The servo-accelerometer is the sensor most widely used today. A product of the space program, the accelerometer can also be used in a static mode to measure angle. A very small, damped pendulous mass suspended by an elastic flexure is electrically nulled, with the voltage proportional to angle. The servo-accelerometer is accurate, reliable, reasonably robust, and compact, and two of them can be mounted in a slender biaxial probe for vertical casings.

The probe body (Figure 6) should be waterproof and preferably of stainless steel, with a well-engineered wheel carriage that has little or no side play in the wheels, axle, and swing arm assemblies. Preferably, the wheels should have double ball bearings and a wheel rim profile that is compatible with the casing groove geometry. Self-centering wheels are better than the earlier fixed-spring wheel assemblies. Wheel geometry and carriage design control the precision with which the probe can be relocated each time in the casing. This factor is particularly important with the currently used biaxial probes, which utilize only one pair of grooves and rely on the sides of the grooves for B-direction location. A sealed electrical connector should be located at the top of the vertical probe and at either end of a horizontal probe to permit it to be turned end for end. Inclined probes are available in which the sensor is set with its axis vertical and at an angle to the inclined probe (*18*), but these cannot be reversed end for end to eliminate zero errors and are less convenient and accurate in use. At the same time, it should be recognized that the accuracy of vertical or horizontal probes degrades severely when they are used in steeply inclined casings (*7*).

The electrical cable should be flexible, waterproof, untwisted, durable, easy to grip, and permanently and accurately graduated. It must not stretch under load or with time, and the outer sheath must not creep over the inner core or serious errors will result. These requirements are stringent and some inclinometer cables do not meet all of them. Connection to the probe requires a heavy-duty waterproof electrical connector, but a nonwaterproof connector is adequate at the readout end of the cable. For cable lengths up to 200 ft, a cable reel is not required because the cable is most easily handled in a manner similar to that used with a climbing rope. For cable lengths of 200–1,000 ft, a manual or powered reel is essential.

The readout unit should be robust, reliable, easy to read, portable, insensitive to temperature, and weather resistant (Figure 7). Traditionally, manual data recording on a field sheet is employed. Battery-powered digital readouts that display one or two sensor axes sequentially or simultaneously in a boxed or clipboard package are available. The amount of data that must be recorded is voluminous, and errors are easily made when manual transcription or computation is employed. Direct field reading to a solid-state memory unit with field data checking facilities is preferred. Field or office computer processing of the data recorded on magnetic tape or disk, including transmission by telephone modem (if needed) and automatic plotting reduce the labor, cost, and chance of errors. Most manufacturers now provide automated readouts of various types, of which the Slope Indicator Digitilt RPP (Recorder, Processor, Printer) is an example (see Figure 7). This instrument can record, store, file, and reduce inclinometer data, spiral data, and azimuth angle input. Built-in software allows the data to be corrected for systematic errors and permits spiral and inclination readings to be combined. This software also allows coordinate system rotation, and all of these procedures can be done on the spot, in

FIGURE 7 Digitilt inclinometer in use with recorder, processor, and printer (RPP).

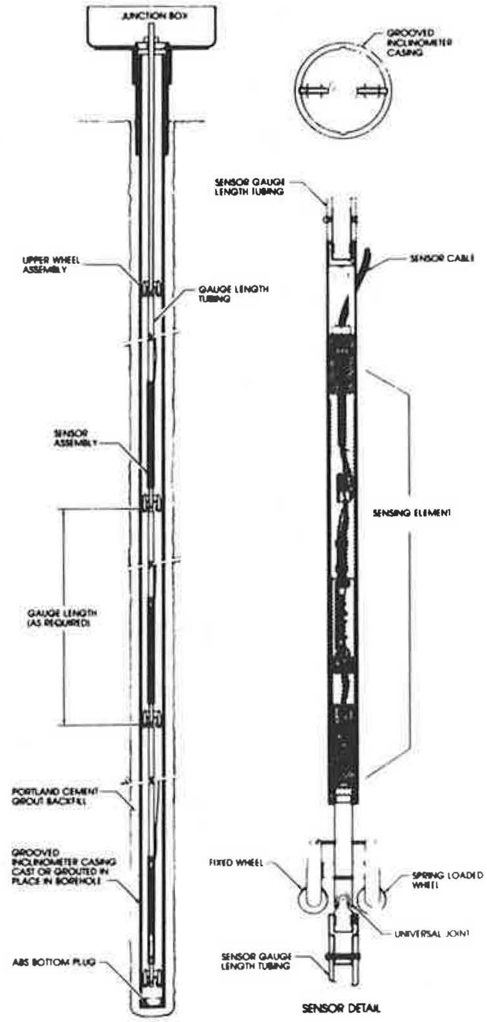


FIGURE 8 Fixed-in-place inclinometer.

the field, if needed. Results are tabulated and graphed on a built-in electrostatic printer, and data files are stored on magnetic tape. The Digitilt RPP can operate as a computer terminal and can send data via a telephone modem to a remote computer, or it can be connected to an external printer or plotter.

Fixed-in-place inclinometers (*13*) consist of a series of long-gauge length probes (Figure 8) permanently installed to provide continuous automatic and remote deformation data at critical locations and where the relatively high cost is justified. These instruments will not provide a detailed profile of ground movements unless sensors are closely spaced. Sensor zero drift with time can be difficult to detect and is not eliminated as it is with the portable probe inclinometer, which can be reversed through 180 degrees. Available accelerometer sensors are fairly reliable, and fixed-in-place inclinometer systems have been reasonably successful (*19*). Fixed-in-place inclinometers can be installed permanently in casings adjacent to probe inclinometer casings or may be installed during a critical period in a probe inclinometer casing and then removed. Even a single sensor fixed-in-place unit can be installed across an identified thin shear zone.

As noted, a casing spiral or twist can occur due to manufacturing, storage, or installation deficiencies. Serious spiraling has, on occasion, been identified in the field. Spiral should be measured (a) on deep casings, (b) where measured movements are in suspect directions, (c) where absolute casing position is required to good accuracy, or (d) where installation difficulties have occurred. A spiral survey is normally required only once, and computerized data reduction is essential. One type of spiral survey tool consists of a long torque rod mounted between wheel assemblies with a rotary displacement transducer.

FIGURE 9 Installation tools and accessories.

Accessories are needed to install and operate inclinometer systems (Figure 9), including installation tools, pulley wheel and cable clamp, grout valves, and end caps. Protective enclosures, a calibration facility, instructional material including videotapes, data sheets, software package, calculator or computer, telephone modem, and printer are also available. The calibration of the probe inclinometer may change with time, and calibration frames that provide a functional check are available from some manufacturers. On-site limited calibration checks may also be performed by using short lengths of fixed casing cast into an immobile concrete block or by extending some casings to well below the movement zone so that regular surveys of immobile casing are performed. Otherwise, calibration should be done periodically by the manufacturer on a dividing head. Inclinometer system accuracy is dependent on a number of factors, including sensor design and construction, installation technique, casing quality and orientation, care and attention given when taking readings, and instrument maintenance (*10*). True independent checks on system accuracy are difficult to perform and rarely done. An inclinometer system check test setup (*12*) is shown in Figure 10, but this excludes field related factors that can degrade measurement accuracy (*20*).

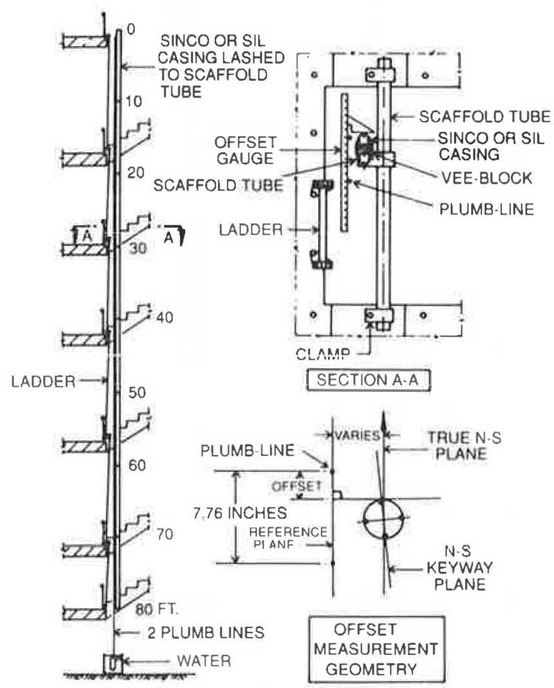


FIGURE 10 Inclinometer casing test set up in a stairwell (*12*).

surveys are proportional to the product of casing inclination and angular changes in sensor alignment. For inclined casings, sensor alignment changes of 1 to 2 degrees may produce errors in the measured displacement of several inches per 100 ft of casing (Figure 11). Sensor alignment change occurs with time due to wheel play in a groove, wheel carriage wear, internal changes in the sensor, and changes in alignment between the sensor and wheel carriage. Thus tight specifications on borehole verticality and quality drilling techniques are preferable.

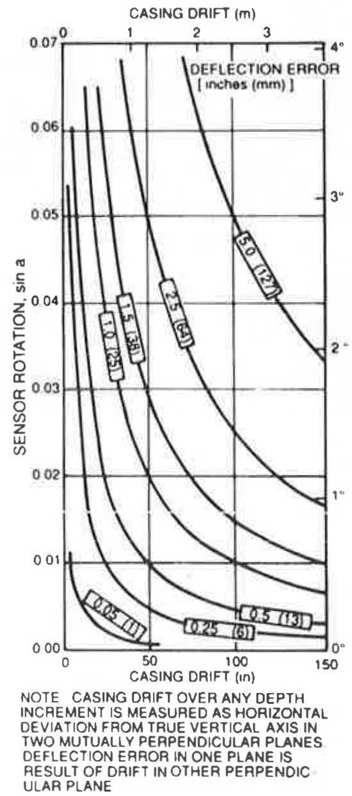


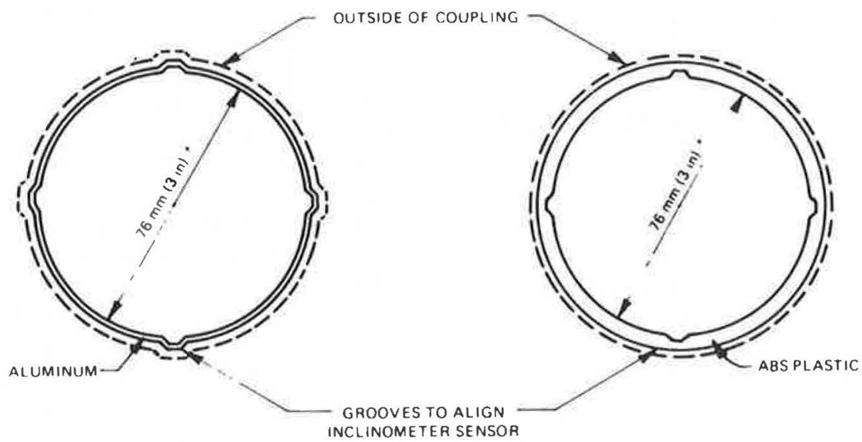
FIGURE 11 Measurement error as a result of casing inclination and sensor rotation (*14*).

In selecting appropriate equipment for inclinometer measurement, instrument accuracy, quality, reliability, manufacturer reputation, and availability of service, manuals, and software should be the prime considerations. Hardware costs are only a small proportion of the total measurement cost, which includes drilling, installation, data processing, and evaluation. Accurate, reliable instruments are essential. Many have discovered this too late, and low cost is a poor basis of choice.

INSTALLATION PRACTICE

Casing installation procedures are described in detail in manufacturer's instruction manuals (*13, 15*) and discussed elsewhere (*10, 14, 21*). The inclinometer casing is commonly installed in soil or rock in vertical boreholes drilled by a variety of methods, depending on the material type, borehole stability, amount of water present, drilling equipment available, casing size, and cost. For observational accuracy, the borehole should be as close to vertical as practicable. Errors in inclinometer

The largest inclinometer casing size should be used where hole size permits, because this size will accommodate the largest movement before probe access is obstructed. Drilling costs or structural constraints may necessitate using smaller sizes, for example, the 1.9-in. O.D. plastic flush coupled casing in an NX-borehole. The casing is coupled together (typically in 5- to 10- ft lengths), the joints are sealed (including the bottom of the casing), and the casing is lowered into the borehole. If the borehole is filled with water or drilling mud, water must be added inside the casing, and extra weight may be needed to over come buoyancy. The casing should be oriented so that grooves are aligned in the anticipated predominant movement direction, and twisting of the casing should be avoided during insertion. Groove alignment at couplings is maintained by keyways of varying types (Figure 12). Where settlement or heave in excess of 1 percent is anticipated, telescopic couplings are required and should be set during installation at the appropriate position for the anticipated movement direction. Telescopic couplings complicate both installation and reading procedures



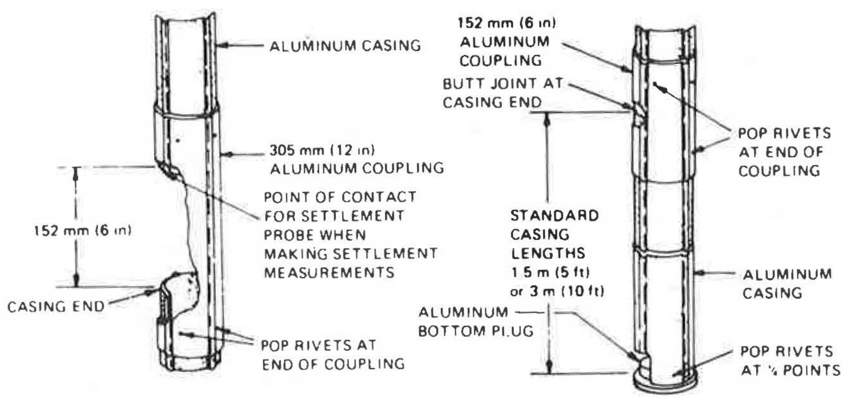


FIGURE 12 Inclinometer casing details (*14*).

and should be used only when required. Care is needed during installation to avoid extending or collapsing the preset assembled joints. All casings should extend 10 to 20 ft below the zone of anticipated movement to provide a stable reference section. Inclinometer measurements are usually more accurate and reliable than surface survey check measurements made on the tops of the casings, and reliable casing base fixity is vital.

A casing installation in a borehole must be backfilled around the casing with sand, pea gravel, or grout (Figure 1) to ensure conformity with the surrounding ground movements. Incomplete backfilling or backfill settlement causes spurious casing movements that are best avoided. In some cases, for instance, soft ground, the backfill strength should be matched to the ground strength so that conformity is achieved. Uniform, coarse clean sand or pea gravel can be used to backfill the annular space in stable shallow holes, and clean sand can be sedimented through water or flushed through a tremie pipe. Granular backfill is more prone to bridging and settlement. Grout pumped through a tremie pipe extending to the bottom of the borehole is the preferred method, but this may not work in open granular materials. An external grout pipe along side the casing can be used if the borehole is large. It is more convenient to grout through a drill rod inside the inclinometer casing, connected temporarily to a one-way grout valve attached to the bottom of the casing (Figure 13). The drill rod also serves to keep the casing straight and helps overcome buoyancy effects until the grout sets. Where large movements on well-defined shear zones are anticipated, a large borehole and weak grout backfill should be used so that the casing will shear locally through the backfill, maintain probe access longer, and provide continuity of readings. The localized shear movements will be redistributed over a larger casing length, but this is preferable to getting no data. Completed installations should be washed out to clean the casing, and adequate surface protection should be installed and locked, if appropriate, to guard against damage or vandalism. Casings can be successfully installed in deep holes (200 to 1,000 ft) by using a safety cable attached to the bottom of the casing and multistage grouting with an external tremie pipe.

Vertical inclinometer casings installed during construction of earth or rockfill dams must (a) incorporate telescopic couplings, (b) be adequately protected from damage by construction traffic, (c) be extended section by section as the embankment rises, (d) avoid providing a zone of increased permeability, and (e) settle by the same amount as the surrounding production machine-placed fill. These needs are often difficult to achieve, and great care and quality control are required. This is particularly true if other instruments are also being installed, as is often the case (*7*).

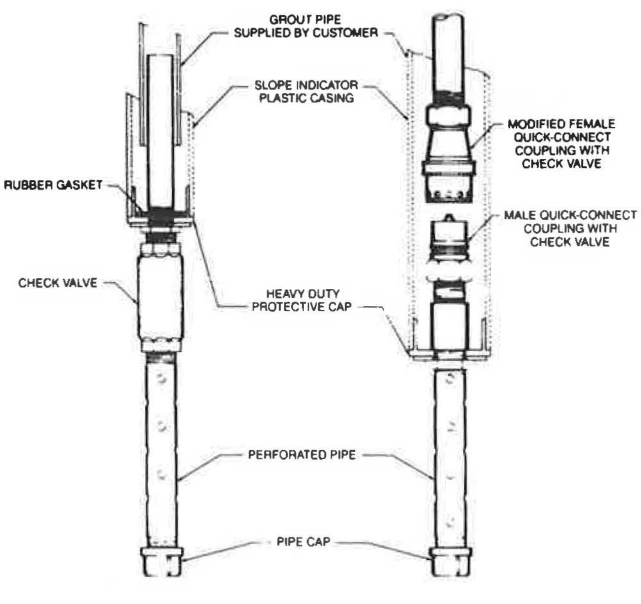


FIGURE 13 Grout valve for inclinometer casing installation.

Horizontal inclinometer casing is usually placed in compacted fill beneath an embankment or structure (*4, 5*). The casing must be straight, with one pair of grooves vertical and preferably on a constant flat, free-draining grade so that the casing can be washed out. The instrument is pulled into the casing with a steel cable. If access is available only at one end, the cable should be passed around a dead-end pulley and returned in a second small PVC pipe alongside the casing.

Inclinometer casings can be cast into concrete piles, grouted into hollow core concrete piles after driving, attached to steel sheet, H, or pipe piles, installed after driving in steel tubes, or grouted into slurry walls. When the inclinometer is being used to monitor wall deflections of any type, it is advisable to extend some, if not all, of the casings beyond the base of the wall. This usually requires drilling 10 to 15 ft beyond the base of the installed wall through a steel pipe.

Inclined inclinometer installations are used in sloping cores and concrete membranes in dams and for drillhole alignment surveys with temporarily installed casing. Examples and methods for dams are discussed by Penman and Hussain (*22*) and errors are analyzed by Mikkelsen and Wilson (*7*). Inclined installations in drillholes should be avoided where possible because measurement errors are too great, due to casing groove alignment uncertainties and sensor errors. With care, short holes can be surveyed with modest accuracy, and position surveys of tieback holes can sometimes be obtained with sufficient accuracy. Where inclined installations on sloping cores are required, groove alignment must be very closely controlled. The trolley and track described by Penman and Hussain (*22*) is an elegant, if complicated, technique that uses a conventional vertical probe that can be reversed (Figure 14).

Fixed-in-place inclinometers are installed in vertical casing, which should be installed as described previously. The fixed-in-place inclinometer assembly is suspended in the casing, and a waterproof electrical cable from each sensor is connected to a junction box at the surface. A conveniently located automatic data acquisition system (ADAS) monitors each sensor sequentially. The system may be hard wired or radio telemetered to the ADAS, which can be accessed by telephone as required. Fixed-in-place inclinometers can be transferred from one casing to another and can be salvaged at the end of the project.

MONITORING PROCEDURES

Installed casings should be monitored regularly by a trained two-person crew who are thoroughly familiar with correct procedures and have read and digested the manufacturer's instruction manual. Inappropriate procedures produce poor-quality data and a disillusioned client. A reading interval equal to the distance between the probe wheel carriages is usually

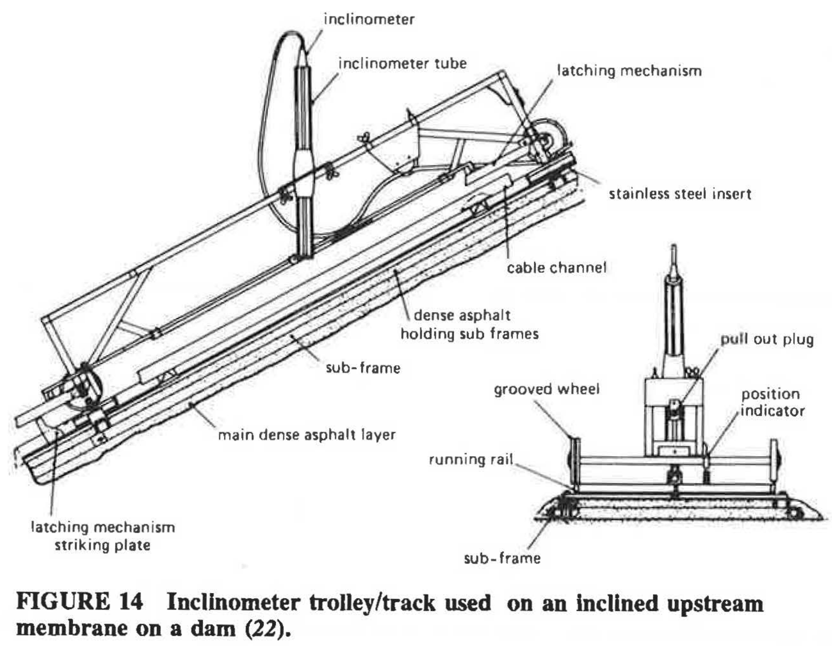


FIGURE 14 Inclinometer trolley/track used on an inclined upstream membrane on a dam (*22*).

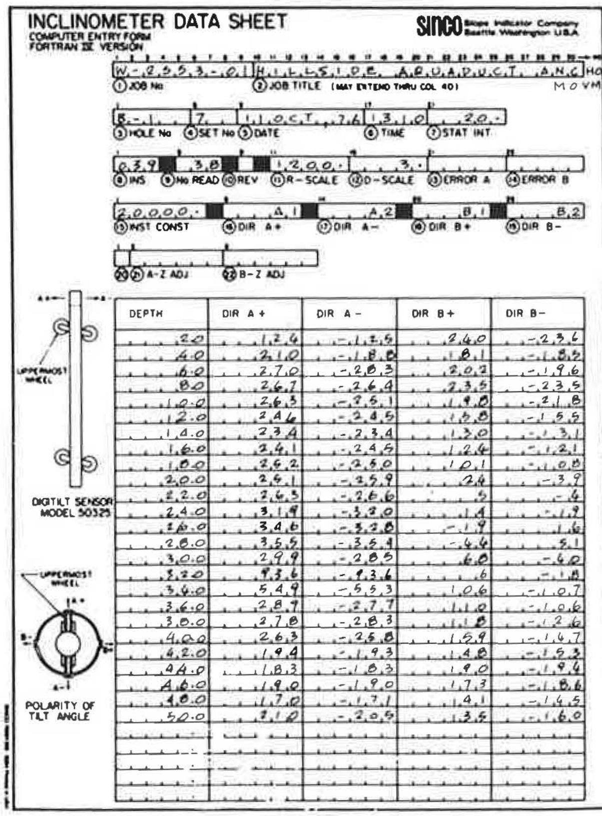
most appropriate, although a greater interval may sometimes be used with little loss of accuracy, provided that thin shear zones are absent. Preferably, the same instrument should be used throughout the job because probes are not interchangeable without generating systematic errors. Instrument damage or manufacturer recalibration may necessitate reinitialization of the readings. Readings should be taken from the bottom of the casing up, with close depth control (±1/4 in.), by using a pulley wheel and cable clamp or similar device attached to the casing collar. In vertical casings, a uniaxial probe requires four passes up the casing, whereas only two passes are needed with a biaxial probe, which saves field time. The B-sensor data obtained with the biaxial probe are less accurate than those of the A-sensor (i.e., parallel to the wheels) because the side of the casing groove controls the B-axis sensor alignment. It is essential to obtain readings on both faces, that is, with the probe turned 180 degrees, to eliminate zero shift errors and enable data quality "checksums" to be computed. Checksums must always be computed and scrutinized for errors in the field. Where errors are suspected, repeat readings must be taken on the spot. In horizontal casings, the uniaxial probe must be disconnected from the cable and reconnected to the second socket at the other end of the probe to reverse the probe, and only two passes are needed.

If casing spiral is suspected to be of significance, a spiral survey tool should be used to survey the casings. All portable equipment should be handled with care and protected during day-to-day use. The back of a pickup truck is no place for a delicate instrument on a rough site; instruments require "tender loving care" for good results. The cable should be protected from nicks, and connectors must be kept clean and dry. Although most probes and readouts are rated for a reasonably broad temperature range, neither should be left unshaded in hot climates or exposed to prolonged temperatures close to or below freezing point. Battery life at low temperatures is considerably reduced, dependent on type (*23*). Electronic component malfunction is more likely under extreme environmental conditions.

Function checks on the instrument are necessary. Hanging the probe freely on 6 ft of cable gives a quick daily zero check, and regular readings in fixed short casings are desirable. Manufacturer adjustment, maintenance, and recalibration are advisable once per year or if instrument performance is suspect. The instrument must be cleaned and wheels checked for excessive wear and oiled daily. Where practicable, periodically washing out the casing will remove grit that can collect in the grooves and cause increased groove wear, degrading data quality. Periodic surface surveys of both line and level at the casing collar provide an important check on measured ground movements and should always be made. If possible, where high accuracy is needed, duplicate sets of inclinometer readings should be taken and an average data set determined.

Data should be reported in an initial installation report, followed by monitoring reports (*13*). The installation report should include equipment model and serial numbers, calibration, description of installation, site plan with casing location, elevation, groove orientation, convention adopted for the sign of the movement and probe orientation, initial data sets, and a spiral survey, if one was taken. Each monitoring report should include equipment model and serial number, data set, a diary of field activities, and observations related to the installation. Jobs may extend over many months or years, and detailed records can be crucial when personnel changes or problems arise.

Where manual readings are taken, data are recorded on a field sheet formatted for computation or computer key entry (Figure 15). Face errors for pairs of corresponding readings (i.e., A+, A- and B+, B-) must be determined and should be relatively constant at each depth. This provides a critical field check on data reliability. A similar procedure must be followed with an electronic notebook or a computer-based readout, such as the Digitilt RPP. Field equipment or procedures that do not permit on-the-spot data quality checks should never be used. Two or three initial data sets should be obtained on a casing before any construction activity to ensure reliable baseline data and confirm that backfill movements are absent.


INCLINOMETER DATA SHEET
COMPUTER ENTRY FORM
FORTRAN IV VERSION
SINCO Slope Indicator Company Seattle Washington U.S.A.

(1) JOB No: W-2553-01
(2) JOB TITLE (MAY EXTEND THRU COL 40): HILLSIDE AQUADUCT ANC HOR MOVMT
(3) HOLE No: B-1
(4) SET No: 7
(5) DATE: 11OCT76
(6) TIME: 1310
(7) STAT INT: 2.0
(8) INS: 039
(9) No READ: 38
(10) REV
(11) R-SCALE: 1200.
(12) D-SCALE: 3.
(13) ERROR A
(14) ERROR B
(15) INST CONST: 20000.
(16) DIR A+: A1
(17) DIR A−: A2
(18) DIR B+: B1
(19) DIR B−: B2
(20) (21) A-Z ADJ
(22) B-Z ADJ

DEPTH	DIR A +	DIR A −	DIR B +	DIR B −
2.0	124	−125	240	−236
4.0	210	−188	181	−185
6.0	270	−283	202	−196
8.0	267	−264	235	−235
10.0	263	−251	140	−218
12.0	246	−245	158	−155
14.0	234	−234	130	−131
16.0	241	−245	124	−121
18.0	262	−250	101	−108
20.0	261	−259	24	−39
22.0	263	−266	5	−6
24.0	319	−320	14	−19
26.0	346	−328	−19	16
28.0	355	−354	−44	51
30.0	299	−285	60	−60
32.0	436	−436	6	−18
34.0	549	−553	106	−107
36.0	289	−277	110	−106
38.0	278	−283	118	−126
40.0	263	−258	159	−147
42.0	194	−193	148	−153
44.0	183	−183	190	−194
46.0	190	−190	173	−186
48.0	170	−171	41	−145
50.0	210	−205	35	−160

FIGURE 15 Field data sheet formatted for keypunch data entry.

Computer-based field readouts are being used more widely because they provide convenience, reliability, elimination of data transcription errors, on-the-spot computation of casing position or displacement, and cost savings. A recent cost comparison (*24*) indicated a tenfold reduction in monitoring costs when use of a Digitilt RPP was compared with use of manual equipment.

Monitoring telescopic casing where large axial displacements occur poses difficulties in repositioning the probe in the casing. If readings are made in the normal way at uniform depth intervals, the absolute profiles must be compared graphically. If a separate settlement survey is performed, computational techniques can be used to compare the two data sets.

Alternatively, the probe may be relocated at the same position in a casing section by "feeling" the casing coupling as the wheels pass through. Again, a separate settlement survey is required. Additional bookkeeping complications occur when casing is built into an earth or rockfill dam because the reference point from which measurements are made is constantly changing (*25*) and reliance is placed on surface surveys. As yet, there is no better solution to this problem.

Fixed-in-place inclinometers should be read daily after installation to check for zero drift of a bad sensor. Depending on the readout arrangements, threshold readings and alarms may be set and telemetry systems may be checked. Automatically collected data should be scrutinized periodically for errors, particularly drift, and periodic probe inclinometer surveys should be made on an adjacent casing (Figure 16).

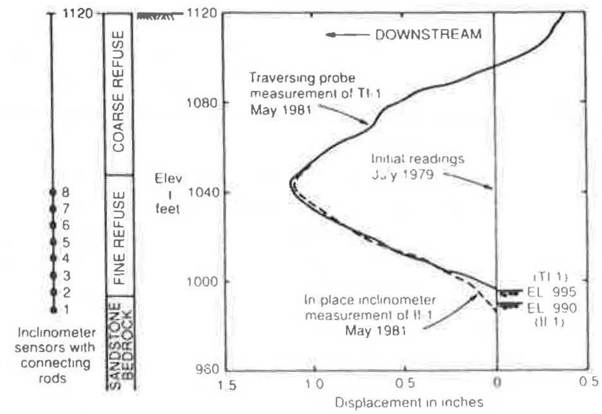


FIGURE 16 Horizontal movements measured with probe inclinometer and with fixed-in-place inclinometer in adjacent casings (*26*).

DATA PROCESSING, ERROR SOURCES, AND ACCURACY

Data checksums (i.e., algebraic sum of pairs of readings 180 degrees apart) must be computed and scrutinized in the field, and data should be processed as soon as possible. Manual data reduction is tedious even with the help of a programmable calculator, and many users recognize the advantages of computerized analysis (*24, 27*). IBM-PC software is now available; for example, Slope Indicator/Geo-Slope PC-SLIN, with keyboard or RS-232C serial input, tabular and graphic output, error

Field Data and Displacements

GEO-SLOPE : P C - S L I N :

80526

BENCHMARK EXAMPLE NO. 1
DIGITILT DATA

	PAST	PRESENT
FILE NAMES	A:SI1.SN	A:SI11.SN
JOB NUMBER	W-2553-01	W-2553-01
HOLE NUMBER	B-1	B-1
DATA SET NUMBER	7	7
DATE	SEPT 20/73	OCT. 11/76
TIME	13:10	12:30
READINGS PER DIRECTION	2	2
INSTRUMENT NUMBER	039	039
CONSTANT	20000.	20000.
ERROR ANGLE - A COMP.	.000	.000
ERROR ANGLE - B COMP.	.000	.000
ZERO SHIFT - A COMP.		0
ZERO SHIFT - B COMP.		0

Total Deflection between the PAST and PRESENT sets of data

PAST DATA			PRESENT DATA					
A+	A-	PAST DIFF.	A+	A-	PRESENT DIFF.	CHANGE	DEFL. IN.	DEPTH FT.
177	-179	356	124	-125	249	-107	1.0149	2.0
202	-195	397	210	-188	398	1	1.1112	4.0
266	-283	549	270	-283	553	4	1.1106	6.0
259	-261	520	267	-264	531	11	1.1082	8.0
254	-249	503	263	-251	514	11	1.1016	10.0
247	-248	495	246	-245	491	-4	1.0950	12.0
237	-236	473	234	-234	468	-5	1.0974	14.0
234	-247	481	241	-245	486	5	1.1004	16.0
255	-256	511	252	-250	502	-9	1.0974	18.0
243	-257	500	251	-259	510	10	1.1028	20.0
267	-279	546	263	-266	529	-17	1.0968	22.0
313	-319	632	319	-320	639	7	1.1070	24.0
335	-328	663	346	-328	674	11	1.1028	26.0
362	-362	724	355	-354	709	-15	1.0962	28.0
288	-294	582	299	-285	584	2	1.1052	30.0
265	-266	531	936	-936	1872	1341	1.1040	32.0
283	-279	562	549	-553	1102	540	.2994	34.0
290	-234	524	289	-277	566	42	-.0246	36.0
273	-281	554	278	-283	561	7	-.0498	38.0
271	-269	540	263	-258	521	-19	-.0540	40.0
194	-195	389	194	-193	387	-2	-.0426	42.0
182	-187	369	183	-183	366	-3	-.0414	44.0
190	-192	382	190	-190	380	-2	-.0396	46.0
176	-175	351	170	-171	341	-10	-.0384	48.0
200	-203	403	210	-205	415	12	-.0324	50.0
221	-223	444	219	-219	438	-6	-.0396	52.0
224	-237	461	225	-239	464	3	-.0360	54.0
254	-260	514	253	-250	503	-11	-.0378	56.0
192	-197	389	188	-191	379	-10	-.0312	58.0
240	-261	501	253	-267	520	19	-.0252	60.0
224	-231	455	213	-215	428	-27	-.0366	62.0

FIGURE 17 Tabulated computer output.

Data Quality Check and Statistical Routine

HILLSIDE AQUADUCT,ANCHOR MOVMT

DIGITILT DATA

INSTRUMENT PERFORMANCE CHECK

HOLE NUMBER B-1 | INITIAL SET NUMBER 1 | INITIAL DATE 20SEP,73 | CURRENT SET NUMBER 7 | CURRENT DATE 11OCT,76

INITIAL DATA A1	A2	INIT SUM	CURRENT DATA A1	A2	CURR SUM	DIFF	DEPTH FT.	INITIAL DATA B1	B2	INIT SUM	CURRENT DATA B1	B2	CURR SUM	DIFF	DEPTH FT.
177	-179	-2	124	-125	-1	1	2.0	271	-276	-5	240	-236	4	9	2.0
202	-195	7	210	-188	22	15	4.0	182	-178	4	181	-185	-4	-8	4.0
266	-283	-17	270	-283	-13	4	6.0	194	-193	1	202	-196	6	5	6.0
259	-261	-2	267	-264	3	5	8.0	231	-238	-7	235	-235	0	7	8.0
254	-249	5	263	-251	12	7	10.0	198	-220	-22	198	-218	-20	2	10.0
247	-248	-1	246	-245	1	2	12.0	145	-157	-12	158	-155	3	15	12.0
237	-236	1	234	-234	0	-1	14.0	126	-136	-10	130	-131	-1	9	14.0
234	-247	-13	241	-245	-4	9	16.0	121	-130	-9	124	-121	3	12	16.0
255	-256	-1	252	-250	2	3	18.0	101	-105	-4	101	-108	-7	-3	18.0
243	-257	-14	251	-259	-8	6	20.0	25	-46	-21	24	-39	-15	6	20.0
267	-279	-12	263	-266	-3	9	22.0	10	-22	-12	5	-4	1	13	22.0
313	-319	-6	319	-320	-1	5	24.0	10	-27	-17	14	-19	-5	12	24.0
335	-328	7	346	-328	18	11	26.0	14	-1	13	-19	16	-3	-16	26.0
362	-362	0	355	-354	1	1	28.0	48	-49	-1	-44	51	7	8	28.0
288	-294	-6	299	-285	14	20	30.0	60	-42	18	68	-40	28	10	30.0
265	-266	-1	936	-936	0	1	32.0	124	-138	-14	6	-18	-12	2	32.0
283	-279	4	549	-553	-4	-8	34.0	120	-154	-34	106	-107	-1	33	34.0
290	-234	56	289	-277	12	-44	36.0	110	-118	-8	110	-106	4	12	36.0
273	-281	-8	278	-283	-5	3	38.0	116	-126	-10	118	-126	-8	2	38.0
271	-269	2	263	-258	5	3	40.0	150	-146	4	159	-147	12	8	40.0
194	-195	-1	194	-193	1	2	42.0	141	-155	-14	148	-153	-5	9	42.0
182	-187	-5	183	-183	0	5	44.0	187	-198	-11	190	-194	-4	7	44.0
190	-192	-2	190	-190	0	2	46.0	171	-186	-15	173	-186	-13	2	46.0
176	-175	1	170	-171	-1	-2	48.0	144	-159	-15	141	-145	-4	11	48.0
200	-203	-3	210	-205	5	8	50.0	125	-155	-30	135	-160	-25	5	50.0
221	-223	-2	219	-219	0	2	52.0	173	-176	-3	177	-180	-3	-0	52.0
224	-237	-13	225	-239	-14	-1	54.0	152	-159	-7	156	-161	-5	2	54.0
254	-260	-6	253	-250	3	9	56.0	148	-160	-12	145	-154	-9	3	56.0
192	-197	-5	188	-191	-3	2	58.0	96	-98	-2	98	-94	4	6	58.0
240	-261	-21	253	-267	-14	7	60.0	146	-166	-20	162	-176	-14	6	60.0
224	-231	-7	213	-215	-2	5	62.0	183	-188	-5	181	-183	-2	3	62.0
143	-146	-3	136	-143	-7	-4	64.0	128	-139	-11	137	-146	-9	2	64.0
167	-184	-17	173	-198	-25	-8	66.0	219	-227	-8	233	-219	14	22	66.0
207	-214	-7	204	-206	-2	5	68.0	204	-213	-9	184	-210	-26	-17	68.0
180	-191	-11	181	-190	-9	2	70.0	111	-93	18	120	-95	25	7	70.0
204	-210	-6	205	-203	2	8	72.0	149	-164	-15	157	-168	-11	4	72.0
204	-207	-3	198	-202	-4	-1	74.0	167	-179	-12	171	-180	-9	3	74.0
197	-198	-1	191	-188	3	4	76.0	143	-157	-14	154	-161	-7	7	76.0

HILLSIDE AQUADUCT,ANCHOR MOVMT

DIGITILT DATA

INSTRUMENT PERFORMANCE CHECK
SUMMARY STATISTICS

HOLE NUMBER B-1 | INITIAL SET NUMBER 1 | INITIAL DATE 20SEP,73 | CURRENT SET NUMBER 7 | CURRENT DATE 11OCT,76

	A1, A2 COMPONENT: INITIAL SUM		CURRENT SUM		DIFFERENCE		B1, B2 COMPONENT: INITIAL SUM		CURRENT SUM		DIFFERENCE	
INTERVAL	MEAN	STD. DEV.	MEAN	STD. DEV.	MEAN	STD. DEV.	MEAN	STD. DEV.	MEAN	STD. DEV.	MEAN	STD. DEV.
2.0 TO 20.0	-4.	8.	1.	10.	5.	5.	-9.	8.	-3.	9.	5.	7.
22.0 TO 40.0	4.	19.	4.	8.	0.	17.	-6.	15.	2.	11.	8.	12.
42.0 TO 60.0	-6.	7.	-2.	7.	3.	4.	-13.	8.	-8.	8.	5.	3.
62.0 TO 76.0	-7.	5.	-6.	9.	1.	5.	-7.	11.	-3.	16.	4.	11.
ENTIRE HOLE	-3.	12.	-6.	9.	3.	9.	-9.	11.	-3.	11.	6.	9.

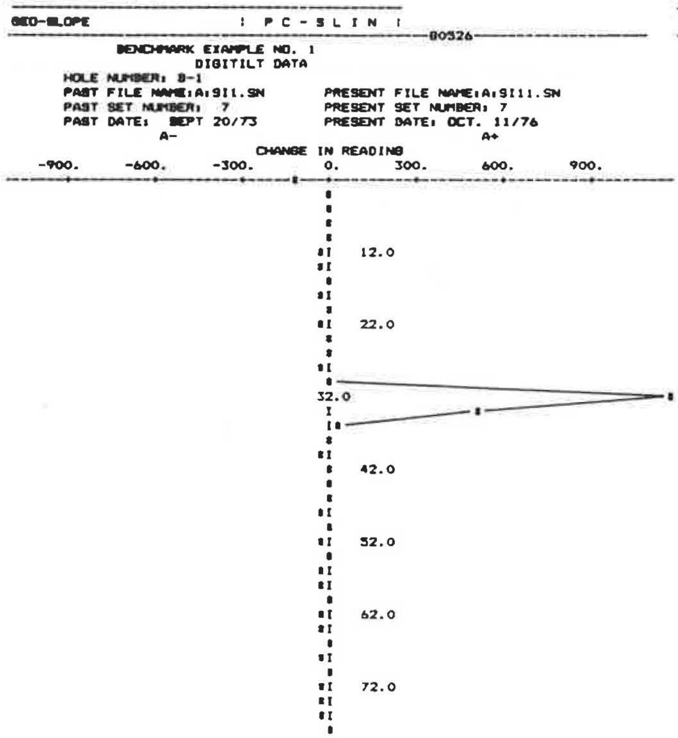
FIGURE 17 continued.

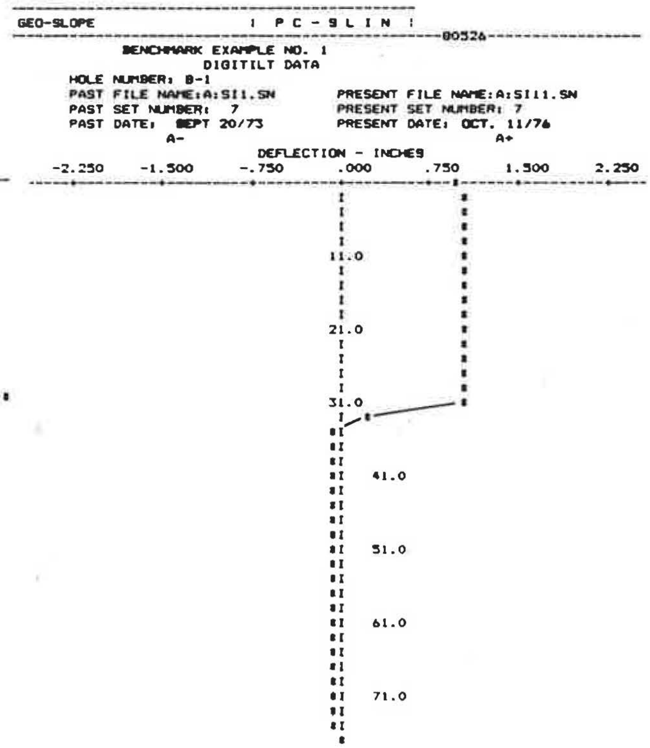
detection routines, and systematic error correction features. Readings can be input manually from hand-recorded data sheets or by RS-232C serial input from magnetic tape when recorded automatically, as with the Digitilt RPP.

Processed data (Figure 17) should be scrutinized by using error detection and correction routines to optimize the data quality and enable correct interpretation of engineering behavior. Graphical data presentation is essential and may include (a) slope change/depth, (b) displacement/depth, (c) displacement/time for a specific depth interval, and (d) displacement vector plots on a site plan, in that order (Figures 18, 19, and 20). Construction activities and instrument field crew observations should be noted on the plots to aid interpretation. Ground movements are usually progressive and continuous in one direction, although the rate of displacement will generally not be constant. Erratic displacement/time plots should always be carefully investigated. Figure 20 shows steady state creep in a reactivated ancient slide, whereas Figure 21 presents a decreasing deep-seated movement rate, that is, a more stable condition developing.

A wide range of random or systematic errors can occur, both obvious or subtle. Experience indicates that data interpretation is often difficult and that wrong conclusions are drawn concerning magnitude and locations of movement. Both authors have extensive experience in trying to diagnose errors and draw rational engineering conclusions from conflicting and inexplicable data, not always as successfully as desired. Errors arise due to equipment faults, user misuse, or mistakes, as well as recognized system limitations.

Equipment problems include sensor malfunction, wheel bearing wear, low batteries, moisture ingress to cable connections or readout, mechanical shock damage, nicked cables, improper casing installation, calibration changes inherent in manufacturer servicing, and cable stretch or marker movement with use. User errors arise due to reading errors and data





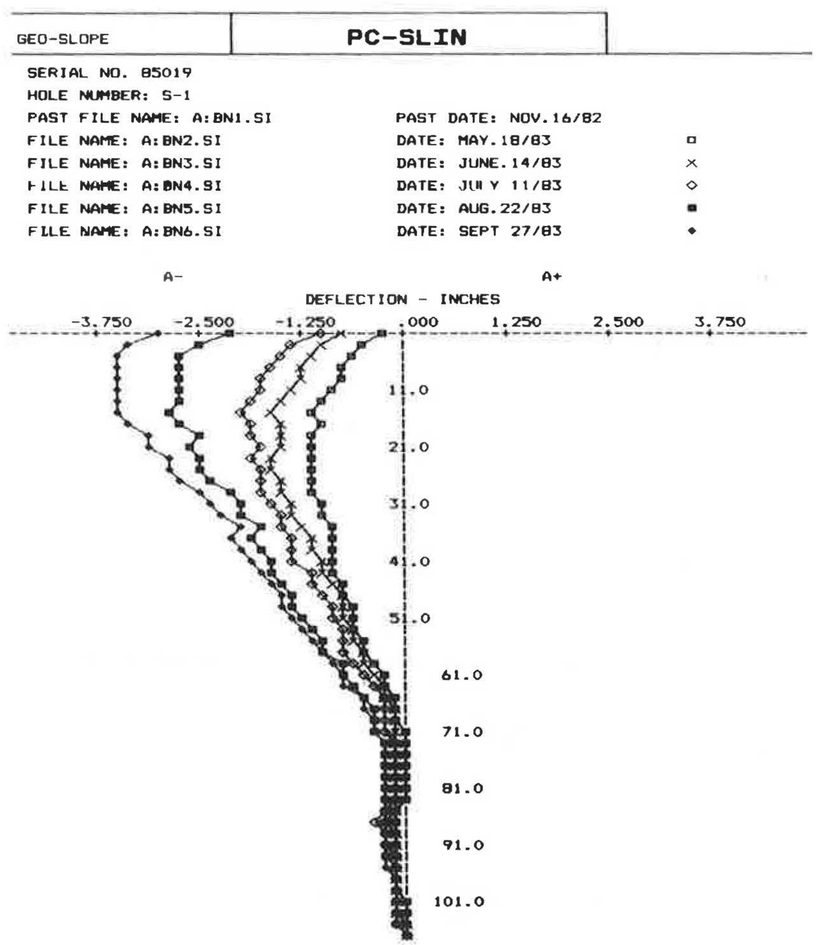


FIGURE 18 Graphical computer output: (top) single data set, (bottom) multiple data set.

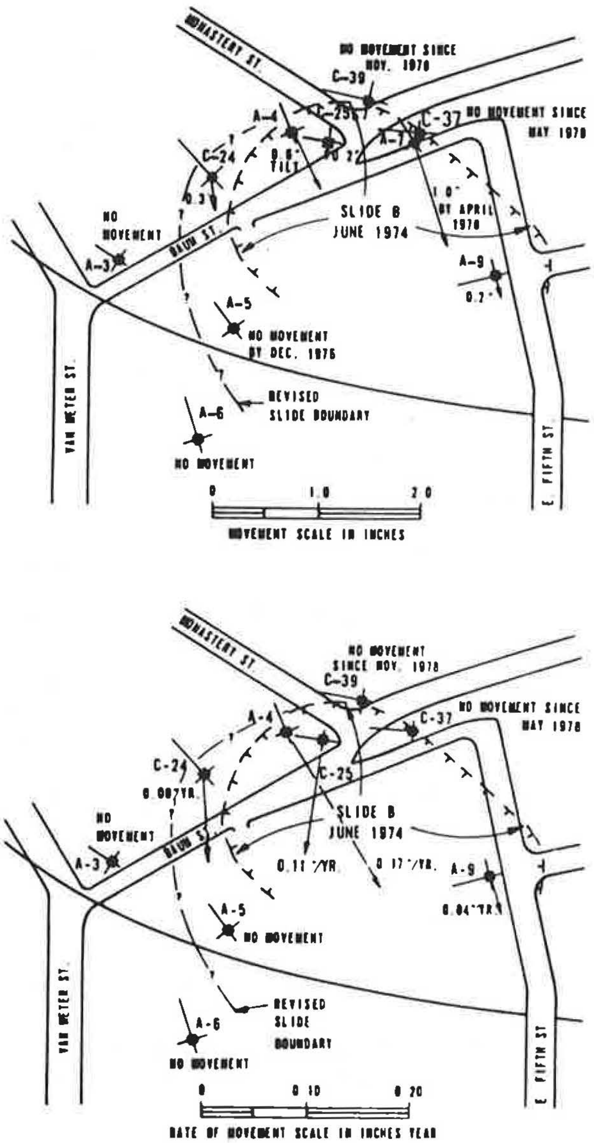


FIGURE 19 Vector plots of slide movements showing magnitude of movement January 1976–April 1979 and average rate of movement January 1978–April 1979.

transcription errors where manual methods are used; incorrect probe orientation in the casing grooves; probe depth position errors due to careless reading techniques, surface casing disturbance, or incorrect matching of readings 180 degrees apart; incorrect assumption of casing base fixity; unrecognized settlement or heave; improper reading techniques; or lack of survey control when telescopic couplings are used or when large settlements or heave must be accounted for. This is a formidable list of problems that can and do occur, and good project planning, equipment selection, training, data scrutiny, and awareness of the pitfalls are needed to avoid them.

