NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP Report 393

Design and Construction Guidelines for Downdrag on Uncoated and Bitumen-Coated Piles

Transportation Research Board
National Research Council

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Design and Construction Guidelines for Downdrag on Uncoated and Bitumen-Coated Piles

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Subject Areas

Bridges, Other Structures, Hydraulics, and Hydrology
Soils, Geology, and Foundations

Research Sponsored by the American Association of State Highway and Transportation Officials in Cooperation with the Federal Highway Administration

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY PRESS
Washington, D.C. 1997

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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NCHRP REPORT 393

Project 24-5, FY'88

ISSN 0077-5614

ISBN 0-309-06065-6

L. C. Catalog Card No. 97-60938

Price $29.00

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

and can be ordered through the Internet at:

http://www.nas.edu/trb/index.html

Printed in the United States of America

FOREWORD

by Staff
Transportation Research Board

These guidelines are the result of a study using bitumen-coated piles to reduce downdrag. The guidelines provide a comprehensive description of the downdrag problem, an analysis of the behavior of piles subjected to downdrag, a selection of the bitumen coating, and a step-by-step design procedure. The PILENEG software described herein is available on the World Wide Web at http://civilgrads.tamu.edu/briaud. The research agency's final report and a video on the design and construction of bitumen-coated piles are available from the NCHRP upon request.

The settlement of soils surrounding foundation piles can cause downdrag forces on the piles significantly larger than the structural loads that the piles must carry. This additional load may result in unacceptable settlements of the piles or even failure of part of the pile group. Downdrag forces can be reduced by the use of bitumen-coated piles.

Under NCHRP Project 24-5, *Downdrag on Bitumen-Coated Piles,* the Texas Transportation Institute was assigned the task of developing practical guidelines for using bitumen-coated piles to reduce downdrag. The research team reviewed relevant domestic and foreign literature and conducted laboratory and field testing. A practical procedure for the design of bitumen-coated piles was developed.

The design procedure with examples is included in these guidelines. The PILENEG software developed as part of this project and described in these guidelines allows users to analyze a pile subjected to downdrag. PILENEG is available on the Internet World Wide Web at http://civilgrads.tamu.edu/briaud. Readers will note that the agency final research report, "Research on Uncoated and Bitumen-Coated Piles Subjected to Downdrag," which describes the research that led to the guidelines, is not published herein. This report and a video, which describes the elements of design and emphasizes the proper steps in the construction process, are available for $15.00 each on request to the NCHRP, 2101 Constitution Avenue, N.W., Washington, DC 20418.

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ACKNOWLEDGMENTS

This work was sponsored by the American Association of State Highway and Transportation Officials in cooperation with Alberta Transportation and Utilities, the Louisiana Department of Transportation, the Federal Highway Administration and the National Timber Pile Council. It was conducted through the National Cooperative Highway Research Program, which is administered by the Transportation Research Board of the National Research Council.

The NCHRP Project 24-5 panel was very helpful throughout the study and consisted of George Machan (Chair), Paul F. Bailey (New York State DOT), Murty S. Devata (Ontario Ministry of Transport and Communication), John B. Gilmore (Colorado Dept. of Highways), Walter A. Kuroiwa (Hawaii DOT), Richard Long (University of Connecticut), Harold C. Maltrow, Jr. (U.S. Geological Survey), V. P. Puzinauskas (Asphalt Institute), Al DiMillio (FHWA), G. P. Jayaprakash (TRB), Frank R. McCullagh and D. Wm. Dearasaugh (NCHRP).