If it is assumed that equipment and user related errors can be recognized and controlled, measurement errors also exist because of the inherent limitations of the best available instrument and casing. The data quality tabulation (Figure 17) is an essential aid and should always be used. Variation in checksums can be quickly scanned, errors detected, and data quality assessed. The statistical analysis provides a useful comparative instrument performance rating. If opposite walls of the inclinometer casing are not quite parallel, if any wheel is in a coupling, or if depth control is imprecise, the checksum may vary randomly. Small variations are tolerable.

Ideally, checksums should remain constant with depth within a given data set for both A and B sensors. If a change in the zero offset occurs between sets of readings, the checksum will change, but this does not matter. If a change occurs in the zero offset during a particular set of readings, either during a traverse or between opposite traverses due to temperature or shock-induced changes to the sensor suspension, the result will

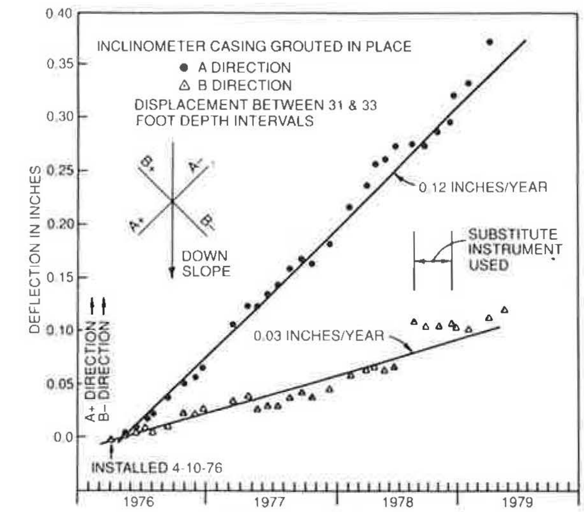


FIGURE 20 Creep movements on a thin shear zone in a reactivated ancient slide.

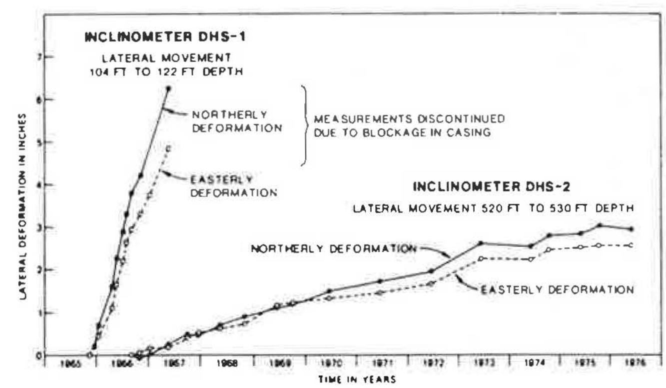


FIGURE 21 Deformations measured on deep shear zones on a landslide in Canada.

be errors that can only be eliminated by careful scrutiny and computerized error detection routines (Figure 22). When readings are obtained in slightly inclined casing, a correction for changes in the servo-accelerometer azimuth orientation may be required. Slight changes in the sensor due to shock occur from time to time. These changes show up as irregular rotation of the displacement/depth profile (Figure 22) and can mask shear occurring at discrete zones. The combination of sensor azimuth rotation with inclined casing causes systematic errors. Individual correction angles can be applied to the A and B sensors of each data set by using computerized routines and shear zone movement correctly identified (Figure 21). Although sensor zero shifts are a significant problem and must be properly dealt with during data processing, the scale span or slope of currently available servo-accelerometers is fairly stable, and errors due to this source appear to be less serious. They cannot be eliminated by routine procedures and require instrument recalibration by the manufacturer. Some manufacturers provide simple field calibration frames, but these appear to be of limited value.

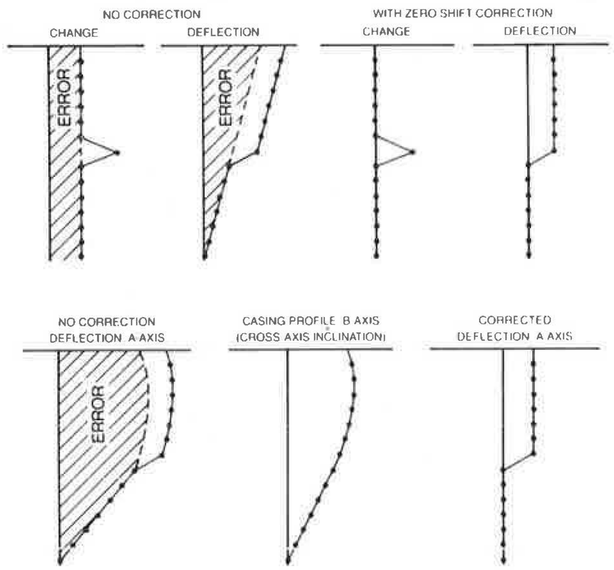


FIGURE 22 Typical zero shift and rotation errors.

Errors can arise immediately after installation due to settlement of the backfill. Erratic casing movements develop but usually cease with time. An extreme type of error can occur where drilling and grouting upset the stress and thermal regime of the surrounding material. Casings installed on a permafrost slope to monitor slope creep have shown extraneous movements due to recovery of thermal equilibrium around the casing, effects of stratigraphy and settlement, and heave of the casing (*28, 29*). High-quality monitoring procedures and painstaking analysis were employed that enabled definitive slope velocities of 0.30 cm/yr to be established. Other data filter techniques have been suggested (*30*); however, caution is urged in using statistical techniques.

Similar principles to the foregoing apply to horizontal inclinometer casings. Where access is available at both ends of the casing, first-order leveling is used to convert the inclinometer survey to a closed traverse, and settlement measurement accuracy is greatly increased. Inclined casings used to monitor displacement pose special problems (*7*) that require consideration of (a) groove spiral, (b) initial groove roll angle at the casing collar, and (c) sensor azimuth changes. In addition, if absolute position is desired, the azimuth of the vertical reference plane is determined by surveying the casing collar. Fixed-in-place inclinometers are subject primarily to sensor drift errors, and slope change/time plots for individual sensors should be scrutinized.

Measurement accuracy achieved with inclinometer systems is highly dependent on (a) the equipment selected, (b) the installation and monitoring procedures used, and (c) correct data scrutiny and processing. The system accuracy or precision achieved is quite different from the quoted resolution or sensitivity of the probe and readout. The inclinometer measures relative movements, so it may be more appropriate to discuss performance in terms of precision, that is, the repeatability with which the instrument can determine the position of one end of the casing relative to the other. A servo-accelerometer-type inclinometer system is capable of a precision of ±0.05 to 0.25 in. over 100 ft in vertical and horizontal casings but much less in inclined casings (*7*). It is, of course, rare for an accurate independent check to be possible (*12*), and estimates of precision or accuracy achievable are based on a combination of direct measurement, experience, and reasonableness of the data.

PLANNING AND EXECUTION OF INCLINOMETER MONITORING PROGRAMS

As with all geotechnical monitoring programs, inclinometer measurements must be properly planned and executed. Too often instruments are installed for ill-defined reasons, and data are collected and filed away, unused (*10, 31*). Instrumentation has become fashionable. In contrast, real needs should be assessed, behavior predicted, and tasks assigned following a systematic approach. What needs to be measured, the accuracy required, and if and how this can be achieved should be considered. Specialist advice on instrumentation system design should be sought where in-house skills are lacking. The engineer is usually the party most interested in obtaining reliable monitoring data and should retain as great a control as is possible. In general, instrumentation should not be left up to the contractor.

Descriptive or performance specifications for instrument procurement may be used (*10, 31, 32*). Too often a single lump sum bid item appears in a specification for supply and installation of a variety of instruments, and the low bidder wins (*7*). Under these conditions the engineer often ends up with something far less than is needed, even though considerable sums of money have been expended. It is preferable for the engineer to select the make and model number of instrument believed to be most suitable and to specify with no substitution allowed. Sole source justification may be required and can be used (*33*). The inclusion of "or equal" leads to excessive emphasis on low cost so that high-quality instruments are excluded (*34, 35*). There is often no good, legally defensible way of excluding substitute instruments. An entire inclinometer monitoring program may fail if an inferior instrument is selected. This is not an acceptable risk and hence must be avoided. Detailed procedures for minimizing these risks are discussed elsewhere (*10, 31*).

CONCLUSION

The inclinometer is widely and successfully used to measure ground movements and is capable of considerable accuracy. Successful projects require good planning, high-quality and appropriate instruments, proper installation, diligent reading, correct data processing, and intelligent scrutiny and interpretation. Cooperation and understanding between engineers, clients, and contractors is essential for success.

ACKNOWLEDGMENTS

We are indebted to our colleagues for support. We are also grateful to our clients, through whose projects we have come to better understand many of the problems of instrument performance and subtleties of data analysis. These experiences led to the improvements discussed in this paper. The substance of this paper was originally presented at a symposium on field instrumentation organized by the Nanyang Technological Institute, Singapore, in November 1986.

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Publication of this paper sponsored by Committee on Soils and Rock Instrumentation.

Dilatometer Experience in Washington, D.C., and Vicinity

PAUL W. MAYNE AND DOUGLAS D. FROST

For 3 years now, the flat dilatometer has been used to supplement standard soil borings and cone penetration tests during routine geotechnical investigations at project sites located within a 300-km radius of Washington, D.C. Although criticized for its mostly empirical nature, the dilatometer test (DMT) appears to provide very reasonable interpretations of soil properties in a diversity of geologic formations including residuum, marine sediments, and alluvium. Moreover, the DMT is an expedient and cost-effective method of profiling subsurface conditions, except in very dense or gravelly deposits where insertion is difficult. Several case histories are presented in support of these conclusions.

Over 160 dilatometer soundings have been completed in the vicinity of Washington, D.C., Northern Virginia, and Maryland during the past 3 years. These dilatometer tests (DMTs) were performed in conjunction with routine geotechnical studies using soil borings and cone penetration tests. Several sites are located as far north as Baltimore, Maryland, and Philadelphia, Pennsylvania, and as far south as Newport News, Virginia, and Wilmington, North Carolina. In addition to simplicity and economy of operation, the primary advantage of the DMT is a direct interpretation of engineering parameters for use in analysis.

The test involves thrusting a flat steel blade into the ground and, at predetermined depth intervals, measuring the lift-off and expansion pressures of a flexible membrane located on one face of the blade. Test procedures are described by Marchetti (*1*) and Schmertmann (*2*). The hydraulic unit on a standard drill rig is used to advance the dilatometer attached to the end of cone rods. No borehole is required, unless very dense, hard, or gravelly layers are encountered. Each test takes approximately 1 min, including measurement of the thrust, contact pressure (P_o or A reading), and expansion pressure (P_1 or B reading). Recently, the recording of the closing pressure (C-reading) of the membrane has been included, which adds another 15 sec duration to each test. For production, tests are normally taken at 0.3 m intervals. An example of DMT records taken in soft organic alluvial clay is presented in Figure 1. The fact that these soundings are about 100 m apart and were taken by three different operators indicates the repeatability of the test, as well as the uniformity of the deposit.

Much of the interpretation centers around three indices, which are calculated from P_o, P_1, the effective overburden stress (σ_{vo}'), and the ambient hydrostatic pressure (u_o). Actually, in sands, the C-reading has been shown to be a measure of u_o (*2*). The DMT indices are

$$I_D = \text{classification index} = (P_1 - P_o)/(P_o - u_o) \quad (1)$$

$$K_D = \text{horizontal stress index} = (P_o - u_o)/(\sigma v_o') \quad (2)$$

$$E_D = \text{dilatometer modulus} = 34.7\ (P_1 - P_o) \quad (3)$$

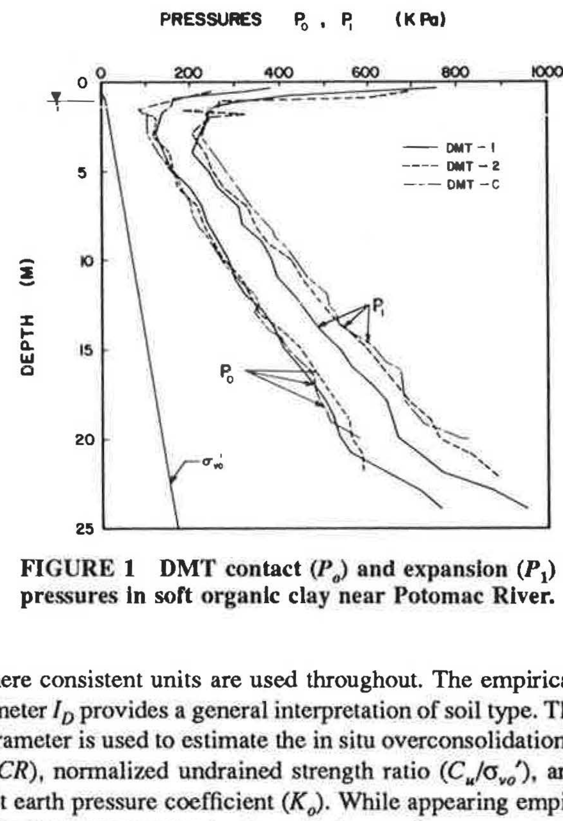

FIGURE 1 DMT contact (P_o) and expansion (P_1) pressures in soft organic clay near Potomac River.

where consistent units are used throughout. The empirical parameter I_D provides a general interpretation of soil type. The K_D parameter is used to estimate the in situ overconsolidation ratio (OCR), normalized undrained strength ratio (C_u/σ_{vo}'), and at-rest earth pressure coefficient (K_o). While appearing empirical, it has been shown that in clays, K_D is actually a measure of the normalized excess pore pressure ($\Delta u/\sigma_{vo}'$) caused during blade penetration (*3*). The modulus E_D is derived from elastic theory considerations and serves as the basis for estimating the constrained modulus ($M = 1/m_v$). In sands, a theoretical development has allowed the calculation of the effective stress friction angle (ϕ') from the thrust measurements (*4*). Details concerning DMT data reduction are given by Bullock (*5*). An example of the reduced computer output is presented as Figure 2.

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LAW ENGINEERING                                                    TEST NO. DMT-17
FILE NAME:     CEBAF
FILE NUMBER:   NK5-1182

RECORD OF DILATOMETER TEST NO. DMT-17
USING DATA REDUCTION PROCEDURES IN MARCHETTI (ASCE,J-GED,MARCH 80)
KO IN SANDS DETERMINED USING SCHMERTMANN METHOD (1983)
PHI ANGLE CALCULATION BASED ON DURGUNOGLU AND MITCHELL (ASCE,RALEIGH CONF,JUNE 75)
PHI ANGLE NORMALIZED TO 2.72 BARS USING BALIGH'S EXPRESSION (ASCE,J-GED,NOV 76)
MODIFIED MAYNE AND KULHAWY FORMULA USED FOR OCR IN SANDS (ASCE,J-GED,JUNE 82)

     LOCATION: NEWPORT NEWS, VIRGINIA
     PERFORMED - DATE:  9-26-86
                  BY:   FROST/TROUT

     CALIBRATION INFORMATION:
     DA=   .13 BARS     DB=   .30 BARS     ZM=   .00 BARS     ZW=  1.93 METERS
     ROD DIA.=  3.70 CM    FRICTION RED. DIA.=  4.80 CM    ROD WEIGHT=  6.50 KG/M    DELTA/PHI=   .50

     1 BAR = 1.019 KG/CM2 = 1.044 TSF = 14.51 PSI         ANALYSIS USES H20 UNIT WEIGHT =  1.000 T/M3
```

Z (M)	THRUST (KG)	A (BAR)	B (BAR)	ED (BAR)	ID	KD	UO (BAR)	GAMMA (T/M3)	SV (BAR)	PC (BAR)	OCR	KO	CU (BAR)	PHI (DEG)	M (BAR)	SOIL TYPE
.61	600.	5.50	12.60	243.	1.32	48.15	.000	1.950	.110	65.43	*****	6.19		23.2	969.2	SANDY SILT
.91	600.	5.50	13.80	287.	1.58	31.28	.000	1.950	.167	38.07	*****	4.17		23.6	1026.4	SANDY SILT
1.22	600.	5.60	18.80	465.	2.63	22.38	.000	2.000	.227	24.41	*****	3.11		24.0	1518.1	SILTY SAND
1.52	600.	4.50	15.40	381.	2.68	14.34	.000	2.000	.286	10.22	35.69	2.10		26.5	1083.9	SILTY SAND
1.83	600.	2.00	3.40	35.	.49	6.10	.000	1.600	.341	1.94	5.70	1.33	.303		70.5	SILTY CLAY
2.13	600.	2.00	6.50	148.	2.24	5.13	.020	1.800	.372	1.76	4.74	.89		31.6	279.9	SILTY SAND
2.44	600.	1.20	2.00	13.	.31	3.21	.050	1.600	.393	.82	2.09	.83	.156		18.0	CLAY
2.74	600.	1.00	2.20	28.	.80	2.47	.079	1.600	.411	.57	1.39	.66	.117		30.4	CLAYEY SILT
3.05	800.	1.50	4.20	83.	1.69	3.27	.110	1.700	.430	.92	2.15	.60		34.9	118.6	SANDY SILT
3.35	1500.	5.20	15.80	371.	2.28	10.28	.139	2.000	.455	6.47	14.20	1.44		35.2	936.2	SILTY SAND
3.66	2150.	7.40	22.60	538.	2.34	13.63	.170	2.000	.486	11.35	23.36	1.81		36.3	1503.2	SILTY SAND
3.96	2700.	6.60	21.60	531.	2.64	11.26	.199	2.000	.515	7.68	14.91	1.46		38.9	1386.9	SILTY SAND
4.27	2400.	8.80	21.40	443.	1.58	14.85	.230	1.950	.545	15.30	28.08	1.98		35.9	1274.6	SANDY SILT
4.57	2200.	4.80	9.80	167.	1.08	7.79	.259	1.800	.571	4.75	8.33	1.57			374.9	SILT
4.88	1800.	5.40	7.20	50.	.28	8.69	.289	1.800	.595	5.89	9.90	1.68	.821		117.6	CLAY

END OF SOUNDING

FIGURE 2 Reduced DMT data for Norfolk and Yorktown sediments at Newport News, Virginia.

In practice, the DMT results are used to predict the behavior of foundations and embankments. Some of the common types of problems and suggested methods of analysis for these situations are summarized in Table 1. The dilatometer can provide estimates of the various required parameters. Problems involving elastic settlements, however, may require an assumed value of Poisson's ratio ($\overline{\mu}$), which fortunately does not greatly affect the calculated results. The elastic modulus $E = (1 - \overline{\mu}^2)E_D$ for undrained conditions may be obtained using $\overline{\mu} = 0.5$. In order to illustrate the applicability of the DMT, several case studies are presented in subsequent sections.

HANGAR 91, WASHINGTON, D.C.

At the confluence of the Anacostia and Potomac rivers, a thick alluvial deposit of soft organic silty clay (highly organic or "OH") extends to typical depths of 18 to 25 m. The land was reclaimed about 1910 when 1 to 2 m of sandy fill were placed to form Potomac Park as well as two military reservations. In 1941, an aircraft hangar was constructed, which added about 1 m of new fill, a 0.45-m-thick at-grade slab, and about 10 KPa floor loading. The structural columns were supported on timber piles. The floor slab has exhibited significant settlements of up to 1 m since that time (as shown by Figure 3), requiring a major reconstruction of the floor and finally, the design of a new replacement structure to be built soon near the area.

The properties of the clay have been previously characterized for several subway crossings (*14*). Typical index properties include $LL = 83$, $PI = 37$, $w_n = 68$, and $e = 1.68$. Typical standard penetration test (SPT) resistances of about 0 to 3 blows/30 cm are indicated in Figure 4. Piezocone data reported by Mayne (*3*) show cone resistances (corrected for pore pressure effects on unequal areas) increasing with depth from about 450 to 900 KPa. The stress history profile shown in Figure 4 is believed typical of undeveloped areas beneath the original fill. Apparently, based on time rate of consolidation calculated from oedometer data, the soft clay directly beneath Hangar 91 may actually still be undergoing the process of primary consolidation as a result of the imposed floor loading.

The undisturbed clay appears to be lightly overconsolidated, probably as a result of a combination of some erosion of the overburden and aging effects. The portion resulting from aging may be approximately estimated as

$$OCR = (t/t_p)\ C_\alpha/C_c \tag{4}$$

where

t = time of consideration,
t_p = time for primary consolidation,
C_α = coefficient of secondary consolidation, and
C_c = virgin compression index.

TABLE 1 APPLICATIONS OF DMT DATA

Geotechnical Problem	Required Parameters	Method of Analysis
Settlement		
Embankment	M, P_c'	Schmertmann (*2*)
Spread footing	E	Poulos and Davis (*6*)
	M	Schmertmann (*2*)
Structural mat	$E, \bar{\mu}$	Fraser and Wardle (*7*)
Pile foundation	$E, \bar{\mu}$	Poulos and Davis (*8*)
Undrained distortion	E_u, OCR, K_o, C_u	Foott and Ladd (*9*)
Stability analyses	C_u, ϕ', γ	Siegel (*10*)
Pile capacity	C_u, ϕ'	Poulos and Davis (*8*)
	K_o	Marchetti et al. (*11*)
Time rate of settlement	c_v	Lutenegger and Kabir (*12*)
Permeability	k	Robertson et al. (*13*)

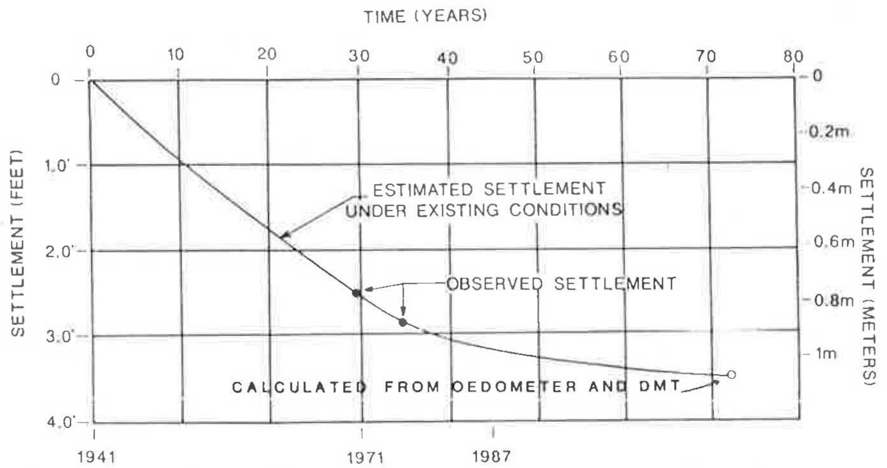

FIGURE 3 Observed settlement of Hangar 91 floor over 30-year period.

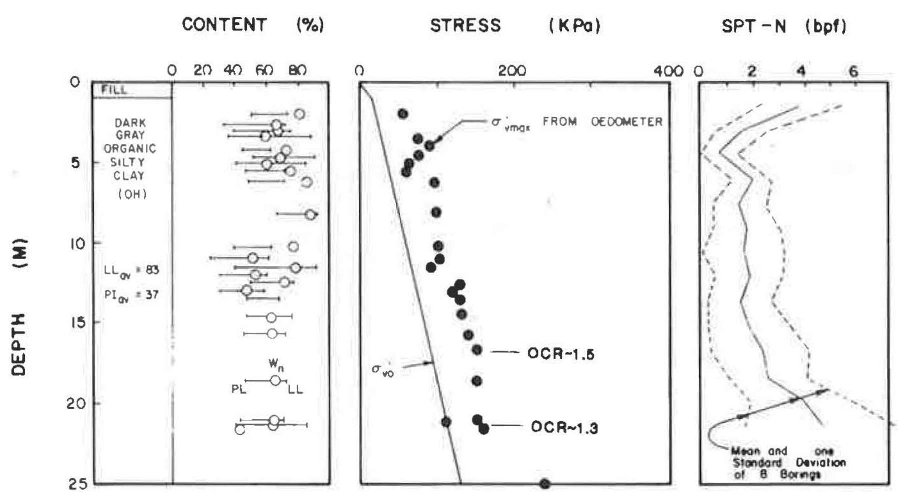

FIGURE 4 Stress history profile and index properties of Potomac River alluvium.

As shown by Mesri and Castro (*15*), the ratio of C_α/C_c may be assumed to be 0.05 for organic plastic clays. For a 50-year period and $t_p = 0.01$, the calculated *OCR* as a result of creep is about 1.5.

The profiles of *OCR* from 5 DMT soundings are shown to be in good agreement with standard oedometer tests conducted on thin-walled tube specimens of the clay (Figure 5). The Casagrande method of determining σ_p' was used for oedometer data. The values of constrained moduli from the DMTs are also shown to compare well from those determined from consolidation tests.

The long-term performance of Hangar 91 allows a backcalculation of soil parameters from measured settlements. An analysis of the induced stresses under the 48-by-73-m building reveals that the upper 5 to 6 m of the clay are forced into a normally consolidated state corresponding to virgin compression. In this regard, the "special case" of settlement calculation described by Schmertmann (2) is warranted. In

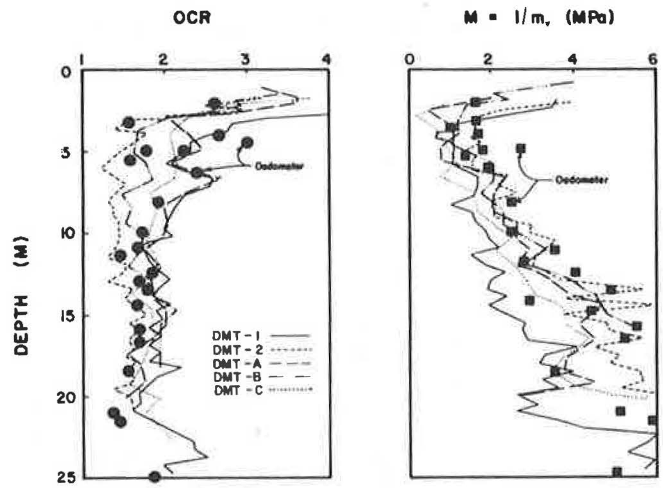

FIGURE 5 Profiles of overconsolidation ratio (*OCR*) and constrained modulus (*M*) from DMT soundings and laboratory oedometer tests.

addition, significant settlements caused by undrained distortion would be expected for loadings beyond the maximum past pressure (*9, 16*).

By definition, the constrained modulus ($M = 1/m_v$) equals the change in vertical stress ($\Delta\sigma_v$) divided by change in vertical strain ($\Delta\Sigma_v$) under conditions of one-dimensional consolidation ($\Delta\Sigma_h = 0$):

$$M = \Delta\sigma_v'/\Delta\Sigma_v \qquad (5)$$

The virgin compression index (C_c) is commonly used to characterize settlements in the normally consolidated range:

$$C_c = \Delta e/\Delta \log \sigma_v' \qquad (6)$$

and since strain is related to void ratio (e) by the expression $\Delta\Sigma = \Delta e/(1 + e)$, it may be derived that

$$M = (1 + e) \ln 10\ \sigma_v'/C_c \quad \text{for } \sigma_v' > \sigma_p' \qquad (7)$$

The results of some 28 consolidation tests indicate a mean value $C_c = 0.83 \pm 0.25$ for $\sigma_v' > \sigma_p'$, corresponding to normally consolidated states. Consequently, the constrained modulus for the "special case" was taken as $M = 7.4\ \sigma_v'$ for calculations involving stresses that exceeded the in situ preconsolidation pressures.

Based on the DMT data, the calculations suggest an undrained distortional settlement of 0.34 m and drained settlement under primary consolidation of 0.64 m. The predicted total settlement of 0.98 m is comparable to the observed value of 0.86 m after 35 years from construction (Figure 3).

Although no dissipation tests were taken at the Potomac River site, methods of evaluating the time rate of consolidation and coefficient of consolidation (c_v) from DMT readings are now available (*12, 13*).

ACCELERATOR FACILITY, NEWPORT NEWS, VIRGINIA

The design of a new continuous-wave electron beam accelerator required stringent tolerances for differential settlement. For protective shielding, the beam tunnels would require 6-m-high earth berms constructed along the 1700-m-long facility. The project site is underlain by approximately 7 m of variable sands, silts, and clays of the Norfolk formation over preconsolidated sandy (low plasticity) clays (CL) and silts (ML) of the Yorktown formation.

The stress history profile of the site as determined from one-dimensional oedometer tests and triaxial shear tests is shown in Figure 6. In order to better define the apparent yield stress corresponding to the preconsolidation pressure, a dead weight consolidometer was modified to permit loading as high as 5000 kN/m². This ensured compression into the normally consolidated range and reduced the uncertainty of interpreting σ_p'.

The results of isotropically consolidated undrained triaxial compression tests (CIUC) were also used to determine the in situ *OCR*s. This method involves a stress history and

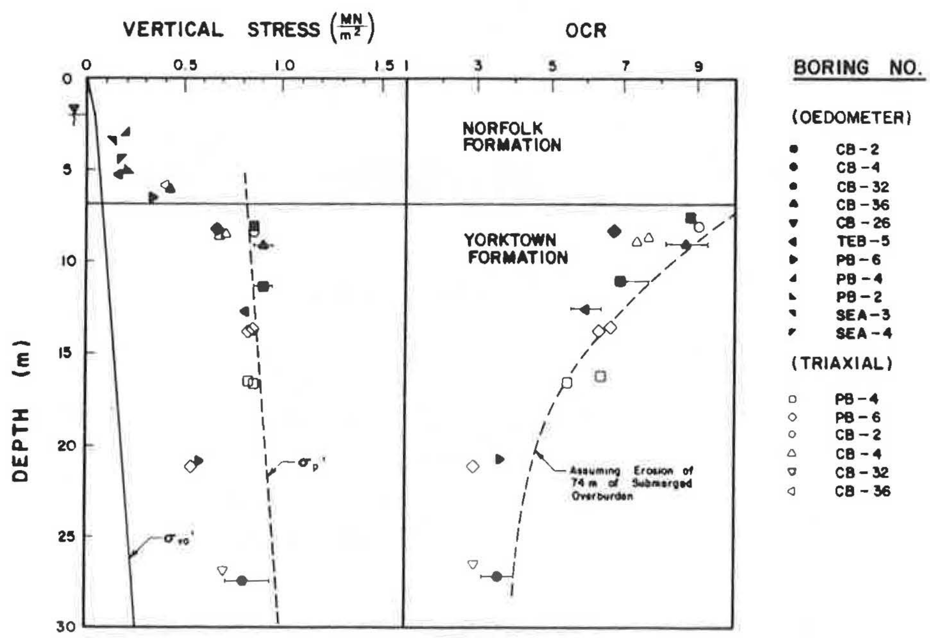

FIGURE 6 Stress history of Norfolk and Yorktown formations.

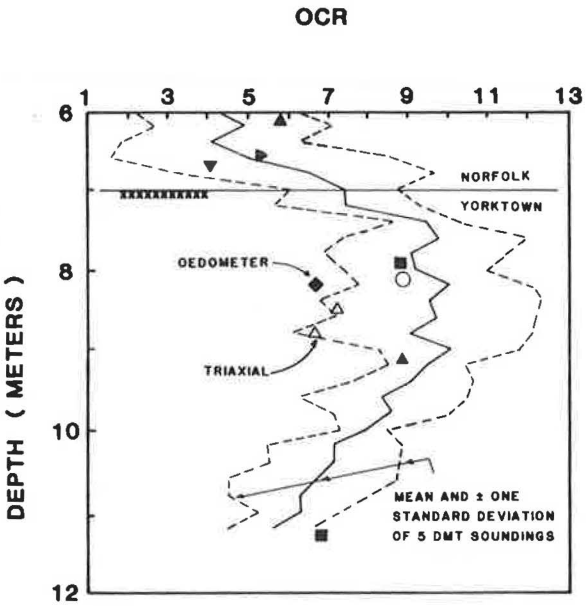

FIGURE 7 OCR profile of Yorktown from DMT data.

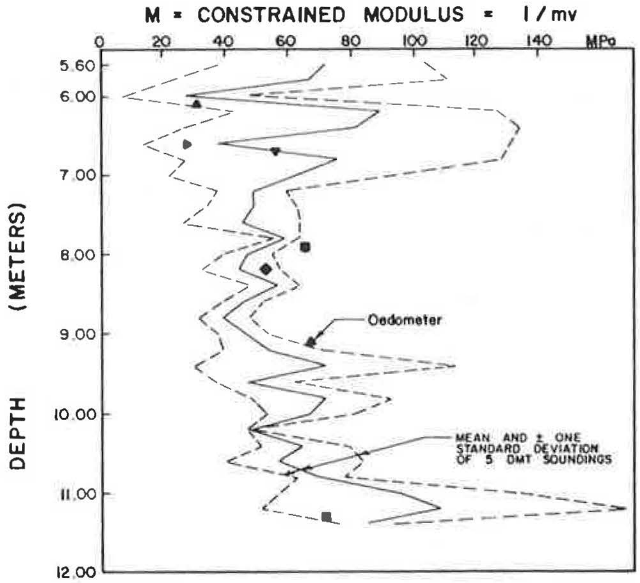

FIGURE 8 Constrained moduli of Yorktown from DMT and oedometer data.

normalized engineering parameter (SHANSEP) (*9*) data base and normalized strength parameters to determine the degree of overconsolidation (*17*). These laboratory test series imply a mean σ_p' of about 830 ± 140 KPa in the Yorktown formation. The results of DMTs show comparable profiles of *OCR* at the Newport News site (Figure 7). Although 17 DMT soundings were advanced at this site, only 5 were able to penetrate a very dense shell layer that separates the Norfolk and Yorktown formations.

The expected magnitudes of primary settlement under the proposed berm loading were evaluated using four separate approaches: oedometer, DMT, finite element method (FEM), and full-scale instrumented test berm. Conventional analyses using oedometer data suggested total settlements on the order of 5.0 cm for the area of the site designated for construction of the test embankment. The DMT sounding from this area (DMT-17 shown in Figure 2) predicted 3.7 cm for primary compression based on the established *M* values (Figure 8). An FEM study was initiated using a plane-strain model developed by Duncan et al. (*18*), which indicated a maximum settlement of 4.3 cm under the center of the embankment. Actually, values of σ_p' and *M* from oedometer tests on the sands and clays of the

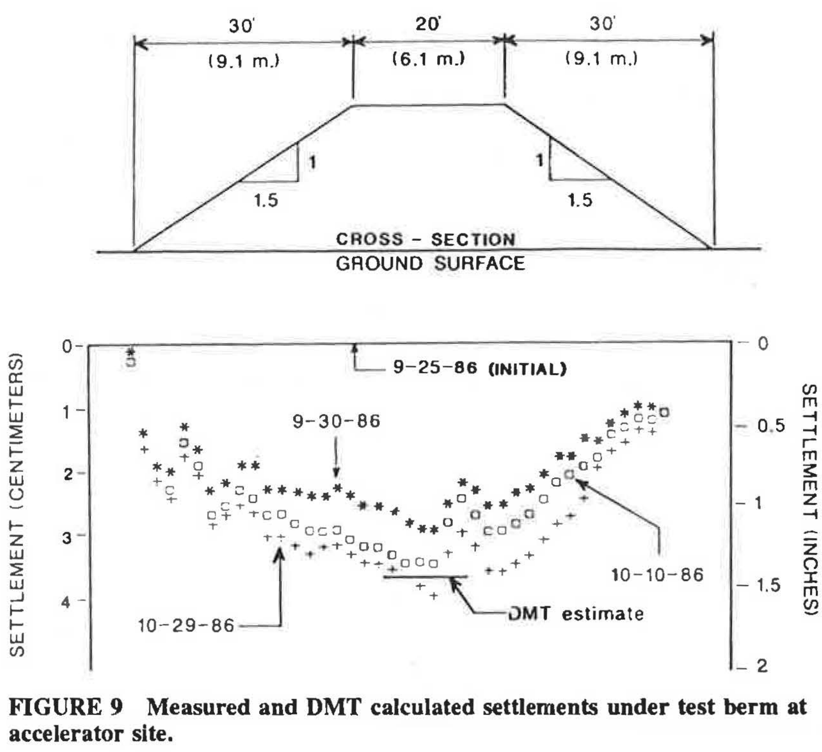

FIGURE 9 Measured and DMT calculated settlements under test berm at accelerator site.

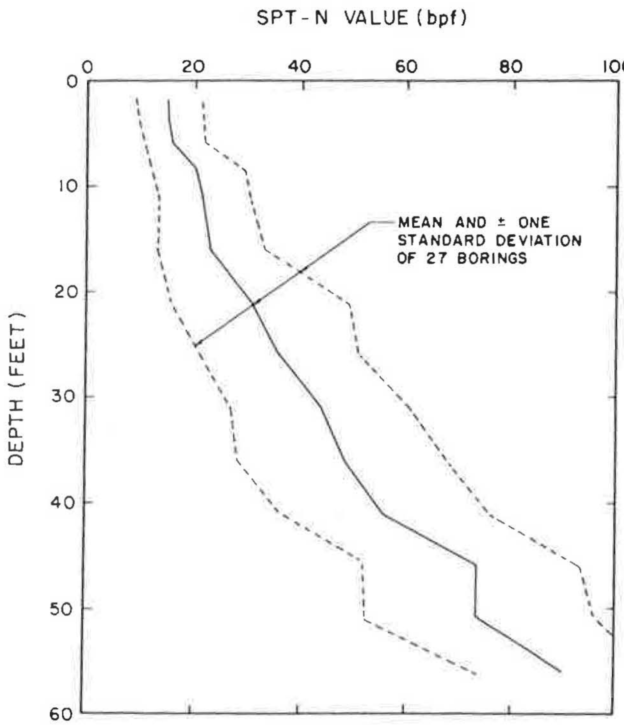

FIGURE 10 Statistical variation of SPT resistance with depth in Piedmont residuum at Fairfax Hospital.

upper Norfolk formation may have been slightly underestimated because of sample disturbance effects. Furthermore, most of these samples were not tested with the high-stress-level oedometer.

The test berm was constructed in September 1986 and instrumented with horizontal inclinometer pipe, settlement plates, borros points, pneumatic piezometers, and open standpipes. Maximum recorded settlements of 3.5 cm were observed in May 1987, although as much as 85–95 percent of all settlements had apparently occurred as recently as October 1986. The settlement profile recorded by the horizontal slope indicator along the test embankment's long axis is shown in Figure 9.

FAIRFAX HOSPITAL, FALLS CHURCH, VIRGINIA

A geotechnical exploration for a new 13-story hospital wing included dilatometer soundings, soil borings, and cone penetration tests. With design loads in axial compression of up to 9 MN, drilled shafts were selected as the appropriate foundation system, having been used for the existing main hospital wing, as well as providing minimal noise and vibration during installation.

The site lies within the Piedmont Geologic Province, which extends from Philadelphia to beyond Atlanta. The overburden soils are residuum, having been derived from the in-place weathering of the underlying metamorphic and igneous bedrock (primarily schist and phyllite at this site). In this regard, penetration test data typically increase with depth, reflecting the lower degree of weathering and saprolitic structure of the formation (Figure 10). The residuum typically classifies as a fine sandy silt (ML) and contains varying amounts of mica.

In addition to geotechnical data being collected for the new hospital wing, the results of two previous load tests were reviewed. The tests included a drilled shaft foundation (d = 0.91 m and L = 19.8 m) tested for the main wing construction in 1967 and a bored, or augered, concrete pile (d = 0.36 m and L = 12.8 m) tested for the ambulatory wing in 1977.

The results of two soundings advanced at the site indicated statistical mean values from 64 individual DMTs: I_D = 1.8, E_D = 35 MPa, and M = 74 MPa. Using the method of Poulos and Davis (*8*), the E_D modulus may be used to provide a prediction of the load-deflection response of deep foundations subjected to axial compression loading. As shown by Figure 11, reasonable predictions of the pier and pile performances are obtained with the DMT results.

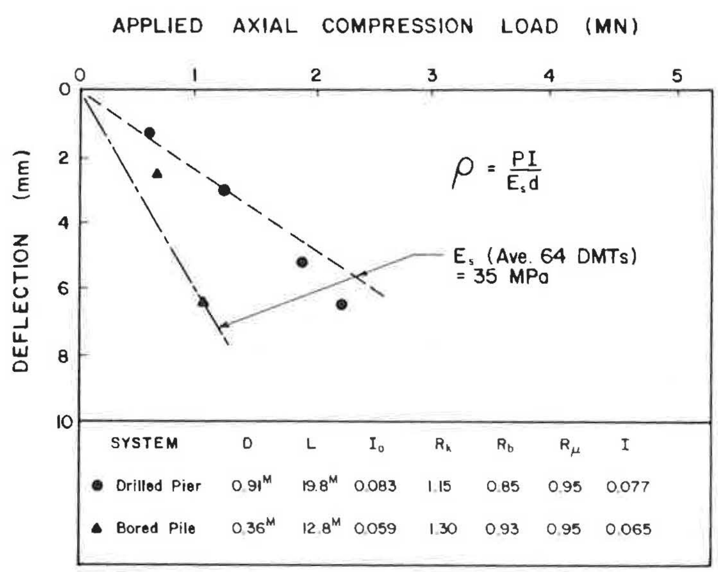

FIGURE 11 Measured and predicted load deflections of drilled shaft and bored pile at Fairfax Hospital.

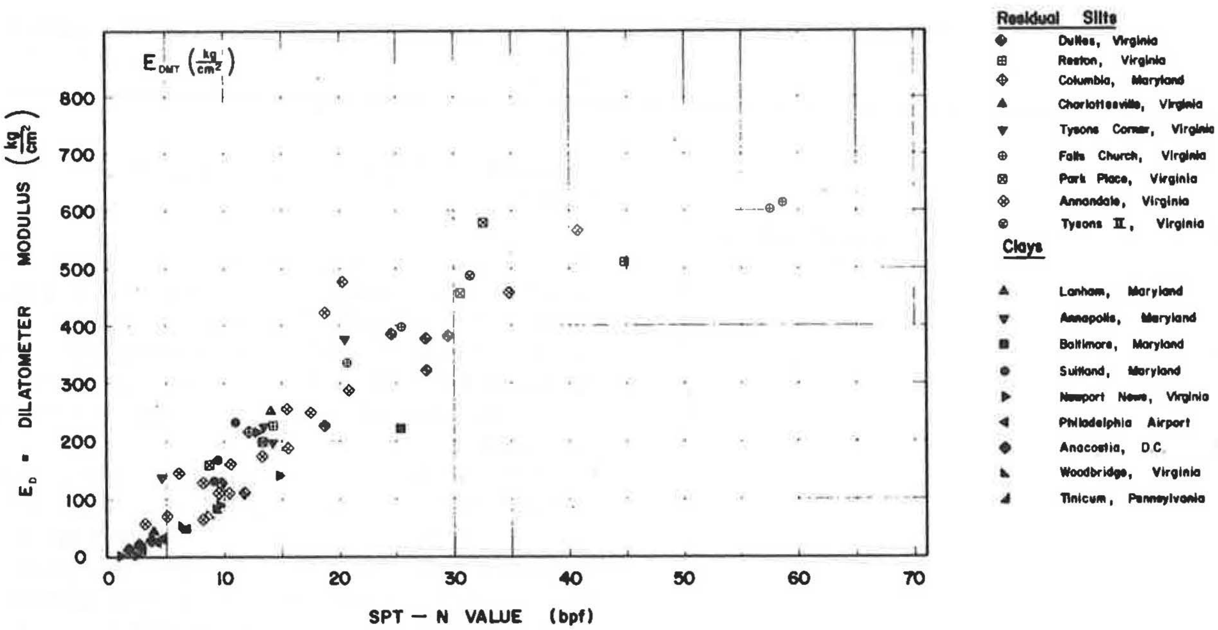

FIGURE 12 Observed correlation between dilatometer modulus (E_D) and SPT resistance in Piedmont residuum near Washington, D.C.

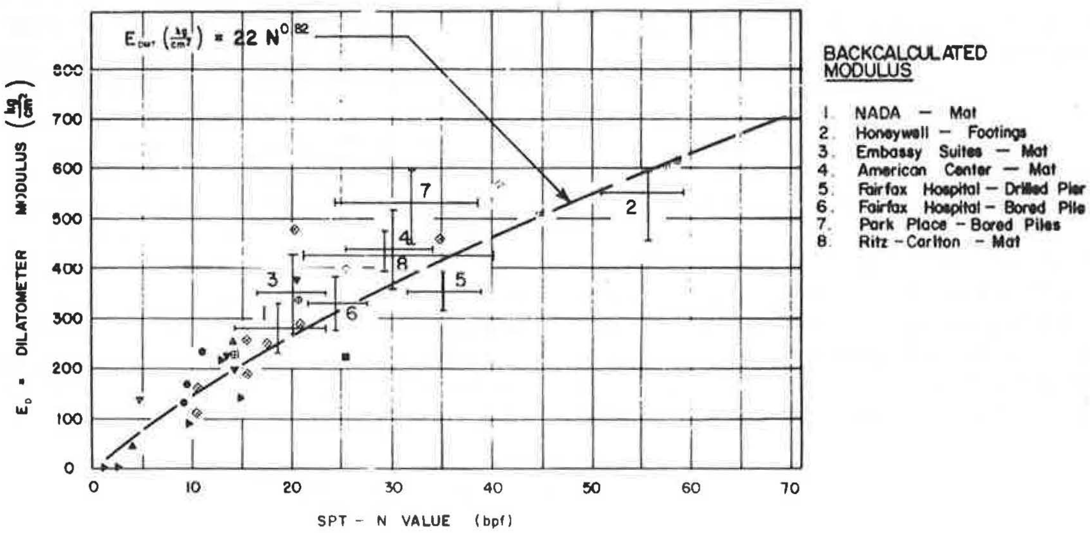

FIGURE 13 Backcalculated moduli from foundation performance data in Piedmont residuum.

ADDITIONAL SITES

Field performance data from several other projects located in Piedmont residuum have been collected by the authors and from published sources (*19, 20*). For routine use on small projects in the residual silts of the Washington, D.C., area, a correlation was developed between the dilatometer modulus (E_D) and SPT resistance because of the widespread use of the latter. Data points represent averages of E_D from DMTs and *N*-values from SPT. The observed correlation (Figure 12) bears a remarkable similarity to the mean relationship determined by Martin (*20*) between pressuremeter modulus and standard penetration tests. The DMTs suggest that for this sandy silt residuum, the local correlation is

$$E_D = 22P_a N^{0.82} \tag{8}$$

where P_a is referenced equal to atmospheric pressure (1 bar = 1 tsf = 100 kPa). Measured settlement data from four mat foundations were reviewed and used to backcalculate soil moduli using elastic theory (*6, 7*). Although DMT data were not directly available for those particular sites, these properties are located adjacent to sites from which the E_D correlation with *SPT* was developed. Figure 13 includes data from the performance of spread footing foundations (*20*) and two additional bored pile load tests performed in Tysons Corner, Virginia. DMT data from the latter site gave good predictions of pile performance under axial compression loading, similar to those described for the Fairfax Hospital project.

CONCLUSIONS

Over 160 dilatometer test soundings have been used to supplement routine soil boring and cone penetrometer soundings in the vicinity of Washington, D.C., during the past 3 years. The DMT estimates of overconsolidation ratio and moduli compare reasonably with values obtained from laboratory oedometer tests as well as backcalculated values from field performance data.

ACKNOWLEDGMENTS

Appreciation is extended to Law Engineering from which much of the field and laboratory data were collected and to Daad Pierce, Bobbie Maier, and Ken Ellis for assisting in preparation of the manuscript.

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Publication of this paper sponsored by Committee on Soils and Rock Instrumentation.

Ground Vibration Monitoring Instrumentation and Computerized Surveillance

CHARLES H. DOWDING AND DALE JESSEN

Computerized surveillance is the high ground to which this paper leads. Data acquisition and processing now possible with "watch dog" computers in the field linked by telephone to monitoring engineers in the office are best appreciated by an investigation of the components of that instrumentation system. This paper presents the details of the likely input, critical components of such a system, proper placement of sensing transducers, development history of these instruments, and the new considerations necessary with computerized digitization of analog signals.

During construction, ground-motion-induced relative displacement of structures causes strains that can lead to cracking. These strains can be measured directly or they can be estimated from measurements of the motion of the structure or the ground and air adjacent to the structure. These motions also elicit human response, which normally manifests itself in annoyance and complaints. This annoyance can be assessed with the same instruments used to monitor and control the possibility of cracking.

This paper describes the characteristics of the instruments that measure the motions (acceleration, velocity, and displacement). Since there are many excellent sources for information on instruments, the principal characteristics of available systems will be summarized rather than exhaustively reviewed. The most complete single reference for detailed instrumentation information is the *Shock and Vibration Handbook* (*1*). Specific information on construction and blast vibration monitors is contained in specific publications by the Bureau of Mines (2) and the Office of Surface Mining (*3*).

Blasting and pile driving are the two principal sources of ground motion and noise associated with construction. To a lesser extent the motions from equipment can be important but usually only at very small distances. The short duration of a blast event and single-pile-driving impact, as well as the relatively high frequencies associated with the resulting ground motion, restrict the types of appropriate instruments.

Significant construction blast motions last less than a second and, therefore, they must first be permanently recorded in order to be analyzed at a later date. Furthermore, the measuring and recording system may have to sense motion with frequencies as high as 500 Hz. However, typical blast motions involve dominant frequencies below 200 Hz and usually below 100 Hz. Because the natural frequency of a two-story house could be as low as 4 Hz, the system of measurement must be able to record accurately motions at frequencies as low as 3 Hz. Larger structures have still lower natural frequencies and may require special instrumentation.

Pile driving can be continuous (vibratory drivers) or intermittent (impact drivers), but generally the motions occur at frequencies between 10 and 30 Hz. The motion from any one impact may last less than $^1/_4$ sec. Thus, these motions are similar to a short, 20-Hz, blast-induced ground motion and can be monitored with the same instrumentation.

An idealized, field-portable blast-monitoring system operating on a 12-V battery is illustrated in Figure 1. It consists of transducers that convert physical motion or pressure to an electrical current, which is transmitted through cables to an amplifying system, and a tape, digital, or paper recorder that preserves the relative time variation of the original signal for eventual permanent "hard-copy" reproduction by a pen or light-beam galvanometric recorder or dot-matrix printer. As can be imagined, there is an almost endless variety of different configurations of these five basic components (see Figure 1). There are additional systems, which operate off 110-V alternating current, that transmit data via telephone or radio, as well as systems that involve computerized digital recording, storage, and reproduction of the signal. However, even these advanced systems still involve the five basic components, each of which will be described in the following sections.

TYPICAL BLAST VIBRATION RECORDS

A typical particle velocity time history at a surface coal mine is shown in Figure 2. The most important parameters that describe the time history are peak amplitude, principal period (1/principal frequency), and duration of the vibration. All of these parameters are dependent on the blast sequence and transmission medium. In normal blasting operations for tunneling, surface mining, and construction, these parameters vary in the ranges presented in Table 1 (*4*). In special cases, such as very close-in blasting, the particle velocities and frequencies are at the higher end of the range. Unfortunately, typical monitoring equipment is often not able to withstand the extreme environment of close-in blasting, and measurement of these motions requires special care with ordinary blast monitors.

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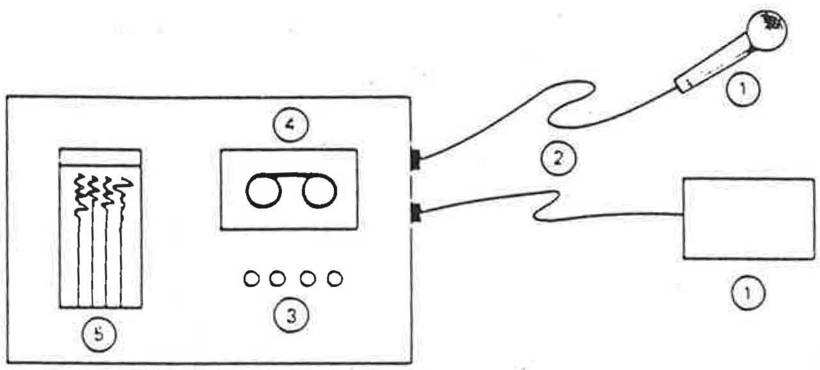


1 Velocity (3 orthogonal) and sound pressure transducers
2 Cables
3 Amplifier
4 Recorder (tape, disk or memory)
5 Light beam oscilloscope or dot matrix printer

FIGURE 1 Basic components of a generic, field-portable, vibration recording system.

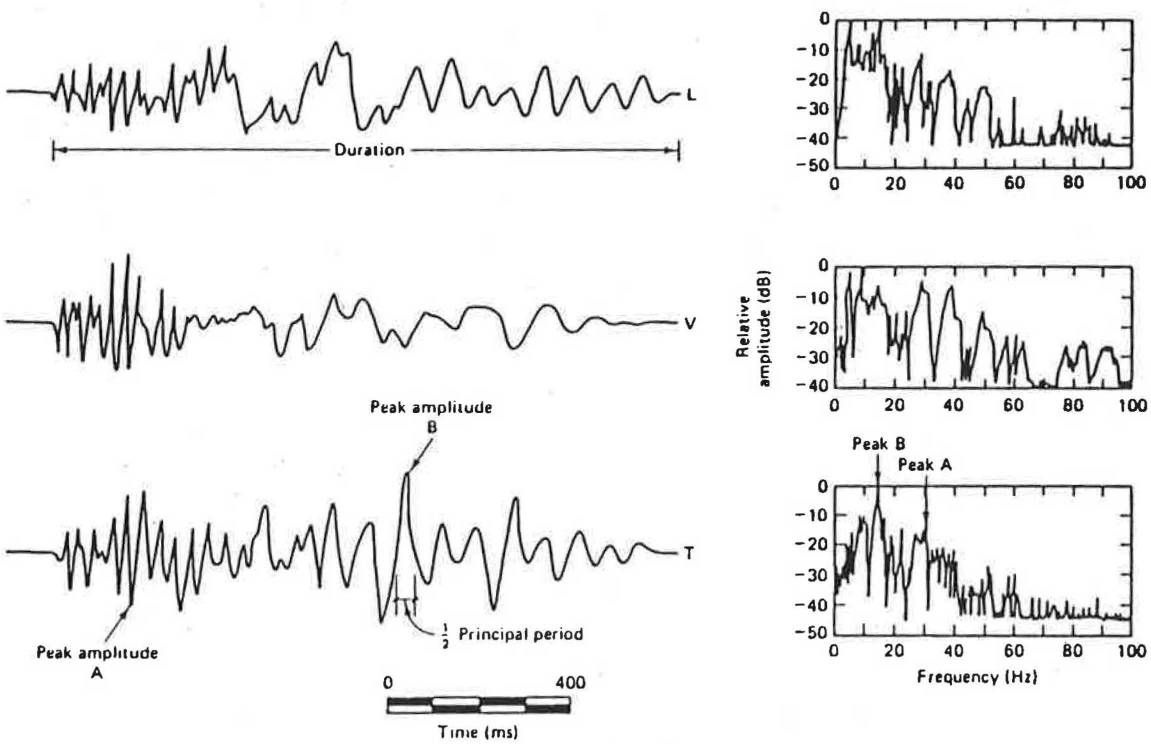


FIGURE 2 Basic characteristics of ground-motion time histories and comparison with Fourier frequency spectra: ground motions recorded near surface coal mining activity (*2*).

PRINCIPAL FREQUENCY

As shown in Table 1, the principal frequency of a blast vibration can vary between 0.5 and 200 Hz, but certain types of blasting tend to produce frequencies in a more limited range. When the principal frequency is defined as that associated with greatest amplitude pulse, as shown in the inset in Figure 3 (*5*), it varies by industry as shown. The relatively large explosions produced by surface coal mining, when measured at typically distant structures, tend to produce vibrations with lower principal frequencies than those of construction blasts. Construction blasts involve smaller explosions, but the typically small distances between a structure and a blast, as well as rock-to-rock transmission paths, tend to produce the highest frequencies. Such high-frequency motions associated with construction blasts are advantageous in reducing the potential for cracking in adjacent structures (*6*).

Principal frequencies also depend on the transmission medium. As will be shown later, high-frequency motions tend to be filtered out or attenuated over shorter distances in soil than in rock. Furthermore, stiff layers can propagate certain frequencies further because of waveguide effects. For instance, it can

TABLE 1 RANGE OF TYPICAL BLAST PARAMETERS (*4*)

Blasting Operation	Parameters
Displacement	10^{-4} to 10 mm
Particle velocity	10^{-4} to 10^3 mm/s
Particle acceleration	10 to 10^5 mm/s^2
Pulse duration	0.5 to 2 s
Wavelength	30.0 to 1500 m
Frequency	0.5 to 200 Hz
Strain	3.0 to 5,000 μin./in.

be shown that shear waves will resonate within a soil layer (or rock layer for that matter) at a frequency described as

$$f = \frac{c_s}{4H}$$

where c_s is the propagation velocity of the shear wave, and H is the thickness of the layer. As a result of these considerations, at typical distances the principal frequency ranges from 1 to 50 Hz when measured on soil profiles with thicknesses greater than 2 to 3 m and 10 to 200 Hz on rock.

Use of frequency-based safe blasting criteria has caused many to ask, "What is the correct frequency for use with the criteria?" The answer, of course, depends on the record, and the vibration records fall into three categories. The easiest is the record with one dominant frequency. For instance, in Figure 4, the peaks of the ground motion generated by the quarry blast occur at approximately 20 Hz. This dominant frequency can be determined through the hand calculation shown in the inset in Figure 3 or by reference to the Fourier frequency spectra in Figure 4. The hand calculation is more direct and much easier than calculating the Fourier frequency spectrum.

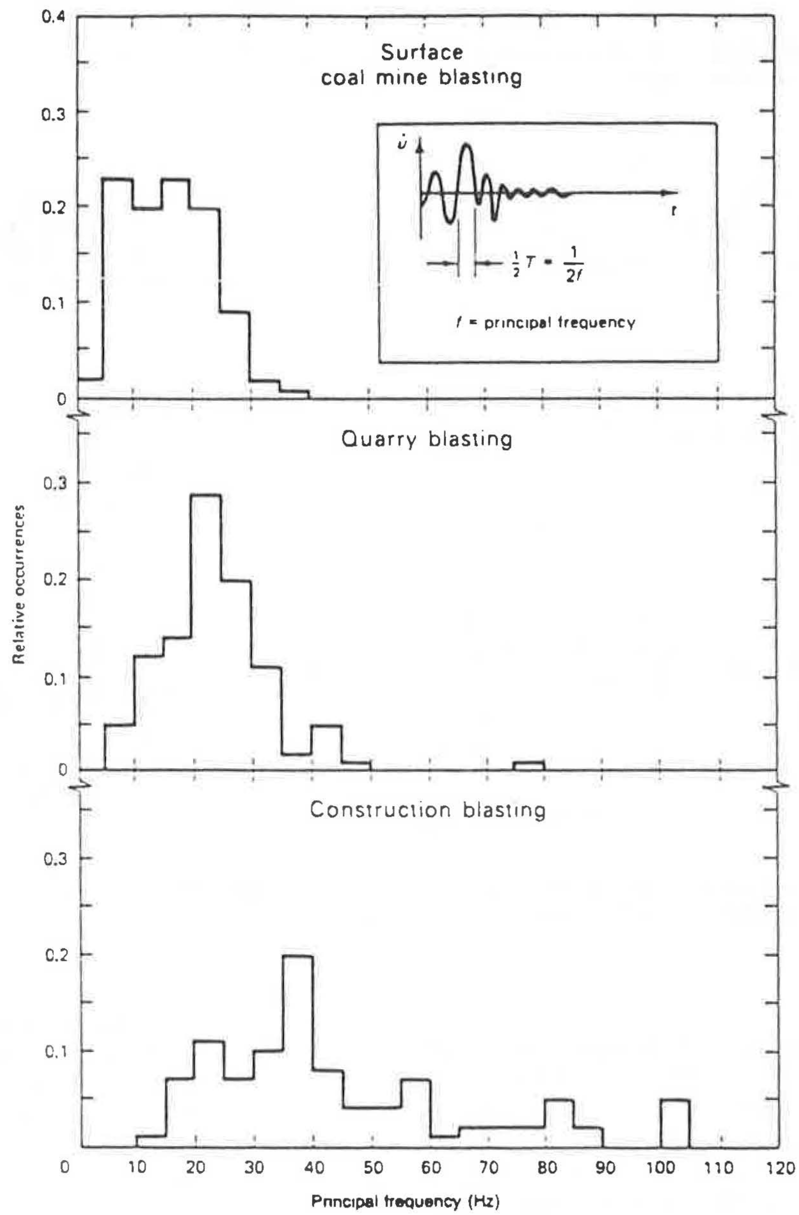


FIGURE 3 Comparison of predominant frequency histograms at structures of concern for mining and construction, which shows construction frequencies to be higher (*5*).

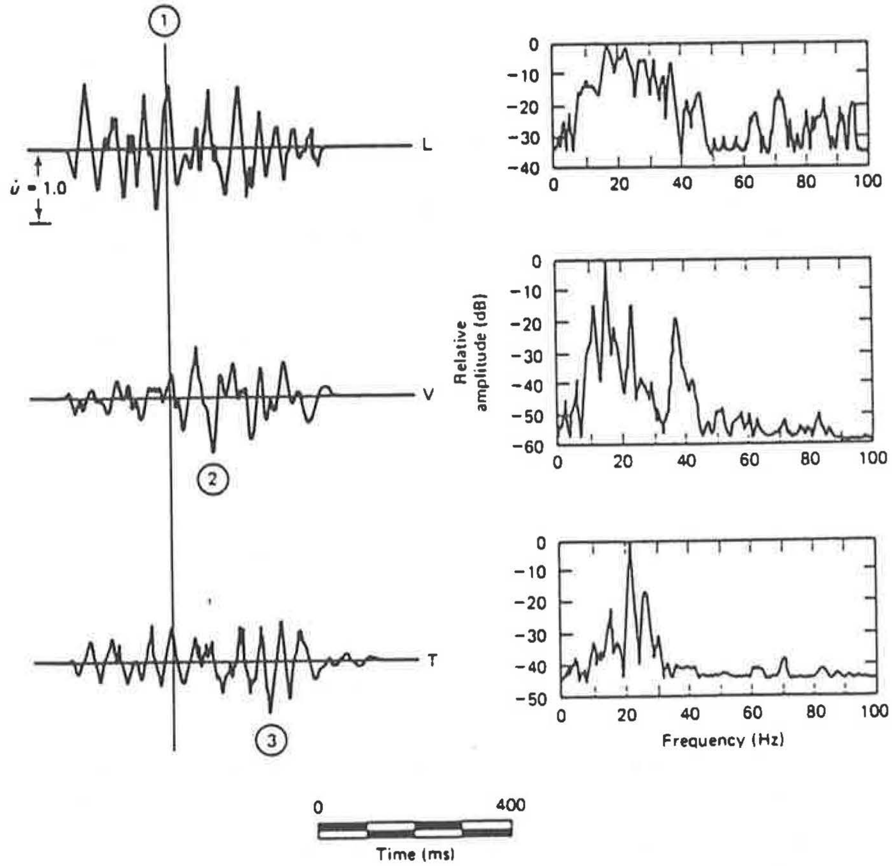


FIGURE 4 Quarry blast vibration time histories with frequency contents similar to those from pile driving (*2*).

A more difficult type of record to interpret is that which contains unequal peaks at two dominant frequencies, such as that in Figure 2. The two dominant frequencies are the initial 30- to 40-Hz portion (Peak A) and the later 15- to 20-Hz portion (Peak B). In this case, the low-frequency portion of the record dominates because it contains the largest peak.

The most difficult type of record to interpret is that which has equal peaks at differing frequencies. No hand calculation will replace either a response spectrum or Fourier frequency analysis. The response spectrum is preferred because it can be related to structural strains; its calculation is described by Dowding (*6*) and in most soil dynamics and structural dynamics textbooks.

TRANSDUCER TERMINOLOGY

Transducers are one of the weaker links in the measurement system because they must translate mechanical motions to electrical signals. The remaining components transform electrical signals or light beams and are not restricted by mechanical displacement. The main characteristics of transducers that affect their performance are sensitivity (axis and cross-axis) and frequency response.

Sensitivity of an instrument is the ratio of its electrical output to its mechanical displacement, velocity, or acceleration for energy-converting transducers (i.e., they do not require an energy source). Since allowable limits are specified in terms of ground particle velocity, all blast monitors come equipped with velocity gauges, most of which have sensitivity of 740 mV/mm/s (or 1 V/in/s) particle velocity.

Frequency response is the frequency range over which the electrical output is constant with a constant mechanical motion. This constancy is normally expressed in terms of decibels (dB). For instance, linear within 3 dB between 5 and 200 Hz means that the transducer produces a voltage output that is constant within 30 percent between 5 and 200 Hz. Generally, it is better to look at the transducer's response spectrum (such as those shown in Figure 5) to determine the frequencies in which this difference occurs. For example, the difference occurs at low frequencies for the velocity transducers in the figure.

TRANSDUCER ATTACHMENT

One of the most critical aspects of vibration monitoring is the mounting of the transducers in the field. The importance of mounting is a function of the particle acceleration of the wave train being monitored. The type of mounting is the least critical when the maximum particle accelerations are less than 0.3 g. In this range, the possibilities of rocking the transducer or the transducer package are small, and the transducer may be placed on a horizontal measurement surface without a device to supply a holding force. When the maximum particle accelerations fall between 0.3 g and 1.0 g, the transducer or transducer package should be buried completely when the measurement surface consists of soil (*7*). When the measurement surface consists of rock, asphalt, or concrete, the transducers should be fastened to

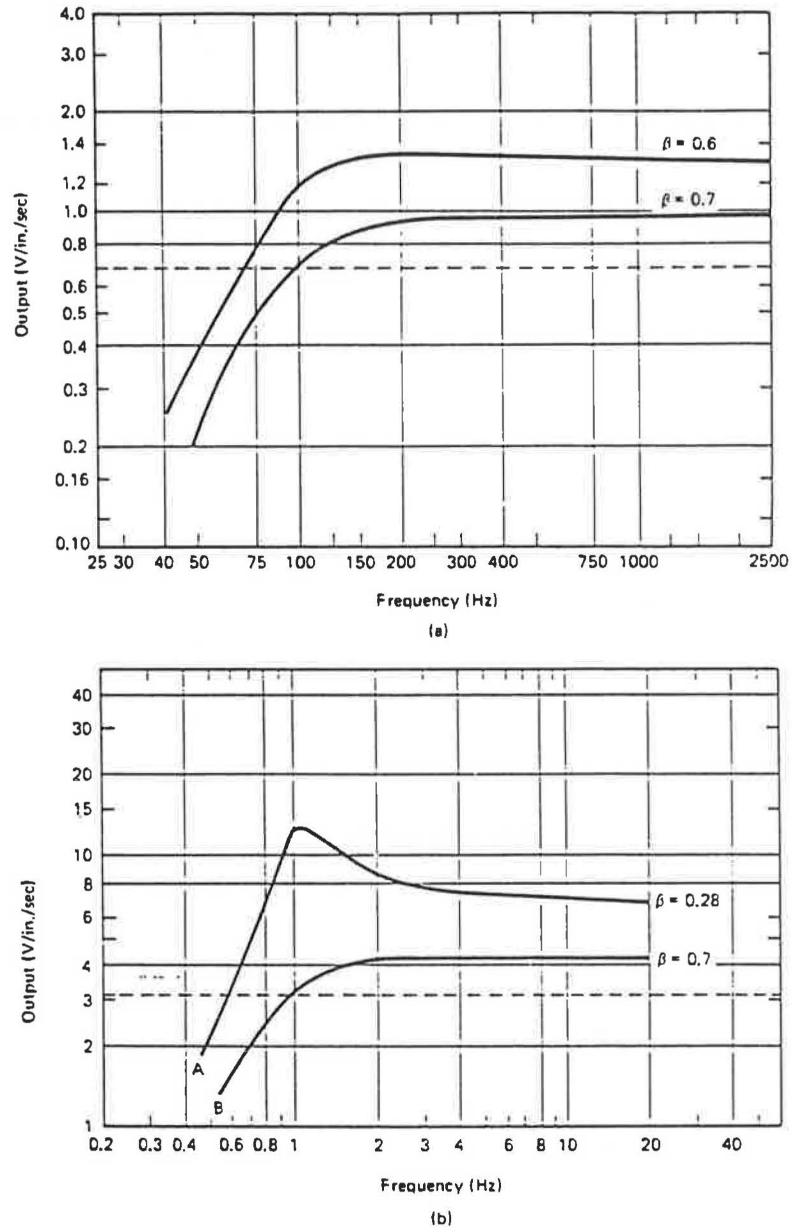


FIGURE 5 Frequency response of 70 percent critically damped (β = 0.7) velocity transducers related to the –3 dB criterion: (a) –3 dB at 100 Hz; (b) –3 dB at 0.9 Hz.

the measurement surface with double-sided tape, epoxy, or quick-setting cement (the authors found Hydrocal to be a good, quick-setting cement). If these methods are unsatisfactory or accelerations exceed 1.0 g, only cement or bolts are sufficient to hold the transducer to a hard surface.

VELOCITY TRANSDUCERS

There are relatively few velocity transducers on the market compared with accelerometers and strain gauges. In most industries, velocity is normally found through integration of acceleration time histories. Even though the velocity transducers are a minority, they are the principal type of transducer used for blast monitoring because allowable vibration limits are specified in terms of particle velocity.

The components of the most common seismic transducer, the geophone, are shown in Figure 6 (*8*). The output from a velocity transducer is generated by a coil moving through a magnetic field. The voltage induced in the coil is directly proportional to the relative velocity between the coil and the magnetic field. Either the coil or the magnet can be part of the vibrating mass, with the other component attached to the transducer frame.

The velocity transducer is convenient for field use because its output is usually high enough (in the range 4 to 8 mV/mm/s) so that amplification is not required. Furthermore, its output impedance is easily matched by other standard components.

Velocity transducer response becomes nonlinear at low frequencies, as shown in Figure 5. Typically, the transducer frequency range is broadened by many manufacturers of blast

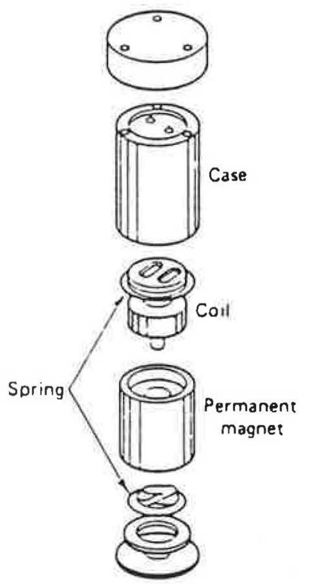


FIGURE 6 Schematic diagram of the components of a velocity transducer (*8*).

monitoring systems with a low-frequency amplifier (2), as shown in Figure 7. In this case the amplifier and the velocity transducer must be matched closely to provide the correct compensation in the desired frequency range.

TAPE, DIGITAL, AND HARD-COPY RECORDERS

Of those blast monitoring systems with tape recorders, most use compact FM cassettes. Many of the systems involve separate record and reproduction modules to reduce the complexity of recording modules. Care should be exercised to determine the exact details of the system before purchasing, as tape recorder performance varies at low temperature.

Digital recording systems now dominate technical recording because of the ease of computer linkage. The signal is sampled at a certain rate, say, 1,000 times per sec, and each sample is converted to a single magnitude. This magnitude and its associated time are then stored in computer memory. Such a system has several advantages. It is very accurate because variation in the speed of the tape, if it is used, has no effect, and the system can be directly accessed by a computer. Details of the digitization process are discussed in later sections.

A permanent record or "hard copy" of the vibration time history is usually made on photographic film or paper. Almost all present film-based recorders use special field-developable, ultraviolet-light-sensitive paper in combination with light beam galvanometers to record high-frequency motions. The newest generation recorder uses dot-matrix printers with microcomputers. Unfortunately, those monitors that print after a vibration event may not be recording another while printing. If multiple shots are likely, this reset time should be determined. Furthermore, printer behavior in cold weather is variable and should also be investigated. Paper hard-copy recorders that use a mechanical lever system to move a pen or a heated stylus over the advancing paper, while reasonably priced, cannot faithfully record large amplitude signals at frequencies above 25 Hz. This restriction greatly diminishes their usefulness for analyzing blast vibrations, whose dominant frequencies often exceed 50 Hz.

Most recorders can be bought as either single-channel or multichannel units. A four-channel unit is necessary in blast monitoring to simultaneously record the three components of the ground motion (longitudinal, vertical, and transverse) and the air blast. The present trend in vibration equipment is to include a signal-conditioning amplifier in the recorder to allow flexible amplification of the signals.

CALIBRATION

It is obvious that the entire vibration measurement system should be calibrated, because it is futile to record data if they cannot be exploited because of a lack of reference. Manufacturers supply calibration curves with their instruments that are similar to the response spectra for transducers shown in Figure 5. Recalibration or checking requires special vibrating platforms where frequency and displacement are controlled and, in the field, a calibrating circuit to pulse the magnetic core of the geophone.

BLAST MONITORS

Blast monitors are complete instrumentation systems like the one shown in Figure 1 and consist of transducers, amplifiers, power supply, and recorder. The user merely plugs the trans-

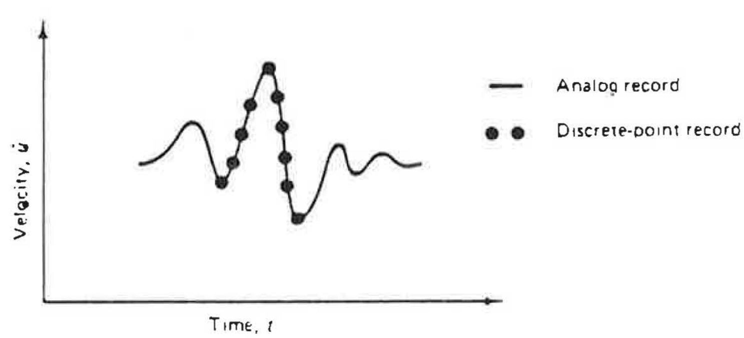


FIGURE 7 Comparison of digitized (or discrete) points and analog (or continuous) records.

ducer cables into the recorder box. Because of the ease of operating these systems and their typically robust design, they should be used in all but the most sophisticated monitoring situations.

Listed in Table 2 are the characteristics of a variety of blast monitors to show the evolution in design and record output. The examples are real equipment without manufacturers' names, to focus discussion on the technical aspects of the equipment. Technical details on specific systems are contained in the Bureau of Mines and Office of Surface Mining comparisons (*2, 3*). The evolution in function has been controlled by the availability of portable equipment. The field-portable galvanometer system and field-developable light-sensitive film enabled analog System 2 to provide four channels of time-history information immediately after the event rather than the peak values given by System 1. The cassette FM recorder allowed analog System 3 to provide the time histories without the human operator triggering required for analog System 2.

The advent of digital electronics and field-portable computers allowed the development of Systems 4 and 5. System 4 is essentially the digital version of self-triggering System 3 with the same permanent photographic paper record as System 2. System 5 links the immediate digitization of field data to central processing and recordkeeping via a telephone line. Already there is a System 6, which is a combination of Systems 4 and 5 with a laptop computer for sophisticated field analysis. Amazing as it seems, the computerized systems cost no more than their noncomputerized counterparts.

Probably the biggest labor-saving (and thus cost-saving) development has been the perfection of inexpensive self-triggering systems (Systems 3, 4, and 5). An operator is no longer required to turn on the equipment just before the blast or pile driving. This also means, of course, that no one can detonate blasts in violation of ordinances by blasting when the seismograph operator is not present.

Frequency analysis of records requires a time history. Unfortunately, frequency analysis with the early self-triggering devices (i.e., System 3) requires that the tape be replayed through equipment costing more than the monitor or that it be sent away for interpretation. Sending records through the mail results in a delay interpretation of 5 days, and sometimes up to 1 month. Systems 4 and 5 allow immediate interpretation of frequency without additional costly equipment.

DIGITIZATION OF TIME HISTORIES

Computerized processing of blast vibrations requires that time histories first be digitized. Digitization is the process whereby the continuous analog signal shown in Figure 7 is transformed into a series of velocity and time coordinates (the dots in the figure) that describe discrete points along the curve. Digital computers can only process such discrete points or digitized data.

Paper or hard-copy records are digitized by hand with the help of the graphics tablet, which translates the location of cross hairs into velocity and time similar to the analog/digital (A/D) converter coordinates. FM tape records or direct transducer output, on the other hand, can be digitized with little human interaction by an A/D converter that samples and records the tape's output voltage at constant time intervals of, say, $^{1}/_{1000}$ of a sec.

TABLE 2 BLAST MONITORS: HISTORICAL PERSPECTIVE

	System					
Parameter	1: Analog Storage	2: Analog Photo Paper	3: Analog Self-Trigger	4: Digital Self-Trigger	5: Digital Self-Trigger Tele-Comm	6: Future
System						
Number of channels	1 or 3	4	4	4	4	
Frequency range (Hz)	5–200	2–200	1–200	4–150[a]	5–200	
Sensitivity (mm/s/mm)	0.6–2.4	0.2–20	NA	1–10	NA	
Transducer						
Linearity (Hz)	−3 dB at 5	−3 dB at 1.8	−3 dB at v	−3 dB at 5	−3 dB at 5	
Natural frequency (Hz)	4.5	2	10	4	4.5	
Damping (% of critical)	60	60	60	60	60	
Recorder						
Record type	Paper peak	Light-sensitive	FM	Light-sensitive	Dot-matrix	
Speed of recording (mm/s)	$^{1}/_{60}$	100	47 (tape)	100	Variable	
Physical						
Weight (kg)	5	26	13	13	18	
Dimension (cm)	12 × 28 × 14	–	38 × 32 × 20	32 × 53 × 16	45 × 45 × 25	
Power supply	ac, 30 days; 12 V dc. 12 days	12 V dc	12 V dc	12 V dc	110 V ac, 12 V dc	
Digitizing						
Rate (samples/s)	–	–	–	~500	250–1,000	
Resolution	–	–	–	1/125[b]	1/2,000[b]	

NOTE: NA = not available.
[a]Limited by digitizing rate.
[b]Fraction of the dynamic range.

It is useful at this point to define proper sampling or digitization density. This discussion is a simplification of the concepts presented by Hudson (*9*) in his in-depth treatment of reading earthquake accelerograms. There are two basic requirements. First and most obviously, the peak velocity must be sampled and, second, the time between samples or data points (dots) must be small enough to allow proper calculation of single-degree-of-freedom response for systems with high natural frequencies (i.e., greater than 40 Hz) (*6*).

The difficulty of sampling the peak velocity depends on the source of the velocity time history. If the source is a paper record, as shown in Figure 8, five points per cycle chosen by a human at a digitizing pad will adequately define the pulse. If the source is an FM tape or voltage output of the transient event itself, the peak is not seen by an operator and the samples are obtained regardless of peak location. Thus the sampling must be rapid enough (approximately 7 to 10 points per period, T, in Figure 9) so that a sample is assured near the peak (error less than 10 and 5 percent, respectively). For a 100-Hz vibration, proper digitization corresponds to a rate of 7 to 10 samples per cycle times 100 cycles per sec, or 700 to 1,000 samples per sec. Table 3 contains the percentage of error possible with differing rates of sampling.

The rate of digitization has an important impact on blast monitoring equipment. Consider System 4 in Table 2. Even though its transducer response is linear past 200 Hz, the system has a 30 percent or 3-dB error at 150 Hz because data are digitized at only 500 samples per sec. Table 3 shows the maximum errors associated with various rates of sampling. For System 4 the number of samples per period of a 150-Hz signal is the sampling rate (500 per sec) multiplied by the period, $T = 1/f$:

$$500\left(\frac{1}{150}\right) = 3.3$$

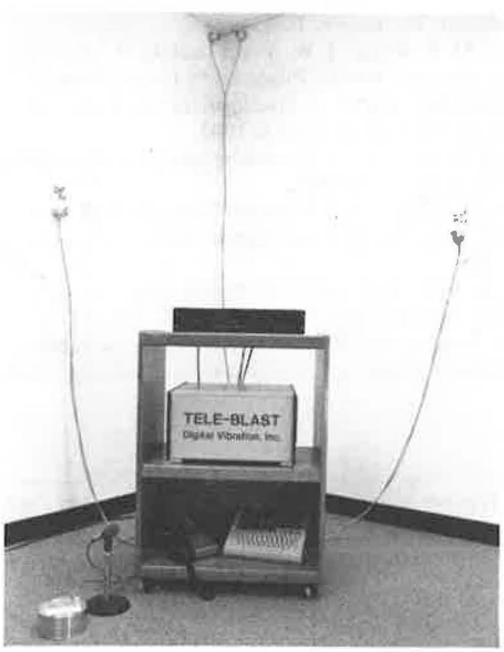
When the sample rate drops below 4 per period, the error increases above 30 percent or 3 dB.

COMPUTERIZED SURVEILLANCE

Linkage of computerized vibration monitoring by phone with a system such as the TELE-BLAST shown in Figure 8 has a number of advantages. The most important in terms of cost is the elimination of on-site technical support. Once several vibration monitors are installed around a construction site, all vibration events will be automatically recorded. Details of these events can be accessed from any phone to obtain either an immediate telephone report (a summary of the event times, peak motions, and dominant frequencies) or time history to be processed by the central computer. The central computer automatically polls all instruments once a day and produces a monthly summary of activity.

This TELE-BLAST system has been used by Digital Vibration, Inc. for performance monitoring of a wide variety of blasting projects. A 14-channel version has been combined with crack width monitoring of a test house to compare weather- and blast-induced crack movements. Presently the system is monitoring construction adjacent to a historic structure in New York City to allow analysis of response in Chicago. Six monitors are being placed above a long-wall mining site to locate vibration sources.

All of this monitoring is conducted automatically via phone without any vibration specialists in the field. Repair work such as board replacement can be undertaken by nontechnical personnel with guidance via the telephone line. Obviously



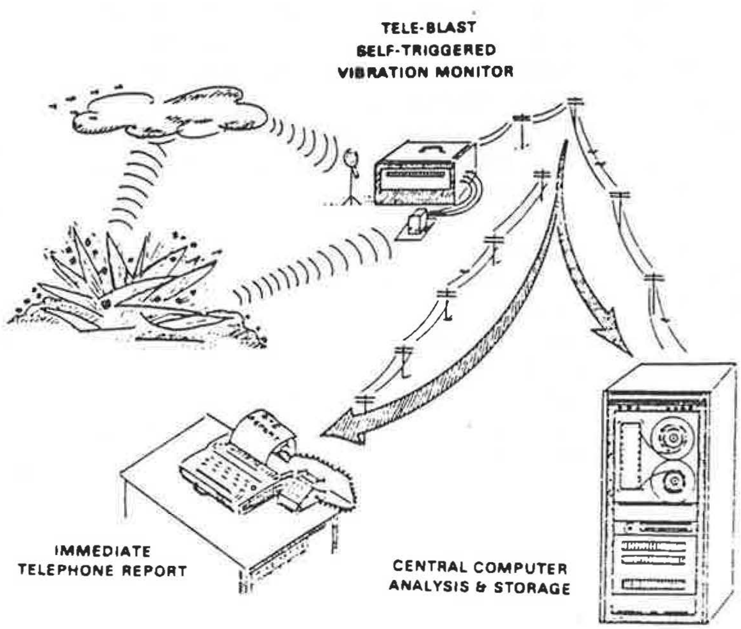


FIGURE 8 TELE-BLAST system continuously monitors wall response and allows instantaneous on-site and remote analysis of data. TELE-BLAST instrument showing computer (middle shelf), teletype access (lower shelf), and transducers for response (on wall) and excitation (on floor) (left); TELE-BLAST operation allows access from any telephone for immediate reporting or detailed analysis (right).

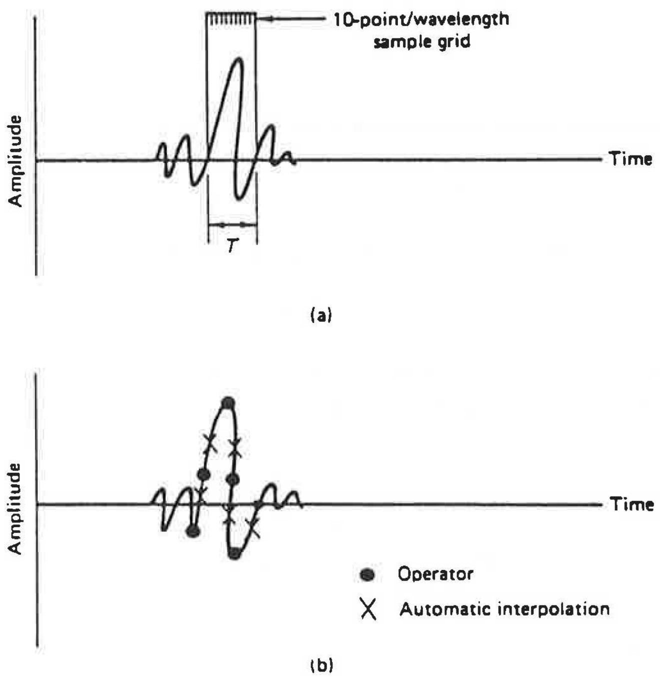


FIGURE 9 Comparison of sample density with system response: (a) FM tape sampling (random); (b) digitization pad sampling (interactive).

future miniaturization of computer equipment, linkage to cellular phones, and so on will allow for even more efficient monitoring of construction vibrations.

CONCLUSIONS

This paper has explored the general nature of construction blast-induced vibrations in order to define the vibration environment to be monitored. This environment determines the requirements of the instrumentation needed for efficient monitoring. Characteristics of these instruments were outlined, and the historical development of self-contained vibration monitors was traced up to the present computerized telecommunicating systems. Within the framework of this discussion the following conclusions are tentatively advanced:

- Construction blasts tend to produce relatively high-frequency vibrations, and therefore, lower-frequency mining-based criteria may not be strictly applicable.
- Frequency-based vibration amplitude criteria will replace single-amplitude criteria.
- Blast and pile-driving vibration have the same frequency character and can be monitored with the same instruments.
- Digital computer equipment will come to dominate vibration monitoring.

TABLE 3 SAMPLE RATE ERRORS

No. Samples per Period	SI[a] (degrees)	*A* (90 degrees − ½ SI[a])	sin *A*	Error (1 − sin *A*) (%)
2	180	0	0.0	100
4	90	45	0.707	29
6	60	30	0.87	13
7	51	65	0.91	9
8	45	68	0.93	7
9	40	70	0.94	6
10	36	72	0.95	5

NOTE: Sample rate errors assume a sinusoidal wave form.
[a]SI = sample interval.

- Accuracy of digital units is a function of the digitizing rate and number of bits of accuracy in analog-to-digital conversion, wherein sampling rates of at least 500 samples per sec are necessary to monitor blasting vibrations.
- Phone linkage of self-triggering, computerized vibration monitors allows labor-efficient monitoring.

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Publication of this paper sponsored by Committee on Soils and Rock Instrumentation.

Instrumentation for Tests of Piles Subjected to Axial Loading

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Load tests of piles under axial loading are performed for two purposes: to prove a particular design and to gain information to allow a redesign. Instrumentation needed for evaluation of proof loading can be placed above ground. To obtain information for redesign, data must be obtained on the manner in which load is transferred in both end bearing and skin friction; therefore, instrumentation must be placed along the length of the pile. The importance of providing the appropriate instrumentation for proof testing is obvious, and failure to do so can lead to serious consequences. However, the far more challenging problem and the potentially more significant one is to select the proper instrumentation for determining the load transfer in skin friction and end bearing as discussed herein. Significant contributions are being made in the development of innovative techniques for these measurements. Recent advances in modeling show the value of determining the unit load transfer, f_z, as a function of axial pile movement, w_z, at points along a pile and unit end bearing, q_B, as a function of tip movement, w_B. The design of piles under axial loading in the near future will be based on predicted f_z–w_z and q_B–w_B curves. Investigation of the integrity and deformation behavior of axially loaded piles by use of dynamic tests (stress wave measurements) is gaining credence. Instrumentation for such testing can be rather complex, but essentially all instrumentation is composed of off-the-shelf items that are readily available. Generally, testing is performed at very small strains. However, the restriking of driven piles, or the use of special equipment to impact cast-in-place piles, is becoming more common, although instrumentation, such as the pile-driving analyzer, for such applications is more custom-made.

Two types of tests are run to obtain the behavior of piles subjected to axial loading: a proof test to learn if the design that was preselected is satisfactory and a test to obtain information for redesign. The instrumentation for the first of these tests is specified by the standards of the American Society for Testing and Materials (*1*) and is not discussed further. The instrumentation for the second type of test is nonstandard, is of considerable importance, and is discussed briefly in this paper. In addition, instrumentation and interpretation of stress wave tests (dynamic testing) to evaluate pile integrity and deformation behavior are presented.

An analytical model for the pile-soil system must be selected before instrumentation for load testing is selected. Therefore, a brief description of several analytical models is first presented. The variety of instrumentation that is available for load testing is then discussed. Finally, a discussion of the interpretation methods of the test results and the benefits of performing instrumented tests is presented.

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The discussion on static load testing is followed by a discussion of instrumentation for integrity testing. As with static load testing of piles, the selection of instrumentation for integrity testing depends on the analytical model used to analyze the results. Methods based on stress wave propagation are most often employed in integrity testing. Therefore, these methods are discussed, and the associated instrumentation and analyses are presented.

MODELS FOR AN AXIALLY LOADED PILE

The pile and the supporting soil can be modeled as a continuum by use of the finite element method, or the models that are shown in the following paragraphs may be used. Selection of a model before selecting instrumentation and performing field tests is essential. The engineer should make use of information on soil properties, pile geometry, construction procedures, instrumentation, and loading arrangement to perform preliminary analyses of the expected results of the load-test program. The accuracy, sensitivity, and ruggedness of the instrumentation can be judged and modifications made if necessary. Selection of appropriate models for both dynamic and static loading and the use of the models in preliminary analyses are essential before making a final decision on instrumentation.

The well-known model of a pile under dynamic loading (pile driving) by Smith (*2*) is shown in Figure 1. The pile is modeled by a series of masses separated by linear springs. The soil is modeled by a series of nonlinear springs and dashpots. The energy of a falling weight is used, along with the nature of the appurtenances at the top of the pile, to obtain a time-dependent force. A differential equation is then solved in difference form to obtain the motion of each of the masses and the downward movement of the top of the pile. One aspect of this approach that continues to be improved is the mechanism used to describe nonlinear soil behavior and its relationship to measurable soil parameters.

Full instrumentation of a dynamically loaded pile is complex, and the usual procedure to date is to use a restricted amount of instrumentation near the top of the pile. This instrumentation allows a simplified version of the wave equation to be used to obtain the capacity of a pile, or to determine the integrity of a concrete pile, as discussed later.

If a static load is applied, the masses and dashpots in Figure 1 are of no consequence, and the pile can be modeled as shown in Figure 2 (*3*). The pile is simply replaced by a spring whose stiffness can vary from point to point along the pile. The soil is

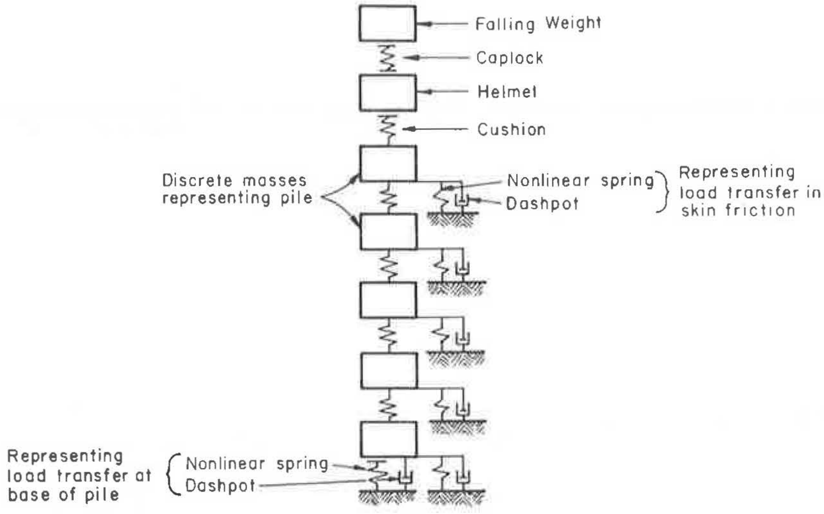

FIGURE 1 Smith model for pile driving (2).

replaced by a nonlinear spring and a friction block. The family of curves at the right of the figure depict a typical set of load-transfer curves for the soil.

The potential value of load-transfer curves in Figure 2 is easy to understand. The engineer can assume a downward movement, w_B, of the tip of the pile and use the curve to obtain the unit end-bearing, q_B, and, hence, obtain the load at the tip. A length of the pile above the tip can be selected, the downward movement, w_z, of the increment can be estimated, the unit load transfer in side resistance, f_z, can be obtained by using the appropriate curve, and the load taken in side resistance by the selected increment can be computed. The shortening of the selected increment can then be computed, which allows the downward movement, w_z, of the increment to be found. The procedure can be repeated until convergence is achieved for the selected tip movement. Successive increments of the pile can then be analyzed until the top is reached. The analysis will reveal a top load and a top movement. Other tip

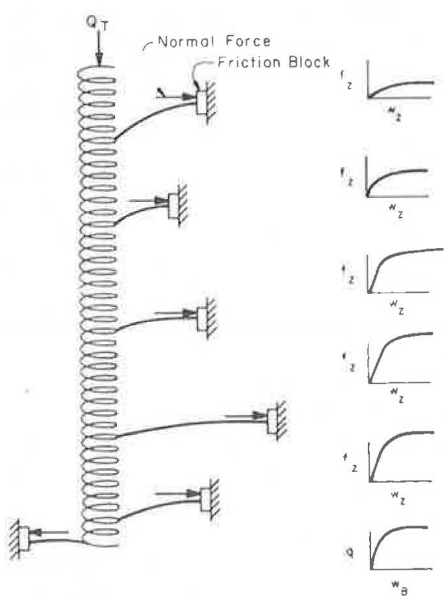

FIGURE 2 Model of pile for static loading (3).

movements can be selected, the procedure repeated, and the load-settlement curve for the pile can be obtained analytically.

Thus, if a sufficient number of tests of instrumented piles can be performed, the development of methods of predicting the load-transfer functions is possible. The ability to predict such functions with reasonable accuracy is an enormous benefit to the geotechnical engineer. Furthermore, the selection of appropriate instrumentation to obtain the load in a pile as a function of length is facilitated even if the load-transfer functions can only be approximately predicted.

Some analysts have criticized the models shown in Figures 1 and 2 because the soil is not shown as a continuum. However, as more information becomes available on load transfer, the curves can be depicted if necessary as multivalued to reflect the continuum effect.

INSTRUMENTATION FOR STATIC LOADING

The requirement of the instrumentation is to produce data showing load versus settlement for the top of the pile and a family of curves showing the distribution of load as a function of depth. It is the latter role of the instruments that is of particular interest in this paper.

The simplest form of instrumentation that will produce the load-distribution curves is the telltale or unstrained rod (4). The rods are inserted into tubes that extend to a series of depths along the length of the pile. The pile will shorten under an axial load and the rods will not. The amount of deformation of points along the pile can be determined by dial indicators, and a plot of the deformation as a function of depth can be made. The strain in the pile can be found from the slope of the plot, and the internal load can be found by multiplying the strain by the pile stiffness. Telltales perform well if there is a sufficient number and if the relative stiffness between pile and soil is such that there is an appropriate amount of strain in the pile.

Another method that will yield the deformation from point to point along a pile is shown in Figure 3. A French agency developed an ingenious system to be placed into a metal tube that is built into the pile (5). The system consists of a series of air-operated clamps that are connected by metal strips on which

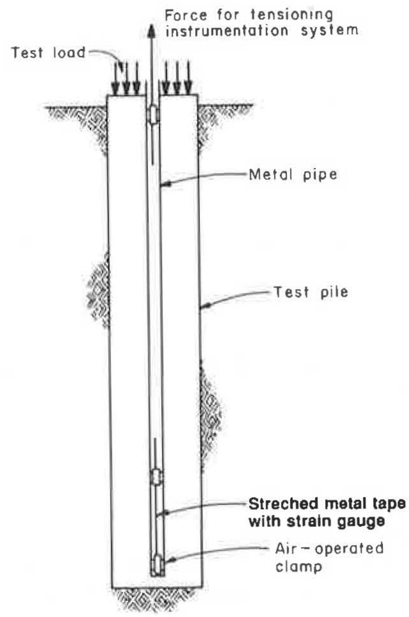

FIGURE 3 French system for instrumenting a pile for obtaining load as a function of depth.

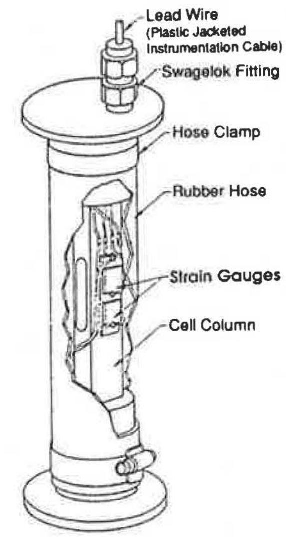

FIGURE 4 The Mustran cell.

electrical resistance strain gauges are fixed. The assembly is lowered to the bottom of the pipe after the pile has been installed, the lower clamp is actuated to fasten the assembly, a tensile force is applied to the top of the assembly to put all of the strain gauges in tension, and then the other clamps are actuated. As load is applied to the top of the pile, the strain in the gauges is reduced and the deformation can be obtained by use of a calibration curve. After the load test is completed, the air-operated clamps are released and the assembly is recovered and available for use another time. Interpretation of the data proceeds as with the use of telltales.

A load cell can be constructed for the bottom of a drilled shaft and installed before the placement of the fluid concrete. The cell consists of two stiff plates that are separated by remote-reading, load-measuring devices so that the load transmitted to the tip of the drilled shaft can be measured directly (*6*). Osterberg (*7*) has described a similar device in which the plates are separated by hydraulic fluid. This system allows fluid to be pumped into the load cell until either the end load or the side resistance is exceeded, whichever is smaller. The separation of the plates can be found from the volume of fluid that is pumped. The measurement of the movement of the bottom of the shaft by use of a telltale and the movement of the top by use of a dial gauge allow the plotting of a load-settlement curve for the tip and a load-uplift curve. Only one of the curves will be fully developed, of course. A similar system was described by Lizzi (*8*). Installation of the bottom-hole load cell should be planned carefully so that construction of the drilled shaft is not seriously delayed.

The basic device for use in obtaining the internal load in a pile is the strain gauge. Two types are in common use: the vibrating-wire and the electrical-resistance. The gauges can be fastened to the interior of a steel pipe, put inside a small channel or angle that is welded to a steel H-pile, fastened to a rebar in a precast or cast-in-place concrete pile, or used to construct a special cell such as shown in Figure 4 (*9*). The Mustran cell is constructed with electrical-resistance strain gauges and can be installed in precast concrete piles or fastened to the rebar cage of a drilled shaft. A serious problem with strain gauges is that they must be kept perfectly dry; only a few molecules of moisture will result in electrical leakage with a loss of accuracy or a complete failure. To combat the moisture problem, dry nitrogen, a desiccant, is forced through the sheath on the electrical cable leading to the Mustran cell and pressurizes the interior of the cell, thereby preventing the intrusion of moisture. Hundreds of Mustran cells have been built for use with load tests.

The strain in a pile can be found with Mustran cells, and the load can be obtained simply by multiplying the strain by the axial stiffness of the pile. The preferred procedure to obtain the value of pile stiffness is to establish a level of Mustran cells at or near the ground surface. If care is taken to ensure that the stiffness of the pile is constant with depth, or nearly so, the top level of instruments can serve as a calibration level. A curve can be developed that shows load in the pile as a function of the Mustran-cell reading. That curve can then be used to obtain the internal load at all other measurement levels that are below the ground surface.

In all of this testing, the modulus of the pile, E, is quite important in evaluating the load-test results. The modulus of steel piles can be considered quite constant and is well-known. Concrete piles and drilled shafts, on the other hand, can exhibit variability in the modulus with load level and from site to site. It is helpful, therefore, to evaluate the modulus of the pile being load tested.

The most direct way of evaluating E for a test pile is to install instrumentation in the unembedded length. This typically requires on the order of 5 to 10 ft of length for telltales or Mustran cells. Another method is to use the speed of compression waves to evaluate the small-strain modulus, E_{max}, of the concrete as discussed in the section on integrity testing. The modulus of concrete is, however, relatively nonlinear at relatively high concrete stresses. Schmertmann (*10*) recommends that the ratio of E_{static} to $E_{dynamic}$ ($E_{dynamic} = E_{max}$) on the order of 0.84 be used in the absence of better data. He also recommends that this ratio not be used above from 0.33 to $0.5 f_c'$ (the unconfined compressive strength). If larger stresses are applied during the load test, other means may have to be used to account for possible significant nonlinear behavior in E, not only during the test but also along the length of the pile under a given high load.

RESULTS FROM STATIC LOADING OF AN INSTRUMENTED PILE

A set of load-distribution curves that were obtained by the use of Mustran cells is shown in Figure 5 (*11*). This test was performed on a drilled shaft that was about 100 ft in length. Curves of this type are useful even before being interpreted to obtain detailed information on load transfer. The curves show that virtually all of the early loads were carried in skin friction, that virtually no load was removed from the pile by the top 10 ft of soil, and that the maximum load transfer in skin friction occurred at about the midheight of the drilled shaft. The last curve shows that the applied load was about 1,800 kips, that about 250 kips were carried by the tip of the drilled shaft, and that about 1,550 kips were carried in skin friction.