The following individuals and companies contributed time, money, materials and services to make this project possible: Mark Morvant and J. B. Esnard (Louisiana DOTD), Vishnu Diyaljee and Tim Morrison (Alberta T&U), Carl Ealy (FHWA), Len Chase and Bill Sadoway (Alberta Research Council), Eric Ng and Steve Tsang (GEO/Resource Consultants), Jim Graham (National Timber Pile Council), Albert Pittman (Pittman Corporations), and Henry Whitty and Steve Whitty (S. K. Whitty & Co., Inc.). The project included full-scale testing on 6 piles in Edmonton, Alberta, Canada; full-scale testing on 8 piles in New Orleans, Louisiana, U.S.; and laboratory testing, computer simulation and data analysis at Texas A&M University. The research team at Texas A&M University was composed of Jean-Louis Briaud (Principal Investigator), Randy Bush, Rajan Viswanathan, Sangseom Jeong, Mohamed Quraishi, Zaid Al Gurgia, Larry Tucker, Albert Yeung, Alan Letton, Dallas Little, Robert Lytton, and Yavuz Corapcioglu. In addition, Michael O'Neill and Tom Davisson were special consultants.

DESIGN AND CONSTRUCTION GUIDELINES FOR DOWNDRAG ON UNCOATED AND BITUMEN-COATED PILES

SUMMARY

These guidelines are based on the results of a large research project complemented by a literature review and discussions with other experts. The objective of this project was to develop practical guidelines for the use of bitumen-coated piles to reduce downdrag. To address the objective of the project, the following tasks were performed:

Field Tests

Two sites were selected, one in Edmonton, Alberta, Canada, to address cold climate concerns and one in New Orleans, Louisiana, to address hot climiate concerns. In Edmonton, six full-scale steel pipe piles were instrumented and installed. Four were coated with various bitumens and two were uncoated. In New Orleans, eight full-scale piles were instrumented and installed. Five were coated with various bitumens and three were uncoated. The pile types included two steel pipe piles, three timber piles, and three prestressed concrete piles. All piles were load tested in compression and in tension, and were monitored for downdrag over a 2-year period. The soil at the site was tested and instrumented.

Laboratory Tests

Nine different bitumens were selected and subjected to a series of conventional tests. Then special tests were designed to simulate field problems including the flow test, the rod shear test, and the particle penetration test. Rheometer tests were also performed and were chosen as the basis for the bitumen-selection process.

Analysis

The analysis part of this study started with a literature review, documented in Briaud et al. (1989). It was followed by the analysis of data accumulated in the field and in the laboratory. Some parallel computer simulations were performed to study the group effect. The PILENEG program was developed based on fundamental observations of the behavior of a single pile. Finally, the knowledge accumulated in all aspects of the study was organized to formulate a fundamental and simple procedure

to select a window of satisfactory bitumens for any given situation. This procedure was validated by comparison against the field observations. The outcome of this research effort is a comprehensive step-by-step design procedure to solve the problem of uncoated and bitumen-coated piles subjected to downdrag.

Environmental Study

The concern of water and soil contamination by the bitumen was studied at the Edmonton and New Orleans sites. Monitoring wells were installed and monitored. Water and soil samples were retrieved and analyzed for Polycyclic Aromatic Hydrocarbons (PAHs). Laboratory tests were also performed at Texas A&M University to simulate the potential leaching of PAHs from bitumen.

CONCLUSIONS

The tasks, field tests, laboratory tests, environmental studies, and analyses undertaken in this study have led to the following major conclusions.

Field Tests

1. There was essentially no difference between the pile driving blow count for the coated piles and the uncoated piles.
2. In New Orleans, wood, steel, and concrete piles were load tested; the load test results on the uncoated piles showed that there was practically no difference in unit skin friction between the different pile, with a slightly higher value for the timber pile.
3. The compression and tension load tests show that the long-term bitumen friction values cannot be evaluated by such rapid tests. There was essentially no difference in capacity between the coated and uncoated piles; the failure occurred in the clay itself.
4. For the uncoated piles the compression and tension load tests gave maximum unit skin friction values which were comparable to the maximum unit skin friction values obtained in the long-term downdrag monitoring tests.
5. Seven different types of bitumens were used to coat the piles. The percent reduction in the downdrag load because of the bitumen coating varied from 0% to 100%. In Edmonton, two of the four bitumens chosen were effective. In New Orleans, none of the bitumens chosen were effective.
6. The β values back calculated from the downdrag monitoring on the uncoated piles averaged about 1.0 in the fill to about 0.2 in the soft clays. The variation of β with depth and from one pile to another was significant.
7. The friction values back calculated from the downdrag monitoring on the uncoated piles were close to the undrained shear strength of the clays.
8. For the uncoated piles and the piles coated with bitumens which did not reduce the downdrag, the downdrag load after 2 years was larger than the downdrag load after 2 months by a factor which varied from 1.1 to 1.5.
9. For the piles coated with a bitumen which reduced the downdrag significantly, the downdrag load after 2 years was about the same as the downdrag load after 2 months and sometimes was smaller.