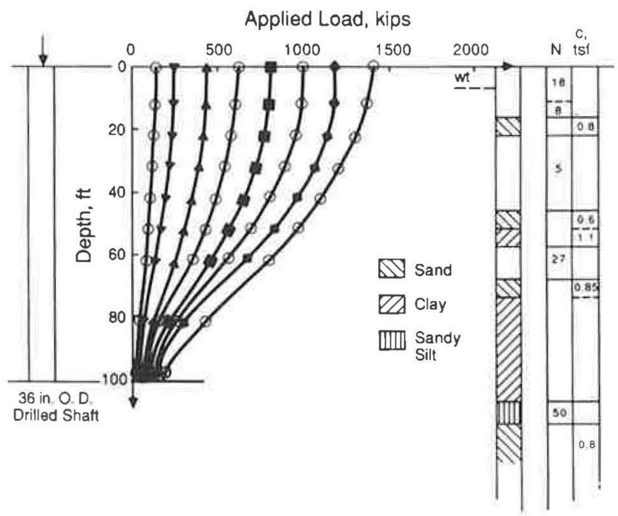

FIGURE 5 Typical set of load-distribution curves for a drilled shaft obtained from use of Mustran cells (*11*).

A detailed analysis can be made of the curves shown in Figure 5 to obtain information on transfer of load as a function of movement of the pile. The unit load transfer in skin friction, f_z, for one of the given curves in Figure 5 can be found for a particular depth, z, by using the following equation:

$$f_z = \frac{1}{C_z}\frac{dQ_{nz}}{dz} \qquad (1)$$

where C_z is the circumference or perimeter of pile at depth z and Q_{nz} is the load in the pile at depth z for nth load. The corresponding downward movement, w_z, of the pile with respect to its initial position can be found by using the following equation:

$$w_z = \delta_n - \int_0^z \frac{Q_n dz}{A_z E_z} \qquad (2)$$

where

δ_n = settlement of the top of the pile for the nth load,
A_z = cross-sectional area of pile as a function of z, and
E_z = modulus of elasticity of the pile as a function of z.

The value of the modulus of elasticity, E, may not be linear with the compressive stress, as discussed earlier. If so, the symbol should be written E_{qz} where q denotes the unit stress in compression in the pile.

The unit load transfer in end bearing, q_B, may be computed from the following equation:

$$q_B = \frac{Q_B}{A_B} \qquad (3)$$

where Q_B is the load at base of the pile and A_B is the area of the base of the pile. The corresponding downward movement w_B of the base of the pile may be found from the following equation:

$$w_B = \delta_n - \int_L^0 \frac{Q_n dz}{A_z E_z} \qquad (4)$$

where L is the total length of the pile.

The curves shown in Figure 5 were analyzed by using Equations 1 through 4 and the results are shown in Figure 6. The curves in Figure 6 are valuable to the designer if drilled shafts of other diameters are needed at the test site. Furthermore, a collection of such data will allow correlations to be developed between soil properties and load transfer.

The application of the analytical procedure that is presented is for the results of a test of a drilled shaft (bored pile) in which

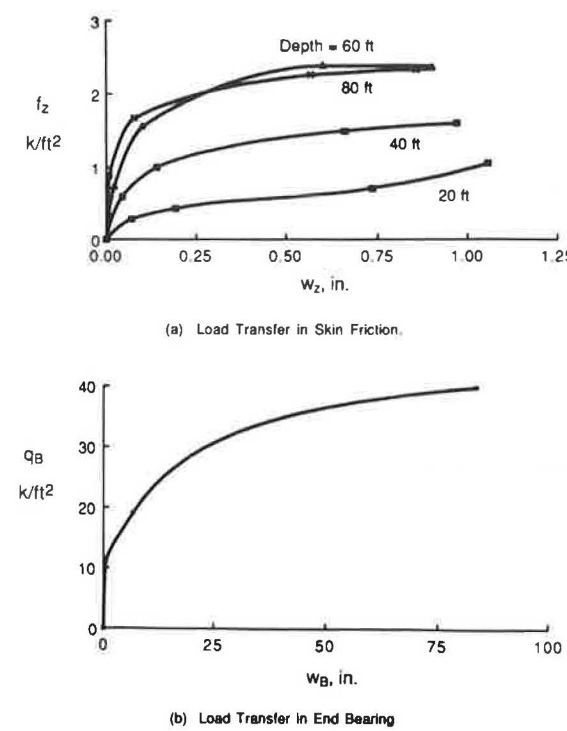

FIGURE 6 Load-transfer curves for test of a drilled shaft.

there were no residual loads and displacements in the pile. The analytical procedure must be modified to deal with the initial conditions in the pile. The determination of the residual loads and displacement for a driven pile requires the use of assumptions about the behavior of the pile during the last blow of the hammer or, preferably, the use of instrumentation for measurement of internal load whose zero reading does not shift as a result of pile driving.

As a further example of the value of instrumenting piles during static tests, the curves in Figures 7 and 8 are presented.

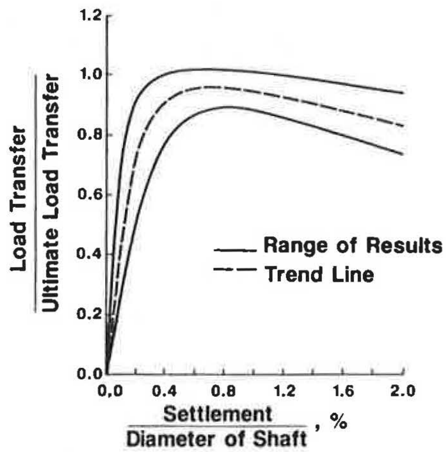

FIGURE 7 Normalized curves showing load transfer in side resistance versus settlement for drilled shafts in clay (*11*).

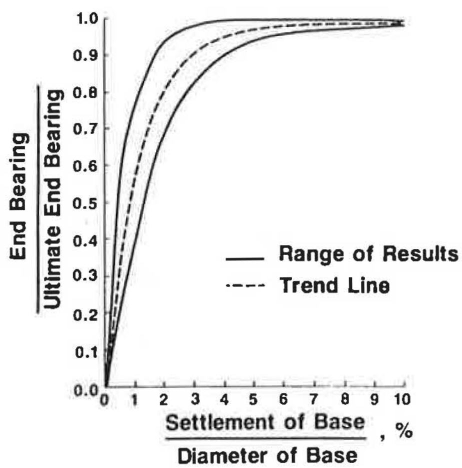

FIGURE 8 Normalized curves showing load transfer in end bearing versus settlement for drilled shafts in clay (*11*).

The curves were obtained by analyzing the results of several tests of instrumented drilled shafts that were installed in clay (*11*). It is of interest to note the difference in the scales of the abscissas for Figures 7 and 8. It is generally known that more relative settlement of the base of a pile is needed to develop a given percentage of load transfer than the relative settlement of the sides. The results from instrumented tests dramatically confirm this important point.

The procedure for using Figures 7 and 8 to get a curve that gives top load versus settlement is essentially the same as described earlier except that the entire drilled shaft is selected as a single element. The resulting curve will not likely be as accurate as if load-transfer curves were available at all points along the drilled shaft. However, the axial stiffness of drilled shafts is usually so great that the shortening of a shaft under axial load is small. The curves in Figures 7 and 8 can allow the engineer to make a reasonably accurate estimate of the settlement of a drilled shaft under a given axial load. This estimate would not have been possible if load tests of instrumented drilled shafts had not been performed.

INTEGRITY AND OTHER DYNAMIC TESTING

Two general types of integrity tests are used on drilled shafts or driven piles that have been installed: routine nondestructive tests that are considered a part of the inspection procedure and special tests in response to a suspected defect. Routine tests are performed on drilled shafts that are uninstrumented or contain some inexpensive transducers that are cast in place; these tests are not costly and thousands of them have been performed in recent years in the United States and abroad.

Special tests that are performed on drilled shafts or piles with a suspected defect, on the other hand, will normally be time-consuming and can be rather expensive. Generally such testing is undertaken only in unusual circumstances and often requires significant preparation of the members to be tested.

Some of the techniques that can be used to perform routine nondestructive tests to investigate pile integrity include the following:

- Vibrate or transiently load the pile externally and record signals with transducers at the pile head,
- Vibrate or transiently load the pile externally and record signals with transducers that are embedded in the concrete, and
- Install access tubes and use down-hole instrumentation to investigate the concrete between the access tubes.

Some of the special techniques that can be used to investigate the integrity of piles include the following:

- Use drilling and coring to investigate for cavities or contaminated concrete and to investigate the condition of the contact at the base of the pile or shaft,
- Use geophysical logging tools or video cameras in drilled or cored holes to examine the condition of the concrete,
- Perform a load test on the questionable pile, and
- Dynamically load the pile by dropping a heavy weight on the top of the pile or by using a pile-driving hammer and then employ wave-equation or impulse-response methods to compute the load capacity or stiffness.

These routine and special methods are discussed briefly in the following sections.

EXTERNAL VIBRATION OF THE PILE

Vibration with Hammer Blow—External Instrumentation Only

A procedure that is simple in concept was developed by the TNO Dynamics Laboratory in Delft, the Netherlands (*12*), and is illustrated in Figure 9. The head of the pile is struck with a hand-held hammer. A compression wave is generated that travels down the drilled shaft, is reflected from the bottom of the shaft (or from a defect), and is picked up by an accelerometer at the top of the shaft. In operation, however, the method can require skilled operators. The signal must be processed to eliminate unwanted waveforms, and the resulting signal must be displayed rapidly for convenient analysis.

An example of an idealized result from an integrity test is shown in the inset in Figure 9. As may be noted, the time for the compression wave to travel down the shaft and back again can be read from the signal as it is displayed on the screen of an oscilloscope. With knowledge or an assumption of the compression wave velocity in concrete (V_c), the constructed length of the drilled shaft (or the distance from the top of the shaft to a defect or irregularity) can be found from the simple equivalence shown.

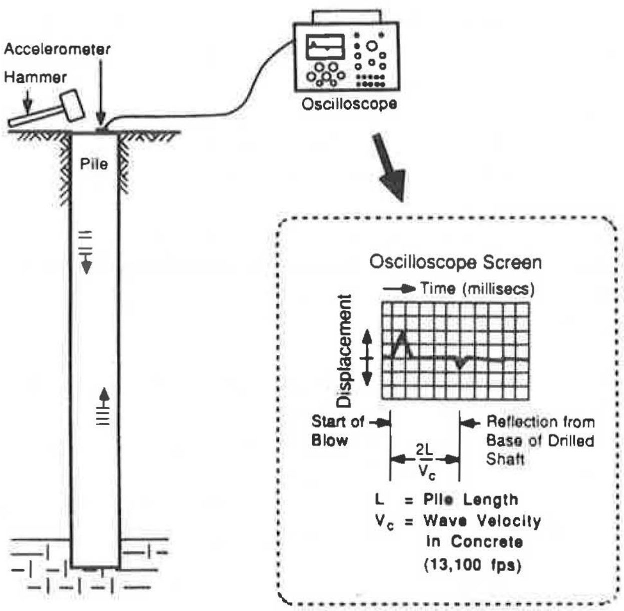

FIGURE 9 General arrangement for TNO testing method (12).

The advantages of the method are that a test can be performed rapidly and inexpensively. The disadvantages are that the interface bond is not investigated, that specialized equipment is necessary, and that the operators must be skilled in interpreting the waveforms that are recorded.

Vibration with Hammer Blow—Embedded Instrumentation

Investigators at The University of Texas at Austin have done studies by using the same concept as was used by TNO except that the receivers were embedded in the drilled shaft (*13, 14*). This test arrangement is illustrated in Figure 10. In addition to the embedded receivers, an accelerometer (or vertical velocity transducer) can be used at the top of the drilled shaft to provide further data.

The two receivers that are shown in the figure, plus a surface accelerometer, yield a significant amount of data. With such an

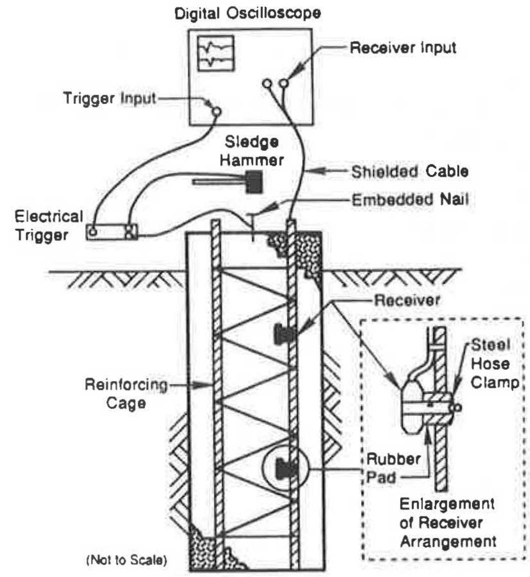

FIGURE 10 Compression wave propagation method with embedded receivers (13).

arrangement, direct evaluation of wave velocity, V_c, is performed and confirmation of any possible irregularities can be done by comparisons between the transducer signals. Direct evaluation of V_c allows computation of the small-strain modulus of elasticity, E_{max}, of the pile from

$$E_{max} = \frac{\gamma}{g} V_c^2 \tag{5}$$

where γ is the total unit weight of the concrete or steel and g is the acceleration of gravity.

The receivers are vertical velocity transducers (geophones) sealed in plastic cases. An electrical cable extends from the transducer case to about 10 ft beyond the top of the shaft. The transducers are relatively inexpensive (about $50 each), and the instrumentation can generally be installed rapidly with no delay in the construction process.

Typical results from a test shaft in Houston, Texas, are shown in Figure 11 (*14*). The shaft is 30 in. in diameter, 50 ft in length, and contains four embedded receivers. Evaluation of the quality of the concrete, in terms of compression wave

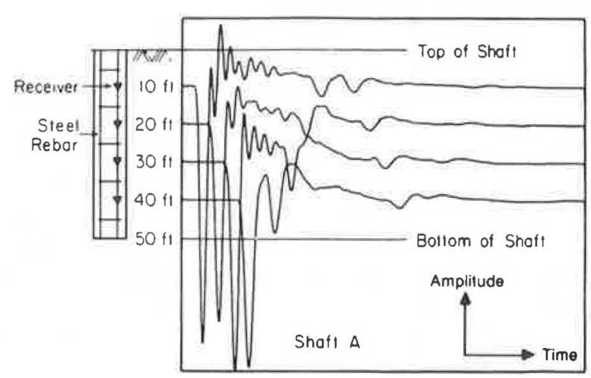

FIGURE 11 Seismic waveforms recorded with embedded receivers in a 50-ft-long sound shaft (14).

velocity, is illustrated in Figure 12. Although compression wave velocity cannot be directly related to concrete strength (because of mix design variations), measured P-wave velocity for a given mix design provides a relative indication of concrete quality of the shaft or pile. A suggested rating of concrete quality based on P-wave velocity is provided in Table

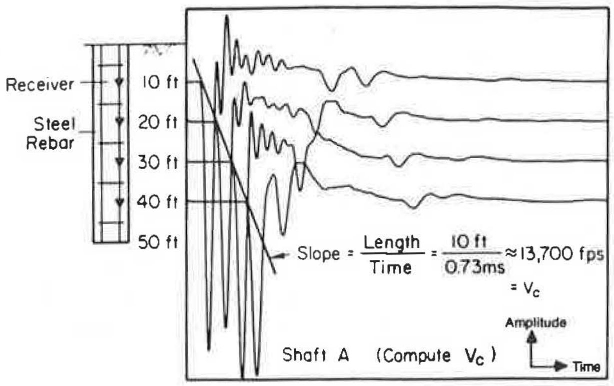

FIGURE 12 Illustration of wave velocity determination from initial arrival of compression wave monitored with embedded receivers (*14*).

TABLE 1 SUGGESTED COMPRESSION WAVE VELOCITY RATINGS FOR CONCRETE FROM ULTRASONIC TESTS (*15*)

Compression Wave Velocity		
Measured in ft/sec	Measured in m/sec	General Conditions
Above 15,000	Above 4570	Excellent
12,000–15,000	3660–4570	Good
10,000–12,000	3050–3660	Questionable
7,000–10,000	2133–3050	Poor
Below 7,000	Below 2130	Very poor

1 (*15*). The rating scale shown in Table 1 is based on ultrasonic pulse tests (high-frequency tests), which were used to measure P-wave velocity through a concrete medium. Wave propagation velocity in a rod differs from wave propagation velocity in a continuous medium of an identical material if the wavelength of the pulse is greater than the rod diameter. Since a drilled shaft or pile behaves like a rod in this testing, wave propagation velocity as determined herein will be less than the velocity measured at ultrasonic frequencies, the reduction being approximately 10 percent for a Poisson's ratio of 0.25. Therefore, velocities listed in Table 1 have been reduced by 10 percent and are presented in Table 2 so that they can be compared with wave velocities determined by this test method.

Identification of wave reflections in these records is illustrated in Figure 13. It can be seen in the figure that reflections have occurred only from the ends (top and bottom) of this straight-sided shaft. Such identification shows that the shaft contains no defects or irregularities.

Two advantages of using embedded receivers in comparison with surface receivers are that the background noise level is much reduced with embedded receivers (reducing or eliminating the need for signal processing) and any number of embedded receivers can be installed. The disadvantages of using the wave propagation method with embedded receivers are the same as those listed for wave propagation testing with external

TABLE 2 SUGGESTED COMPRESSION WAVE VELOCITY RATINGS FOR CONCRETE FROM WAVE PROPAGATION METHOD (*14*)

Compression Wave Velocity (ft/sec)	E^a (psi)	General Conditions
Above 13,500	5.90	Excellent
10,800–13,500	3.77–5.90	Good
9,000–10,800	2.62–3.77	Questionable
6,300–9,000	1.28–2.62	Poor
Below 6,300	1.28	Very poor

NOTE: Figures are assuming the wavelength is greater than two times the pile diameter.
^{a}Assuming the unit weight of concrete = 150 pcf.

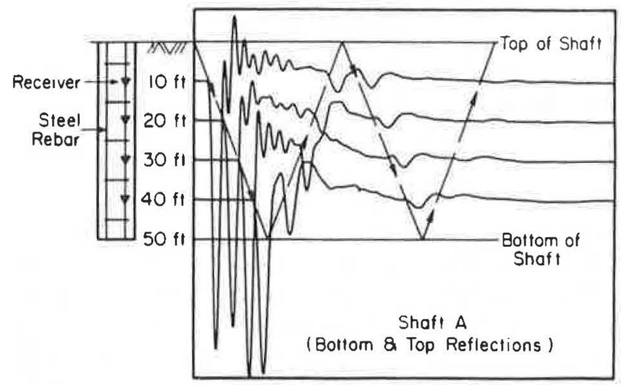

FIGURE 13 Identification of wave reflections from top and bottom of a sound shaft (*14*).

instrumentation. In addition, use of embedded receivers involves a somewhat greater cost and the decision to test must be made before construction.

Steady-State Vibration

Davis and Dunn (*16*) report that the method of forced vibration (steady-state vibration) was developed by the French Building Institute (CEBTP) in the 1960s. Preiss et al. (*17*) report that several thousand bored piles (drilled shafts) have been routinely tested with the method. The test is performed by placing an electrodynamic vibrator over a load cell in a specially prepared region at the center of the top of the shaft as shown in Figure 14 (*18*). The vibrator is driven by a sinusoidally varying current over a frequency range from about 20 to 1,000 Hz. The output from the load cell at the top of the drilled shaft is fed into a regulator that adjusts the amplitude of vibration to keep the force level constant. The vibrator force is applied to the head of the drilled shaft, and the vertical velocity at the shaft head from a velocity transducer is monitored for each measurement frequency.

The curve of V_0/F_0 is plotted as a function of frequency, where V_0 is the maximum velocity at the head of the drilled shaft and F_0 is the applied force. This plot, shown in Figure 15 (*18*), expresses the mechanical admittance or mobility of the shaft head. The frequency difference between successive resonance peaks, Δf, is equal to

$$\Delta f = V_c/2L \tag{6}$$

where L is the intact length and V_c is the compression wave velocity in the concrete (about 13,000 fps for sound concrete).

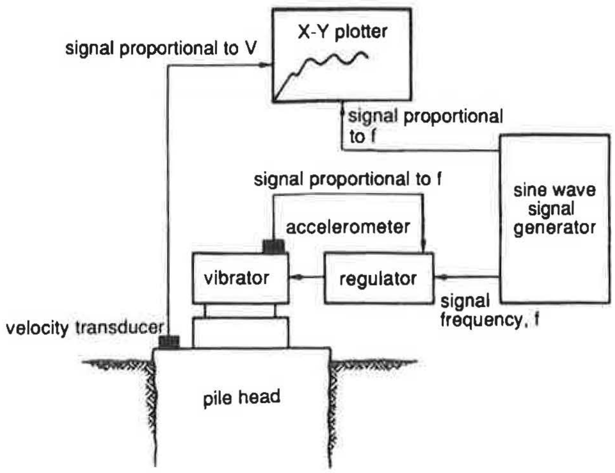

FIGURE 14 Experimental arrangement used in steady-state vibration testing (*18*).

If the shaft length is known, the quality of the concrete in the drilled shaft can be inferred from the wave velocity. One problem that can arise is that interpretation can be complicated by "noise" in the electronic monitoring systems created by variations in diameter of the drilled shaft, variations in concrete quality, variations in the stiffness of the soil around the drilled shaft, and any exposure of the top part of the drilled shaft above the ground.

In addition to predicting reflector depths, the forced vibration test also measures dynamic stiffness at the foundation head. Dynamic stiffness reflects the soil-foundation interaction conditions and may be thought of as a spring constant with units of kips per in. Identical foundation members will be much less stiff when placed in soft soil than in bedrock. Defects such as necking and soil inclusions in drilled shafts and breaks in driven piles also result in comparatively lower dynamic stiffnesses (*19*). Correspondingly, enlargements and bulbs in shafts produce comparatively higher dynamic stiffnesses.

The forced vibration test can characterize the nature of defects, such as necking or bulbing, on a comparative basis for similar foundations. Although the dynamic stiffness measured by the forced vibration test is a low-strain measurement, correlations with static stiffness at low loads in axial load tests indicate dynamic stiffness is typically one to two times the static stiffness. Such correlations have been used to make reasonable predictions of foundation settlement under working loads. However, the forced vibration test does not predict the capacity of deep foundations.

The advantages of the method are that no preselection is needed of the shafts to be tested and that an experienced operator can detect both discontinuities and major faults. The disadvantages are that more equipment and instrumentation are required than for the other methods where vibration is used and that little information can be obtained below a significantly enlarged region in the drilled shaft.

Impulse Response

Impulse-response testing is a technique in which the force of a vertical impact is compared to the vibration response of the shaft head using a spectral analyzer. Stain (*20*) reports that results and interpretation from the impulse-response or "transient dynamic response" method are in every way identical to the forced vibration method just discussed. The impulse-response method has several advantages over the forced vibration technique in that the bulky vibrator is eliminated, minimal preparation of the shaft head is required, and the impulse-response test is much faster. Consequently, the impulse-response method has now largely replaced the forced vibration technique in practice.

Equipment used to perform the impulse-response test consists of a spectral (signal) analyzer, impulse hammer (with

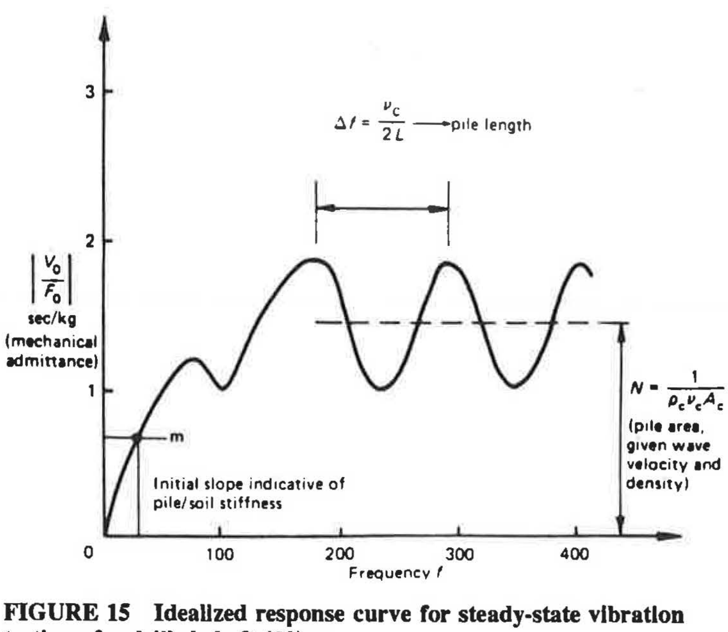

FIGURE 15 Idealized response curve for steady-state vibration testing of a drilled shaft (*18*).

built-in force transducer), and a geophone as indicated in Figure 16 (*19*). The test consists of hitting the shaft head with an impulse hammer and measuring the vibration response of the shaft head with a vertical velocity transducer (geophone). Several tests are conducted on a single pile, and the impact force and vibration response are averaged and processed with the signal analyzer to determine the dynamic response of the soil-foundation system.

Results of impulse-response tests of the sound shaft at the research site of The University of Texas (*14*) are presented in Figure 17. (Wave propagation testing of this shaft, Shaft A, has already been shown in Figures 11–13.) The test records are plots of V_0/F_0 versus frequency, or mobility plots, as discussed previously. Dynamic stiffness can be calculated from the initial straight-line portion of the plot at low frequencies or directly by integration using the signal analyzer. At low frequencies the soil and shaft move together, which permits the measurement of dynamic stiffness as a result of the soil-structure interaction effect (*19*). Resonant peaks for the shaft are also apparent in the mobility plot. An average change in frequency between the peaks of 138 Hz was calculated as illustrated. A reflector depth of 49.6 ft was calculated for the shaft as indicated on the mobility plot. The actual reflector depth (the bottom of the shaft) was 50 ft, which is in close agreement with the prediction.

In summary, the impulse-response method provides data on reflector depths and soil-foundation interaction effects. Comparative evaluation of test results can also indicate the nature of an irregularity such as a bulb or neck. The impulse-response method is much more portable and faster than its predecessor, the forced vibration method. Like the TNO stress wave method, reflections may not be identified from long slender shafts because of excessive attenuation, and reflections may also not be identified from shafts founded in competent bedrock with density and compression (stress) wave velocities that are similar to the shaft concrete. In these cases, embedment of geophones or tubes is necessary to confirm the continuity of such shafts. However, experience has shown that significant defects will produce identifiable reflections at depth in even long slender shafts, depending on the soil conditions (*21*).

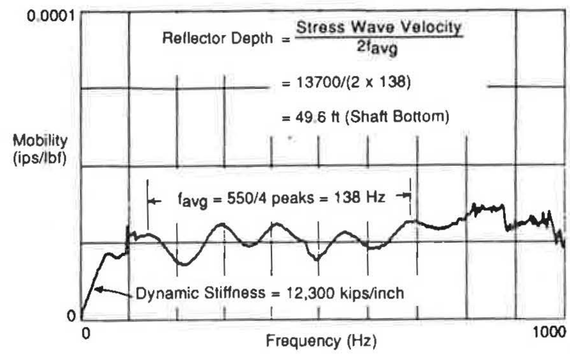

FIGURE 17 Mobility plot for Shaft A (sound shaft) shown in Figure 11 (*19*).

STATIC AND DYNAMIC LOAD TESTS OF SUSPECTED DRILLED SHAFTS

In some instances, the only positive way to prove the integrity or behavior of a particular deep foundation is to perform a static load test. However, only in unusual instances is such a procedure economically feasible for drilled shafts. The assembly of the loading equipment would, in most instances, be more expensive than the replacement of the questionable shaft.

A general exception exists where a drilled shaft is designed for a relatively light load and where existing drilled shafts in

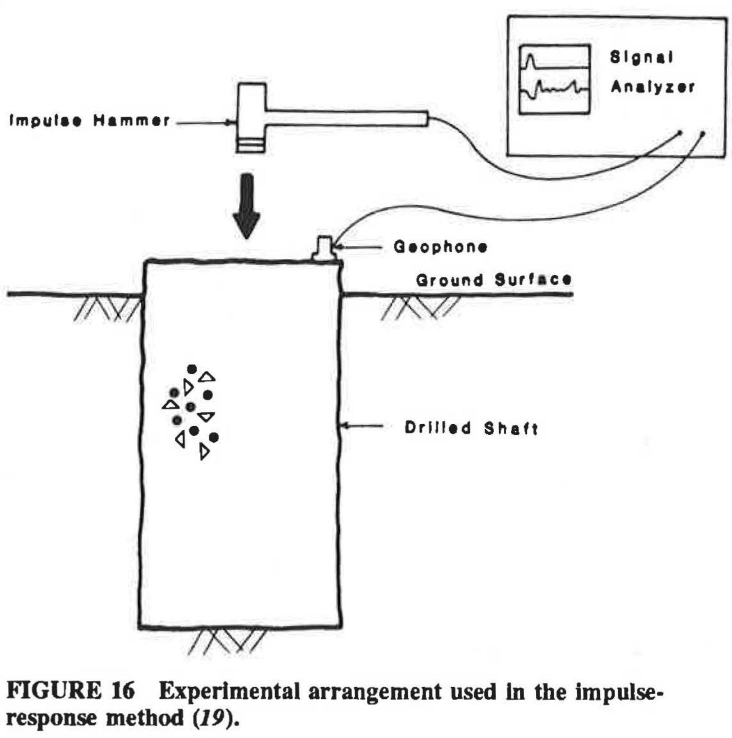

FIGURE 16 Experimental arrangement used in the impulse-response method (*19*).

line and on each side of the questionable one can be used for the reaction. Such a favorable situation is thought to be rare.

A procedure that has been developed involves the performance of a dynamic load test (*12*). A testing arrangement is made that allows a mass of sufficient magnitude to be dropped onto the top of the drilled shaft. Measurement of the applied dynamic load and movement of the top of the shaft allows the performance of the foundation to be deduced by the wave-equation method (*22*). While the method has been used to a limited extent in the United States and abroad, it has promise as a tool to be used in special cases and even routinely on important projects.

OTHER METHODS OF INTEGRITY TESTING

There are several other methods for investigating the integrity of piles or drilled shafts. Methods that involve the use of access tubes and down-the-hole instruments, such as sonic or nuclear logging tools, have been used successfully. The use of drilling and coring has also been done. These methods are, however, not discussed herein because of space limitations.

CONCLUSIONS

- The performance of static-load tests of instrumented piles, or drilled shafts, is feasible and can yield valuable results.
- The selection of instruments and their placement along a pile should take into account the analytical model of the pile-soil system.
- A variety of instruments is available for yielding data giving internal load or displacement in a pile as a function of depth.
- The Mustran cell and the telltale have proven to perform well in the testing of drilled shafts under axial load.
- With the collection of a sufficient body of data concerning the distribution of axial load with depth, it is probable that load-transfer functions can be developed that will allow designers to predict by numerical techniques the load versus settlement for an axially loaded pile.
- Investigation of the integrity of a concrete pile by the use of dynamic methods (stress wave measurements) is feasible. In addition, such measurements can also be used to evaluate the small-strain modulus of the member if embedded measurement points are used.
- Off-the-shelf transducers and digital recording equipment are available that can be used to perform embedded wave velocity measurements in a pile.
- Steady-state and impulse-response tests (dynamic tests) can also be used to evaluate the integrity of piles and drilled shafts. In addition, the deformation behavior of these members at small strains to obtain a sense of the initial load-settlement curve of the pile can be determined with these tests.
- Only on rare occasions are static or dynamic load tests used specifically to evaluate pile integrity.

ACKNOWLEDGMENTS

This work was sponsored by the Texas State Department of Highways and Public Transportation in cooperation with the FHWA, U.S. Department of Transportation. The support of these sponsors is gratefully acknowledged.

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Publication of this paper sponsored by Committee on Soils and Rock Instrumentation.

Use of the Wave Equation by the North Carolina Department of Transportation

J. F. LEDBETTER, JR.

Use of the wave equation analysis method offers the most complete and comprehensive control of pile driving of any method available today. It provides a means of increasing the engineer's confidence that not only is the required capacity being achieved but also that the pile is not being overstressed and that the pile-driving hammer is capable of driving the pile to the desired depth in the ground. An outline of the development of the wave equation analysis to control pile driving for the North Carolina Department of Transportation is presented. This process occurred over several years and culminated with the purchase of a pile driving analyzer in July of 1987. This paper contains three case histories demonstrating the use of this method.

The use of the wave equation to control pile driving is relatively new and is not widespread among state highway departments. The wave equation in conjunction with the pile driving analyzer (PDA) is gaining in popularity and will eventually displace other, less reliable means of determining pile capacity and driving stresses. This paper outlines the development of pile driving control through the use of the wave equation in the North Carolina Department of Transportation (NCDOT).

In 1977 the NCDOT sent two employees to Atlanta, Georgia, to attend a Federal Highway Administration Pile Wave Equation Seminar. The NCDOT then acquired the 1976 Wave Equation Analysis of Piles (WEAP) computer program and used it unofficially on pile projects to gain experience and develop confidence. Presently the NCDOT is using the 1986 WEAP computer program (*1*). With encouragement from the local FHWA bridge engineer, methods of correlating the wave equation computer output with pile load test results were practiced on at least four projects a year. The pile load test sites were chosen with regard to subsurface conditions and pile type. Around 1980 the NCDOT let to contract a large bridge project in the coastal plain using 54-in. prestressed concrete cylinder piles. The piles were 120 ft long with a 60-ft free length above finished grade. This was the first time NCDOT had used this type of pile; so, to ensure structural integrity during driving and to control the length and ensure capacity, the contract stated that the wave equation would be used instead of the *Engineering News* (EN) formula. The wave equation results were correlated with the pile load tests.

Since 1980, the NCDOT has used the wave equation to control pile driving on all the large coastal bridges and on other projects across the state that were judged to require pile-driving control. For about 90 percent of the bridges on which the wave equation is used, a load test or tests are conducted.

A considerable portion of North Carolina lies in the Blue Ridge Mountain and Piedmont Provinces, and many bridges are founded on spread footings or steel H-piling driven to rock. The EN formula is still generally being used for these structures, especially if the total linear feet of piling is less than about 4,000 ft. However, it is anticipated that North Carolina will use the wave equation exclusively for pile-driving control in the near future.

PILE DESIGN PROCEDURES

There are many different types of piles available and several types may be suitable for a given situation. The rationale behind pile selection, pile length, and allowable load is usually based on consideration of geotechnical data, bridge location, type of superstructure, and engineering judgment. Although the factors are interrelated, they can be accounted for in three pile design categories. A description of each follows.

Static Analysis

Once the structural load has been determined for proposed foundation alternates, the pile type, length, and size required to support that load are chosen from the results of bearing capacity and settlement analyses. Static analyses are made using methods after Vesic (*2*). Settlement analyses using laboratory consolidation test data are made, and the effects of drag-down on the pile or piles are computed. Generally, a bearing capacity safety factor of two is used and settlement is limited to 1 in. under dead plus live load.

Soil Driving Resistance

The soil resistance to be overcome in placing the pile to the required tip elevation is computed to determine drivability. This resistance is obtained after the design pile length is established by summing the side friction computed in the static analysis for all soil layers. The soil driving resistance in general will be greater than the soil support computed for a given pile length since compressible layers and soil above the scour line, which are ignored for support, will provide resistance to pile driving. A wave equation analysis is performed using damping parameters and a stress distribution corresponding to the static

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bearing capacity computation and assuming a commonly used hammer. The results of the analysis indicate the pile stresses to be expected as a result of driving and whether the pile can be driven to the full depth. Allowable driving stresses are taken from Vanikar (*3*).

Lateral Load

Based on the type of substructure selected, piles may be subject to substantial horizontal as well as vertical loads. For instance, piles that extend from the ground line to the superstructure must be designed for horizontal live loads. However, this type of bent is often the most economical. Horizontal load analysis is performed with the aid of the COM624 computer program (*4*). Allowable deflection is generally limited to 1 in. depending on the bridge with a safety factor of two against overturning. Also, piles in soft ground or piles with a free length are checked for buckling. Piles adjacent to embankments installed in soft ground are checked for the likelihood of being affected by lateral squeeze. Lateral squeeze is the horizontal pressure that may bend or push the piles outward. This thrust is as a result of the unbalanced fill load. The approach taken is after Cheney and Chassie (*5*).

CONTRACT SPECIFICATIONS

From the foregoing considerations, an economical pile is chosen with a foundation capacity and estimated length. These are shown in the general notes of the structure plans. The NCDOT Standard Specifications state that "the estimated length of piles shown on the plans and in the itemized proposal are for bid purposes only." Unless otherwise stated, the contractor is required to drive the piles to the design capacity and install them not less than 10 ft into natural ground.

Pile load test locations are shown in the plans with minimum tip elevations. The general notes also state that the wave equation will be used to determine bearing capacity in lieu of the EN formula, and that lengths for the piles to be installed will be determined by the engineer after the load tests are concluded. The pile load tests are shown in the bid items as per each. The average cost of a pile load test using the quick method in 20 ft of water is approximately $20,000 and half that for tests on land.

The contract special provisions state that the contractor shall submit for approval the specifications on the pile-driving hammer, cap block, and cushioning material that are proposed for driving the piles 2 weeks before work begins. The load test setup and equipment must also be submitted for review 2 weeks before load testing. The special provisions further state that the contractor shall use the same pile-driving hammer for production and test piles.

On long coastal bridges, test piles for length are used with pile load tests and listed by location in the special provisions. Test piles for length are used to verify that proposed production pile lengths will achieve capacity with a minimum of cutoff. Usually test piles for length are placed in bents on either side of a load test and are intended to provide information at the limits of a range of bents having similar subsurface conditions. The lengths of these piles are determined by the engineer after the pile load tests are satisfactorily completed and the test pile has been restruck. Based on the results of the load test and with confirmation from the test pile for length, the contractor is given production order lengths. Test piles for length are incorporated into the structure and paid for as production piles. Hammer blows are counted for the entire length and the pile may be restruck with a warm hammer, if necessary, to determine freeze.

PILE-DRIVING CONTROL WITH THE WAVE EQUATION

The NCDOT has found that the most reliable means for assuring pile capacity and predicting length of production piles is to load test a few piles and analyze them using the wave equation. The field control then consists of providing bearing graphs to the resident engineer and order lengths to the contractor. The NCDOT pays the contractor for the length installed in the ground to pile cut-off elevation. For any length that is cut off, the NCDOT Standard Specifications state,"When the engineer has determined the length of piles to be furnished and driven, the department will reimburse the contractor for pile cutoffs; however, the cutoffs will remain the property of the contractor."

The pile to be load tested is driven and driving data recorded as the number of blows per foot (BPF) for each foot of penetration. After a minimum period of 36 hr, the pile is tested vertically (in accordance with ASTM D1143-81—quick method). The pile is tested to two or three times the design load and every effort is made to determine the failure load. The failure load is determined by the use of Davisson's limit. Davisson's limit is defined as the load corresponding to the movement that exceeds the elastic compression of the pile by a value of 0.15 in., plus a factor equal to the diameter in inches of the pile divided by 120. After the ultimate load carrying capacity of the pile is determined and the pile is restruck with a warm hammer to determine the freeze, a wave equation analysis is performed. The soil damping parameters for the wave equation analysis are modified until the wave equation results in a value corresponding to the ultimate load as determined by the pile load test. The Smith damping values are adjusted while the soil quake of 0.1 remains unchanged. In general, if the side resistance is less than 30 percent of the total capacity as determined by the static analysis, only the tip damping is adjusted until the hammer BPF and ultimate capacity as given by WEAP agree with the pile load test failure load and final blow count. The stress distribution and the ratio of side resistance to total capacity for input in WEAP are taken from the static analysis. After this is achieved, a capacity table or bearing graph showing capacity versus BPF is constructed and used to determine the bearing capacity of production piles. Before the NCDOT determines the probable production order lengths, several test piles for length are driven to capacity according to the derived bearing graph. Any modifications to the length of production piles judged necessary are then made.

The wave equation is also used to limit damage to the pile during driving by analysis and approval of the cap block, cushion, and pile-driving hammer. The contractor is required to submit this information through the resident engineer, and on approval, the resident engineer and contractor receive instructions on inspection of the cushion material and at what decreased thickness the cushion is required to be changed. The maximum and minimum BPF to ensure that the pile is not overstressed in either compression or tension are included. If the hammer has different stroke settings, the blow count before a change is determined. Tentative approval for the pile-driving equipment is given before the load test, but final approval must wait until the wave equation has been correlated with the load test results. The test pile must be installed using methods the contractor proposes to use on the production piles.

CASE HISTORIES

US-17 over the Intercoastal Waterway

The first bridge project on which the wave equation was used to control pile driving was the dual bridges on US-17 over Lake Drummond Canal in Camden County, North Carolina. This site is located in the Coastal Plain Province. Lake Drummond is an alternate route of the Intercoastal Waterway and required 65 ft of vertical clearance. The contract was let in 1980 for $8,156,777 and contained 20,000 linear ft of 54-in. prestressed concrete cylinder piles at a unit bid of $100/ft. The unit bid price for a pile load test was $75,000. The load tests were carried to 849 tons or three times the design load.

Two pile load tests were conducted. Results of the load tests indicated that 60 ft of penetration would provide adequate bearing capacity in fine to coarse sand with a standard penetration test (SPT) of 20+ BPF. The bearing graph is shown in Figure 1.

The contractor received approval to use a Conmaco 300 single-acting steam hammer and one 5½-in. plywood cushion. The cap block was Dura-Cush. Graphs were provided that showed the tensile stresses versus blow count for both the short and long hammer strokes. (Figures 2 and 3, respectively). The contractor requested and received approval to prejet approximately one-half the pile penetration to shorten the installation time. The approval was based on the contractor's use of the short hammer stroke.

After installation was begun, tensile cracking of the piles was detected in the upper 50 ft of the 120-ft pile. The contract provided for inspection inside the piles. An investigation of the pile-driving operation revealed that the cracking was occurring under soft driving either under the short hammer stroke or

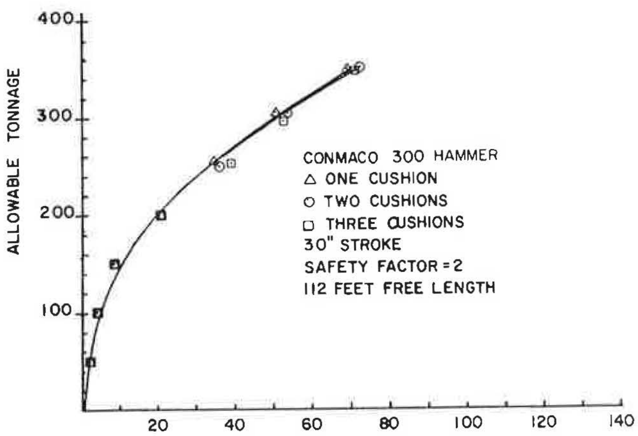

FIGURE 1 Bearing graph for 54-in. cylinder pile: Allowable tonnage versus hammer blows.

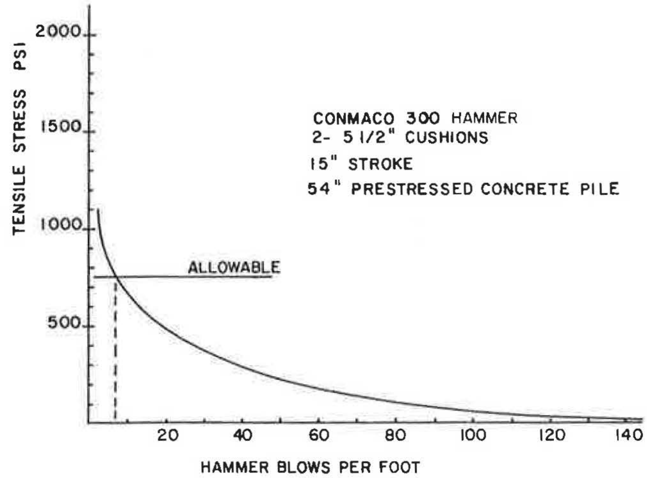

FIGURE 2 Tensile stress under short hammer stroke.

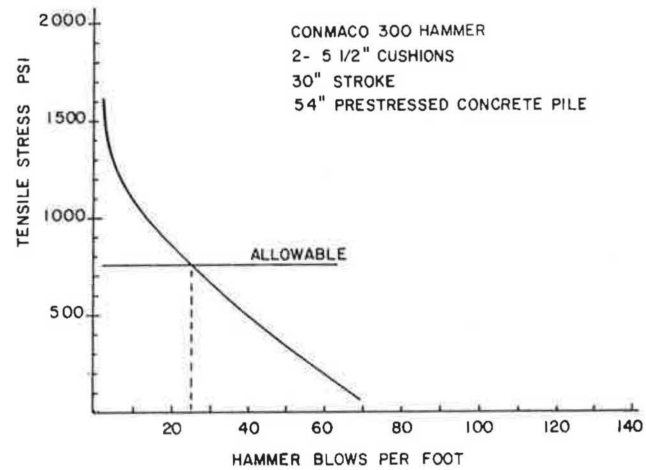

FIGURE 3 Tensile stress under long hammer stroke.

when the long stroke was first initiated. The hammer approval specified that the short stroke was to be used until a blow count of 52 per ft was reached. It was not ascertained that this was the case. Further investigation revealed that the cushion was not being changed or inspected often enough to prevent its complete deterioration.

Additional wave equation analysis resulted in calling for the use of two 5½-in. plywood cushions, with the provision that, provided the top cushion is compressed to a thickness no less than 4¾ in., the cushion may be reused as the bottom cushion on the next pile to be driven. The remaining piles were driven without incident and the cracked piles were repaired and left in place. Presently, when approval is given for the cushion, a thickness is specified at which the cushion must be replaced.

NC-32 over Albemarle Sound

A recent project on which the wave equation is being used to control pile driving is the bridge over Albemarle Sound on NC-32 in Washington and Chowan Counties. This project was let to contract in 1985 and is an 18,465-ft bridge providing 65 ft of vertical clearance. The contract is for $22,389,850 and contains 156,840 ft of 20- and 24-in. prestressed concrete piles at unit bid prices of $35 and $38/ft, respectively. The unit bid price for a pile load test is $25,000. The load tests were carried to two times the design load of either 100 or 135 tons depending on the bent location. This bridge site is located in the Coastal Plain Province of North Carolina. Five pile load tests have been conducted. Results of the load tests indicate that 70 to 80 ft of penetration will provide adequate bearing capacity in silty fine sand and clayey silt with an SPT of 10 to 25 BPF. The contractor received approval to use a Delmag-D 62-22 diesel hammer, one 10-in. plywood cushion, and 3.5 in. of Micarta cap block.

Three of the five load tests exhibited freeze effects. Preliminary wave equation analyses for hammer approval indicated that 15 BPF with the pile hammer were required; however, the final blows per foot on the test piles ranged from 7 to 13. After 36 hr, the piles sustained 200 to 315 tons ultimate load. This range was sufficient for a safety factor of two. The restrike blow count ranged from 19 to 39.

An example of correlating the WEAP program with a pile load test and using the results to control pile driving and to determine pile order lengths is as follows.

The static capacity computations for a 24-in. prestressed concrete pile installed to elevation −82 in materials similar to those shown in Figure 4 indicated an ultimate capacity of 186 tons. The capacity at the pile tip was 132 tons, whereas the side resistance was 54 tons, or 29 percent of the total. The side resistance was assumed to be distributed in a rectangular shape along the pile.

The test pile driving record is also shown in Figure 4. The blow count at the end of driving was 13 (BPF) and after 4 days the restrike blow count was 19 (BPF).

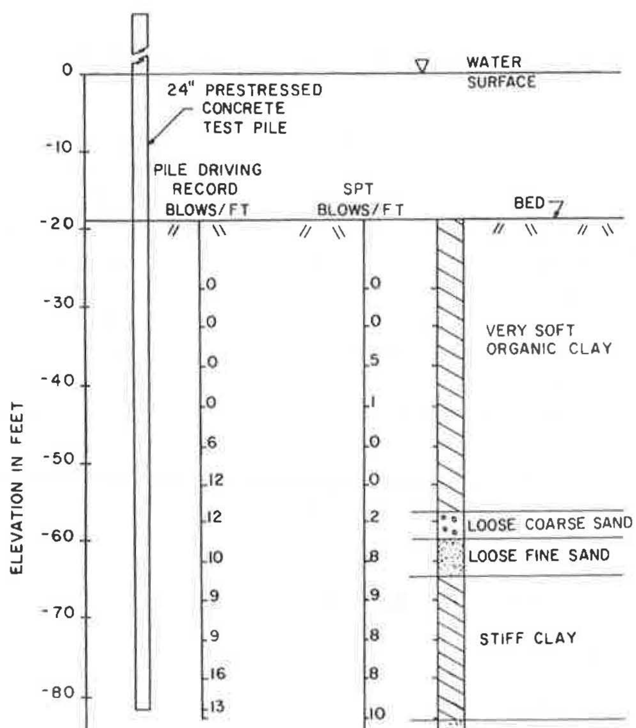

FIGURE 4 Boring log-pile load test site.

The pile load test graph is shown in Figure 5 and indicates an ultimate load of 197 tons. The correlation of WEAP with the ultimate test load is shown by the bearing graphs in Figure 6. Two curves are shown, one labeled "Prior to Load Test" and one labeled "Adjusted for Load Test." The damping and quake values assumed are shown in Table 1. The adjusted curve was used to control pile driving for 86 pile trestle bents, including 3 test piles for length. The average tip elevation of the test piles for length was −88. The subsequent production pile order lengths were based on this tip elevation, with an extra 4 ft added to the length.

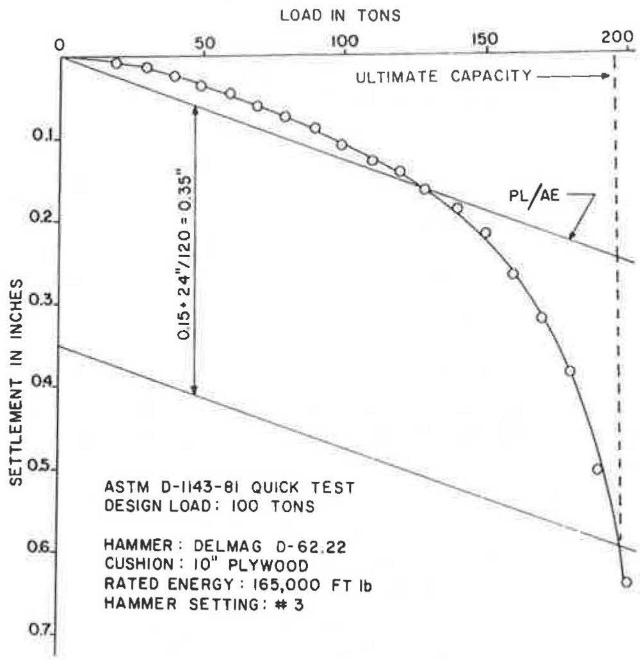

FIGURE 5 24-in. prestressed concrete pile load test.

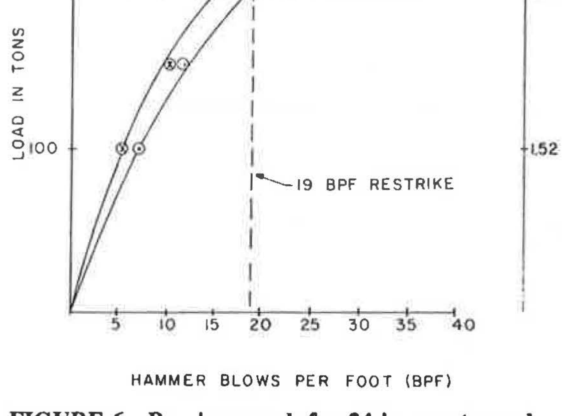

FIGURE 6 Bearing graph for 24-in. prestressed concrete pile.

TABLE 1 ASSUMED DAMPING AND QUAKE VALUES FOR PILE LOAD TEST

Soil Parameters	Before Load Test	After Load Test
Smith side damping	0.20	0.20
Smith toe damping	0.10	0.24
Side quake	0.10	0.10
Toe quake	0.10	0.10

NOTE: Percent side resistance from static analysis = 29 percent.

To verify the capacity of production piles, a schedule of restriking the test piles for length between 1 and 24 hr was devised. The desired result was to determine how quickly a pile would gain capacity. The test pile was left 4 ft high and driven 6 in. after 1, 3, 7, and 24 hr. From this data a chart of strength gain versus time was developed (Figure 7).

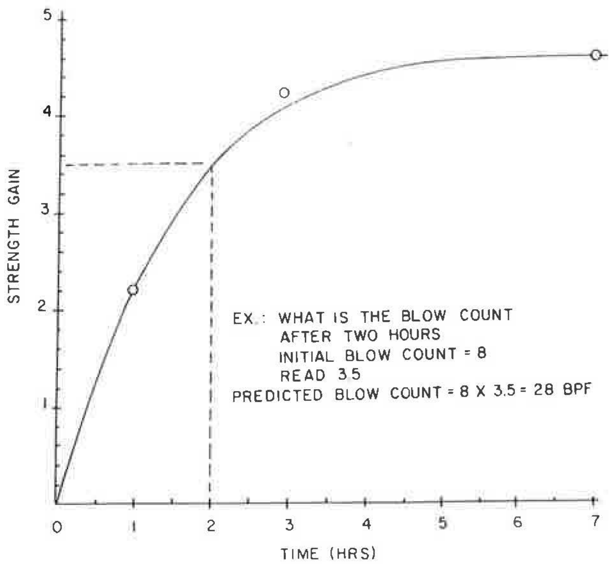

FIGURE 7 Strength gain versus time.

Following the time-dependent tests, the resident engineer was given the following information to schedule redriving of selected piles to verify the capacity of any pile or group of piles. Generally, in a group only one pile was redriven that did not exhibit the required capacity at the end of initial driving.

- If the blow count for the final count is greater than 12 BPF but less than required blow count, redrive the pile 6 in. after a 1-hr period.
- If the blow count for the final count is less than 12 BPF but greater than 8 BPF, redrive the pile 6 in. after a 2-hr. period.
- If the final blow count is less than 8 BPF, redrive the pile 6 in. after a 3-hr period.
- If the required capacity is not obtained after the initial waiting period, use the redrive blow count to choose the next appropriate waiting period, or redrive after 24 hr.

At the time of this writing, all piles in place have achieved the required capacity. In the event the required capacity was not achieved, the addition of piles or pile splicing would have been considered.

It is felt that the pile-driving control based on a wave equation analysis instead of the current NCDOT Standard Specifications and EN formula provided a more logical, and justifiable, engineering method to accept pile lengths that did not show immediate bearing capacity and thereby saved considerable time and money. The NCDOT Standard Specifications would have required that the pile be driven until capacity was achieved according to the EN formula with no time allowed for freeze.

A pile-driving analyzer was purchased in July 1987 and will be used on this project to further verify strength gain.

US-64 and US-264 over Roanoke Sound

A project was let in the fall of 1987 for a bridge on US-64 and US-264 over Roanoke Sound between Manteo and Nags Head in Dare County. The bridge will be 5,544 ft long and provide 65 ft of vertical clearance. The contract contains 50,560 linear ft of 22-in. prestressed concrete piles, with four pile load tests to be carried to 300 percent of the required bearing of 100 tons.

In addition to pile-driving control with the wave equation, the contract special provisions state that the pile-driving analyzer will be used on this project, and that the engineer will require approximately 1 hr/pile to install the measuring equipment. It further states that it is anticipated that 50 piles will be dynamically tested.

The contractor is instructed to notify the engineer at least 5 working days before driving piles at any location where a dynamic test is anticipated. The contractor is also to supply a source of electrical power and an air compressor. For those piles on which redriving is required, a freeze period of 24 hr minimum is specified.

This is the first contract that contains special provisions regarding the pile-driving analyzer, and future contracts will doubtless be modified to reflect the experiences on this bridge.

CONCLUSION

The wave equation analysis method provides a useful means to control pile-driving, especially when all concerned in a state agency are informed and are acquainted with the information that is required from the contractor. The wave equation analysis is a substantial improvement over dynamic formula, but there are still unknowns regarding hammer performance and soil parameters. It is hoped that the pile-driving analyzer, in conjunction with available pile analysis programs, will provide information on the hammer energy transferred to the pile and the soil constants required in the wave equation analysis.

It is anticipated that the acquisition of the pile-driving analyzer by the NCDOT will further refine construction control and reduce the number of pile load tests required, as well as permit the contractor to substitute different hammers for the one used in driving load test piles without conducting an additional pile load test.

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Publication of this paper sponsored by Committee on Foundations of Bridges and Other Structures.

NYSDOT's Construction Control of Pile Foundations with Dynamic Pile Testing

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New York State Department of Transportation tests piles dynamically, and the results are used to determine final pile-driving criteria. Approximately 20 percent of projects having piles include dynamic testing. Although the majority of the tests are specified for in design, a number of dynamic tests are needed each year to solve construction problems. The dynamic test is used to verify predictions made using static analysis and the wave equation analyses. The intent of the pile program is to avoid and control pile construction problems. Dynamic testing is done on special projects that have peculiar or unique soil conditions, or when it is difficult to estimate soil parameters in design or in preconstruction analyses. Testing is also done on projects with soil conditions that are found to be different from those assumed in design and to troubleshoot pile-driving problems. Examples from field tests demonstrate how the testing program is used to check on concerns for pile capacity, pile length, and pile driving stress, as well as hammer operation.

New York State Department of Transportation (NYSDOT) uses four tools to estimate and verify pile resistance: the wave equation analysis of piles (WEAP) computer program, dynamic test, Case pile wave analysis program (CAPWAP), and occasionally a static test.

NYSDOT normally uses WEAP, dynamic test, and CAPWAP instead of the more traditional static test. There is a concern that the three methods are the same, implying a lack of redundancy in the program. The three methods are related, but by no means are they the same. They are all based on the principles of Newtonian physics, an analysis of instantaneous impacts. However, the problem of pile driving does not occur instantaneously. Therefore, any solution has to account for this fact and factor in a time of occurrence.

WEAP is a computer program that solves this problem by dividing the pile into a number of elements. It accounts for time by looking at each element of the pile. Soil parameters are applied to each element and combine with pile and hammer models to enable prediction of results. WEAP analyzes the effects of one segment on another during the short time interval it takes for a stress wave to travel through the pile segments. This program is normally used before driving any piles, to check on hammer and pile suitability to the anticipated driving conditions.

Dynamic testing, on the other hand, analyzes the pile as a whole, but not at the time of impact. The test is performed during pile driving. In essence, the test views the pile from the instant the stress wave begins at the top of the pile after the pile is hit by the hammer until after the wave completes its journey through the pile. Dynamic testing compares compression stress wave at impact and upon return to the pile head to determine what effect soil resistance forces have on the situation. This results in a capacity prediction. In this analysis, the time is an occurrence of the passage of a stress wave.

CAPWAP is a combination of the two previous procedures. By taking the data from the dynamic test and using it as input to a wave equation program, the user can compute, among other values, the capacity of the pile. This is accomplished by applying soil resistance, quakes (limits of static resistance), and damping constants (temporary dynamic forces). The end result is a prediction of capacity similar to that from a constant rate of penetration (CRP) static test. In fact, by applying increments of loads, a CRP plot can be computer-generated.

The static test is usually used only on large projects with high-capacity piles. These substructures rely on only a few high-capacity piles to support a large structure, resulting in a low redundancy factor. When the static load test is used, it is usually a CRP test method to failure.

PILE CONTROL

WEAP is used both during the design process and during construction. When piles are required because of possible scour, the piles must be driven into competent material, which usually necessitates hard driving. This has the potential for causing overstress of the pile, especially for cast-in-place (CIP) piles. Generally, the equipment the local contractors are apt to use is known. A hammer is assumed and a WEAP is run to check pile stresses. If an overstress problem is likely, a heavier pile section is required. The wall thickness or pile size is increased until a pile section is found that will not likely be overstressed. This type of analysis allows specifying the proper size pile in the bid documents. The other option, not as desirable, is to wait until the equipment is submitted for approval by the contractor and then find out that a heavier pile is required. This last situation often results in requiring an order-on-contract for a change in pile size.

WEAP also is used during design for concrete piles. An analysis is performed to check tensile and compressive stresses. This may result in specifying limits on hammer size or type or specifying certain thicknesses or kinds of pile cushions.

The most common and routine use of the wave equation is during the construction of a project. The state requires that

New York State Department of Transportation, Soil Mechanics Bureau, 1220 Washington Avenue, Albany, N.Y. 12232.

the contractor submit his or her hammer-pile system for approval. Part of the approval process is the use of WEAP. It is used to check the ability of the hammer to drive the pile without overstress. A blow count is calculated for the inspector to use. In the case of diesel hammers, a means to determine that the hammer is working at the proper stroke is established. This could be either a blow rate (single acting hammers) or a bounce chamber pressure (double acting hammers). This procedure ensures that a minimum energy is used in driving the piles.

Again with concrete piles, the proposed pile cushion is checked. This is usually a requirement to limit tensile stresses, because a very minimum amount of cushion is necessary to guard against compression stresses.

Dynamic testing is used for a variety of reasons. It is used not only to determine capacity but also to monitor stresses, hammer performance through measurement of energy, and pile integrity and to determine lengths of existing embedded piles and sheeting. NYSDOT uses the pile driving analyzer made by Pile Dynamics Incorporated. The model of the analyzer currently used is GC, with many previous tests made using models EB from the mid-1970s and GA from the early 1980s.

A dynamic test begins by attaching strain transducers and accelerometers at the top of a pile by tapping in or bolting. Electric leads connect the instruments to the analyzer. The contractor then drives the pile with an impact hammer. Each blow creates stress waves that are picked up by the instruments and sent to the analyzer. The analyzer uses wave theory to evaluate the blow and show or print the results. The results, in part, can include resistance, energy transferred to the pile—as well as maximum and minimum forces—and velocities.

CAPWAP was developed as an adjunct to the Case Western Reserve piling program that produced the pile driving analyzer. This is an interactive computer program used to model the soil and how it acts on the pile. The digitized data from one blow from the dynamic test are input into the computer via a modem. This blow is then analyzed.

The majority of soils in New York State produced problems during dynamic testing. These problems are usually a result of dynamic resistances, or large toe quakes. Therefore, it is necessary to perform CAPWAP analysis on most of the dynamic test results. This ultimately allows a refinement of the damping parameter used during dynamic testing and results in improved confidence in the capacity predictions made during design.

CASE HISTORIES

To identify specific ways to use dynamic testing of piles in actual field situations a number of case histories are presented. The examples are situations or problems that presented themselves for interpretation and decision making. The cases are in four groups: pile capacity, pile stress, hammer energy problems, and existing piles.

Pile Capacity

Determining soil resistance is the primary reason for performing most dynamic tests. The capacity prediction cases have been divided into three groups, to show how

- Pile length can affect capacity (two cases),
- Setup is very evident in some soils (one case), and
- The dynamic test can allow a lower safety factor (two cases).

A project at Schroon Lake in the Adirondack Mountains shows how pile length affects pile capacity. A dynamic load test on the first estimated 50-ft-long, 35-ton CIP pile indicated capacity was reached in the medium dense sandy soil at 50 ft. As pile driving progressed on other piles, driving became harder and harder. A review of the driving records and the dynamic load tests indicated that, as driving progressed, piles began attaining capacity at 40 ft, and later on at 30 ft in some locations.

A static analysis shows that for each 1 degree increase in soil friction angle the pile should reach capacity 10 ft shorter. A 32-degree friction angle resulted in a 50-ft-long pile, 33 degrees in a 40-ft-long pile, and 34 degrees in a 30-ft-long pile. The conclusion is that the medium dense sand was becoming more dense from either the volume displaced by the CIP piles or the vibrations generated by pile driving, or both.

Another example of determining the required penetration is a procedure used on a bridge over the West Canada Creek. Here the profile consisted of 30 ft of medium compact silty sand underlain by 60 ft of soft lacustrine soil over a very compact till. Although the bearing capacity of the soil was sufficient to support a spread footing, piles were required to protect the structure against scour. However, it was not necessary to drive to the till layer if piles could be terminated in the silty sand layer. To accomplish this it was decided to use tapered monotube CIP piles. The final length was determined by dynamic test.

The piles were instrumented and monitored throughout the driving. As would be suspected, the resistance steadily increased. At about 5 ft short of estimated length the required capacity was achieved. The pile driving continued, and about 2 ft deeper the capacity peaked and began to drop off. This was to be expected as the pile began to lose tip capacity on entering the lacustrine layer. As it happened the desired capacity was achieved before the capacities peaked.

An example of determining setup is at a bridge over the Oswegatchie River. The foundation was to have 35-ton 12-in.-diameter CIP piles driven into a loose sandy silt soil. Pile lengths were estimated at 50 ft. WEAP indicated that 28 blows per ft (BPF) would result in an ultimate resistance of 140 kips. Two dynamic tests showed that the piles missed achieving capacity, with a capacity of 105 kips and a safety factor of 1.5 at a blow count of 10 BPF. Below a depth of 50 ft, the soil changed to a clayey silt. The capacity of the pile in this layer dropped to 60 kips. On reaching another sand layer 20 ft deeper, the capacity increased again. Piles were stopped at 50 ft, and another dynamic test was scheduled on a retap of the pile. Without a dynamic test, the pile would have been driven an additional 20 ft to reach blow count. Results of a dynamic test indicate that the required capacity was achieved.

With the dynamic testing results, confidence in pile capacity is greatly increased. Although NYSDOT's procedure calls for piles to be driven to a capacity of two times the allowable load, occasionally a lower safety factor is used in construction, because of this confidence. For the Mohawk Street Bridge over the New York State Barge Canal, 45-ft-long, 50-ton, 12-in.-diameter CIP piles were being driven into a silty sand with some clayey silt layers. WEAP results indicated a blow count of 77 BPF, corresponding to a capacity of 100 tons. The first pile was driven to 75 ft before reaching this blow count. The contractor was instructed to drive one pile to the estimated length with a dynamic test performed. The blow count at 45 ft was 20 BPF. The retap test at 45 ft showed that capacity was reached, with a first inch blow count of 6 blows per inch (BPI). This is close to the WEAP results. Therefore, all the piles were driven to 45 ft.

There were only 26 piles to be driven in this last substructure. By the time the dynamic test was performed, almost all the piles had been driven to the estimated length. By the next day, driving was done, and the piles were poured and capped soon after. A review of the dynamic test data in the office, however, turned up a problem. Due to operator error, incorrect transducer calibrations were used. This occurred with the old model EB analyzer, with which the oscilloscope cannot be directly used to visually check the incoming information. The result was that the true capacity of the piles was 75 tons, resulting in a safety factor of 1.5 on the allowable load. Interestingly, if the piles had been stopped at 38 ft, the blow counts were higher and the set-up capacity may have been better. The piles were accepted.

Now, with a recent model analyzer the incoming information is verified so this error should technically not occur again. All field results are also immediately reviewed, and operators now send in data from the field for main office reviews. The analysis is usually finished before the return of the field personnel from the field to the office.

Another example of accepting a lower safety factor is the Sunnyside Street Bridge. The soil was a 22- to 25-ft-thick layer of sand over a 75-ft soft clay layer. The static analysis showed that 35-ton, 12-in.-diameter CIP piles could be stopped in the top layer having mobilized a resistance equal to the allowable load with a safety factor of 2.0. The dynamic test, however, resulted in a safety factor of only 1.7. The only way to get a safety factor of 2 would be to drive the piles 95 ft long. A reduced safety factor was used which avoided a considerable added cost.

Pile Stress

Occasionally pile stress is of concern, particularly with short H-piles to rock, as well as all concrete and all timber piles. Often a dynamic test is required to check on pile stress as well as pile capacity. Pile stress is verified as a matter of course on all dynamic tests. Three cases are presented to show how

- Overstress can be controlled with testing,
- Through testing, pile damage can be identified, and
- Hoop stress can be monitored.

Dynamic testing was used primarily for overstress control at a project at the Waterloo Prison facility. There, 10 by 42 H-piles were intended for an allowable load of 45 tons being driven 13 ft to 30 ft to rock through a silty clay soil. It has been common practice in driving piles to rock to drive to 20 BPI, a general termination criterion. When the driving system is marginal, a minimal increase in capacity will result in a large blow count increase. This condition is accounted for by specifying a lower refusal criterion on a project-by-project basis. This criteria is based on the WEAP results.

On this project, two test piles were driven to a 20-BPI criterion. The first pile was 30 ft long and was driven to a capacity of 200 tons determined by the pile driving analyzer. This resulted in a maximum stress of 28 kips per square inch (ksi). The second pile was 15 ft long. It reached an ultimate capacity of 200 tons; but as a full 20 BPI was reached, overstress occurred and the top 1 ft of pile was damaged. The dynamic test indicated a stress of 45 ksi at a location 2 ft down from the top of the pile. A review of the dynamic test results showed that as long as the pile was driven only to 3 to 5 BPI, damaging the piles was unlikely, and the pile capacity increased minimally beyond that point. Of the production piles, the short piles were driven to the reduced blow count.

A project in New York City serves as an example of detecting pile damage. H-piles were required to penetrate through a bouldery till to rock. Most of the 70-ton-allowable-load 12 by 53 H-piles were driven to the estimated length of 50 ft and reached termination criteria. Low headroom required that only 24-ft sections of pile could be placed at a time, with subsequent sections spliced on. One pile showed an increasing penetration resistance up to 50 BPF at 50 ft. At that depth the blow count dropped off to 10 BPF. Driving was stopped at 103 ft, without reaching the required penetration resistance. The field personnel thought the pile may have dropped into a bedrock valley, which is possible in portions of New York City. Dynamic measurements indicated that the pile was damaged at the splice location 40 ft down from the head of the pile. A second pile went 74 ft before reaching termination criteria. It had a splice at 24 ft. The remaining piles reached termination criteria at 50 ft. Apparently the piles hit boulders on the way down and deflected enough not to reach the 50-ft depth of bedrock. Where is the tip of that 103-ft pile?

Another stress that has been identified is a hoop stress. It is monitored indirectly in concrete cylinder piles as a compressive stress. Hoop stresses come about from the radial component of the compressive stress acting outward from the pile wall. In order to monitor these stresses the allowable tensile stress permitted by the spiral reinforcing in the cylinder itself must be found. This varies with the pitch and diameter of the spiral wire. Once the allowable tensile stress is calculated, then an appropriate value of Poisson's ratio will translate that into an allowable compressive stress. This allowable compressive stress will be considerably less than that which is normally found in the literature for driving concrete piles. This stress is unique to hollow piles. However, it is reasonable to assume that the same circumstances may exist in square and octagonal piles, which have voids cast in them to reduce their weight. For square and octagonal piles, detection and monitoring become more difficult because the void does not exist at the top of the pile as with the cylinder. The cylinder usually exhibits longitudinal cracks at the top since this is the area of highest compressive stress. However, with the American Concrete Institute (ACI) standard precast prestressed piles with hollows, the stress picture becomes more complicated because of the nonuniform cross section. This complicates the compressive stresses, as well as where the cracks will appear. If it appears that hoop stresses may be a problem, it may be advisable to dynamically monitor the pile by attaching the instruments at the top of the void and below the solid cross sections.

Hammer Energy

Hammer problems occur more often than many engineers realize. A decrease in hammer energy can trick a blow count pragmatist into being convinced of a good pile capacity. A dynamic test gives a capacity of the pile and the energy delivered to the pile.

Dynamic tests work only when there is a sharp impact. A dynamic test was scheduled for the piles supporting a 90-ft-span bridge over Hemlock Creek. The 90-ft-long, 35-ton CIP piles were to be driven with a Link-Belt 440 diesel hammer into a 120+ ft deep lacustrine clay and silt deposit. The graphical traces of force and velocity indicated a large cushion effect, which showed that the force had a shaky rounded top and no peak at impact. The diesel hammer was preigniting, and although it was able to drive the piles, the dynamic load test results were meaningless.

Coincidentally, the blow count was a little higher than the wave equation predicted, indicating a good pile. In order to verify the pile capacity, the contractor was asked to change hammers. He willingly did, since he knew the hammer was not working well, and the retap with an air-steam hammer showed the pile did have capacity.

Existing Piles

In addition to the normal dynamic testing, NYSDOT has used the pile driving analyzer to determine the length of sheet piling in the ground. This was done on two occasions. The first was on a number of sheet-pile check dams in a stream in Erie County just south of Buffalo. The designers were concerned that the sheeting was not long enough to withstand any additional scour. The pile driving analyzer was used as an integrity testing device to measure the time of travel down the pile and back and thus determine the length of pile in the ground. Another case was in the Bronx, New York, where the NYSDOT was planning to install a new trestle next to an existing bulkhead. The question arose as to the depth of the sheet-pile bulkhead. This was a concern in making a decision as to whether to replace the bulkhead or whether, if it was left in place, it would interfere with inclined piles. In both cases the lengths were found to be adequate.

Another use of the analyzer has been in the testing of existing piles. Many of NYSDOT's larger, more recent projects have been in upgrading interchanges to handle larger traffic volumes and widening Interstate highways in urban areas. Both deal in improvements adjacent to existing structures. On occasion, this involves building directly over an existing foundation. Because piles are already present, it is occasionally proposed to reuse some of the existing piles in the foundation. However, in order to do this the capacity of the piles has to be verified. This situation has been present on two Interstate projects in Syracuse. The piles for these bridges were installed in the late 1950s. They were installed as mandrel-driven thin wall shells. The piles were installed to twice the blow count determined by the *Engineering News* formula. Therefore, they were driven to many times the required design load, with this very efficient driving system. All of the piles were driven into either a till soil or decomposed shale.

On one project the foundations were to be CIP piles, the same as the existing foundations. This required the existing piles to retain the hook bars for attachment to the reinforcing steel in the cap. Dynamic testing required straightening the hook bars and using a follower to drive the pile. The follower consisted of a piece of $^3/_8$-in. wall pipe, with a splicing collar on the bottom. A plate was attached into the collar, fitted with holes to accommodate the straightened hook bars. Beneath the plate on the pile head was fitted a cushion of $^3/_4$-in.-thick plywood. The pile-driving hammer for the redrive was the same one used for the project piles. This was a Link-Belt 440, a closed-end diesel hammer. This appeared to be a very reasonable driving system. However, as the test proceeded it became obvious that the hammer was not powerful enough to mobilize all of the soil resistance supporting the existing pile. On returning to the office, and examining the data further, it was found that a great part of the energy was in fact lost in the follower system, partially because the pile being redriven was so much stiffer than the follower.

Another project used H-piles as the foundation for the new structure. However, since the footprint of the new bridge foundation overlapped the existing foundation, it was decided to reuse some of the piles. The existing foundation was higher in elevation than the proposed foundation, therefore the hook bars could be eliminated.

With the hook bars cut off, the need for a follower was eliminated. The hammer for this project was a Link-Belt 520 closed-end diesel. However, since the existing piles were embedded in the decomposed shale it was difficult with that hammer to move the pile sufficiently to mobilize all of its capacity. The lower-bound prediction was, however, sufficient to indicate that the piles had a satisfactory capacity to be reused.

The lessons to be learned from these experiences are the following:

- There is a potential for the reuse of existing foundations when the opportunity arises.
- When testing existing piles, make it clear in the contract documents that a larger pile-driving hammer may be necessary for the redriving and testing of the existing piles. It may even be necessary to specify a minimum hammer energy for this purpose.
- If it is necessary to use a follower, it should be designed such that its impedance (AE/c) is as close to the existing pile being tested as possible. This also may have to be specified in the contract documents.

The way to avoid the use of the follower is to have the hook bars cut off and drill and grout new ones in after the test. The other option is to choose an existing pile that is not in the footprint of the new foundation and can be tested and abandoned. This last option is the game plan for an upcoming test to be performed in Utica.

SUMMARY

NYSDOT's intent is to avoid and control pile problems. The pile program has the same level of concern with the pile-driving stage as there is in the design stage. The dynamic test is used to verify capacity predictions made by the static analysis and the WEAP. The strength of the program is the continuity between design and construction. The designer designs the pile, runs the wave equation analysis, and goes out in the field to dynamically test the piles.

WEAP is used on all pile projects. Dynamic testing is done on special projects that have peculiar or unique soil conditions or when it is difficult to model soil parameters. Testing is also done on projects with field soil conditions different from those assumed in design and to troubleshoot pile driving problems.

The long-term objective is to be able to modify both computation stages, static design and the wave equation analysis, through the CAPWAP results, as a way of bettering the designs. Examination of some factors like H-pile tip bearing capacity in soil and correlating damping and quake values with soil parameters has begun. Using CAPWAP makes these studies practical.

Publication of this paper sponsored by Committee on Foundations of Bridges and Other Structures.

Discussion of Procedures for the Determination of Pile Capacity

RICHARD L. ENGEL

The Ohio Department of Transportation (ODOT) uses the following methods in determining pile capacity: static analysis, *Engineering News* driving formula, static load tests, dynamic load tests, wave equation program, and Case pile wave analysis program. The ODOT's procedures in using these methods are discussed in this paper along with a case history. A preview of responses from a survey on pile testing and installation practices is also presented. The need to load test and to document load test results is emphasized.

In light of today's technological advancements, which include static analysis and wave equation methods performed by computerized procedures and the use of relatively reliable dynamic-load-testing equipment, why is it still acceptable to support some highway structures on piles having unknown load-resistance capabilities? Why is the *Engineering News* (EN) pile-driving formula still a widely used method for predicting pile capacity (*1*)? Topics relevant to these questions as well as pile foundations as used by the Ohio Department of Transportation (ODOT) are subjects addressed in this paper.

In reviewing pile design and installation procedures as they are currently being used, it can be concluded that there is a need for a knowledgeable organization to prepare updated pile design guidelines and construction specifications. As is the case with most research efforts, the use of quality data obtained from full-scale models is important if accurate mathematical expressions are to be developed. A standardized load-test data base should be developed and made available to all parties interested in pile-resistance prediction methods. Note that much of the information provided herein is being offered not because of its correctness, but rather as documentation of the disarray of existing design and construction methods.

ODOT's PRACTICE

Piles generally used for the support of ODOT's structures are 40 to 70 ft long HP 12 by 53 steel "H" sections or 12-in.-diameter steel pipe piles that are filled with concrete after installation.

The chronological events that are generally followed for the design of a pile foundation are

1. Prepare a subsurface profile from soil sampling and testing information.
2. Determine pile type and estimated length necessary to develop the design load resistance.
3. Provide plan pay items for static load tests and dynamic load tests as per the ODOT guidelines.
4. Install piles to a length that satisfies the specification EN formula.
5. After obtaining initial driving experience at the structure site, decide if static or dynamic load testing, (or both) should be conducted.

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Estimated Pile Lengths

The estimated pile lengths provided on project plans are derived by using engineering judgment, "as built" records from similar foundations, and the results of static analyses performed in accordance with the Federal Highway Administration's *Manual on Design and Construction of Driven Pile Foundations* (2). The plan-estimated pile lengths serve two functions: to compute the total bid quantity so a unit price per linear foot can be established through the bidding process and to flag potential problem situations in which pile load tests may be appropriate because there is an excessive difference between driven lengths and estimated lengths.

The contractor is given the responsibility of determining the lengths of piles to be ordered. Since the contractor is not specifically paid for pile splices or unused pile lengths, his or her profits are directly related to the ability to furnish pile lengths compatible with the state inspector's driving requirements.

Guidelines offered in the Standard Specifications by the American Association of State Highway and Transportation Officials (*3*) recommend that the furnishing of piles and the driving of piles should be separate pay items. This may be a more equitable procedure for the contractor and would also give the designer more incentive to be accurate in estimating pile lengths. More emphasis might then be given to conducting static or dynamic load tests (or both) during the early stages of projects for the purpose of obtaining measurements that would be useful in establishing appropriate pile-order lengths and pile-hammer limitations.

Dynamic Driving Formula

All piles that are not driven to refusal on bedrock are required to be installed to a penetration that satisfies the specification EN blow count (Appendix A). The EN blow count criteria may be modified if information is obtained from a load test. Although the EN formula contains a theoretical factor of safety of six, it has been found that when the hammer energy is such that the computed required blow count is in the range of 30 to 70 blows per foot, the installed piles often have a failure resistance that is reasonably close to providing a safety factor of two. Piles that are installed in cohesive soils may require time for developing set-up resistance in order to achieve the safety factor of two.

Static Load Tests

Static load tests generally are conducted only on piles for relatively large projects (Appendix B). During the past 20 years, 90 percent of the projects that had static load test pay items provided did not have these items performed during construction. The number of documented static load tests that have been performed on ODOT projects is shown in the barchart in Figure 1. The rationale for permitting the nonperformance of load tests was that the conditions of the pile installations were typical, the expense of the load test could be saved, and the progress of the project would not be delayed.

Dynamic Load Tests

The use of dynamic load testing methods for the prediction of pile capacity has been available to the ODOT since the mid-1970s. A pile-driving analyzer (PDA) became the property of the ODOT at the conclusion of a pile-capacity research project conducted at Case Western Reserve University. In 1982, a PDA (model GA) was purchased by the ODOT with assistance from FHWA.

Typical steps that encompass a dynamic load test (Appendix C) are as follows:

1. The project engineer furnishes an advanced notification of the contractor's proposed pile-driving schedule to the central office construction engineer. This notification is provided to enable the testing personnel to plan their work activities around a potential testing date. Dynamic pile tests are conducted by two engineers in the foundation section of the Bureau of Bridges.
2. After two piles have been installed, the driving logs are reported to the foundation engineer of the Bureau of Bridges. The subsurface conditions are reviewed, and the driven pile lengths are compared to the estimated pile length. If the pile installation behavior is judged to be typical, the dynamic load test is generally nonperformed. For relatively large projects, the dynamic load test is almost always conducted.
3. When a dynamic load test is to be conducted, the project engineer is given instructions for installing the piles that are to be dynamically load tested. If the subsoils are cohesive and it is appropriate to attempt to determine the magnitude of setup, the dynamic testing is delayed as long as is practicable so that the driven piles can gain resistance (setup) before being subjected to a restrike.
4. On the day that the test is to be conducted, the testing team generally arrives at the project site between 9:00 a.m. and 10:00 a.m. The pile-driving logs are examined and a testing scheme is then developed.
5. Piles are made ready for testing by drilling holes at appropriate locations in the piles and, if necessary, threads are cut in the sides of the holes with a tap. Transducers are then attached to the pile after the hammer has been positioned on the pile. Computations are performed using specific pile property values to determine input data for the PDA. The PDA, oscilloscope, and data tape recorder are appropriately situated and all required interconnections are made.
6. As the pile being tested is driven by the contractor's pile hammer, dynamic measurements from the strain transducers and accelerometers are processed by the PDA. The information generated by the PDA is reviewed as the pile is being driven.
7. After three piles are dynamically tested, the test results are assessed and the project engineer is given pile installation instructions, which generally consist of a minimum blow count or a minimum pile penetration requirement.

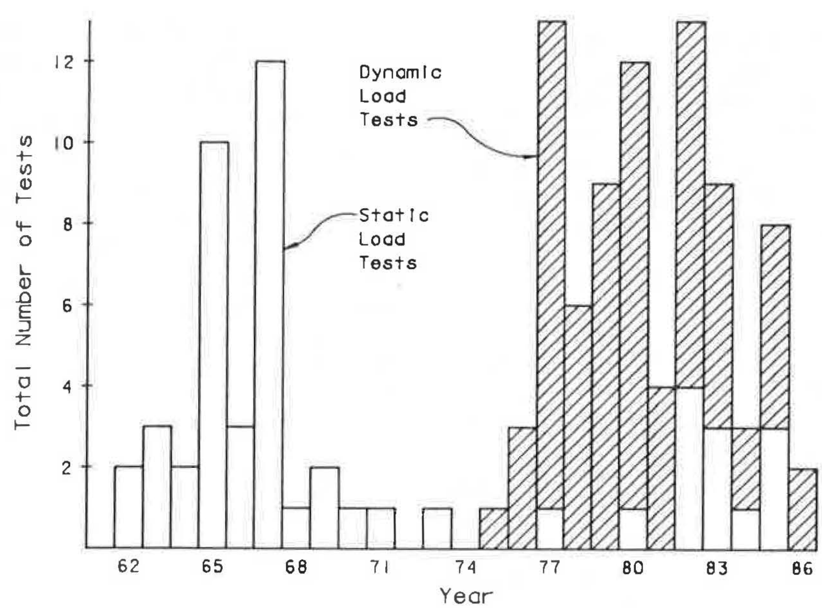

FIGURE 1 Load tests per year.

8. A dynamic load test report is prepared by the Bureau of Bridges' foundation engineer and submitted to the Bureau of Construction.

A common situation that the ODOT has encountered when conducting dynamic load tests is outlined as follows:

1. Piles for a bridge substructure are being installed to a penetration depth that is substantially different from the plan-estimated pile length.
2. In the course of conducting dynamic load tests the piles are driven to a high blow count (over 120 blows per foot).
3. The results of the dynamic load tests indicate that a factor of safety of at least 1.3 can be predicted from the PDA measurements. This predicted pile capacity is probably conservative because the pile hammer is not adequately mobilizing the test pile.
4. The subsoil conditions at this site are such that the pile is expected to gain resistance as time passes (setup).
5. To put this situation in perspective, note that the project is relatively small with only 80 piles, each having a 100-kip design load and estimated to be 45 ft long. If a larger pile hammer is required, the contractor would have to make arrangements for furnishing a larger hammer and mobilizing the hammer to the project site.
6. Is the contractor at fault in any respect or was the type of pile load testing inappropriate? A static load test, or possibly a delayed restrike dynamic load test performed after an appropriate waiting period, may have found the pile resistance to be satisfactory. At this time does this project warrant additional testing of any type?
7. The resolution of these dilemmas has generally been to accept the piles. The rationale for accepting the installed piles is that the factor of safety is 1.3 or more and expected to increase with time. A restrike test could cause delays in production and the personnel performing the tests would be required to return to the project site.
8. The project engineer is instructed to install the piles to an appropriate tip elevation since blow count controls are not relevant.
9. Restricting the hammer size to not less than a specified energy is a means currently being used to reduce the occurrences of some of these awkward testing situations. Potential pile load testing dilemmas should be anticipated and eliminated during the plan design stage by preparation of thorough pile installation specifications. Wave equation controls may be beneficial when attempting to avoid this situation.

Upon reviewing the results of 60 ODOT dynamic load test reports, the following patterns were found:

- On 20 projects, the project engineer was instructed to continue installing the piles as per the specification EN formula. The subsoils at these projects are generally nonplastic.
- On 20 projects, the specification EN pile-driving blow count was required to be increased by 10 to 60 blows per foot. The subsoils at these project locations contain various percentages of cohesive material.
- Most of the remaining projects consisted of restrikes on previously driven piles and a comparison to the testing results of a freshly driven similar pile. Blow count or minimum penetration requirements were then based on measured set-up values.

When consideration is being given to purchasing the PDA and accessories, consideration must also be given to providing personnel for operating the equipment. Usually, an experienced foundation engineer must be present at the site to see that the testing is conducted on piles driven to depths and blow counts that will provide data appropriate for use in establishing driving criteria. The personnel performing the dynamic load tests must have a sincere interest in doing this type of work. Carelessness in conducting dynamic load tests will lead to poor quality measurements. The field portion of dynamic pile testing sometimes requires physical effort that may have to be performed during inclement weather and under dirty working conditions.

The ideal way to perform dynamic load tests is to have trained personnel whose first priority in work assignments is to conduct pile tests. A mobile van or a temporary shack at the test site equipped with shelves, a stool, and a table is suggested for providing a convenient testing environment.

As shown in Figure 1, the ODOT has conducted between 2 and 12 dynamic load tests per year. Approximately 50 bridges are constructed each year that have dynamic load test work items (Appendix D). Most of the load tests are not performed because they are deemed unnecessary or because the testing personnel are unavailable because of commitments to perform their other duties. In an effort to ensure that more dynamic load tests can be conducted, the ODOT is in the process of entering into an annual contract with a dynamic load-testing consultant who will be available as a substitute for the ODOT testing personnel. The consultant would be paid a flat fee (predetermined by bid) per day for testing. The content of the contract will be similar to the plan note provided in Appendix E except that the testing consultant will be reporting directly to ODOT rather than working through the contractor.

Case History

The project that is the subject of this case history is the West Third Street bridge reconstruction located adjacent to Cleveland Municipal Stadium. The foundation for this bridge is 0.25-in.-wall, 14-in.-diameter, closed-end steel pipe piles having a 140 kips design load. The plan-estimated pay length for the piles is 80 ft. The subsoil is very stiff silty clay.

The piles were initially installed to a penetration of 50 ft with a Foundation Equipment Corporation Model 1500 (FEC 1500) pile hammer. At 50 ft of penetration, the blow count per foot was 60. In order to reduce the time of driving and to increase the contractor's ability to install the piles to an appropriate depth, on May 21, 1987, an FEC 3000 was used to install the piles from a penetration of 50 ft to 100 ft. The blow count per foot with this hammer was 35 at 51 ft and 65 at 100 ft of penetration.

After installation of the piles to 100 ft, a static load test was conducted on May 29, 1987, and the failure load was determined to be 200 kips (1.4 factor of safety, 8 days for setup). Piles driven into the silty clay subsoils at this bridge location have a history of gaining strength (setup) during a time period of approximately 2 weeks following pile installation.

On June 3, 1987, a dynamic load test was conducted for the purpose of obtaining additional information that would be used for establishing the appropriate driving requirements. An anchor pile, installed to the same penetration and to a similar blow count as the pile tested statically, was load tested using the dynamic methods. Measurements were obtained while driving the pile from a penetration of 100 ft to 110 ft. The pile's failure load resistance was predicted to be 260 kips at the beginning of restrike and 200 kips at the end of restrike. The 60 kips that were dissipated during the restrike driving are assumed to be an approximate measure of the set-up resistance for a pile in these subsoil materials. A Case damping factor of 1.0 was used by the PDA in the field when determining the pile's static bearing capacity and was later confirmed by the Case Pile Wave Analysis Program (CAPWAP) performed by a consultant.

The measurements found by the static load test and subsequent dynamic load test indicated that approximately 60 kips or more of set-up load resistance should be expected to develop by the piles installed in the subsoils at this bridge site. The contractor was instructed to install all piles to a penetration of 115 ft. The developed pile resistance at this penetration is anticipated to satisfy the required failure load resistance of 280 kips (safety factor of two).

The piston weight for the FEC 3000 is 6,600 lb and the estimated maximum stroke during driving was 4 ft (26,400 ft-lb). The maximum energy transferred to the pile during driving was determined by the PDA to be 19,000 ft-lb (70 percent transferred energy to the pile). The maximum driving stress occurring in the pile at the location of the gauges, as determined from the measured compressive force, was 25,000 lb/in.^{2}. CAPWAP analysis results for a blow occurring at approximately 102 ft penetration found the failure load resistance to be 250 kips (170 kips shaft resistance and 80 kips base resistance).

A similar magnitude of set-up resistance was found at a nearby bridge site having a pile foundation consisting of 200,000 linear ft of 14-in.-diameter closed-end pipe piles. Eight special static load tests had recently been conducted. The static load tests were considered special because each test pile was loaded at 3 days and reloaded 14 days later for the purpose of verifying the amount of set-up resistance. The pile penetrations were approximately 80 ft and the failure loads were approximately 200 kips at 3 days and 260 kips at 17 days.

Hammer Energy

In 1955 the ODOT organized a hammer energy study using full-scale equipment. The study compared the driving capabilities of single acting air-steam hammers with single acting diesel hammers. The conclusion drawn from the comparison of driving similar piles with these hammers was that if 70 percent of the diesel hammer energy, as recommended by the manufacturer, is used for "E" in the EN formula (Appendix A), the predicted pile capacity for both hammer types would be similar at equal pile penetrations. Although this was a crude and limited study, for the past 32 years minimum required blows per foot have been computed by using this reduced hammer energy rating for all diesel hammers.

When conducting dynamic load tests the ability of the pile hammer system to transfer energy to the pile can be determined. The measurements obtained during the ODOT dynamic load tests have found the transferred energy to vary from 25 to 70 percent of the manufacturer's rating.

On two occasions when dynamic load tests were conducted, the energy transferred from the pile hammer was found to be low and inconsistent. The contractor was instructed to replace or repair his pile hammer. After examining the hammers in the shop, the contractors confirmed that mechanical deficiencies were restricting the performance of the hammers. A trained pile inspector may be capable of detecting that a pile hammer is malfunctioning, but generally a confrontation develops when the pile inspector raises questions concerning performance of a contractor's pile hammer. Contractors have not contested a single request by ODOT for hammer repairs or hammer replacement, when measurements are available from the PDA.

During the past 30 years, the ODOT's typical pile design loads have increased from 70 kips up to 120 kips. This increase in design load reflects a need for contractors to provide larger pile hammers that can develop adequate energy for installing piles to greater penetrations for developing the required resistance.

The ODOT has found that the reliability of the specification EN formula (Appendix A) in predicting the correct pile capacity diminishes as the blow count increases beyond 70 blows per foot. At the ODOT it has not yet been decided to undertake the efforts necessary to incorporate the wave equation into the construction program, therefore a plan note specifying a minimum hammer energy rating has been used for the past 2 years as a means to ensure that a reasonably appropriate hammer size will be furnished by the contractor. The required minimum hammer energy is chosen so that its corresponding blow count for the plan pile design load resistance does not exceed 70.

SURVEY

A survey entitled "Pile Testing and Installation Practices" (*4*) has been conducted by the ODOT. Thirty-eight states have furnished their response and a summary of the harvested information is offered herein.

Load Tests

The following list of statements are criteria used for determining when pile load testing should be conducted:

- There must be over 100 piles and the design load must be relatively high.
- There must exist a potential for a cost savings.
- There must be in excess of 10,000 linear ft of piling and the design load must be relatively high.
- At least one load test is to be conducted per structure.
- At least one load test per structure is to be conducted when friction piles are installed in cohesive soils.
- A load test is necessary when soil conditions are such that a static analysis may not be reliable.
- When piles are installed in soils that contain boulders that can damage driven piles, a load test is necessary to verify the integrity of the piles.

- After considering the information in the geotechnical report, the magnitude of the pile design load, previous experiences, the pile proposed length, and tolerable settlement limitations, a determination is made.
- Piles that are installed in rivers must be tested for capacity.
- A load-test data base is available for reference. If a structure is to be built where there are no records of a nearby load test in similar soils, a load test is required to verify capacity.
- Load tests are not specified. However, construction personnel may require load tests when the test is considered to be beneficial.
- Load tests should not be avoided just because there have not been any problems with pile foundations.
- Load tests are needed when the design load is significantly higher than the typical design loads.
- Many states have not developed criteria for requiring load tests.

The goal adopted by most states is to attempt to install piles to a failure resistance equal to twice the design load. In general, pile design loads that are currently being used by departments of transportation vary from 80 kips to 300 kips. Nine of the states sometimes require a safety factor of three. Eight states require the contractor to attempt to load test the piles to failure. Ten states require that a load cell be used to measure the applied load.

Dynamic Load Tests

Nine states indicated that they have purchased the PDA for pile testing. Four other states plan to purchase a PDA in the near future and five states have had piles tested dynamically by consultants.

Typical bridges having pile foundations were described in the questionnaire and each state was asked to indicate its recommended type of load testing, if required. Five states preferred that dynamic load tests be conducted, eight states would perform a static load test, five states would require both types of testing, and twenty states would not require any load testing.

The following is a list of positive and negative comments addressing the use of the PDA:

Positive

- Testing can be performed quickly.
- It is less expensive than a static load test.
- Since static load testing is seldom done, the determination of the capacity of the piles by dynamic load testing will provide confirmation of capacity, which may allow for a reduction of typical pile lengths.
- It is primarily used to ensure that a proper amount of hammer energy is being provided.
- Stresses that occur in the pile during driving can be monitored.
- It is useful on problem projects for aiding in interpreting unusual driving conditions.
- It can be used to obtain field measurements that can be compared with the wave equation analysis of piles output values (WEAP 86).
- Pile damage can be detected during installation.

Negative

- CAPWAP is required in order to get reliable results.
- Predictions are too conservative for hard driving conditions.
- Long-term pile creep data cannot be obtained.
- It is difficult to maintain a staff of trained PDA operators.
- It delays the contractor's operations.
- Personnel conducting the test may sometimes be subjected to hazardous working conditions.
- It is less reliable than a static load test.
- Using CAPWAP delays turn-around time for providing driving criteria to the project personnel.
- There is limited knowledge in the use of damping constants.
- The PDA operator's normal duties are disrupted at short notice when testing is required.

Engineering News Formula

Seventy percent of those responding stated that the EN formula was being used in some manner to control pile installations. Fifty percent agreed with the 1948 Terzaghi-Peck statement regarding the variability of results that can occur with the use of the EN formula and that "the continued use of the EN formula can no longer be justified." The remaining 50 percent defended their use of the formula with arguments such as

- Piles are thought to be installed to a conservative capacity.
- The procedure is relatively easy for field personnel to administer.
- The EN formula is probably as good a method as wave equation or dynamic measurement methods.
- The formula is considered to be relatively reliable.
- When installing piles in granular soils the factor of safety is between 2.0 to 3.0. For cohesive soils the factor of safety is generally 1.1 to 1.4 (may increase with setup), therefore in cohesive soils a minimum tip elevation must be achieved as per a plan requirement.
- The formula is satisfactory provided the person using this method understands its limitations.

Wave Equation

Approximately half of the states responding to this survey have used the wave equation method. Three of the states use the wave equation to determine the required blow count and also to preapprove the contractor's hammer. Most states are presently experimenting with the wave equation to determine to what extent the predictions by the wave equation will affect their current pile installation practices.

DISCUSSION AND CONCLUSIONS

After many years of near stagnation in the development of improved pile foundation design methods to be used by departments of transportation, the past decade has brought about a renewed emphasis in geotechnical engineering. Research projects and workshop programs promoted by the FHWA have been responsible for much of this new activity. Interest is being shown in providing an updated foundation section in the AASHTO Standard Specifications. AASHTO is probably responsible for many of the long-standing policies or lack of policies currently being used by departments of transportation.

Wave Equation

The wave equation is considered to be a more rational approach for determining pile installation blow counts than the EN dynamic formula. On select projects, wave equation blow counts have been determined by WEAP-86 methods (*5*) for evaluating how the wave equation predictions compare to current practice. When the subsoils at a project site consist of granular materials, the wave equation blow count is similar to the blow count required by present methods. Frequently the wave equation results indicate that the pile cannot reasonably be installed to the required resistance by the contractor's hammer (the hammer is too small), although the hammer has been used (successfully?) many times on other similar projects. As shown by the blow counts tabulated in Table 1, it should be expected that if the wave equation controls are not correctly used, the contractors will be required to sometimes furnish larger pile hammers than they have become accustomed to providing. When high blow counts are required by wave equation methods, the engineer must consider if the effects of residual stress (*6*) and setup have been properly addressed.

Dynamic Load Testing

As indicated by the responses to the survey, dynamic load testing is available to 18 of the departments of transportation. If their staffs are interested in performing this type of testing and dynamic load testing is designated as a priority work item, a successful practice can be expected to develop. As more private testing companies obtain the capability to perform dynamic load tests, contracting their services may be a more appropriate avenue for using dynamic load test methods.

Dynamic load testing offers the contractor an economical testing method that can be used as a defense against unreasonable driving requirements. Generally, contractors are at the mercy of the inspector when high blow counts and longer piles are required. As contractors become successful in their rebuttals, it may be prudent for transportation departments to improve their pile design and installation practices.

When dynamic load tests are conducted, the equitable payment units are per hour for the use of the contractor's pile installation equipment and personnel and per day for a testing company that furnishes and operates the PDA and related accessories. The contractor may hesitate to fully cooperate without appropriate compensation. Testing consultants do not have total control over how many piles can be tested per day because they are dependent on cooperation from the contractor and they may be restricted by project constraints.

Load Tests

Advancements in the geotechnical engineering profession are now technologically accelerating; however, some of the fuel being used for these advancements is being extracted from a deficient pile-load-test data base. The load-test data base, which should be the genesis of all theory and correlation studies, must be appropriately developed and made readily available. Although hundreds of load tests are performed each year, those who have put out a call for load-test data reports have received only a few responses consisting of generally incomplete information. There should exist a requirement to furnish a standardized load test report that is considered to be an integral part of the load-test work. Final payment for the load test should not be made until the report has been furnished. Some of the basic topics contained in the load test report should be

- Subsurface soil properties,
- Load versus deformation data,
- Interpretation of failure (preferably by standard methods),
- Pile-driving blow-count logs,
- Pile-hammer system performance information,
- Wave equation analysis,
- Static analysis computations,
- A discussion of set-up resistance,
- PDA prediction of capacity and other pertinent measurements (when available), and
- CAPWAP analysis results (when available).

The format of the report must be developed so that all information can be easily transferred into the standard computerized data bank.

One standard static load test procedure should be used so that consistency is used in determining the load-deformation values. A reasonable attempt should always be made to load the test pile to a plunging failure condition. A reloading of the test pile for determining the magnitude of setup should be done whenever practical. A load cell is needed to measure the

TABLE 1 BLOW COUNTS FROM WAVE EQUATION AND ODOT EN

DELMAG D-12			240 KIPS (SF=2, 60 TON DESIGN LOAD) 14" CLOSED-END PIPE PILE				
MFR'S RATING	ODOT RATING	WEAP-86 RATING	ODOT EN BLOW COUNT	WEAP-86 SMITH DAMPING			
				SIDE = 0.05 TOE = 0.15	SIDE = 0.2 TOE = 0.15		
				SAND	CLAY	CLAY RESIDUAL STRESSES	CLAY 192 KIPS 48 KIPS SET-UP
22,500 FT-LBS.	16,500	18,870	68	68	272	87	88

applied load. Only when a load cell is being used can the accurate calibration of the jack be verified. In the past 5 yr ODOT has experienced two pile load tests using jacking systems that were furnishing totally erroneous loading values. Other load tests have experienced a 10 percent difference between load cell and jack pressure readings.

There should exist some standard rational guidelines for determining when to load test. Basing the need to load test on criteria such as the total number of piles or the number of feet of piles to be installed or the magnitude of the design load permits many bridges to be constructed without load testing controls. Engineers should be capable of producing documentation that offers evidence as to how the pile load resistance was determined for each project.

After a detailed load-test data base has been established, a means then exists for studying the relationships between full-scale pile load tests, static analysis methods, wave equation predictions, dynamic load test predictions, soil strength parameters, and field sampling techniques.

There is a trend toward varying the factor of safety from two to three depending on the confidence level of the controls used for the installation of the piles. When using a safety factor of three for pile designs, the consequences of additional costs that may be encumbered because of a need for a bigger pile section, longer piles, and a larger pile hammer must not be overlooked. Often the use of a load test and a safety factor of two will be the economical procedure for the construction of a pile foundation. Note that conservative methods used in superstructure designs generally will result in extending the life span of the superstructure. Conservative pile foundation designs are a waste of monetary resources because pile foundations are relatively slow to deteriorate.

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APPENDIX A
ODOT Specification EN Formula

507.05 Determination of Capacity

The safe bearing value (R) of a driven pile (considered as a single isolated pile) shall be determined by means of the following capacity formula, unless this formula is modified as a result of a static load test or a dynamic load test: For a single-acting, differential-acting, or double-acting, steam (or air-operated) hammer, or a diesel hammer

$$R = 2F/(S + 0.1) \tag{1}$$

where

- R = safe bearing value, in pounds (corresponding with the design load resistance per pile called for on the plans). By using this formula the piles are assumed to be driven to a failure load that is two times the design load.
- W = weight of striking parts of hammer, in pounds.
- H = height of fall of striking parts, in feet.
- F = WH for single-acting steam hammer, in foot pounds, or
- F = approved rated energy of hammer in foot, pounds. For a differential-acting or double-acting steam or air-operated hammer, the manufacturer's rating is used. For diesel hammers, 70 percent of the manufacturer's rating is used.
- S = penetration, in inches per blow (generally determined from the rate of penetration for the last several inches of penetration).

APPENDIX B
When to Furnish Static Load Tests as Bid Items

A static load test item should be included in the structure's estimated quantities if the design load for the piles is 90 kips or more, except as follows:

- A static load test is not necessary if piles are driven to refusal on bedrock.
- A static load test is not necessary if the estimated linear feet of piles is less than 6,000 ft.

Structures that require a static load test item may also require subsequent static load tests as follows:

Estimated Length of Piles (linear ft)	*No. of Subsequent Static Load Tests*
0–10,000	0
10,001–20,000	1
20,001–30,000	2
30,001–40,000	3

APPENDIX C
Construction Specification for Dynamic Load Testing

523.01 Description

This item shall consist of a dynamic load applied by a pile hammer to a pile while transducers obtain measurements for predicting the static capacity of the pile. Waiting periods may be required so that soil set-up and relaxation characteristics can be determined.

523.02 General

The contractor shall notify the engineer of his intent to drive piling at least 3 days before the installation of the first pile. The engineer shall inform the director of the contractor's pile-driving schedule. The director shall determine if the test is to be performed or if some pile-driving experience at the proposed site is to be obtained before a decision can be made. The director will establish a date for the tests and will also determine the location of all piles to be dynamically load tested.

The hammer selected for driving the test-loaded piles shall be used for driving all piles represented by the load-test piles. If the contractor subsequently finds it necessary to use a different hammer, the director will determine if an additional dynamic load test is necessary. Any such test shall be completed at no additional cost to the department.

523.03 Equipment

The contractor shall supply all personnel and equipment required for striking the test pile with the pile hammer. The contractor shall also supply a source of 115-V, 1500-VA, 60-Hz electrical power and extension power cords.

The department will provide the transducers, the Pile Driving Analyzer, and the personnel to install and operate this equipment.

523.04 Test Procedures

Approximately three piles will be tested in one day. Department personnel will drill holes into the piles to be tested so that electronic transducers (two accelerometers and two strain gauges) can be attached. When the transducers have been placed in position and the Pile Driving Analyzer has been made ready to receive the acceleration and strain measurements, the contractor shall strike the pile with the pile hammer as many times as is required to obtain adequate measurements as determined by department personnel.

After the dynamic testing measurements have been obtained and reviewed, the department will provide instructions for driving the piles.

523.05 Method of Measurement

The hours to be paid for under this item will be the sum of the time intervals that the department has requested the contractor to discontinue his normal production pile-driving operation so that the dynamic load tests can be performed. The engineer will measure and record the time needed to perform the tests to the nearest one-tenth of an hour.

APPENDIX D
When to Furnish Dynamic Load Tests as a Bid Item

A dynamic load test item should be included in the structure's estimated quantities if the design load for the pile is 70 kips or more, except as follows:

- A dynamic load test is not required if piles are driven to refusal on bedrock.
- A dynamic load test is not required if the design load is less than 90 kips per pile and the estimated linear feet of piles is less than 1,500 ft.

Estimated Length of Piles (linear ft)	*Estimated Pay Quantity (hr)*
0–5,000	3
5,001–10,000	6
10,001–20,000	9

APPENDIX E
Plan Note for Requiring Dynamic Load Tests by a Consultant

This item is provided to compensate the contractor for using a testing consultant to conduct dynamic load tests on service piles as required by the director. Testing instrumentation and personnel are to be furnished by the testing consultant. The testing consultant's personnel shall have had successful experience in performing dynamic load tests on piles for at least 10 projects. The contractor shall furnish the name of his testing consultant, along with a list of the company's work experiences, at the preconstruction meeting. The director will review the testing company's background and will either approve or reject the contractor's testing consultant.

The testing consultant shall furnish a report summarizing all testing results and stating his conclusions. The report shall be furnished within 1 week after the conclusion of the pile-testing work.

The contractor shall drive piles and assist the consultant during the dynamic pile tests as per Item 523 (Appendix C).

The unit of payment for this work by the testing consultant shall be per day. The testing consultant is expected to test as many piles as practical in an 8-hr workday time period. The testing consultant shall also perform one CAPWAP for each day's work. The contractor will be paid for lapsed time during all dynamic load testing as per Item 523.

Publication of this paper sponsored by Committee on Foundations of Bridges and Other Structures.

Modern Specification of Driven Pile Work

RICHARD S. CHENEY

In the past 20 years, extensive research and implementation efforts have been directed at improved construction control of driven piles. The feasibility of techniques such as wave equation analysis, quick load testing, and dynamic pile testing has been thoroughly evaluated and generally accepted by the engineering profession. The mechanics of the pile-driving operation have been studied with results confirming the importance of controlling the properties of pile driving appurtenances such as hammer cushion, drive head, pile cushion, followers, and so on. Finally, the elements of risk in bidding pile work have been learned through numerous court of claims cases on topics such as estimated lengths, fixed-cost items bid as variable costs, delays in furnishing proper equipment, or required pile lengths. Modernization of current specifications for construction control of driven piling requires philosophical as well as technical changes. Five areas of potential improvement that need to be addressed in detail are specifying ordered pile lengths, using ultimate pile capacity, approving driving equipment, field verifying pile capacity, and devising a pile payment method.

Nearly all pile specifications currently in use by U.S. highway agencies were developed many years ago and have been continually revised within the original format. The resulting specifications often contain a mixture of outdated and state-of-the-art requirements. What is needed is an entirely new modern specification that has state-of-the-art continuity yet allows state-of-the-practice introduction into current procedures. The specification should be restricted only to driven piles because control procedures for installation of drilled shaft foundations are radically different than for driven piles. Five key elements must be included in the modern pile specification to achieve successful implementation and equitable payment for work performed. Those elements are described in this paper with suggested wording (indicated by headings containing the word "Specifications:") for a modern construction specification.

PROVIDING ORDERED LENGTHS OF PILES

The objective of a pile specification is to provide criteria by which the owner can ensure that designated piles are properly installed and the contractor can expect equitable compensation for work performed. The owner's responsibility is to estimate the pile lengths required to safely support the design load. Pile lengths should be estimated on the basis of subsurface explorations, testing, and analysis, which are completed during the design phase. Pile contractors who enter contractual agreements to install piles for an owner should not be held accountable or indirectly penalized for inaccuracies in estimated lengths. The contractor's responsibility is to provide and install designated piles, undamaged, to the lengths specified by the owner. This work is usually accomplished within an established framework of restrictions necessary to ensure a "good" pile foundation. The price bid for this item of work will reflect the contractor's estimate of both actual cost to perform the work and perceived risk. The following statement is an example of how to implement this concept in a pile specification.

Federal Highway Administration, Office of Engineering, Hydraulics and Geotechnical Branch, 400 7th Street, S.W., HNG-31, Washington, D.C. 20590.

I. SPECIFICATIONS: DESCRIPTION