10. All the field tests were very useful in gathering information on the dos and don'ts in the construction aspect of bitumen-coated piles. These construction aspects included preparation of the pile surface, application of the primer and of the bitumen, handling and storage, and driving. This information on the construction aspects was very helpful in the preparation of guidelines for specifications.

Laboratory Tests

1. Nine bitumens were selected based on a number of factors including past experience, manufacturer's opinion, and limited testing. Conventional classification tests were performed; the penetration at 25° varied from 25 to 75 mm and the softening point from 45° to 156°C.
2. A series of new laboratory tests were developed to simulate the field conditions in an effort to find a simple test for bitumen selection. The tests included the flow test for storage simulation, the rod shear test for downdrag simulation, the particle penetration test for long-term penetration of large particles, and the freezing test for bitumen behavior under very cold temperatures. On the basis of these tests, the researchers developed an understanding of the behavior of bitumen and selected the bitumens for the field tests.
3. The flow test showed that bitumen deformation is extremely sensitive to temperature; for example, at a reasonable shear strain rate for pile storage of $10^{-6}s^{-1}$, if the temperature is doubled from 10°C to 20°C, the viscosity can be divided by a factor of 10 from 3×10^{7} Pa.s to 3×10^{8} Pa.s (Bearing Pile Lubricant).
4. The rod shear test showed that bitumen deformation is quite sensitive to shear strain rate; for example, at a reasonable soil temperature of 10°C, if the shear strain rate is doubled from $1 \times 10^{-6}s^{-1}$ to $2 \times 10^{-6s}s^{-1}$, the viscosity can be doubled from approximately 2.5×10^{8} Pa.s to 5×10^{8} Pa.s (Bearing Pile Lubricant).
5. The particle penetration test showed that the larger the particles of soil are the more rapid the penetration is into the bitumen coating. These tests allowed the researchers to develop some guidelines for selecting bitumens, on the basis of a tolerable particle penetration after 50 years.
6. The freezing tests indicated that at -20°C the softer bitumens crack and spall less than the stiffer bitumens. These tests also showed that if no primer is applied or if the primer has not cured when the bitumen is applied, the tendency to crack and spall at -20°C is significantly increased.
7. While all these tests helped the researchers understand bitumen behavior and the influence of key factors, none of them alone yielded the kind of fundamental information needed. This information should allow the engineer to handle the bitumen problems associated with pile storage, pile driving, downdrag loads, and particle penetration. The rheometer tests give viscosity as a function of temperature and shear strain rates. Such tests were performed on seven of the nine bitumens.

Analysis

1. The amount of data accumulated was reduced and analyzed. It led to the choice of the following bitumen behavior model:

$$\tau = \eta\dot{\gamma}$$

where τ is the shear stress applied (Pa), $\dot{\gamma}$ is the shear strain rate (s^{-1}) and η is the secant viscosity (Pa.s). Note that 1 Pa.s is equal to 10 poises, a common unit for viscosity values.

2. Several criteria were established for the bitumen selection; the bitumen should not sag excessively during storage, the bitumen should not sag excessively during driving, the bitumen should offer very little resistance to soil downdrag, and the bitumen should resist soil particle penetration.
3. These criteria were converted into requirements on the viscosity of bitumen. Using the modeling law above and fundamental equations, means of calculating the shear stress, the shear strain rate, and the temperature were established for each of the criteria: storage, driving, downdrag. The requirement for each criterion would state that the appropriate bitumen should have a viscosity at this temperature and at this shear strain rate larger (storage and driving) and smaller (downdrag) than the calculated required viscosities. The particle penetration problem was handled separately.
4. These critera were finalized late into the project. They were applied to the case histories of Edmonton and New Orleans. The fact that findings on the efficiency of the various bitumens to reduce downdrag in the field matches the predictions according to the proposed criteria gave credibility to the proposed approach.
5. The decision process was developed and outlined in the design guidelines in a step-by-step procedure together with a complete example. This example used much of the soil data and pile data of the two field sites and gives an evaluation of seven of the nine bitumens used.
6. Overall, it is easier to find an appropriate bitumen when the air temperature is lower or equal to the soil temperature (winter months); indeed, in this case the bitumen needs to be "hard" when it is cooler (storage and driving in air) and "soft" when it is warmer (downdrag in soil). This corresponds to the natural behavior of bitumen. It is sometimes difficult or even impossible to find an appropriate bitumen for summertime construction in a hot climate. In very cold climates, precautions in the construction process become very important. The input from a bitumen expert is very valuable in all cases.
7. The behavior of a single pile uncoated or coated with bitumen can be handled with the PILENEG program. This program takes the soil settlement profile with depth and the soil friction profile with depth as major input. The output consists of the pile settlement profile with depth and the pile load profile with depth. Of the many options that exist, one allows the user to get a complete load settlement curve.
8. The behavior of pile groups under downdrag was addressed in a parallel study (Jeong and Briaud 1992). It was found that piles in a group carry less downdrag than single piles. This was obtained by a series of three-dimensional, nonlinear finite element simulation performed after calibrating the model against a full-scale case history. The corner piles in a pile group carry a downdrag load that can be as high as the downdrag load on a single pile; the interior piles carry as little as 15% of the downdrag load on a single pile. This led to the idea, for large pile groups, of placing a curtain of dummy piles on the outside of the pile group to take the brunt of the downdrag and to design the inner group with less concern for downdrag.

Environmental Study

1. After the bitumen-coated piles had been in the ground for at least 2 years, samples of soil and water were taken next to the piles and then far away from the piles (12 m) to provide a background reference. These samples were analyzed by two accredited laboratories in Canada and in the United States. Both soil and water samples were tested for 16 different PAHs. A total of 240 concentrations were measured for the Edmonton site and 256 for the New Orleans site.
2. At the Edmonton site, the 240 concentrations came from 12 soil samples involving 4 soil sampling holes and 2 depths, and from 4 water samples involving 3 water sampling holes and 1 depth. Of the 240 concentrations, 171 were too small to be detected by the instrument (generally <1 ppb). For the other 69 concentrations the mean concentration of the background reference samples were almost always within ± 1 standard deviation of the concentrations obtained from the samples next to the piles; therefore, on that basis, the bitumens did not create any contamination in the soil or in the water.
3. At the New Orleans site, the 270 concentrations came from 11 soil samples involving 4 soil sampling holes and 4 depths, and from 5 water samples involving 3 water sampling holes and 2 depths. For the 176 concentrations measured on soil samples, the mean concentrations of the background reference samples were almost always within ± 1 standard deviation of the concentrations obtained from the samples next to the piles; therefore, on that basis, the bitumens did not create any contamination in the soil.
4. At the New Orleans site, for the 80 concentrations measured on the water samples, the mean concentrations of the background reference samples were almost always within ± 1 standard deviation of the concentrations obtained from the samples next to the piles except for one sample: CPM 25. This water sample was taken at a depth of 7.6 m, 0.3 m away from the concrete pile coated with the U.S. Intec Blue polymer modified bitumen membrane (one of the inefficient bitumens); the concentrations for this water sample were consistently significantly higher than the background reference sample. Yet, the soil sample taken at exactly the same location showed very low concentrations consistent with the background reference sample. Also, the water sample taken 8 m away at the same depth shows very low concentrations consistent with the background reference sample. To date, this odd and surprising measurement remains unexplained. One can surmise that in drilling the hole 0.15 m from that pile, the hole was not perfectly straight, got very close to the bitumen coating at that depth, and a small piece of bitumen was collected with the water sample.
5. At both sites, the PAH concentrations were compared with the soil and the drinking water regulatory levels mandated by the U.S. Environmental Protection Agency and the Canadian Council of Ministers of the Environment. In New Orleans, all 270 concentration levels were below the regulatory levels except for 4 concentrations on the odd sample CPM 25 mentioned above. In Edmonton, of the 240 concentrations measured, 27 were higher than the regulatory levels including most of the reference samples; as pointed out before, there was no significant difference between the concentrations next to the piles and those used for background reference; therefore, this cannot be attributed to the bitumens.