This item shall consist of furnishing and driving foundation piles of the type and dimensions designated including cutting off or building up doundation piles when required. Piling shall conform to and be installed in accordance with these specifications, and at the location, and to be elevation, penetration, and/or bearing shown on the plans or as directed by the Engineer.

The Contractor shall furnish the piles in accordance with an itemized order list, which will be furnished by the Engineer, showing the number and length of all piles. When test piles are required, the pile lengths shown on the plans are for estimating purposes only and the actual lengths to be furnished for production piles will be determined by the Engineer after the test piles have been driven. The lengths given in the order list will be based on the lengths that are assumed after cutoff to remain in the completed structure. The Contractor shall, without compensation, increase the lengths to provide for fresh heading and for such additional length as may be necessary to suit the Contractor's method of operation.

DESIGN AND CONSTRUCTION CONTROL USING ULTIMATE PILE CAPACITY

The ultimate pile capacity during driving is the soil resistance that must be overcome to reach the pile tip elevation where the design load can be mobilized with an acceptable safety factor. The safety factor selected will depend both on design factors, such as quantity of subsurface information and type of geotechnical analysis, and construction factors, such as the use of load tests, wave equation, or dynamic formula to monitor pile capacity. Commonly, design load factors range from 2.0 (comprehensive foundation investigation and construction control) to 3.5 (minimal foundation investigation and construction control). However, the design load should not be used to monitor field installation of piles because only on the most routine pile projects will the ultimate pile capacity be equal to the pile design load multiplied by the design safety factor.

More typically, piles are used to penetrate upper soil layers that are unsuitable for load bearing because of either poor soil characteristics or future loss of support by scour or erosion. In such cases the resistance in the unsuitable layers is not considered in determining the pile embedment necessary to support the design load at the appropriate safety factor. However, the estimated ultimate pile capacity to be encountered during driving must include the resistance to be encountered in penetrating those unsuitable layers in addition to the design load multiplied by the safety factor. This ultimate pile capacity must be shown on the contract documents to permit the contractor to properly size the driving equipment and the engineer to judge the acceptability of the contractor's driving equipment.

Hammer blow count measured in the field has no direct relation to design load, only ultimate capacity. The hammer blow count at any pile penetration reflects the total capacity mobilized by the pile. This total capacity may include capacity mobilized temporarily in soil deposits unsuited for bearing, as well as suitable bearing layers. Therefore, hammer blow count should be established for the ultimate pile capacity that must be overcome to reach the ordered length at which the design load can be supported at the chosen safety factor. Also the maximum pile stress is directly proportional to the ultimate resistance the pile section must overcome to reach final tip. This driving stress is far more critical than the stress caused after installation by the design load.

Note that the following two sections on approval of driving equipment and driven pile capacity contain example specification sections based on ultimate pile capacity.

APPROVING DRIVING EQUIPMENT

The current procedures in most public agency specifications for approval of driving equipment are inadequate because little or no information concerning the equipment is required previous to the actual production pile driving. The lack of proper criteria has frequently resulted in equipment being provided that either cannot drive the pile to the desired elevation or inflicts damage on the pile. Improvements are needed in equipment approval to reduce the problems associated with the "wrong" hammer being furnished. Contractors should applaud such restrictions as both driving time and the chance of pile damage will be reduced. The following model specification section is suggested to provide proper control for impact hammers:

II. SPECIFICATIONS: EQUIPMENT FOR DRIVING PILES

A. Pile Hammers

Piles may be driven with steam, air, or diesel hammers. Drop hammers, if specifically permitted in the contract, shall be used only to drive timber piles. When drop hammers are permitted, the ram shall weigh between 2,000 and 3,500 lb and the height of drop shall not exceed 15 ft. In no case shall the weight of drop hammers be less than the combined weight of drive head and pile. All drop hammers shall be equipped with hammer guides to insure concentric impact on the drive head.

The plant and equipment furnished for steam and air hammers shall have sufficient capacity to maintain at the hammer, under working conditions, the volume and pressure specified by the manufacturer. The plant and equipment shall be equipped with accurate pressure gauges that are easily accessible to the Engineer. The weight of the striking parts of air and steam hammers shall not be less than $1/3$ the weight of drive head and pile being driven, and in no case shall the striking parts weigh less than 2,750 lb.

Open-end (single acting) diesel hammers shall be equipped with a device such as rings on the ram or a scale (jump stick) extending above the ram cylinder, to permit the Engineer to visually determine hammer stroke at all times during pile-driving operations. Also, the Contractor shall provide the Engineer with a chart from the hammer manufacturer equating stroke and blows per minute for the open-end diesel hammer to be used. Closed-end (double acting) diesel hammers shall be equipped with a bounce chamber pressure gauge, in good working order, mounted near ground level so as to be easily read by the Engineer. Also, the Contractor shall provide the Engineer a chart, calibrated to actual hammer performance within 90 days of use, equating bounce chamber pressure to either equivalent energy or stroke for the closed-end diesel hammer to be used.

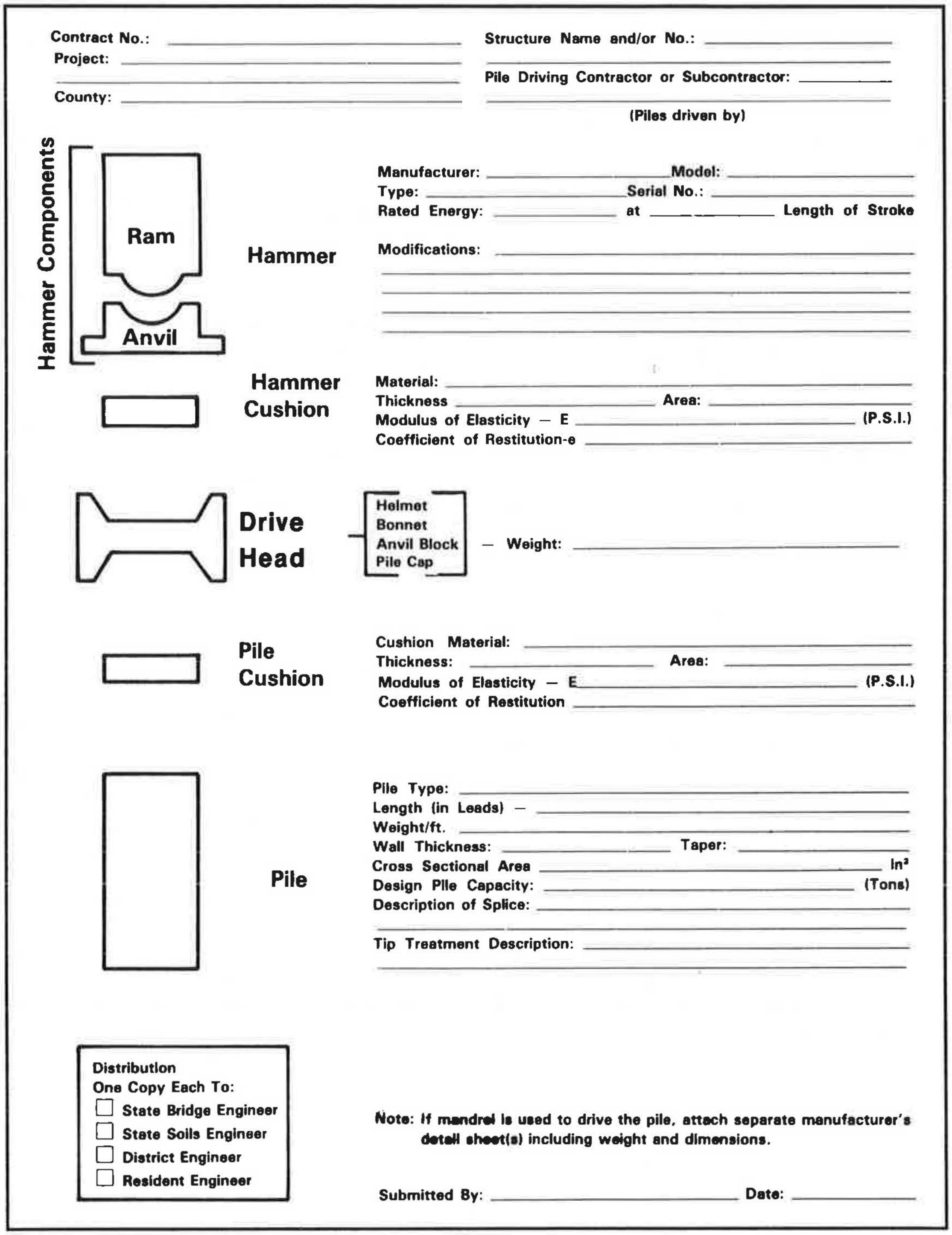
B. Approval of Pile-Driving Equipment

All pile-driving equipment furnished by the Contractor shall be subject to the approval of the Engineer. It is the intent of this specification that all pile-driving equipment be sized such that the project piles can be driven with reasonable effort to the ordered lengths without damage. Approval of pile-driving equipment by the Engineer will be based on wave equation analysis and/or other judgments.

In no case shall the driving equipment be transported to the project site until approval of the Engineer is received in writing. Prerequisite to such approval, the Contractor shall submit to the Engineer the necessary pile-driving equipment information at least 30 days before driving piles. The form that the Contractor shall complete with the previous information accompanies this text [Figure 1]. A full-size form will be included in the contract documents or supplied by the Engineer.

[*Commentary.* Use of wave equation analysis for approval of driving equipment can substantially reduce pile-driving costs and pile-driving claims by insuring that the equipment brought to the job can drive the pile to the required length without damage. Public agencies should encourage contractors to use wave equation analysis to select the optimum hammer for each project. In cases where disputes arise over rejection of pile-driving equipment, the engineer should ask the contractor to submit proof of the adequacy of the pile-driving equipment. Such proof should consist of, but not be limited to, a wave equation analysis of the proposed driving equipment performed by a registered professional engineer. All costs of such submissions, if required, shall be the responsibility of the contractor.

The pile and driving equipment data form should be submitted for approval even if wave equation analysis will not be done for hammer approval. The approved form should be used by the pile inspector to insure that the proper hammer and



Contract No.: ______ Structure Name and/or No.: ______
Project: ______
______ Pile Driving Contractor or Subcontractor: ______
County: ______
(Piles driven by)

Hammer Components

Ram
Anvil

Hammer
Manufacturer: ______ Model: ______
Type: ______ Serial No.: ______
Rated Energy: ______ at ______ Length of Stroke
Modifications: ______

Hammer Cushion
Material: ______
Thickness ______ Area: ______
Modulus of Elasticity — E ______ (P.S.I.)
Coefficient of Restitution-e ______

Drive Head
Helmet
Bonnet
Anvil Block
Pile Cap
— Weight: ______

Pile Cushion
Cushion Material: ______
Thickness: ______ Area: ______
Modulus of Elasticity — E ______ (P.S.I.)
Coefficient of Restitution ______

Pile
Pile Type: ______
Length (in Leads) — ______
Weight/ft. ______
Wall Thickness: ______ Taper: ______
Cross Sectional Area ______ In²
Design Pile Capacity: ______ (Tons)
Description of Splice: ______
Tip Treatment Description: ______

Distribution
One Copy Each To:
☐ State Bridge Engineer
☐ State Soils Engineer
☐ District Engineer
☐ Resident Engineer

Note: If mandrel is used to drive the pile, attach separate manufacturer's detail sheet(s) including weight and dimensions.

Submitted By: ______ Date: ______

FIGURE 1 Pile and driving equipment data form.

driving appurtenances are furnished and maintained during the driving operation. Few agencies currently supply the pile inspector with any such information on which rational inspection can be based.]

The criteria, which the Engineer will use to evaluate the driving equipment from the wave equation results, consist of both the required number of hammer blows per inch and the pile stresses at the required ultimate pile capacity. The required number of hammer blows indicated by the wave equation at the ultimate pile resistance shall be between 3 and 15 per in. for the driving equipment to be acceptable.

In addition, for the driving equipment to be acceptable the pile stresses that are indicated by the wave equation to be generated by the driving equipment shall not exceed the values where pile damage impends. The point of impending damage in steel piles is defined herein as a compressive driving stress of 90 percent of the yield point of the pile material. For concrete piles, tensile stresses shall not exceed 3 multiplied by the square root of the concrete compressive strength, f'_c, plus the effective prestress value (3 $\sqrt{F_c'}$ + prestress), and compressive stresses shall not exceed 85 percent of the compressive strength minus the effective prestress value (0.85 F_c' – prestress). For timber piles, the compressive driving stress shall not exceed three times the allowable static design strength listed on the plans. These criteria will be used in evaluating wave equation results to determine acceptability of the Contractor's proposed driving system.

The Contractor will be notified of the acceptance or rejection of the driving system within 14 calendar days of the Engineer's receipt of the Pile and Driving Equipment Data Form. If the wave equation analyses show that either pile damage or inability to drive the pile with a reasonable blow count to the desired ultimate capacity will result from the Contractor's proposed equipment or methods, the Contractor shall modify or replace the proposed methods or equipment at his expense until subsequent wave equation analyses indicate the piles can be reasonably driven to the desired ultimate capacity, without damage. The Engineer will notify the Contractor of the acceptance or rejection of the revised driving system within 7 calendar days of receipt of a revised Pile and Driving Equipment Data Form.

During pile-driving operations, the Contractor shall use the approved system. No variations in the driving system will be permitted without the Engineer's written approval. Any change in the driving system will only be considered after the Contractor has submitted the necessary information for a revised wave equation analysis. The Contractor will be notified of the acceptance or rejection of the driving system changes within 7 calendar days of the Engineer's receipt of the requested change. The time required for submission, review, and approval of a revised driving system shall not constitute the basis for a contract time extension to the Contractor.

C. Driving Appurtenances

1. Hammer Cushion

All impact pile-driving equipment except gravity hammers shall be equipped with a suitable thickness of hammer cushion material to prevent damage to the hammer or pile and to insure uniform driving behavior. Hammer cushions shall be made of durable, manufactured materials, provided in accordance with the hammer manufacturer's guidelines except that all wood, wire rope, and asbestos hammer cushions are specifically disallowed and shall not be used. A striker plate as recommended by the hammer manufacturer shall be placed on the hammer cushion to insure uniform compression of the cushion material. The hammer cushion shall be inspected in the presence of the Engineer when beginning pile driving at each structure or after each 100 hr of pile driving, whichever is less. Any reduction of hammer cushion thickness exceeding 10 percent of the original thickness shall be replaced by the Contractor before driving is permitted to continue.

[*Commentary.* Mandatory use of a durable hammer cushion material that will retain uniform properties during driving is necessary to accurately relate blow count to pile capacity. Nondurable materials that deteriorate during driving cause erratic estimates of pile capacity and, if allowed to disintegrate, result in damage to the pile or driving system.]

2. Pile Drive Head

Piles driven with impact hammers require an adequate drive head to distribute the hammer blow to the pile head. The drive head shall be axially aligned with the hammer and the pile. The drive head shall be guided by the leads and not be free swinging. The drive head shall fit around the pile head in such a manner as to prevent transfer of torsional forces during driving while maintaining proper alignment of hammer and pile.

For steel and timber piling, the pile heads shall be cut squarely and a drive head, as recommended by the hammer manufacturer, shall be provided to hold the axis of the pile in line with the axis of the hammer.

For precast concrete and prestressed concrete piles, the pile head shall be plane and perpendicular to the longitudinal axis of the pile to prevent eccentric impacts from the drive head.

For special types of piles, appropriate driving heads, mandrels, or other devices shall be provided in accordance with the manufacturer's recommendations so that the piles can be driven without damage.

3. Pile Cushion

The heads of concrete piles shall be protected by a pile cushion made of plywood. The minimum plywood thickness placed on the pile head before driving shall not be less than 4 in. A new pile cushion shall be provided for each pile. In addition, the pile cushion shall be replaced if, during the driving of any pile, the cushion either is compressed more than one-half the original thickness or begins to burn. The pile cushion dimensions shall match the cross sectional area of the pile top.

[*Commentary.* A pile cushion is needed only for the protection of concrete piles. If the wave equation analysis of the Contractor's hammer indicates unacceptable tension stresses, the pile cushion may need to be substantially thicker than 4 in. Pile cushion thicknesses up to 18 in. have been used to mitigate tension stresses. Compressive stresses at the pile head can be controlled with a relatively thin pile cushion. However, cushions may become overly compressed and hard after about 1,000 hammer blows.]

4. Followers

Followers shall be used only when approved in writing by the Engineer, or when specifically stated in the contract documents. In cases where a follower is permitted, the first pile in each bent and every tenth pile driven thereafter shall be driven full length without a follower, to verify that adequate pile length is being attained to develop the desired pile capacity. The follower and pile shall be held and maintained in equal and proper alignment during driving. The follower shall be of such material and dimensions to permit the piles to be driven to the length determined necessary from the driving of the full-length piles. The final position and alignment of the first two piles installed with followers in each substructure unit shall be verified to be in accordance with the location tolerances in Section [XXX] before additional piles are installed.

[*Commentary.* The use of a follower causes substantial and erratic reductions in the hammer energy transmitted to the pile because of the follower flexibility, poor connection to the pile head, frequent misalignment, and so on. Reliable correlations of blow count with pile capacity are impossible when followers are used. Severe problems with pile alignment and location frequently occur when driving batter piles with a follower in a cofferdam unless a multitier template is used. A follower should be a heavy structural section with a good connection at the pile head to reduce energy losses.]

VERIFYING DRIVEN PILE CAPACITY

Current good piling practices dictate use of the wave equation in place of dynamic formula to monitor driven pile capacity for all projects. The hammer blow count and maximum pile stress should be determined for the ultimate pile capacity. Use of the wave equation will permit the use of lower safety factors on the design load and the minimum permissible pile section to resist the driving force. This will result in significant cost reductions from savings in pile lengths and use of smaller pile sections. All highway agencies should begin phasing in wave equation analysis with an ultimate goal of specifying wave equation as the primary method for construction control of driven piles.

III. SPECIFICATIONS: CONSTRUCTION METHODS

A. Driven Pile Capacity

1. Wave Equation

The ultimate driven pile capacity shall be determined by the Engineer on the basis of a wave equation analysis. Piles shall be driven with the approved driving equipment to the ordered length or other lengths necessary to obtain the required ultimate pile capacity. Jetting or other methods to facilitate pile penetration shall not be used unless either specifically permitted in the contract documents or approved by the Engineer after a revised driving resistance is established from the wave equation analysis. Adequate pile penetration shall be considered to be obtained when the specified wave equation resistance criteria are achieved within 5 ft of the tip elevation based on ordered length. Piles not achieving the specified resistance within these limits shall be driven to penetrations established by the Engineer.

B. Load Tests

1. Static Load Test

Load tests shall be performed by procedures set forth in ASTM D1143 using the quick-load test method except that the test shall be taken to plunging failure or the capacity of the loading system. Testing equipment and measuring systems shall conform to ASTM D1143 except that the loading system shall be capable of applying 150 percent of the ultimate pile capacity, or 1,000 tons, whichever is less. The Contractor shall submit to the Engineer for approval detailed plans, prepared by a licensed professional engineer, of the proposed loading apparatus. The apparatus shall be constructed to allow the various increments of the load to be placed gradually without causing vibration to the test pile. When the approved method requires the use of tension (anchor) piles, such tension piles shall be of the same type and diameter as the production piles and shall be driven in the location of permanent piles when feasible, except that timber or tapered piles installed in permanent locations shall not be used as tension piles.

The safe pile load shall be defined as 50 percent of the failure load. The failure load for the pile shall be defined as follows: for piles 24 in. or less in diameter or width the failure load of a pile tested under axial compressive load is that load that produces a settlement at failure of the pile head equal to

$$S_f = S + (0.15 + 0.008\,D)$$

where

S_f = settlement at failure in inches,
D = pile diameter or width in inches, and
S = elastic deformation of total pile length in inches.

For piles greater than 24 in. in diameter or width

$$S_f = S + D/30$$

The top elevation of the test pile shall be determined immediately after driving and again just before load testing to check for heave. Any pile that heaves more than 1/4 in. shall be redriven or jacked to the original elevation before testing. Unless otherwise specified in the contract, a minimum 3-day waiting period shall be observed between the driving of any anchor piles or the load test pile and the commencement of the load test.

[*Commentary.* The load transferred to both the top of the bearing layer and the pile tip should be determined from instrumentation for each static load test pile. Instrumentation commonly consists of strain gauges or telltale rods mounted at varying depths from the pile top. Also, a load cell shoud be mounted between the load frame and the pile head to verify the readings from the hydraulic jack pressure gauge. Because of jack ram friction, loads indicated by a jack pressure gauge are commonly 10 to 20 percent higher than the actual load imposed on the pile.

Lastly, after completion of a load test on a nonproduction pile, the static test pile should be pulled and checked for damage. The examination of the extracted pile will determine driving damage and its effect on capacity.]

2. Dynamic Load Tests

Dynamic measurements will be taken by the Engineer during the driving of piles designated as dynamic load test piles.

[*Commentary.* When static load tests are specified, dynamic load tests are recommended to be performed on at least half the reaction piles before driving the static load test pile. The dynamic test results are used both to verify that the desired ultimate resistance can be attained at the proposed estimated static load test pile length and to fine tune the dynamic test equipment for site soil conditions. When dynamic tests are specified on production piles, the first pile driven in each substructure foundation is recommended to be tested. Where uniform soil conditions exist across a site, the number of dynamic tests may be reduced based on recommendations from the agency's geotechnical engineer.]

Before placement in the leads, the Contractor shall make each designated concrete and/or timber pile available for taking wave speed measurements and for predrilling the required instrument attachment holes. Predriving wave speed measurements will not be required for steel piles. When wave speed measurements are made, the piling shall be in a horizontal position and not in contact with other piling. The Engineer will furnish the equipment, materials, and labor necessary for drilling holes in the piles for mounting the instruments. The instruments will be attached near the head of the pile with bolts placed in masonry anchors for the concrete piles or through drilled holes on the steel piles or with wood screws for timber piles. The Contractor shall provide the Engineer reasonable means of access to the pile for attaching instruments after the pile is placed in the leads. A platform with minimum size of 4 by 4 ft (16 ft^2) designed to be raised to the top of the pile while the pile is located in the leads shall be provided by the Contractor. It is estimated that the Engineer will need approximately 1 hr per pile to install the dynamic test equipment.

The Contractor shall furnish electric power for the dynamic test equipment. The power supply at the outlet shall be 10 amp, 115 V, 55–60 cycle, AC only. Field generators used as the power source shall be equipped with functioning meters for monitoring voltage and frequency levels.

The Contractor shall furnish a shelter to protect the dynamic test equipment from the elements. The shelter shall have a minimum floor size of 8 by 8 ft (64 ft^2) and minimum roof height of 7 ft. The inside temperature of the shelter shall be maintained above 45°F. The shelter shall be located within 50 ft of the test location.

The Contractor shall drive the pile to the depth at which the dynamic test equipment indicates that the ultimate pile capacity shown in the contract plans has been achieved, unless directed otherwise by the Engineer. The stresses in the piles will be monitored during driving with the dynamic test equipment to ensure that the values determined do not exceed the critical values. If necessary, the Contractor shall reduce the driving energy transmitted to the pile by using additional cushions or reducing the energy output of the hammer in order to maintain stresses below the critical values. If nonaxial driving is indicated by dynamic test equipment measurements, the Contractor shall immediately realign the driving system.

When directed by the Engineer, the Contractor shall wait up to 24 hr and, after the instruments are reattached, retap the dynamic load test pile. It is estimated that the Engineer will require approximately ½ hr to reattach the instruments. A cold hammer shall not be used for the redrive. The hammer shall be warmed up before redrive begins by applying at least 20 blows to another pile. The maximum amount of penetration required during redrive shall be 6 in. or the maximum total number of hammer blows required will be 50, whichever occurs first. After retapping, the Engineer will either provide the cut-off elevation or specify additional pile penetration and testing.

[*Commentary.* For purposes of measurement and payment, one dynamic test includes all data collected on one pile during both the initial pile driving and a retap done up to 24 hr after the initial driving. Additional long-term retaps should be paid for as separate tests unless the retap schedule is specifically stated in the dynamic test specification. The retap data taken in a 12 to 24 hr period after initial testing will normally provide the engineer sufficient information on which to determine the long-term pile capacity. Particularly for friction piles in silt or low plasticity clay soils, the capacity measured in the first few blows where full hammer energy was applied may reliably be taken as the long-term capacity.]

3. General

On completion of the load testing, any test or anchor piling not a part of the finished structure shall be removed or cut off at least 1 ft below either the bottom of footing or the finished ground elevation if not located within the footing area.

C. Test Piles (Indicator Piles)

Test piles shall be driven when shown on the plans at the locations and to the lengths specified by the Engineer. All test piles shall be driven with impact hammers unless specifically stated otherwise in the plans. In general, the specified length of test piles will be greater than the estimated length of production piles in order to provide for variation in soil conditions. The equipment used for driving test piles shall be identical to that which the Contractor proposes to use on the production piling. Approval of driving equipment shall conform with the requirements of these Specifications. The Contractor shall excavate the ground at each test pile to the elevation of the bottom of the footing before the pile is driven.

Test piles shall be driven to a hammer blow count established by the Engineer at the estimated tip elevation. Test piles that do not attain the hammer blow count specified above at a depth of 1 ft above the estimated tip elevation shown on the plans shall be allowed to "set up" for 12 to 24 hr, or less if directed by the Engineer, before being redriven. A cold hammer shall not be used for redrive. The hammer shall be warmed up before driving begins by applying at least 20 blows to another pile. If the specified hammer blow count is not attained on redriving, the Engineer may direct the Contractor to drive a portion or all of the remaining test pile length and repeat the "set up" redrive procedure. Test piles driven to plan grade and not having the hammer blow count required shall be spliced and driven until the required bearing is obtained.

A record of driving of test piles will be prepared by the Engineer that includes the number of hammer blows per foot for the entire driven length, the as-driven length of test pile, cut-off elevation, penetration in ground, and any other pertinent information requested by the Engineer. The Contractor shall provide the information listed in Figure [X] to the Engineer for inclusion in the record. If redrive is necessary the Engineer will record the number of hammer blows per inch of pile movement for the first foot of redrive. The Contractor shall not order piling to be used in the permanent structure until test pile data have been reviewed and pile order lengths are authorized by the Engineer. The Engineer will provide the pile order list within 7 calendar days after completion of all test pile driving specified in the contract documents.

[*Commentary.* Test piles are particularly recommended on projects in which (a) large quantities or long length of friction piling are estimated, even if load tests are to be used at adjacent footings; (b) large ultimate soil resistance is expected in relation to the design load, and (c) precast concrete piles are used.]

DEVISING A METHOD OF MEASUREMENT AND BASIS OF PAYMENT

Pile payment items should be chosen to separate the major fixed costs from the variable costs. Many highway agencies oversimplify pile payment by including all costs associated with the driving operation into the price per foot of pile installed. Contractors bidding such "simple" items are required to break down the total cost of the mobilization, splices, shoes, and so on, into a price per foot based on the total estimated pile footage. If that footage underruns, the Contractor does not recover the full cost of mobilization, splices, shoes, and so on; if the footage overruns, the owner pays an unfair price for the overrun footage. The use of separate items for piling operations of major fixed cost, such as mobilization, can substantially mitigate the inequitable impact of length variations. Similarly, separate payment for both furnishing piles and driving piles compensates the contractor for actual materials used and installation costs even when overruns or underruns occur. Also, separate items permit the Contractor to obtain payment for work completed in a timely, equitable manner instead of requiring him or her to carry large debts for materials purchased or equipment furnished. The following section on measurement and payment is suggested.

IV. SPECIFICATIONS: METHOD OF MEASUREMENT

A. Timber, Steel, and Precast Concrete Piles

1. Piles Furnished

The unit of measurement for payment for furnishing timber, steel, and precast concrete piles shall be the linear foot. The quantity to be paid for will be the sum of the lengths in feet of the piles, of the types and lengths ordered in writing by the Engineer, furnished in compliance with the material requirements of these specifications and stockpiled in good condition at the site of the work by the Contractor, and accepted by the Engineer. No allowance will be made for that footage of piles, including test piles, furnished by the Contractor to replace piles that were previously accepted by the Engineer, but are subsequently damaged prior to completion of the contract.

When extensions of piles are necessary during construction, the extension length ordered in writing by the Engineer will be included in the linear footage of piling furnished.

2. Piles Driven

The unit of measurement for driving timber, steel, and precast concrete piles shall be per linear foot of piling in place measured below the cut-off elevation. The measured length will be rounded to the nearest foot.

Preboring, jetting, or other methods used for facilitating pile-driving procedures will not be measured and payment shall be considered included in the unit price bid for the Piles Driven pay item.

B. Cast-in-Place Pipe of Shell Concrete Piles

The quantity of cast-in-place pipe of shell concrete piles to be paid for will be the actual number of linear feet of steel pipe or shell piles driven, cast, and left in place in the completed and accepted work. Measurements will be made from the tip of the steel pipe or shell pile to the cut-off elevation shown on the plans.

No separate measurement will be made for reinforcing steel, excavation, drilling, cleaning of drilled holes, drilling fluids, sealing materials, concrete, required casing, and other items required to complete the work.

C. Pile Shoes

The number of pile shoes measured for payment shall be those shoes actually installed on piles accepted for payment by the Engineer.

D. Load Tests

The quantity of load tests to be paid for will be the number of load tests completed and accepted, except that load tests made at the option of the Contractor will not be included in the quantity measured for payment.

Anchor and test piling that are not a part of the permanent structure will be included in the unit price bid for each load test. Anchor and test piling or anchor and test shafts, which are a part of the permanent structure, will be paid for under the appropriate pay item.

E. Splices

The number of splices measured for payment shall be only those splices actually made and required to drive the piles in excess of the ordered length furnished by the Engineer.

F. Furnishing Equipment for Driving Piles

Payment will be made at the lump sum price bid for this item as follows: 75 percent of the amount bid will be paid when the equipment for driving piles is furnished and driving of satisfactory piles has commenced. The remaining 25 percent will be paid when the work of driving piles is completed. The lump sum price bid shall include the cost of furnishing all labor materials and equipment necessary for transporting, erecting, maintaining, making any ordered equipment replacement, dismantling, and removing the entire pile-driving equipment. The cost of all labor, including the manipulation of the pile-driving equipment and materials in connection with driving piles, shall be included in the unit price bid per linear foot for the piles to be driven. The furnishing of equipment for driving sheet piling is not included in this work. Payment for furnishing and using a follower, augers, or jetting will be considered as included in the unit price bid for piles.

V. SPECIFICATIONS: BASIS OF PAYMENT

The accepted quantities, determined as provided in the previous sections, will be paid for at the contract price per unit of measurement, respectively, for each of the particular pay items in the following list that is shown in the bid schedule, which prices and payment will be full compensation for the work prescribed in this Section. Payment will be made under:

Pay Item	*Pay Unit*
Piles, furnished	Linear foot
Piles, driven	Linear foot
Piles, cast-in-place	Linear foot
Test piles, furnished	Linear foot
Test piles, driven	Linear foot
Test piles, cast-in-place	Linear foot
Pile load test (static)	Each
Pile load test (dynamic)	Each
Splices	Each
Pile shoes	Each
Furnishing equipment for pile driving	Lump sum

CONCLUSION

In summary, current pile specifications need to be modernized to include state-of-the-art construction control methods such as wave equation analysis and dynamic and static pile load testing. Consideration also needs to be given to both improvement in the use of modern design analyses such as static analyses on which the construction control depends and the equitable distribution of responsibility and payment for work specified and accomplished. Use of these methods combined with local experience will improve pile foundation reliability, allowing the design and construction of cost-effective foundations with reduced frequency of claims and disputes.

Publication of this paper sponsored by Committee on Foundations of Bridges and Other Structures.

Analysis of Laterally Loaded Piles with Nonlinear Bending Behavior

STEVEN L. KRAMER AND EDWARD J. HEAVEY

Analysis of the response of pile foundations to lateral loads requires accurate characterization of the behavior of the pile and the soil surrounding the pile. The common assumption of pile linearity in bending may not be valid in many cases. A model for representing nonlinear pile bending behavior in the analysis of laterally loaded piles is introduced. The fourth order differential equation governing the response of a laterally loaded pile is solved iteratively by a finite difference technique. The method converges to a solution featuring displacement-compatible soil resistance and curvature-compatible bending moments along the length of the pile. The ability of the method to represent nonlinear pile bending behavior is illustrated. The ability of the model to predict observed behavior is illustrated by application of the model to lateral load test case histories.

The response of pile foundations to lateral loads depends on the interaction between the piles and the surrounding soil. Analysis of this response requires accurate characterization of the behavior of both the pile and the soil surrounding the pile. Commonly used existing methods for analysis of the lateral load response of single piles consider the nonlinearity of the soil resistance but treat the pile as a linear, elastic beam. The assumption of pile linearity may not be valid in many cases. This paper considers the effect of this assumption on the behavior of laterally loaded piles and introduces a method for representing nonlinear pile bending behavior in the analysis of laterally loaded piles.

PREVIOUS WORK

The problem of calculating the response of a single, laterally loaded pile has been studied by many investigators using different approaches including methods of applied elasticity (*1*), finite element analysis (*2, 3*), boundary element methods (*4*), and subgrade reaction methods (*5*). While each of these methods has advantages in particular applications, the subgrade reaction methods have become the most commonly used for practical analysis of routine problems.

Subgrade reaction methods of lateral load analysis, in which the soil resistance is described by *p-y* curves, have been used for many years and have been recently summarized (*6*). Such methods for analysis of laterally loaded piles reported in the literature, however, almost uniformly treat the pile as a linear, elastic material. Nakai and Kishida (*3*) incorporated pile bending nonlinearity into an incremental finite element model, which iteratively solved for the deflected shape of the pile using a Rayleigh-Ritz procedure. Their model showed reasonable prediction of load-deflection behavior when applied to several case histories. For piles loaded into their nonlinear bending range, Reese (*6*) suggests analytical determination of the pile flexural stiffness corresponding to the maximum bending moment and use of that stiffness for the entire pile in a linear pile analysis.

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ANALYSIS OF LATERALLY LOADED PILES WITH NONLINEAR BENDING BEHAVIOR

The response of laterally loaded piles embedded in an elastic medium is usually described by the beam-on-elastic-foundation theory of Hetenyi (*7*). Equilibrium of an element of a beam subjected to axial and transverse loading was shown by Hetenyi to be described by the differential equation:

$$d^2M/dx^2 + Q(d^2y/dx^2) - p = 0 \qquad (1)$$

where

- M = bending moment;
- x = position along axis of beam;
- Q = axial load;
- y = deflection normal to axis of beam; and
- p = transverse loading along length of beam.

If the beam is assumed to exhibit linear bending behavior, the bending moment may be taken as the product of the constant beam flexural stiffness, EI, and the induced curvature, in which case the governing differential equation is of the familiar form:

$$EI(d^4y/dx^4) + Q(d^2y/dx^2) - p = 0 \qquad (2)$$

For the analysis of laterally loaded piles in soil, this equation is commonly expressed in difference form and solved by a numerical procedure that iterates toward a solution in which the unit soil resistance, p, is compatible with the lateral pile deflection, y.

The bending moment induced in a pile exhibiting nonlinear bending behavior is not proportional to the pile curvature. Defining a secant pile flexural stiffness, $(EI)_s$, as the ratio of the bending moment to the induced curvature, the bending moment may be represented as

$$M = (EI)_s\,(d^2y/dx^2) \qquad (3)$$

Substitution of Equation 3 into Equation 1 and expansion of the first differential term leads to the general governing differential equation for a laterally loaded pile with nonlinear bending behavior:

$$(EI)_s(d^4y/dx^4) + 2[d(EI)_s/dx]\,(d^3y/dx^3) + [Q + d^2(EI_s/dx^2)]\,(d^2y/dx^2) - p = 0 \qquad (4)$$

This governing equation may be solved numerically by discretizing the pile into segments of length, h, and expressing the differentials in difference form, which yields a banded system of simultaneous equations expressed, letting $R = (EI)_s$, in the form:

$$A_i y_{i+2} + B_i y_{i+1} + C_i y_i + D_i y_{i-1} + E_i y_{i-2} = 0 \qquad (5)$$

where

$$\begin{aligned}
A_i &= R_i + (R_{i+1} - R_{i-1})/2 \\
B_i &= 2R_{i-1} - 6R_i + Qh^2 \\
C_i &= 10R_i - 2R_{i+1} - 2R_{i-1} - 2Qh^2 + E_s h^4 \\
D_i &= 2R_{i+1} - 6R_i + Qh^2 \\
E_i &= R_i + (R_{i-1} - R_{i+1})/2
\end{aligned}$$

SOLUTION OF PILE RESPONSE EQUATION

An iterative procedure was developed to solve the governing equation in difference form accounting separately for both the nonlinearity of the soil resistance and the bending nonlinearity of the pile. The proposed procedure retains the conventional iteration toward displacement-compatible soil resistance, used in programs like COM624 (*5*), wherein the soil resistance is taken as the negative product of the pile displacement and a secant soil modulus. The procedure iterates until the computed displacement is within some small tolerance of the displacement from the previous iteration, as shown in Figure 1a. The proposed procedure also iterates simultaneously toward a curvature-compatible bending moment. In this process, the actual bending moment at some point along the pile is taken as the product of the curvature of the pile at that point and a secant flexural stiffness. The secant flexural stiffness is varied until the curvature computed by a particular iteration is within some small tolerance of the curvature used in the previous iteration, as shown in Figure 1b. By this simultaneous iteration procedure, the solution converges to one in which the soil resistance is compatible with the pile displacement and the bending moments are compatible with the induced pile curvature.

This simultaneous iteration procedure has a number of attractive features. It is a relatively simple procedure, both conceptually and computationally. It can be incorporated into existing programs for lateral load analysis with a moderate amount of programming effort. The bending characteristics of the pile are described by one or more moment-curvature diagrams and may be varied along the length to accommodate tapered, segmented, or composite piles. While it does not provide a rigorous method for representation of nonlinear pile behavior, the method is consistent with that conventionally used to represent the soil resistance. Considering that the moment-curvature behavior of a pile is likely to be much more reliably known than the unit soil resistance–displacement behavior of the soil, the uncertainty associated with this numerical method of representation of pile nonlinear bending behavior is small.

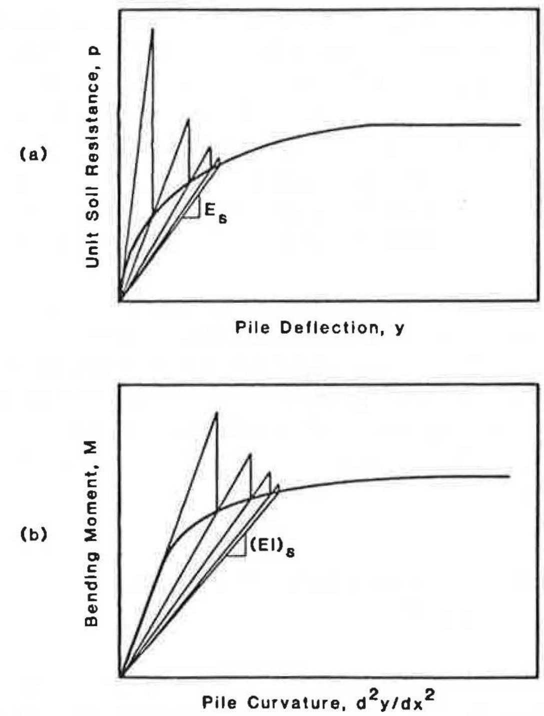

FIGURE 1 Schematic diagram of iteration procedure used to converge to (a) deflection-compatible soil stiffness and (b) curvature-compatible pile flexural stiffness.

MOMENT-CURVATURE BEHAVIOR OF PILES

Piles generally exhibit nonlinear bending behavior as a result of one or both of two mechanisms. First, yielding of the pile material(s) may lead to a nonlinear moment-curvature relationship. Nonlinearity as a result of material yielding can occur in steel, concrete, and timber piles when bending moment-induced stresses in the outer fibers of the pile section exceed the elastic range of the pile material. Second, the flexural stiffness of the pile may vary as a result of changes in geometry of the moment-resisting pile section. This effect is usually the result of cracking of the pile section and is typically observed only in concrete piles.

Steel piles are generally not subject to cracking under normal conditions. Large pile curvatures, however, may induce compressive and tensile stresses that may exceed the yield strength of the steel, particularly in portions of the pile section in which residual stresses are high. If additional loads cause further curvature, the associated bending moment will not increase proportionally. The theoretical maximum bending moment for which a steel pile will have constant flexural stiffness may be determined from simple beam theory. The moment-curvature relationship for steel piles stressed beyond the yield stress of the steel may be calculated based on the cross-sectional shape of the pile and an assumption of elastic, perfectly plastic steel behavior (*8*).

Concrete is used in pile foundations in the form of reinforced concrete, as in drilled, cast-in-place piles, or in precast, prestressed piles. Concrete piles, however, may develop tensile cracking even at moderate curvatures. Since tensile stresses cannot be transmitted across a crack, the moment-curvature relationship becomes nonlinear as soon as cracking develops. The nonlinear portion of the moment-curvature relationship for a concrete pile cannot be calculated as easily as that of steel pile since the bending behavior depends on the constitutive behavior of both the concrete and the reinforcing steel or prestressing strands. Analytical models are available (*6, 9*), however, that are capable of predicting the moment-curvature behavior of reinforced concrete and prestressed concrete sections.

The moment-curvature behavior of timber piles may have significant nonlinearity at moderate curvatures because of the material nonlinearity of wood. Ultimate bending moments and initial flexural stiffnesses appear to vary from pile to pile (*10*). The nonlinear bending model is well suited for incorporation into a probabilistic analysis of laterally loaded timber pile response.

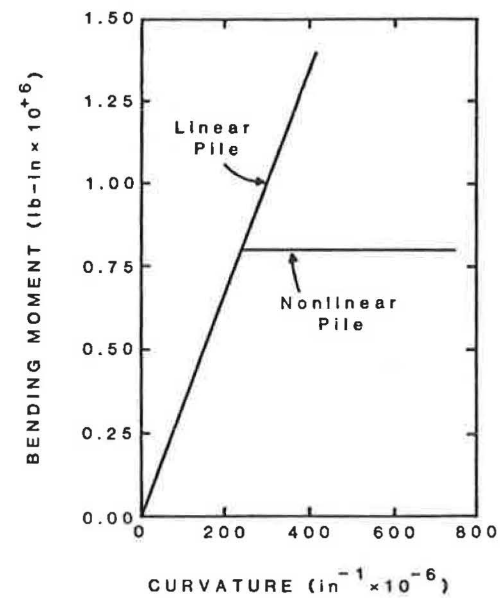

FIGURE 2 Moment-curvature diagrams for linear and nonlinear bending behavior of drilled shaft (*11*).

ILLUSTRATION OF EFFECTS OF PILE NONLINEARITY

The nonlinear bending model was applied to two reported case histories of laterally loaded piles. The case histories were both drilled shafts, one constructed in stiff clay and the other in loose sand, or soft clay. In each case, the response of the pile was analyzed by assuming linear bending behavior and then assuming nonlinear bending behavior. The results are then compared with the observed response of the piles.

Johnson et al. Test

Results of a full-scale field lateral load test on an aged drilled shaft were presented by Johnson et al. (*11*). The shaft was 18 in. (0.46 m) in diameter by 34.5 ft (10.5 m) long including a 36-in. (0.9 m) diameter underream. The shaft extended through expansive clays, which had fractured the shaft at the neck of underream by the time it was tested 18 yr after construction. The shaft was loaded to failure with one intermediate unload-reload cycle. Yielding of the shaft was observed at failure.

Johnson et al. analyzed the response of the shaft with a constant computed flexural stiffness and the Reese and Welch (*12*) stiff clay *p-y* criteria and calculated load-displacement response stiffer than observed. The authors suggested that the actual flexural stiffness was less than computed.

Analysis of the load test with a smaller, constant flexural stiffness as indicated in Figure 2 gave reasonable agreement at lateral loads below about 28 kips (125 kN). At lateral loads above 28 kips (125 kN), however, linear pile analyses are unable to predict the large deflections observed in the field, as shown without the unload-reload cycle in Figure 3. Assuming that the maximum bending moment of the shaft was reached at a lateral load of 28 kips (125 kN), the bilinear moment-curvature relationship marked "Nonlinear Pile" in Figure 2 may be taken to represent the bending behavior of the shaft. Solution with this nonlinear moment-curvature relationship predicts the load-displacement response over the entire range of loading shown in Figure 4. The superior ability of the nonlinear model to predict the observed load-displacement response for this case history is apparent.

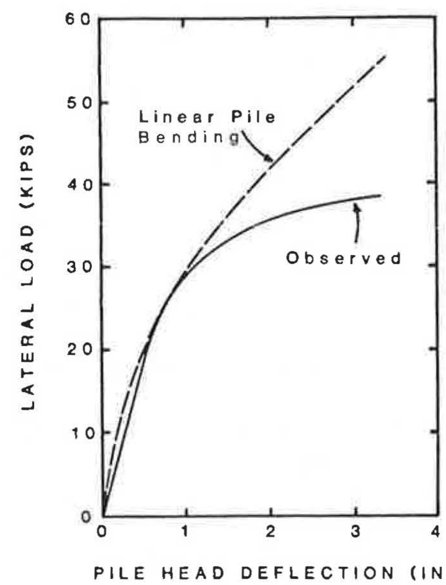

FIGURE 3 Comparison of observed response with response predicted by linear pile bending model (*11*).

Tsuji et al. Test

A lateral load test originally reported by Tsuji et al. in Japanese was described by Tominaga et al. (*13*). The test was performed on a 47.2-in. (1200 mm) diameter, cast-in-place bored pile, installed in loose sand, or soft clay, with standard penetration resistance of 4 to 5.

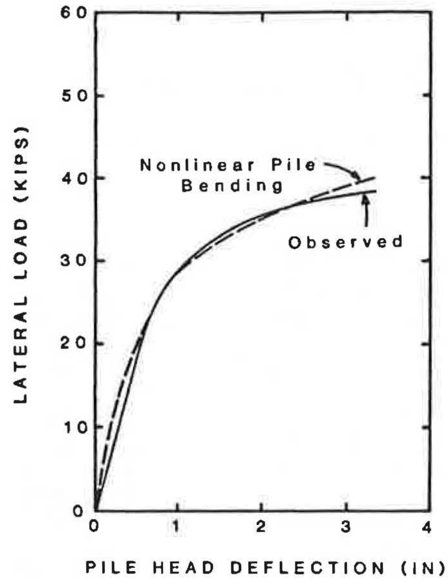

FIGURE 4 Comparison of observed response with response predicted by nonlinear bending model (*11*).

The uncracked pile was reported to have a flexural stiffness (*EI*) of 1.389 × 10^{12} lb-in.^{2}. Tominaga et al. developed an expression for the ultimate unit soil resistance as

$$P_{ult} = 0.1126z^2 + 9.21z$$

in pounds per inch where z = depth in inches. Using these values and assuming bilinear *p-y* behavior, Tominaga et al. computed the response of the pile by varying the initial soil stiffness until reasonable agreement with the observed behavior was obtained. By this procedure, good agreement was obtained at lateral loads below approximately 132 kips. Above this level, however, the linear model is unable to predict the rapidly increasing deformations attributed by Tominaga et al. to cracking of the pile.

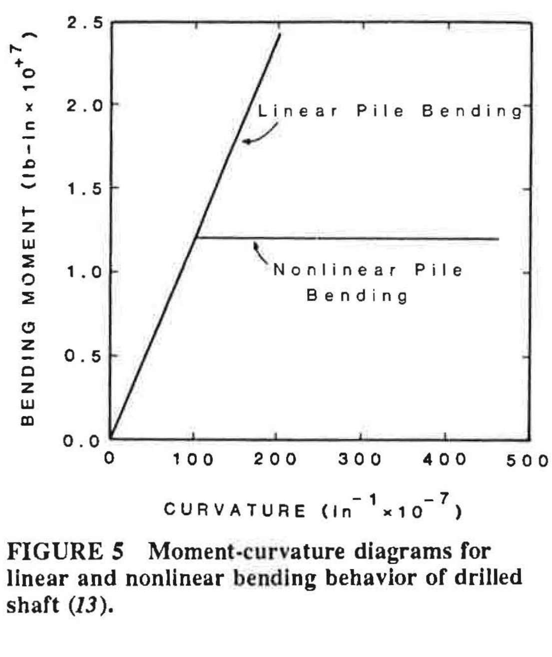

FIGURE 5 Moment-curvature diagrams for linear and nonlinear bending behavior of drilled shaft (*13*).

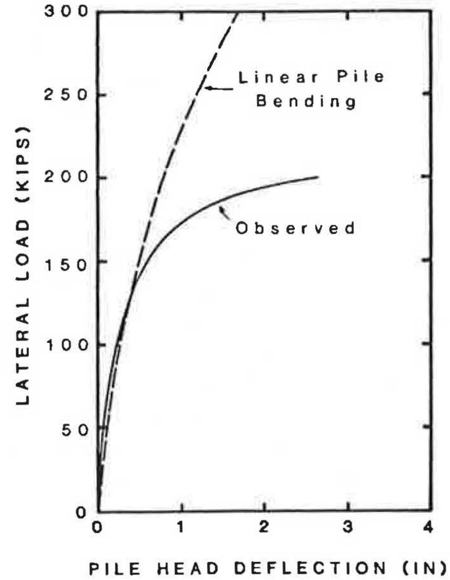

FIGURE 6 Comparison of observed response with response predicted by linear pile bending model (*13*).

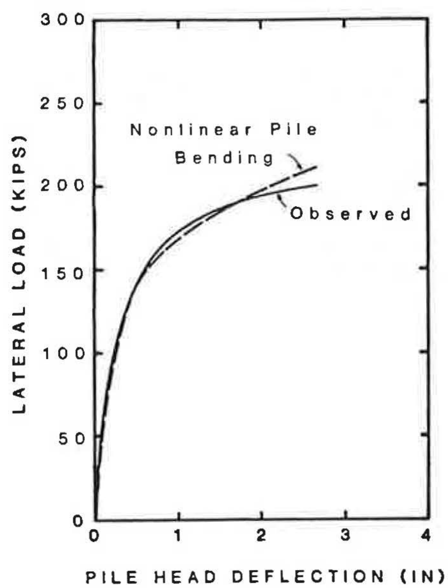

FIGURE 7 Comparison of observed response with response predicted by nonlinear pile bending model (*13*).

A similar procedure was used in the present investigation. Using *p-y* curves of the hyperbolic tangent form (*14, 15*), which is considered to be more representative of the actual nonlinear soil behavior than the simple bilinear model, the response of the pile was computed assuming linear pile bending behavior. The moment-curvature relationship was that indicated for linear pile bending in Figure 5. This analysis also indicates good agreement between computed and observed behavior at lateral loads below 132 kips as shown in Figure 6. Assuming that the ultimate bending moment was reached at a lateral load of 132 kips, the bilinear moment-curvature relationship marked "Nonlinear Pile Bending" in Figure 5 may be taken to represent the bending behavior of the pile. Solution of the pile response with this nonlinear moment-curvature relationship gives the load-displacement curve shown in Figure 7. Again, the ability of the nonlinear model to predict the large pile deflections associated with nonlinear pile bending behavior can easily be seen.

SUMMARY

A methodology for analysis of the lateral load response of piles with nonlinear bending behavior has been developed. Application to case histories of lateral load tests on piles and drilled shafts loaded into their nonlinear bending ranges indicates that the model is capable of predicting the large deflections often associated with the lateral load response of such piles. The model is expected to have particular application to reinforced concrete drilled shafts and to timber piles.

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Publication of this paper sponsored by Committee on Foundations of Bridges and Other Structures.

Unified Design of Piles and Pile Groups

BENGT H. FELLENIUS

A unified design of piles and pile groups is proposed wherein capacity, residual compression, negative skin friction, and settlement are related. First, the location of the neutral plane is determined. Then, the adequacy of the structural strength of the pile is checked and followed by an analysis of the settlement of the pile foundation, applying the concept of an equivalent footing placed at the neutral plane. Finally, the adequacy of the pile bearing capacity is verified. For structural capacity at the neutral plane, dead load and dragload are considered together, but live load is excluded. For settlement, all stress increase in the soil is considered, not just that of the dead load acting on the pile foundation. For bearing capacity, dead and live loads are considered, but dragload is excluded. The design is iterative, inasmuch as the choice of load and pile length will have an interactive influence on all aspects: location of neutral plane, dragload, structural capacity, and settlement, as well as bearing capacity.

Conventionally, or traditionally, when designing piles and pile groups, design for bearing capacity and design for settlement are considered separately and are not influenced by each other. In the simplest principle, design for bearing capacity consists of determining the allowable load—the service load—on the pile by dividing the capacity by a factor of safety. Settlement occurs when the service load on the piles stresses the soil, causing the soil to consolidate and compress. Usually, the methods of calculation are very simple. For instance, a common approach is to take the settlement of piles in sand to be equal to 1 percent of the diameter of the head of an individual pile plus the "elastic" compression of the pile under the load. For the case of an essentially shaft-bearing pile group in homogeneous clay soil, Terzaghi and Peck (*1*) recommended taking the settlement of the group as equal to that calculated for an equivalent footing located at the lower third point of the pile embedment length and loaded to the same stress and over the same area as the pile group plan area (Figure 1). For other approaches, see Meyerhof (*2*).

More complex methods for calculating settlement use elastic halfsphere analysis or finite element techniques. Vesic (*3*) and Poulos and Davis (*4*) presented several such analytical approaches toward calculating settlement on single piles and pile groups. Generally, it is assumed that before load is applied to the pile foundation, no stress is present in the pile or piles.

For the case of piles installed through a multilayered soil deposit, where upper layers settle because of, for instance, a surcharge on the ground surface or a general groundwater lowering, a settlement calculation of the pile group is often not performed. (The design practice seems to be to trust that the settlement will somehow be taken care of by including loads from downdrag in the bearing capacity analysis. Sometimes, on the other hand, the dragload is added to the service load and some settlement calculation is carried out for this combined load—a totally erroneous approach.)

Provided that the piles have been installed to reach well into competent soils and that no weaker soil layers exist below the pile toe elevation, this approach of including the dragloads in the bearing capacity and settlement analyses is mostly safe, albeit excessively costly. However, the problem of negative skin friction is one of settlement and not of bearing capacity (i.e., the magnitude of the dragload is of no relevance to the bearing capacity of the pile). Furthermore, the allowable load on the pile should be governed by a combined (unified) approach considering soil resistance and settlement inseparably acting together and each influencing the value of the other.

LONG-TERM MEASUREMENTS OF LOAD AND SETTLEMENT

Observations show that for piles bearing on very competent material, negative skin friction can result in very large dragloads. Bjerrum et al. (*5*) measured dragloads amounting to about 4,000 kN on 0.5-m-diameter steel test piles installed to bedrock through 55 m of clay soil settling under the influence of a recent surcharge.

If a pile is long enough or if the ratio of its unit circumferential area to its cross-sectional area is large enough, the induced stress could exceed the material strength (i.e., the structural capacity of the pile). In the field tests reported by Bjerrum et al. (*5*), the piles were driven to rock, and the induced dragload exceeded the available toe resistance, forcing the pile to penetrate into the rock. This effect is cyclic, as discussed by Fellenius (*6*). Obviously, the toe force developed during the pile driving must have been smaller than the dragload.

Immediately after a pile is installed in the soil, the soil begins to reconsolidate from the disturbance caused by the installation of the pile, whether the pile was driven or otherwise installed. Fellenius and Broms (*7*) and Fellenius (*6*) reported load measurements in 0.3-m-diameter concrete piles driven into a 40-m-thick clay deposit and into an underlying sand layer. Immediately after the driving, the load in the pile was small, about equal to the free-standing weight of the pile before the driving. The reconsolidation of the clay after the driving took about 5 months. During this time, negative skin friction developed and the dragload induced amounted to about 350 kN corresponding to about one-third of the maximum dragload, which developed during the following several years of observations (*6, 8*). The settlement of the ground surface

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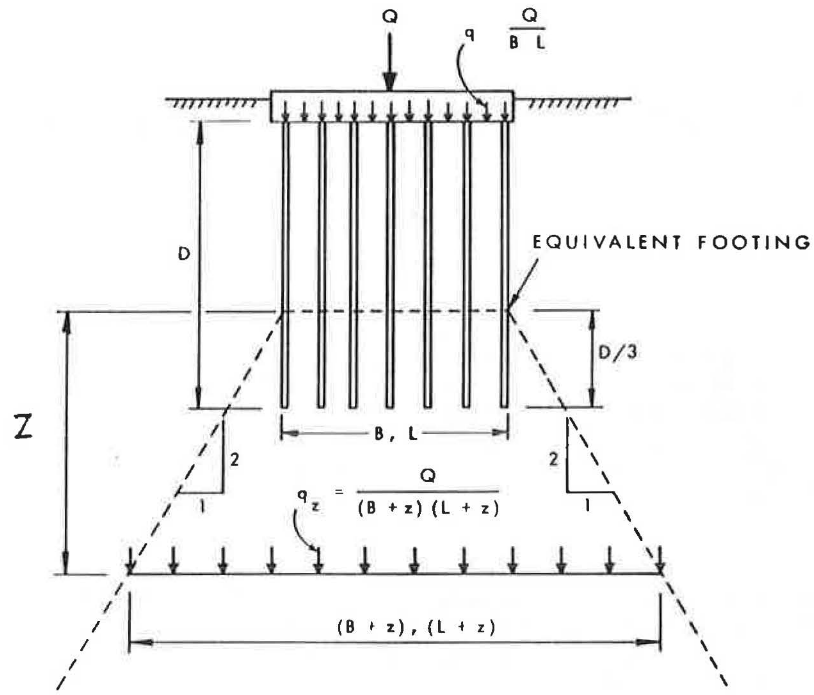

FIGURE 1 Calculations of settlement for a pile group in homogeneous clay soil using the equivalent footing concept (*1*).

associated with the 5-month reconsolidation was interpolated from measurements over a longer period of time and found to be smaller than about 2 mm. The distribution with depth of relative displacement between the pile and the clay was even smaller, of course.

The test is particularly interesting because it involved the effect of applying a static load to the pile head and not just observations of the development of the dragload in the pile. Applying a static load to the pile head caused the dragload in the pile to be reduced by the magnitude of the load applied. As the load became permanent, however, the negative skin friction built up again and the end effect was that the dragload in the pile was added to the load applied to the pile head. At the end of the test, the dragload was fully developed. The maximum load in the pile was 1,750 kN, consisting of a dead load of 800 kN and a dragload of 950 kN.

The negative skin friction was fully mobilized to a depth of about 25 m after a relative displacement of about 2 mm as measured at a distance smaller than about 0.5 m away from the pile. Measured at a distance of 5 m away from the test piles, the mean relative displacement approached 6 mm.

The magnitude of the movement necessary for shear resistance to develop was observed by Walker and Darvall (*9*), who reported that a mere 35-mm settlement of the ground surface due to a surcharge placed around single piles driven in clay was sufficient to develop negative skin friction down to a depth of 18 m. Settlement distribution with depth was not reported.

When loading a 49-m-long instrumented pile that had been driven through an embankment and into consolidating clay and then monitored during a 10-yr period, Bozozuk (*10*) found that a reversal of direction of shear forces down to a depth of 20 m occurred after a relative movement between the pile head and the ground surface of no more than about 4 mm. At depth, the relative movement between the pile and the soil near the pile was much smaller than 4 mm.

Bjerrum et al. (*5*) reported that negative skin friction causing large dragloads developed along about 55-m-long piles driven in clay at a site where the settlement under a recent fill amounted to 2 m. However, the same authors also reported that about an equal magnitude of dragload developed on 41-m-long piles that were driven through an adjacent 70-year-old fill of the same height and in the same type of soil in which an ongoing surface settlement of only 1 to 2 mm per year was observed.

In the referenced observations, which report results from field investigations performed on three separate continents extremely small movement was all that was needed to generate shear stress or to reverse the direction of shear along the pile-soil interface. Of course, on other occasions, larger relative movements could be necessary before the shear forces are fully developed. For instance, Vesic (*3*) suggested the rather large relative movement of 15 mm as a general requirement. However, the evidence suggests that such large movement is the exception and that the very small movement is the rule. Vesic's suggestion should be understood more to indicate when settlement around a pile foundation could start to cause problems.

There is a far-reaching consequence of a very small movement being all that is needed to fully transfer shear between the pile and the soil. The pile is immensely more rigid than the soil; with time, there will always be small movement in a soil, meaning that small relative displacements between a pile and the soil will occur and these are large enough to develop shear forces along the pile. The conclusion is that all piles experience negative skin friction and dragloads.

A consequence of the small displacement required to mobilize or to reverse the direction of shear forces along a pile is that live loads and dragloads are not additive (*6, 10*).

CALCULATION OF THE MAGNITUDE OF UNIT SHAFT RESISTANCE

For the analysis of shaft resistance, Johannessen and Bjerrum (*11*) and Burland (*12*) established that the unit resistance is proportional to the effective overburden stress in the soil surrounding the pile. The constant of proportionality is called beta-coefficient, β, and is a function of the earth pressure coefficient in the soil, K_s, times the soil internal friction, $\tan \phi'$, times the quotient of the wall friction, $M = \tan \delta'/\tan \phi'$ (*13*). Thus, the unit negative skin friction, q_n, follows the following relations:

$$q_n = \beta \, \sigma_v'$$

$$\beta = M \, K_s \tan \phi'$$

(See the Notation section for more information on the symbols used.) Bjerrum et al. (*5*) found that the beta-coefficient in a soft silty clay ranged between 0.20 and 0.30. Kraft et al. (*14*) summarized several methods of determining shear transfer for piles in clay.

PILE LOAD DISTRIBUTION AND LOCATION OF THE NEUTRAL PLANE

There must always be equilibrium between the sum of the dead load applied to the pile head and the dragload and the sum of the positive shaft resistance and the toe resistance. The depth where the shear stress along the pile changes over from negative skin friction into positive shaft resistance is called the neutral plane. The location of this plane is also where there is no relative displacement between the pile and the soil.

Provided the shear stress along the pile does not diminish with depth and that there is some toe resistance, the neutral plane lies below the midpoint of a pile. If the soil below the neutral plane is strong, the neutral plane lies near the pile toe. The extreme case is for a pile on rock, where the location of the neutral plane is at the bedrock elevation. For a predominantly shaft-bearing pile "floating" in a homogeneous soil with linearly increasing shear resistance, the neutral point lies at a depth that is about equal to the lower third point of the pile embedment length (assuming that the negative skin friction is equal to the positive shaft resistance, the toe resistance is small, and the load applied to the pile head is a third of the bearing capacity of the pile). It is interesting to note that this location is also the location of the equivalent footing according to the Terzaghi-Peck approach (*1*) (Figure 1).

With larger toe resistance, the elevation of the neutral plane lies deeper into the soil. If an increased dead load is applied to the pile head, the elevation of the neutral plane moves up.

Figure 2 illustrates the distribution of load in a pile subjected to a service load, Q_d, and installed in a relatively homogeneous soil deposit, where the shear stress along the pile, as induced by a relative displacement, is proportional to the effective overburden stress. It is assumed that any excess pore pressure in the soil has dissipated and the pore pressure is hydrostatically distributed. For reasons of simplicity, the shear stress along the pile is assumed to be independent of the direction of the displacement (i.e., the magnitude of the negative skin friction, q_n, is equal to the magnitude of the unit positive shaft resistance, r_s). It is also assumed that a toe resistance, R_t, has been mobilized.

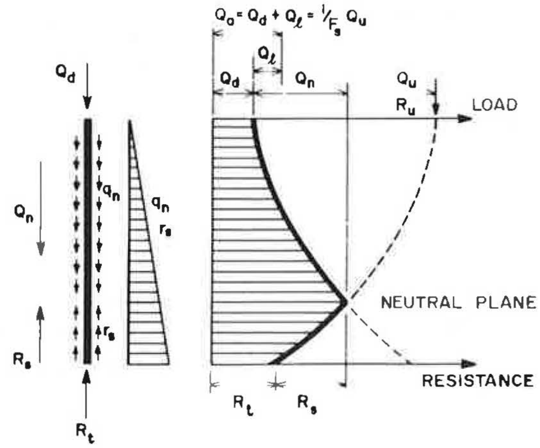

FIGURE 2 Definition and construction of the neutral plane.

The dragload, Q_n, is the sum (integral) of the unit negative skin friction along the pile. The total shaft resistance below the neutral plane, R_s, is the sum of the unit shaft resistance along the pile.

These conditions determine the location of the neutral plane as shown in the diagram.

SETTLEMENT OF A PILE

The neutral plane is, as mentioned, the location where there is no relative displacement between the pile and the soil. Consequently, whatever the settlement in the soil is as to its magnitude and vertical distribution, the settlement of the pile head is equal to the settlement of the neutral plane plus the compression of the pile caused by the applied dead load and the dragload combined.

Illustrated in the "Load and Resistance" segment of Figure 3 is how the elevation of the neutral plane changes with a variation of the load, Q_d, applied to the pile head. Notice also that the magnitude of the dragload changes as Q_d changes.

Assume that the distribution of settlement in the soil around the pile is known and follows the "Settlement" portion of the diagram in Figure 3. As illustrated in the diagram for the case of the middle service load, by drawing a horizontal line from the neutral plane to intersect the settlement curve, the settlement of the pile at the neutral plane and, thus, the settlement of the pile head can be determined. The construction in the figure is valid both for a small settlement that diminishes quickly with depth and for a large settlement that continues to be appreciable well below the pile toe.

The construction in Figure 3 has assumed that the toe resistance is fully mobilized. If the settlement is small, it is possible that the toe movement is not large enough to mobilize the full toe resistance. In such a case, the neutral plane moves

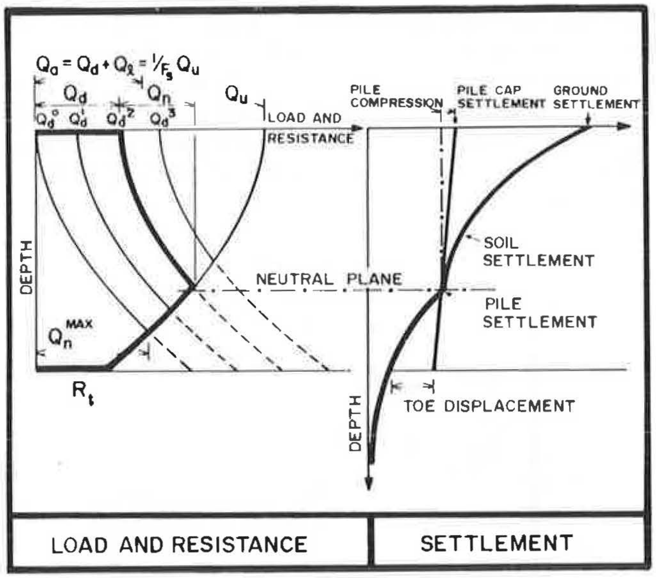

FIGURE 3 Unified design for capacity, negative skin friction, and settlement.

to a higher location as determined by the particular equilibrium condition.

For a driven pile, the toe movement necessary to mobilize the toe resistance is about 1 to 2 percent of the pile toe diameter. For bored piles, the movement is larger. However, in cases where the toe movement is too small for the full toe resistance to be mobilized (less toe resistance results in a raising of the location of the neutral plane), the settlement is normally not an issue.

REDUCTION OF SKIN FRICTION BY MEANS OF BITUMEN COATING

Bjerrum et al. (*5*) demonstrated the efficiency of coating piles with bitumen to reduce the negative skin friction. Walker and Darvall (*9*) presented a comparison between bitumen coated and uncoated steel piles, and Clemente (*15*) reported measurements of dragloads on coated and uncoated concrete piles. Fellenius (*16, 17*) discussed some practical aspects of bitumen coating of piles to reduce negative skin friction.

Other papers comparing measurements in plain and coated piles were published by Endo et al. (*18*), Okabe (*19*), Mohan et al. (*20*), Velloso et al. (*21*), Fukuya et al. (*22*), Lee and Lumb (*23*), and Keenan and Bozozuk (*24*).

DESIGN ASPECTS

Fundamentals

In all the papers referenced in the foregoing, the emphasis was on the dragload. When the referenced authors reported observations of deformation and settlement, the main use of these was to calculate the loads in the pile. As to the consequence of the negative skin friction on the design, it was discussed in terms of reduction of the pile-bearing capacity or of the allowable load, not in terms of settlement.

In contrast, this paper suggests that the problem in designing for negative skin friction is one of settlement and not of bearing capacity (i.e., the magnitude of the dragload is of no relevance to the bearing capacity of the pile). Furthermore, the allowable load on the pile should be governed by a combined, unified approach considering soil resistance and settlement inseparably acting together and each influencing the magnitude of the other.

The published records of measurements of movements associated with negative skin friction indicate that extremely small relative movements—on the order of 1 mm—are sufficient to generate negative skin friction. Because of the considerable difference of stiffness between a pile and soil, all piles are subjected to relative movements of this magnitude. Therefore, all piles are subjected to negative skin friction, not just those in soils that settle significantly, and, in all piles, a neutral plane develops.

As mentioned, at the elevation of the neutral plane, there is no relative movement between the pile and the soil. Consequently, the settlement of the pile head is that of the soil at the neutral plane plus the "elastic" compression of the pile for the dead load and the dragload. To determine the location of the neutral plane, an analysis of the load distribution in the pile must first be performed.

Negative skin friction and the consequent dragload on piles cannot be treated separately from the settlement occurring in the soil, the pile movement relative to the soil, and the shaft and toe resistances of the pile.

The suggested design approach is essentially the same for all piles, whether single or in a group; whether installed in a soil that settles significantly under the influence of a surcharge, groundwater lowering, or other cause, or in a soil that does or does not experience appreciable settlement; and whether the piles are essentially toe bearing, shaft bearing, or both toe and shaft bearing. The design principles are equally applicable on piles in clay as on piles in sand, or in other coarse-grained soil, or in multilayered soil.

To understand the design principle, it is important to realize that the live load and the dragload do not combine and that two separate loading cases must be considered: dead load plus dragload, but no live load, and dead load plus live load, but no dragload. Furthermore, a rigid, high-capacity pile will experience a large dragload but small settlement, whereas a less rigid, smaller capacity pile will experience a smaller dragload but larger settlement. Also, while the dragload is caused by settlement or, rather, relative displacement, the dragload does not generate settlement, and no pile will settle more than the ground surface nearest the pile.

The design is carried out considering four aspects interactively: location of the neutral plane, structural capacity, settlement, and bearing capacity.

Neutral Plane

As a first step in the design, the neutral plane must be determined. The neutral plane is located where the negative skin friction changes over to positive shaft resistance. In other words, the neutral plane is determined by the requirement that the sum of the applied dead load plus the dragload is in equilibrium with the sum of the positive shaft resistance and the toe resistance of the pile. The neutral plane is located at the intersection of two load distribution curves construed as follows. First, as was illustrated in Figure 2, a load distribution curve ("forcing-load" curve) is drawn from the pile head and down, with the load value starting with the applied dead load and increasing with the load because of negative skin friction taken as acting along the entire length of the pile. Second, a load distribution curve ("resistance" curve) is drawn from the pile toe and up, starting with the value of the toe resistance and increasing with the positive shaft resistance.

Correct determination of the two load distribution curves is important for a correct design. Several theories exist whereby the capacity of a pile can be determined from soil data. Most of these theories have been developed by correlation to results of static loading tests. However, most static tests, even the ones that have been well instrumented, share one fallacy, namely, the measurements of the load induced in the test pile by loading the pile head fail to consider the load induced by the installation and the reconsolidation of the soil after the installation—the residual load, or residual compression, in the pile. Every pile will be subjected to a residual compression by the time it is instrumented for the static test. Assuming that the pile and the soil are unstressed before the load was placed on the pile head brings a large error into the interpretation of the test and the correlation of the data with theory.

The sometimes observed reduction of unit shaft resistance with depth can be satisfactorily explained by means of introducing a small residual compression into the observed pile compression under the applied load. So can the concept of the critical depth (*2*), which states that below some depth, called the "critical depth," both the unit shaft resistance and the unit toe resistance are constant and independent of the effective overburden stress below the critical depth. This does not mean that the critical depth concept is wrong, only that the issue is more complicated than thought. For a discussion, see also Vesic (*25*) and Hanna and Tan (*26*). To determine the load distribution curves requires reliable information on the soil strength properties. Then, for most problems encountered in practice, the theoretical analysis using the previously mentioned method of beta-coefficient on the effective overburden stress is preferred over any total stress, or "elastic" method. The analysis should be supplemented with information from penetrometer tests. Results from static test loading, preferably on piles that are instrumented with at least telltales, would be of significant assistance. However, it is beyond the scope of this paper to discuss the method for determining the load distribution or the many factors influencing the results of a static loading test.

In practice, a pile group will consist of piles of varying length installed to a capacity that varies between the piles. If each pile in the group were able to carry an equal portion of the dead load on the pile group, the location of the neutral plane would vary considerably between the piles. In reality, provided the pile cap is reasonably rigid, there is an equalization between the piles inasmuch as part of the dead load is transferred from "softer" piles to "stiffer" piles. In the process, the location of the neutral plane is equalized too. Of course, the neutral plane cannot be a horizontal or even plane, but must be shaped like a rolling surface. The location of the neutral plane determined according to the foregoing is therefore a mean location. This concept can be used to determine the load distribution between piles in a given pile cap considering known pile lengths, installation behavior, and so on.

Structural Capacity

The structural capacity is governed by the structural strength of the pile material at the neutral plane for the combination of dead load plus dragload—live load is not to be included. (At or below the pile cap, the structural strength of the embedded pile is determined as a short column subjected to dead load plus live load, but dragload is not included).

At the neutral plane, the pile is confined and it is suggested that the limiting value of maximum combined load be determined by applying a safety factor of 1.5 on the pile material strength (steel yield or concrete 28-day strength or long-term crushing strength of wood).

It should be realized that if both the negative skin friction and the positive shaft resistance, as well as the toe resistance values, are determined assuming soil strength values "erring" on the strong side, the calculated maximum load in the pile will be on the conservative side (and the neutral plane located deep down into the soil).

As illustrated in Figure 3, a reduction of the dead load on the pile will result in a lowering of the location of the neutral plane but have a proportionally smaller effect on the magnitude of the maximum load in the pile.

Settlement

As also demonstrated in Figure 3, the settlement of the pile head is determined by first calculating and plotting the distribution of settlement of the soil and then drawing a horizontal line from the neutral plane to intersect the settlement curve. The settlement of the pile is equal to the settlement of the soil at the elevation of the neutral plane plus the "elastic" compression of the pile from the sum of the dead load and the dragload.

To predict settlement correctly is difficult, in particular, in view of the dearth of results from field tests on pile groups. Until data become available from well-instrumented field tests having emphasized the measurement of settlement and deformation rather than the loads, the author suggests the following approach.

The settlement calculation should be carried out according to conventional methods for the effective stress increase caused by surcharge, groundwater lowering, and any other aspect influencing the stress in the soil. The calculation should include the dead load acting on the pile(s) as applied at the level of the neutral plane on an equivalent footing having the same size as the pile cap. Neither the dragload nor the live load should be included in the calculation. (Some judgment must be exercised as to whether a live load is of a duration that it should be considered at least partially a dead load or if it truly is temporary.)

A condition for the suggested approach is that the movement at the pile toe must be equal to or exceed the movement required to mobilize the ultimate toe resistance of the pile. (In most soils, this required movement is about 1 to 2 percent of the pile toe diameter of driven piles and about 5 to 10 percent of the toe diameter for bored piles). If the movement is smaller than this, the toe resistance will not be fully mobilized and the neutral plane will move to a higher elevation. In a design case where the toe resistance value is difficult to estimate or where it is variable, for instance in the case of toe-jetted piles, a conservative estimate of the settlement is obtained by disregarding the toe resistance when construing the location of the neutral plane.

The settlement calculations emphasize the settlement of the soil layers located below the neutral plane and must include the compression of silt and sand layers in the soil profile. This makes it important to carry the investigation of the soil conditions at a site to a sufficiently large depth and to include a representative amount of sampling and laboratory testing of the soils located below the pile toe. As a minimum, an investigation should include static cone penetrometer tests and sampling of all layers encountered with undisturbed samples taken of all cohesive soils.

The settlement calculation of noncohesive soils should not be based on the use of a constant "elastic" modulus, but on the tangent modulus approach, which considers that compression of soil does not increase linearly with increase of stress. The Janbu unified settlement theory, as detailed by the *Canadian Foundation Engineering Manual* (*27*), is particularly useful for calculating settlement in deep profiles of cohesive, as well as noncohesive, soils. The manual contains reference values of moduli for use in estimating soil compression.

It should be realized that if both the negative skin friction and the positive shaft resistance, as well as the toe resistance, are determined assuming soil strength values "erring" on the weak side, the calculated location of the neutral plane will be located higher up in the settlement diagram (i.e., the settlement of the pile will be calculated on the conservative side).

As illustrated in Figure 3, a reduction of the dead load on the pile will result in a lowering of the neutral plane and, therefore, a reduction of the settlement of the pile.

Bearing Capacity

The dragload must not be included when considering bearing capacity (i.e., the dragload is of no consequence for the analysis of soil bearing failure). Therefore, for bearing capacity consideration, it is incorrect to reduce the dead load by any portion of the dragload.

The dead load should only be reduced because of insufficient structural strength of the pile at the location of the neutral plane, where the pile is subjected to the combination of dead load and dragload, or by a necessity to lower the location of the neutral plane in order to reduce the amount of settlement.

The consideration of the bearing capacity in the design of a pile, or of a group of piles, amounts to making a check of the safety against plunging failure of the pile(s). In such a case, the pile moves down along its entire length and the negative skin friction is eliminated. Therefore, the load to apply on the pile in the bearing capacity analysis is the combination of dead and live loads. Dragload is not to be included.

Normally, when the pile capacity has been determined from results of a static loading test or analysis of data from dynamic monitoring, a factor of safety of two or larger ensures that the neutral plane is located below the midpoint of the pile. When the capacity is calculated from soil strength values, the factor of safety should not be smaller than three.

Special Considerations

It is clear from the foregoing that all piles will be subjected to negative skin friction and experience dragload. However, piles installed where soil settlement is small will not constitute a problem, unless the structural capacity of the pile is exceeded. Furthermore, the maximum dragload induced in a straight and vertical pile is not dependent on whether the settlement of the soil is large or small. However, for piles that are inclined, large settlement will force the pile to bend. For this reason, where large settlement is expected, it is advisable to avoid inclined piles in the foundation or, at least, to limit the inclination of the piles to values that can accept the settlement without its inducing excessive bending in the piles.

Piles that are bent, doglegged, or damaged during the installation will have a reduced ability to support the service load in a downdrag condition. Therefore, the unified design approach postulates that the pile installation is subjected to stringent quality control directed toward ensuring that the installed piles are sound and that bending, cracking, and local buckling do not occur.

Counteracting Negative Skin Friction

When the design calculations indicate that the settlement could be excessive, increasing the pile length or decreasing the pile diameter could improve the situation. When the calculations indicate that the pile structural capacity is insufficient, increasing the pile section, or increasing the strength of the pile material, could improve the situation. When such methods are not practical or economical, the negative skin friction can be reduced by the application of bituminous coating or other viscous coatings to the pile surfaces before the installation (*15–17*). For cast-in-place piles, floating sleeves have been used successfully.

Design Case History

A pile group consisting of 10 piles is to be installed at a site where the soil profile consists of a 2-m-thick fill recently placed over 8 m of soft-to-firm clay on a 4-m layer of sand below which lies a 20-m-thick layer of slightly overconsolidated, silty clay deposited on dense mixed granular soil, an ablation glacial till. The groundwater table is located at a depth of 2 m and the pore pressure is hydrostatically distributed throughout the profile. Details of the soil properties are given in Table 1.

The 10 piles consist of 300-mm-diameter pipe piles driven closed-toe to a total embedment of 38 m. The pile cap area is 3.5 m by 5.0 m = 17.5 m^2.

The piles will be concrete filled after driving and the allowable structural load of the pile at the neutral plane is 2100 kN. The intended allowable service load on the piles is 1400 kN of which 1200 kN is dead load and 200 kN is live load. Thus, the stress over the equivalent footing area is 686 kPa.

The bearing capacity of the pile is 2520 kN, when calculated in accordance with the beta-method and neglecting the critical depth concept. For the applied load of 1400 kN, the factor of safety against bearing failure is 1.80. The detailed results of the calculations are summarized in Table 2 and graphically presented in Figure 4.

TABLE 1 SOIL PROFILE AND SOIL PROPERTIES

Depth Range (m)	Type	Unit Weight (kN/m^3)	c' (kPa)	β	m	m_r	j	$\sigma_p' - \sigma_0'$ (kPa)
0–2	Fill	18.0	–	0.50	–		–	–
2–12	Clay	15.5	15	0.25	40	200	0	0
12–16	Sand	20.0	–	0.45	250	–	0.5	–
16–36	Clay	17.4	0	0.35	180	550	0	120
36–	Till	21.0	–	0.60	400	–	0.5	–

NOTE: The bearing capacity coefficient, N_t, of the till is 50. The letters m, m_r, and j indicate the modulus numbers and the stress exponent for the soil. The letter c' indicates the effective cohesion intercept and the symbols β, σ_0', and σ_p' indicate the beta coefficient, the existing effective stress in the soil, and the preconsolidation pressure, respectively.

TABLE 2 RESULT OF BEARING CAPACITY AND RESISTANCE CALCULATIONS

Factor	Determined Value
Applied loads	
Q_{tot} (kN)	1400
Q_d (kN)	1200
Q_l (kN)	200
Ultimate resistances	
R_{ult} (kN)	2520
R_s (kN)	1750
R_t (kN)	770
Dragload, Q_n (kN)	660
Maximum load, $Q_d + Q_n$ (kN)	1860
Depth, D_{NP} (m)	24.5

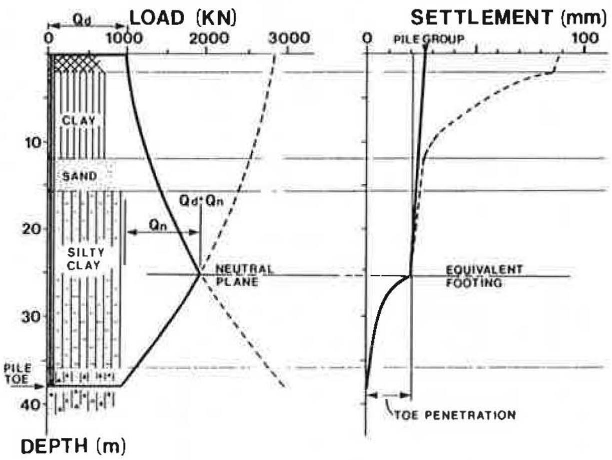

FIGURE 4 Design case history. Graphical presentation of the results of the design calculations.

The settlement for the pile group is calculated for the increased stress from the fill and the dead loads on the piles acting on and below the neutral plane. The stress from the equivalent footing is distributed according to the 2:1 method. The calculations indicate that the pile cap would settle slightly less than 30 mm and that the pile toe penetration into the till is about 15 mm, which movement is enough to develop the toe resistance assumed in the calculation of the neutral plane location.

NEED FOR RESEARCH

The proposed unified design approach shares one difficulty with all other approaches to pile group design, namely, there is a lack of thorough and representative full-scale observations of load distribution in piles and of settlement of pile foundations. With regard to settlement observations, the lack is almost total with respect to observations of settlement of both the piles and the soil adjacent to pile foundations.

In a typical design case, the shaft and toe resistance for a pile can only be estimated within a margin. For instance, in a given case, a designer cannot know whether the critical depth concept should be included in the calculation of resistance even when a routine static loading test is carried out. To provide the profession with reference cases for aid in design, it is very desirable that sturdy and accurate load cells be developed and installed in piles to register the load distribution in the pile during, immediately after, and with time after the installation. Naturally, such cells should be placed in piles subjected to static loading tests, but not exclusively in these piles (*28, 29*).

The greatest perceived need lies in the area of settlement observations. (It is paradoxical that pile foundations are normally resorted to for reasons of excessive settlement. Yet, the design is almost always based on a capacity rationale with disregard of settlement.) Actual pile foundations should be instrumented to determine both the settlement of the piles and the distribution of settlement in the soil near the piles. No instrumentation for study of settlement should be contemplated without an inclusion of piezometers.

CONCLUSIONS

A unified design of piles and pile groups is proposed wherein capacity, residual compression, negative skin friction, and settlement of both pile and soil are related. The design is carried out in four main steps: first, the location of the neutral plane is determined, then the adequacy of the structural strength of the pile is checked, followed by an analysis of the settlement of the pile foundation applying the concept of an equivalent footing placed at the neutral plane, and, finally, the adequacy of the bearing capacity of the pile is verified.

For structural capacity at the pile cap level, dead and live loads are considered together; for structural capacity at the neutral plane, dead load and dragload are considered together, but live load is excluded.

For settlement, all stress increase in the soil is considered, not just that of the dead load acting on the pile foundation. For bearing capacity, dead and live loads are considered, but dragload is excluded.

The design analysis is iterative, inasmuch as the choice of load and pile length will have an interactive influence on all aspects: the location of the neutral plane, the structural capacity, the settlement, and the bearing capacity.

NOTATION

β = beta-coefficient = $M\ K_s$ tan ϕ'
M = quotient of wall friction = tan δ'/tan ϕ'
δ' = angle of effective pile-soil friction
ϕ' = angle of effective internal soil friction
K_s = ratio of horizontal to vertical effective stress
Q_{tot} = allowable or applied total load
Q_d = allowable or applied dead load
Q_l = allowable or applied live load
Q_n = dragload
q_n = unit negative skin friction
R_{ult} = ultimate resistance
R_s = pile shaft resistance
r_s = unit pile shaft resistance
R_t = pile toe resistance
r_t = unit pile toe resistance
D = pile embedment depth
D_{NP} = depth to the neutral plane
m = modulus number–virgin curve
m_r = modulus number–reloading curve
j = stress exponent

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Publication of this paper sponsored by Committee on Foundations of Bridges and Other Structures.

greater depths, the controlling failure mechanism is assumed to be a horizontal flow failure of sand around the pier (*14*). A similar analysis principle was followed in this study to derive the ultimate resistance expression for the sloping ground surface condition (*10*). Although the presence of the ground slope will influence the mobilized passive resistance near the ground surface, the flow failure pattern, which governs at greater depths, is not significant for rigid piers having a relatively short length.

The total lateral resistance is given by the following expression (2):

$$Pu = \gamma H\ [H\ (S_{1\phi} + 3K_0\ S_{3\phi}) + bS_{2\phi} - K_a b] + c\ [H\ (S_{1c} + S_{3c}) + bS_{2c} - 2b\ K_a^{0.5}] \tag{8}$$

where

$$S_{1\phi} = \frac{\lambda 2 \tan \Omega \tan \beta}{(\tan \theta \tan \beta + 1)^2} [(\tan \theta \tan \beta + 1) \times (3 + 4 \tan \phi \tan \beta) - (2 \tan \phi \tan \beta)] \tag{9}$$

$$S_{2\phi} = \frac{2\lambda 2}{\tan \theta \tan \beta + 1} (1 + \tan^2 \phi) \tag{10}$$

$$S_{1c} = \frac{2 \tan \Omega \tan \beta}{(\tan \theta \tan \beta + 1)^2} [\lambda 1 (1 + 2 \tan \theta \tan^2 \beta + \tan \beta) + 2 \tan \beta (\tan \theta \tan \beta + 1) - \tan \beta] \tag{11}$$

$$S_{2c} = \lambda 1 = \frac{1 + \lambda 1 \tan \phi}{\tan \theta \tan \beta + 1} \tag{12}$$

$$S_{3\phi} = (\tan \phi - \tan \Omega) \times \left[\tan \beta - \frac{\tan^4 \beta \tan^3 \theta + \tan^3 \beta \tan^2 \theta}{(\tan \beta \tan \theta + 1)^3} \right] \tag{13}$$

$$S_{3c} = \tan \beta - \frac{\tan^3 \beta \tan^2 \theta + \tan^2 \beta \tan \theta}{(\tan \beta \tan \theta + 1)^2} \tag{14}$$

$$\lambda 1 = K1\ K_{pc} \tag{15}$$

$$\lambda 2 = \frac{K1}{2} \left(K_{p\phi} + \frac{K2}{\cos \beta} \right) \tag{16}$$

$$K1 = \frac{\cos \beta (\tan \theta \sin \beta + \cos \beta)}{H} \tag{17}$$

$$K2 = \frac{\tan \beta \sin \beta}{\tan \theta \tan \beta + 1} \tag{18}$$

$$K_{pc} = \frac{1}{(\tan \theta \sin \beta + \cos \beta)(\sin \beta - \cos \beta \tan \phi)} \tag{19}$$

$$K_{p\phi} = \frac{\tan \beta (\cos \beta + \sin \beta \tan \phi)}{(\tan \theta \tan \beta + 1)(\sin \beta - \cos \beta \tan \phi)} \tag{20}$$

where

b = pier diameter,
H = depth below ground surface,
β = $45 + \phi/2$,
θ = ground surface slope angle,
γ = soil effective unit weight,
ϕ = angle of internal friction,
c = cohesion intercept,
K_a = coefficient of active earth pressure,
K_0 = at-rest earth pressure coefficient
Ω = angle that defines the wedge size in front of the pier.

COMPUTER PROGRAM LTBASE

LTBASE [Lateral pier analysis including base and slope effects (*10*)] is a computer program for the load-deflection analysis of laterally loaded piles and drilled piers. The program uses the finite difference technique to solve the nonlinear simulation model formulated using the subgrade reaction approach. The program is coded in FORTRAN77 computer language, and the source code was compiled using the Microsoft FORTRAN77 version 3.2 compiler. The compiled code is linked to the MS-FORTRAN runtime library FORTRAN.L87, which supports an 8087 math coprocessor. The Microsoft 8086 object linker version 3.02 was used in the linking process. Double precision arithmetic is used throughout the program to enhance the accuracy of the solution.

The computer program was written for an IBM-compatible PC, and solves for deflection, bending moment, soil reaction, and soil moduli values as functions of depth under a given set of loads. The base resistance model and the ultimate resistance expressions presented in the previous section were implemented in the program. The structure of the computer program, LTBASE, consists of a main program and seven subroutines. The authors modified the subroutines LPILE1 and SOIL 2, given in the computer program LPILE (*8*), and used them in developing LTBASE.

The solution technique adopted in LTBASE requires that the soil be replaced by a set of nonlinear springs that conceptually define soil response curves. Such curves define the soil resistance, *p* (force per unit length along the pile) as a function of pier deflection, *y*. In general, the shape of a soil response curve, usually referred to as a *p-y* curve, is defined by the coefficient of lateral subgrade reaction, K_{ho}, and the ultimate soil resistance, P_u. As of this time, the sloping ground surface ultimate resistance expression has not been validated for the undrained clay soil condition (i.e., $\phi = 0$). Also, it is assumed that the K_{ho} values are unaffected by the presence of a sloping ground surface.

Several methods of formulating *p-y* curves have been developed (*3, 5, 6, 9, 12, 15–18*). Although each method uses many of the same soil parameters, the equations are formulated differently because each method was developed in conjunction with a particular set of field or laboratory tests. The soil response curves generated by LTBASE are according to the procedures described by Reese et al. (*15*) and Murchison and O'Neill (*16*), based on experimental data reported by Parker and Reese (*6*) for sands and according to the unified method developed by Sullivan for clays (*7, 16*).

COMPUTING TECHNIQUE

Although the mathematical formulation of the problem resulted in a number of simultaneous linear equations, the nature of the problem presented here is nonlinear and requires an iterative approach for solution. The reactions that are generated in the soil because of the pier deformations under a given loading condition must be such that the equations of static equilibrium and compatibility are satisfied.

LTBASE initially assumes zero base resistance. Values of lateral soil moduli and associated deflections are successively computed until convergence is achieved under the applied loads. Convergence to the correct solution is judged to have been obtained when the differences between deflections computed in Step $i+1$, y_{i+1}, and Step i, y_i, are less than a specified tolerance criterion.

The moment and horizontal shear resistance at the base are then determined using the evaluated base deformations. New deflection values along the pier are then calculated using the applied loads at the top and the computed moment and horizontal shear resistance at the base.

The computed new deflection values result in different soil reaction along the length of the pier, and different moment and horizontal shear resistance at the base from those used in the previous step. Therefore, using the new computed base resistance, new deflection values are computed along the pier. The procedure is repeated until convergence is achieved. Convergence to the correct solution is achieved when the following convergence criterion is reached:

$$y_{i+1} - y_i < \text{convergence tolerance}$$

where the convergence tolerance is specified by the user. A flow chart that outlines the computational sequence followed to evaluate the load-deflection response is presented in Figure 3.

LOAD DEFLECTION AND FACTOR OF SAFETY

The nonlinear lateral load-deflection response is evaluated incrementally. The calculation algorithm proceeds by computing the response under an initial loading condition equal to one-fourth of the design load specified by the user. The initial loads are then incremented within the program and the lateral response is evaluated. This procedure is continued until the deflection at the top of the pier exceeds a user-specified deflection criterion. The evaluated response for each load increment includes lateral deflection, moment, shear, soil modulus, and soil reaction, along the pier length.

The ultimate resistance of the soil is defined as the pier resistance corresponding to the user's specified limiting deflection. The factor of safety is evaluated by dividing the ultimate resistance, based on the deflection criterion, by the design load specified by the user.

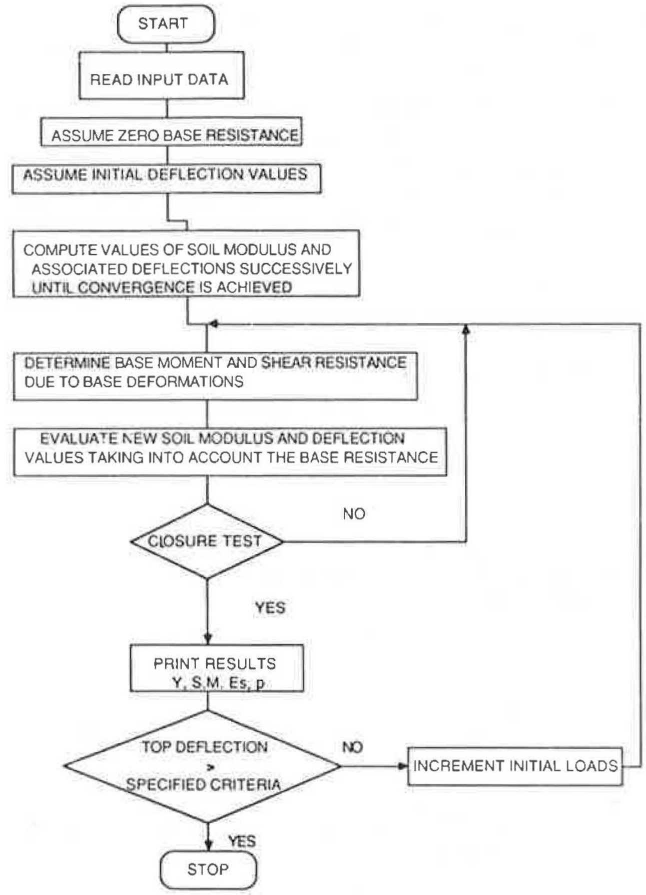

FIGURE 3 Flowchart of the computer program LTBASE.

The program is also capable of internally increasing the pier length. The length of each increment is equivalent to an element length, which is specified by the user in the data input file. The program proceeds by evaluating the factor of safety for the original input length. If this factor of safety is found to be less than a limiting factor of safety criterion, specified by the user, the program internally increments the length. The analysis is then performed using the new length. If the evaluated factor of safety is greater than or equal to the specified criterion, the run is terminated and the results are printed.

The maximum number of elements that can be added to the original length is three. If adding a new element will lead to placing the base of the pier in a soil with lower strength than that initially existing at the base of the pier, the pier capacity will be overpredicted. Caution should be exercised in interpreting the results for such cases.

PROGRAM CAPACITY

The capacity of the program is determined by the size of the variables in the COMMON statements. Currently the storage required for the sum of the code, data, and constants blocks is about 300 K. Maximum number of nodes and elements corresponding to this capacity is 101 and 100, respectively. The capacity could be increased by increasing the dimensions of the variables in the COMMON statement. However, the memory limitations of the computer system will be the controlling factor. Also, it should be noted that as the number of elements

LTBASE: A Computer Program for the Analysis of Laterally Loaded Piers Including Base and Slope Effects

MOHAMMED A. GABR AND ROY H. BORDEN

An improved model for the analysis of laterally loaded piers is presented. The model is based on the subgrade reaction concept and incorporates base shear and moment springs. The model is capable of accounting for the presence of a sloping ground surface. The computer code, LTBASE, which implements the model, is described. A comparison between the predicted and measured response of 16 load tests shows the inclusion of the base resistance in the conventional subgrade reaction approach to be desirable. The ability of the model to predict the behavior of piers on slopes is indicated by a favorable comparison between the predicted and measured response of five test piers.

Rigid pier foundations are commonly used to support a variety of structures subjected primarily to lateral loads. Highway overhead signs, light pole structures, and electrical transmission towers are all examples of situations in which the lateral loads transferred to the piers from the superstructure generally result in high overturning moments and relatively insignificant vertical loads. Although the majority of piers are constructed on horizontal ground surfaces, it is not uncommon to see them constructed in cut slopes or compacted embankments.

In general, the load-deflection analysis of laterally loaded piers is conducted without consideration for the influence of base resistance. Although it can be shown that this assumption is valid for piers with relatively large length/diameter (L/D) ratios, for the case of short rigid piers, the inclusion of base resistance can be significant.

Two theoretical approaches have generally been employed for predicting the lateral movement of piers. The elastic approach (*1*), which assumes the soil to be an ideal elastic continuum, and the subgrade reaction approach (*2–8*), in which the soil reaction at a point is related to the pier deflection at that point through a constant of subgrade reaction referred to as K_{ho}.

A model incorporating base resistance contribution to the lateral response of rigid piers has been presented (*9*). The authors used a linear 3-D finite element parametric study to define numerical values for the base subgrade moduli. Three different L/D ratios were used in the analysis in order to define base spring stiffnesses as a function of L/D ratio. Spring stiffness expressions for the base resistance were formulated by fitting empirical equations to the results obtained from the parametric study.

Using the subgrade reaction approach, the soil-pier interaction mechanism is modeled by treating the pier as a linear elastic beam and the soil reaction as a line load. Using a finite number of elements in a numerical solution, the interaction is represented by discrete nonlinear springs, with the spring stiffness varying as a nonlinear function of pier lateral deformation. The subgrade reaction concept provides a rational approach that permits the description of the nonlinear behavior of the soil-pier interaction system readily, if only approximately.

Presented in this paper is the computer program LTBASE, which was developed for the nonlinear analysis of piers subjected to lateral loads. The analysis is based on the subgrade reaction approach and incorporates the mobilized base resistance (*4*). The program is capable of analyzing cases where the piers are constructed on slopes (*10*).

BASE RESISTANCE MODEL

For rigid piers having relatively small L/D ratios, it has been shown that the soil at the base provides significant moment and horizontal shear resistance (*4*). Considering this effect, a difficulty would arise from the fact that the determination of such boundary condition is dependent on both the soil reaction and the pier response or, in a more commonly used term, is dependent on the soil-structure interaction mechanism. The interaction behavior is explained by the dependence of the magnitude of the base resistance on the amount of deformation at the base; the determination of such deformation requires the knowledge of the amount of the base resistance.

Referring to Figure 1, and assuming rigid body motion, the vertical and horizontal displacements of the base could be correlated to the angle of rotation of the pier, θ_r, as follows (*1*):

$$w_o = 2 * \mathrm{SIN}\,(\theta_r/2) * SQRT[C^2 + (D/2)^2] \quad (1)$$

$$W_o = (\Pi/2) - (\theta_r/2) - \mathrm{ARCTAN}\,[C/(D/2)] \quad (2)$$

$$y_o = w_o * \mathrm{COS}\,(W_o) \quad (3)$$

$$v_o = w_o * \mathrm{SIN}\,(W_o) \quad (4)$$

where

C = distance from the base to the center of rotation,

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D = pier diameter,
θ_r = angle of rotation of the pier,
y_o = horizontal displacement of the base as a result of rotation, θ_r, and
v_o = vertical displacement of the base as a result of rotation, θ_r.

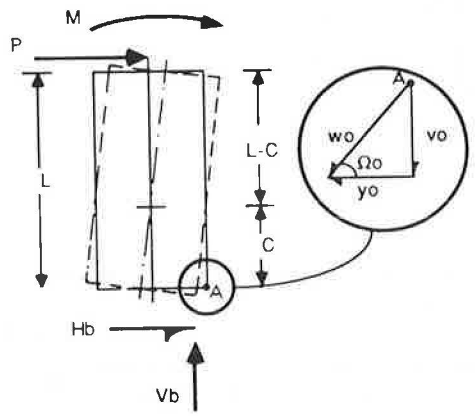

FIGURE 1 Base deformations as a function of pier rotation.

The horizontal shear resistance, H_b, and moment resistance, M_b, mobilized at the base as a result of rotation angle θ_r, are expressed as functions of horizontal and vertical deformations developed at the base. The normal reaction caused by the soil at the base will be a function of the vertical displacement resulting from pier rotation.

The base load–vertical displacement curve is shown in Figure 2. A linear relationship is assumed to exist between base vertical resistance and vertical deformation up to the failure of the soil under the base (*11–13*). Similarly, the shear force–lateral displacement relationship (Figure 2) is also assumed to be linear up to the mobilization of the full lateral shear force (*11–13*). Experiments have revealed that the ultimate shear resistance will develop at a shear movement of approximately 0.2 in., whereas a downward movement of about 5 percent of the pier diameter is necessary to mobilize the ultimate vertical base resistance (*11*).

However, because of the coupled pressure-deflection dependence at the base, the shear spring stiffness is defined as a function of the shear deformation, as well as the normal force developed at the base. The mobilized normal soil resistance is formulated as a function of the pier rotation angle, θ_r, in Figure 1. Accordingly, the resisting moment and lateral shear force developed at the base because of pier rotation, θ_r, were developed as follows (*4*):

$$V_b = \beta(v_o) * K_v * v_{ult} * (D^2/6) \tag{5}$$

$$H_b = \alpha(V_b, y_o) * V_b \tan(\delta) + Ca\,(\Pi D^2/8) \tag{6}$$

$$M_b = V_b * (3\Pi D/32) \tag{7}$$

where

K_v = vertical modulus of subgrade reaction;
V_b = normal soil reaction mobilized at the base as a result of rotation, θ_r;
H_b = horizontal shear resistance mobilized between the base and the soil as a result of rotation, θ_r;
δ = angle of friction between the base and the soil;
v_{ult} = vertical displacement required to mobilize the ultimate vertical base resistance;
M_b = mobilized resisting moment at the base as a result of rotation, θ_r;
$\alpha(V_b, y_o)$ = ratio of the lateral base deformation as a result of rotation angle, θ_r, to the lateral deformation required to mobilize the full lateral shear resistance. This coefficient is a function of the lateral base displacement, y_o, and the magnitude of the normal force, V_b, at the base;
$\beta(v_o)$ = ratio of vertical base deformation as a result of rotation angle, θ_r, to the vertical base deformation required to develop the ultimate vertical base resistance; and
Ca = undrained shear strength × adhesion factor at the base.

ULTIMATE RESISTANCE INCLUDING SLOPE EFFECT

An ultimate lateral resistance expression for piers constructed in cohesionless soil deposits with horizontal ground surface was presented by Reese (*14*). In this formula, two mechanisms of soil resistance are assumed to exist. Near the ground surface, a passive wedge is assumed to provide the lateral resistance. At

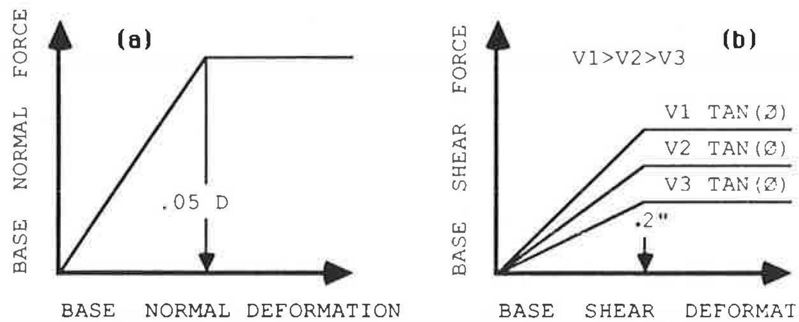

FIGURE 2 Base and shear normal springs.

used in the analysis gets larger, the computational time increases and could be as long as 15 minutes for an analysis using 100 elements, using a computer instrumented with an Intel 8087 numeric data coprocessor that has a clock speed of 4.77 MHz.

OUTPUT INFORMATION

For each successful execution, the program output is directed to three output files. The output files are saved on the default drive (i.e., the drive from which the program is loaded and executed). The name under which each output file is stored and the contents of each are described as follows:

- OUTPUT.PRN. This file contains information about critical input data and the output results for all the loading increments used in the analysis. If the option for internally increasing the length is specified, this file is expected to be relatively large and comprehensive. The size of this file might be as large as 300 K. This is approximately equivalent to the size of a double-density, double-sided floppy disk. During the preliminary analysis, if the option for the search of the appropriate length is selected, it is recommended that the user specify the option to suppress the printout of this file.

Once the appropriate length is found, a single run is executed using this length and the output file OUTPUT.PRN can then be printed. It would be beneficial to glance through this file to verify the input data. The information about the distribution of lateral deflection, moment, shear, and soil modulus, as a function of depth, is printed to this file. Also, the maximum shear and maximum moment in the pier, corresponding to each loading increment, are printed.

- SUMMARY.PRN. This file contains a summary of the applied loads, input soil properties, and the pile dimensions. The computed factor of safety, based on the predicted capacity, is printed whenever applicable. The factor of safety is printed each time the analysis is performed using a new length. A brief glance through this file would help the user to decide upon the appropriate pier length to be used.
- PLOT.PRN. This is a special file prepared for using the output results in association with any graphics software package to create a load-deflection plot. The output to this file consists of three columns. The third column represents the pier top deflection, y, corresponding to different loading increments. The first and second columns represent the lateral load, PT, and the value of the second boundary condition, $BC2$, applied at the top of the pier. In general, $BC2$ will be an applied moment or a specified pier top rotation. Short headings are used to help the user to identify the results.

EXAMPLE PROBLEM

An example problem is presented to demonstrate LTBASE capabilities. The example also serves to illustrate the ease of creating a batch input data file when a simple soil profile is encountered. The soil profile and the pier dimensions are given in Figure 4. The profile consists of a uniform sand deposit that has an angle of internal friction of 30 degrees. The ground surface is horizontal and the water table is located at a depth of 5 ft. The pier is 30 in. in diameter. The initial estimated length

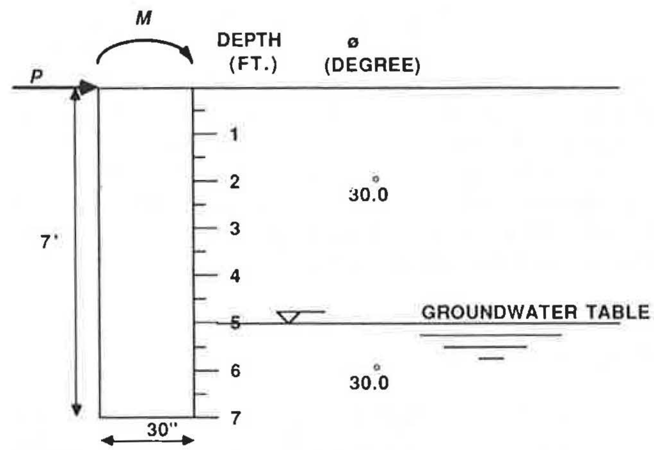

FIGURE 4 Example problem: soil profile and pier dimensions.

is chosen equal to 7 ft. The pier length is divided into seven increments, each increment having a length of 1 ft. In cases where part of the pier is extended above the ground surface, the choice of the element size should satisfy the following conditions:

$$N \times H = L \tag{21}$$

$$NU \times H = Le \tag{22}$$

where

N = number of elements,
NU = number of elements above ground surface,
H = element length,
L = pier length, and
Le = exposed pier length, above the ground surface.

The tolerance of convergence is chosen equal to 1×10^{-3} in. The maximum deflection criterion, at the top of the pier, is taken equal to 3.0 in. The applied design loads at the pier top are assumed to equal 2 kips lateral load, and 60 K-ft applied ground moment. This simulates a condition where a lateral load of 2 kips is applied at the top of a 30-ft column supported by the pier being analyzed. The pier head is assumed to be free to rotate. The limiting factor of safety criterion is chosen to equal 1.5. Once this value is achieved, the program execution is ended and the results are printed.

Because of the groundwater table, the soil profile is divided into two sublayers. The first layer is 5 ft thick (from the ground surface to the groundwater level) and the second layer is 2 ft thick (from the groundwater level to the bottom of the pier). The existence of the water table is accounted for by using the submerged unit weight for the soil below the groundwater level. The unit weight of the soil above and below the water table is taken to equal 120 pcf and 57.6 pcf, respectively. The values of the coefficient of lateral subgrade reaction, K_{ho}, were chosen according to the values given by Reese et al. (7).

The vertical subgrade reaction coefficient, K_v, used at the base of the pier was obtained by assuming that the full base normal resistance will be mobilized at a downward movement of 5 percent D, and equal to 90 psi, according to the bearing capacity expression by Kulhawy (*19*). Based on the assumption of a linear relationship between normal base resistance and downward movement to failure, K_v was computed to be approximately 60 pci. The option for internally increasing the pier diameter ratio was selected. The printout of the output file OUTPUT.PRN was suppressed because of space limitations. Problem input data are shown in Figure 5.

OUTPUT RESULTS

The output file SUMMARY.PRN, given in Figure 6, provides the computed factor of safety as a function of pier length. The pier length was incremented by two elements, each 1 ft long. When the pier length reached 9 ft, the computed factor of safety was found to be 1.98, which is higher than the specified minimum of 1.5. The execution of the program was then automatically terminated and the results printed.

It is clear from the output file SUMMARY.PRN that the length that satisfies the factor of safety criterion is approximately 8 ft. Once this length is found, a single run is executed using the length of 8 ft, and the output file OUTPUT.PRN is created. The file OUTPUT.PRN contains comprehensive information about the input data as well as the analysis results. The input data are printed for user verification. The pier deflection, moment, shear, and soil modulus are printed as a function of depth. Lateral pier top deflection is plotted versus the applied ground moment in Figure 7. The data required to produce such a plot are written to the file PLOT.PRN.

SIGNIFICANCE OF BASE RESISTANCE

The results of a parametric study indicating the significance of base resistance on the predicted ultimate capacity of 2.5-ft-diameter piers are shown in Figure 8. In this study, the ultimate capacity is defined as the moment resistance corresponding to 2 degrees pier rotation. The vertical subgrade reaction coefficient, Kv, at the base of the pier was obtained by assuming that the ultimate base normal resistance, q_{ult}, will be mobilized at a downward movement of 5 percent D and equal to 150 psi. Based on the assumption of a linear relationship between normal base resistance and downward movement up to the deformation corresponding to failure, Kv was taken to equal 100 pci. The surrounding soil was chosen to consist of a

```
 1. LTBASE
 2. EXAMPLE RUN
 3. NCSU
 4. M. A. GABR
 5. 5/18/87
 6. 1
 7. 2.0 60. 1 1.5
 8. 30.0 1.0 .001 3. 7 0   1 0 0
 9. 30.0  60. 00.0
10. 00.0  0.
11. 2
12. 5.0 30. 120.0  30.0 100.  00. .000 0
13. 7.0 30. 57.60  30.0 60.0  00. .000 0
14. 1
15. .293E+11 7.
```

<u>Notes Added for Explanation:</u>

1. Job title
2. Job number
3. Job location
4. Operator
5. Date
6. Option to specify single run or multiple runs using incremented length
7. Lateral load — Moment — Code to indicate that the applied load is a Moment — F.S. Criteria
8. Diam. — Length increment — Convergence tolerance — Deflec. limit
   No. of elements — Option to internally generate P-y curves — Option to printout P-y curves
   No. of elements above G.S. — Option to generate output file "output.prn"
9. ϕ at the base — K_v at the base — C_u at the base
10. Ground surface slope angle in the front — Ground surface slope angle in the back
11. No. of soil sublayers

12,13. Soil properties, pier diameter and option to generate P-y curves

14. No. of different pier EI's
15. E_pI_p

FIGURE 5 Data for the sample run.

LTBASE

PROJECT NO.: Example Run
LOCATION: NCSU
ANALYSIS RUN BY: M. A. GABR
DATE: 5/18/87

SUMMARY

**** INPUT SOIL PROPERTIES AS A FUNCTION OF DEPTH ****

DEPTH, FT	UNIT WEIGHT, PCF	PHI, DEGREE	C, PSI
1.00	120.00	30.00	.00
2.00	120.00	30.00	.00
3.00	120.00	30.00	.00
4.00	120.00	30.00	.00
5.00	120.00	30.00	.00
6.00	57.60	30.00	.00
7.00	57.60	30.00	.00

IMPORTANT NOTE

AS THE PIER IS INCREASED IN LENGTH BEYOND THAT INITIALLY SPECIFIED, THIS PROGRAM ASSUMES THAT THE SOIL PROPERTIES REMAIN CONSTANT WITH DEPTH BELOW THE INITIAL BASE ELEVATION. IF THE SOIL STRENGTH ACTUALLY DECREASES WITH DEPTH, THE LATERAL CAPACITY WILL BE OVER-PREDICTED.

**** INPUT LOADS, LIMITING DEFLECTION AND PILE DIAMETER ****

PT, KIPS	BC2, K-FT	BC CASE	LIMIT DEFL., IN.	DIAMETER, IN.
2.00	60.00	1	3.000	30.000

*** GROUND SURFACE SLOPE ANGLE IN THE FRONT, = .0000 DEGREES ***
*** GROUND SURFACE SLOPE ANGLE IN THE BACK, = .0000 DEGREES ***

PILE LENGTH, FT	FACTOR OF SAFETY
7.00	1.13
8.00	1.48
9.00	1.98

FIGURE 6 Output file, SUMMARY.PRN.

uniform sand deposit with angle of internal friction ϕ of 30 degrees.

For L/D ratios greater than four, Figure 8 indicates that the base resistance accounts for an increase in capacity of approx imately 10 percent. However, as the L/D ratio decreases, the importance of the base resistance is obvious. For an L/D ratio of 2.5, the model indicates that the capacity of a pier could be underpredicted by slightly more than 25 percent if the base resistance is not included. For L/D ratios less than 2.5, the significance is even greater.

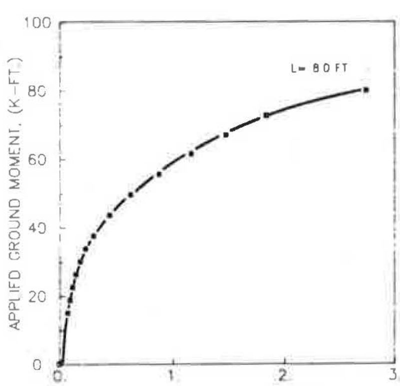

FIGURE 7 Lateral load-deflection response, sample problem.

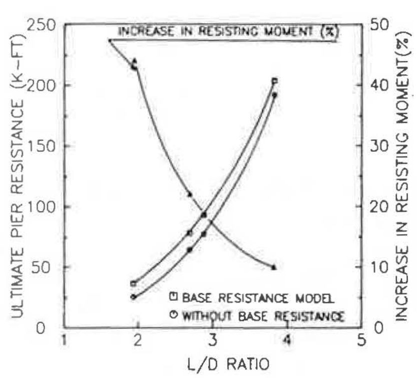

FIGURE 8 Increase in capacity as a function of L/D ratio.

EFFECT OF SLOPING GROUND SURFACE

An analysis performed by the authors (*10*) indicated that the percent reduction in lateral capacity due to the presence of a sloping ground surface was essentially independent of pier dimensions and soil properties. The ratio of sloping ground surface capacity to horizontal ground surface capacity was found to be simply a function of the value of the ground surface slope angle, θ. A parametric study performed on a 2.5-ft-diameter and 7-ft-long pier is shown to illustrate the influence of the slope presence on the overall capacity. An idealized subsurface sand deposit with ϕ equal to 30 degrees was used. The *p-y* curves were internally generated by LTBASE using the procedure described by Reese et al. (*8*). However, the ultimate lateral resistance, used in the construction of the *p-y* curves, was computed using the developed expressions that account for the slope.

A significant decrease in capacity was observed as a result of the slope presence. Figure 9 shows that for a ground surface slope angle, θ, of 15 degrees, a 32 percent reduction in capacity is predicted for a pier rotation of 2 degrees. The reduction is somewhat less for smaller deformations. At a pier rotation of 0.1 degrees, the 15-degree slope angle resulted in only a 5 percent reduction in capacity. The smaller reduction in capacity at the 0.1-degree rotation level is a result of using the same K_{ho} values for both horizontal and sloping ground surface cases. As mentioned earlier, it remains to be investigated whether the slope presence would have an effect on the values of the coefficient of lateral subgrade reaction, K_{ho}.

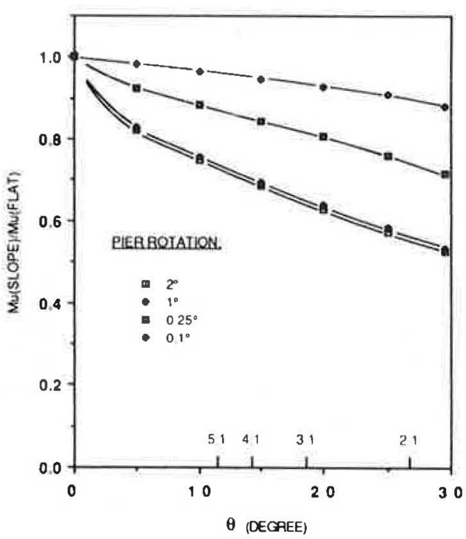

FIGURE 9 Moment capacity ratio as a function of ground slope angle.

COMPARISONS OF PREDICTED AND OBSERVED FIELD BEHAVIOR

Predictions of the lateral response of 16 load tests performed by GAI and ITT (*9*) were performed using LTBASE. An extensive documentation of the load test procedures, soil properties, idealized profile of each site, and the corresponding organization can be found elsewhere (*9*). The soil properties used as input data were those reported by the test performers. The *p-y* curve procedure developed by Reese et al. (*15*) was used whenever a sand profile was encountered and the procedure developed by Sullivan (*7*) was used for clay. The base load-deformation relationships were developed following the procedure described previously in this paper.

To summarize the results and aid in evaluating model predictive capability, the measured versus predicted capacities are plotted for 1 and 2 degrees of pier rotation in Figure 10. Tests for which pier rotation as a function of applied moment was not provided were evaluated assuming that the pier rotated about a point two-thirds of the pier length down from the top. Rotations of 1 and 2 degrees correspond to pier top lateral deflections of 1.4 and 2.8 in., respectively, for a pier length of 10 ft.

The data are shown in conjunction with a 45-degree line, indicating perfect agreement between measured and predicted responses. Lines representing predicted values equal to 1.25 and 0.75 times the measured response are also shown. Linear regression analyses were performed on the data and resulted in slopes of 0.82 and 0.83 for the 1- and 2-degree rotation plots, respectively. It is shown from the foregoing statistics that the predictions are generally conservative and tend to somewhat underestimate the pier capacity. However, it should be noted that, at small deformation, the predicted response would be dependent, mostly, on the coefficient of lateral subgrade reaction and its distribution with depth. As the deformation increases, the predicted response becomes more governed by the ultimate lateral soil resistance and the base resistance.

For comparison, the measured versus the predicted capacities, using the computer program COM624 developed by Reese, are shown in Figure 11. The slope of the regression lines, standard error of the slopes, and the regression coefficients are given in Table 1. The standard error values indicate the variation of the data points around the slope of the best fit line. Using COM624, it was found that approximately 68 percent of the predicted responses were less than 0.75 times the actual measured field behavior, as shown in Figure 11.

TABLE 1 STATISTICAL PARAMETERS EVALUATED AT 1- AND 2-DEGREE ROTATION

	LTBASE		COM624	
	Degrees			
	1	2	1	2
Slope of regression line	0.83	0.82	0.64	0.56
Standard error of slope	0.04	0.04	0.04	0.04
Coefficient of correlation, R^2	0.87	0.85	0.59	0.56
Percent predictions exceed 1.25 the measured capacity	0	0	0	0
Percent predictions less than 0.75 the measured capacity	13	13	68	68

PIERS CONSTRUCTED ON SLOPES

The predicted and measured field behavior of five field load tests of drilled piers constructed in profiles with sloping ground surfaces are presented (Figure 12). One of the test piers was embedded in a cohesionless soil with a 3.5:1 sloping ground surface, whereas the other four were in a residual soil profile with a 2.2:1 sloping ground surface. All the piers were loaded

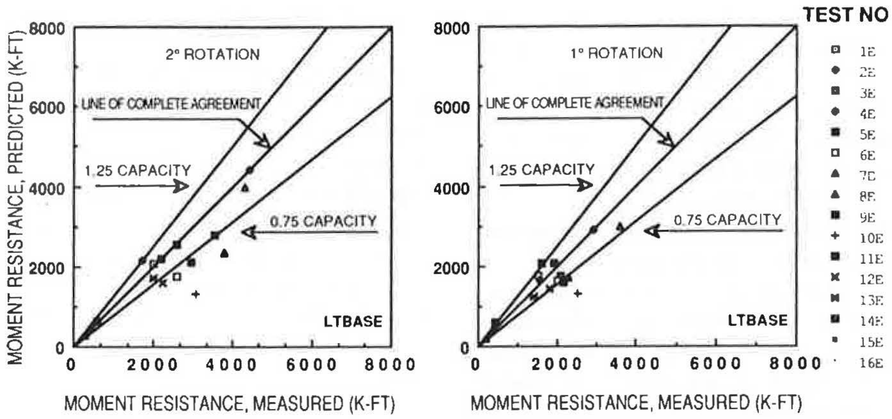

FIGURE 10 Predicted versus measured moment resistance at 1 and 2 degrees rotation: LTBASE.

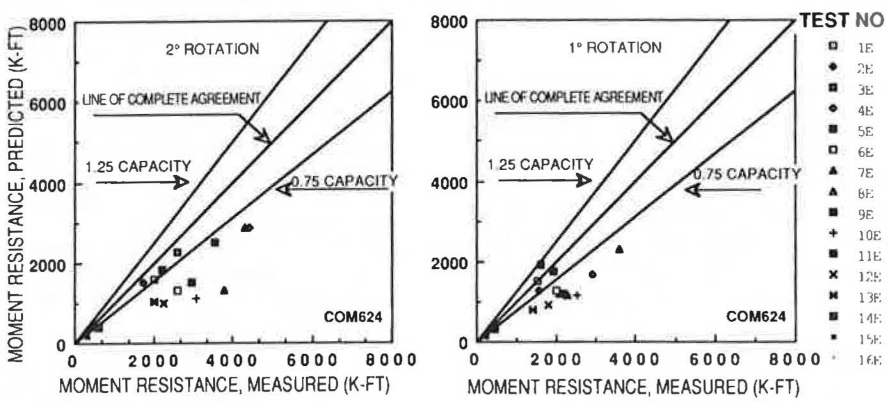

FIGURE 11 Predicted versus measured moment resistance at 1 and 2 degrees rotation: COM624.

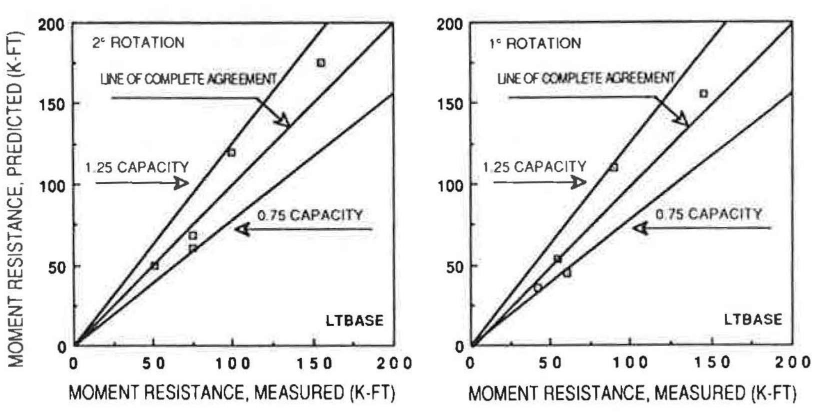

FIGURE 12 Predicted versus measured moment resistance at 1 and 2 degrees rotation for laterally loaded piers on slopes.

in the down slope direction. Detailed descriptions of the test procedures, site soil properties, and load test results can be found elsewhere (*20*).

In general, the measured responses were in reasonably good agreement with those predicted. It should be noted that at the early stage of the moment-deflection curve (for small deflections) the predicted response is highly dependent on the value of the coefficient of subgrade reaction, K_{ho}. This establishes the need of a procedure for the evaluation of K_{ho} in residual soils.

In three of the five piers tested, the measured capacity was somewhat underestimated at large deformations, although the other two predictions overpredicted the capacity by a nearly equal percentage. Nevertheless, it should be noted that the ultimate resistance values, *Pu*, computed using the developed slope expressions (*10, 20*), are modified using an empirical ultimate capacity adjustment parameter introduced by Reese et al. (*7*). This parameter, which is a function of pier diameter and depth, was developed to force *Pu* from theory to match test results. However, this parameter was developed on the basis of test data from piles constructed in a horizontal ground surface profile. It remains to be seen if this parameter is influenced by the presence of a sloping ground surface.

Moreover, the formulation of the theoretical expression included the effect of the active force developed behind the pier (*10, 20*). It has been observed that, at least for partially saturated soil deposits, this active force does not develop in the field (*9, 10, 20*). This active force ultimately has an influence equal to about 5 to 10 percent of the passive resistance. However, for generality it was deemed desirable to include the active force effect. In the final analysis, a solution including this effect is conservative and therefore acceptable at this time.

SUMMARY

This paper presents the capabilities of the computer program LTBASE, which has been developed to evaluate the nonlinear lateral load-deflection response of laterally loaded piers. A procedure to account for the influence of mobilized resistance at the base of the piers on the predicted lateral response is described. Also, a methodology supported by theoretical formulation is implemented in the program for the analysis of cases where the laterally loaded piers are constructed on slopes.

The importance of considering base resistance effects and sloping ground surface has been presented. For a 30-in.-diameter pier with an L/D ratio of 2.5, in loose sand, the base resistance is shown to increase the predicted ultimate moment capacity by more than 25 percent. The corresponding capacity at any design deflection level is also increased, although to a lesser degree for smaller rotations. On the contrary, the presence of a sloping ground surface decreases the pier capacity. For the same pier constructed in a 15-degree slope, 3.7:1, the predicted capacity corresponding to 2-degree rotation is shown to decrease by approximately 32 percent.

The capability of LTBASE to predict the measured field behavior of 16 piers constructed in horizontal ground surface profiles and 5 piers constructed in sloping ground surface profiles was also demonstrated. In general, the predictions obtained from LTBASE agreed reasonably well with the measured field behavior, with all but one prediction within ±25 percent of the measured response.

LTBASE is capable of running on an IBM-compatible PC. Storage required for the sum of the code, data, and constants blocks is about 300 K. Minimum amount of random access memory (RAM) required to run the program is about 200 K. Maximum number of nodes and elements corresponding to this capacity is 101 and 100, respectively. The program is coded in FORTRAN77 computer language. The source code was compiled using the Microsoft FORTRAN77 version 3.2 compiler. The compiled code is linked to MS-FORTRAN runtime library, FORTRAN.L87, which supports an 8087 math coprocessor. The Microsoft 8086 object linker version 3.02 was used in the linking process. Double precision arithmetic is used throughout the program to enhance the accuracy of the solution.

ACKNOWLEDGMENTS

This research was sponsored by the North Carolina Department of Transportation and Highway Safety (NCDOT) in cooperation with the FHWA, U.S. Department of Transportation. A technical advisory committee chaired by W. G. Marley and including R. Martin, J. Ledbetter, and P. Strong of the NCDOT, and D. D. Rhame of FHWA aided in the coordination of the research and are gratefully acknowledged. The authors also wish to thank the NCDOT personnel who supported the geotechnical investigation and lateral load testing.

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Publication of this paper sponsored by Committee on Foundations of Bridges and Other Structures.