6. A series of leaching tests was performed in the laboratory under controlled conditions on 3 one-dimensional columns filled with beads. Two of the columns contained 10 g of bitumen, the third column had none, to serve as a reference. Flow of distilled water took place through the columns during 90 days after which samples of water and glass beads were obtained and analyzed for PAH concentrations. A total of 144 concentrations were measured, 96 on the water samples and 48 on the glass bead samples. The concentrations of the samples spiked with bitumen were always close to the ones of the reference samples and always much lower than the soil and the drinking water regulatory levels.

CHAPTER 1

RECOGNITION OF DOWNDRAG AND JUSTIFICATION FOR BITUMEN

1.1 WHAT IS DOWNDRAG?

In the usual case of pile loading, the structural load applied to the top of the pile causes the pile to move downward with respect to the soil. The mobilized shear stresses along the pile-soil interface act upward and contribute to the bearing capacity of the pile: this is the case of positive skin friction. In this case, the load carried by the pile in friction is F_p and the load carried by the pile point is Q_p (Figure 1. 1). The total load carried at the top of the pile is $Q_t = F_p + Q_p$

If a pile is driven through a layer of soft compressible soil such as soft clay, soft silt, peat, recent fill, or collapsible soil, it is possible for the embedding soil to move downward with respect to the pile. The settlement of the soil layer may be caused by the application of a surcharge, such as a fill or an embankment, lowering of the water table, thawing of frozen soils, consolidation of a recent fill under its own weight, construction work adjacent to the site, or reconsolidation of soft soils disturbed during driving.

Down to a certain point along the pile, called the neutral point, the settlement of the soil is larger than the downward movement of the pile (Figure 1.2b). The shear stresses mobilized along the pile down to the neutral point act downward and are called negative skin friction. They act as downdrag and increase the load applied to the pile (Figure 1.1). Below this point, the downward movement of the pile is larger than the soil settlement and the mobilized shear stresses act upward on the pile (Figure 1.2a). They are referred to as positive skin friction. The downdrag force is F_n (Figure 1.1), the positive skin friction force is F_p and the load carried by the pile point is Q_p. The total load carried at the top of the pile is $Q_t = Q_p + F_p - F_n$, while the maximum load in the pile is $Q_{max} = Q_t + F_n = Q_p + F_p$.

The neutral point is defined as the point along the pile at which the relative pile-soil movement is zero, that is to say the settlement of the soil is equal to the downward movement of the pile (Figure 1.2b). In the case of a pile with its point resting on a hard, unyielding layer, the only downward movement of the pile will be due to the pile compression. The neutral point will tend to be located close to the bottom of the compressible layer, leading to a large amount of downdrag (large F_n).

If the underlying layer is as deformable as the consolidating layer, the pile point will penetrate into that layer by an amount related to the load at the pile point and to the properties of the bearing layer. The neutral point will then be located above the bottom of the compressible layer, and its position will be a function of the settlement profile for the compressible layer and of the movement profile for the pile. As a rule of thumb, the neutral point location, in this case, is at a depth approximately equal to two-thirds of the pile length.

1.2 WHY IS DOWNDRAG A PROBLEM?

The settlement of pile foundations in the case of positive friction is usually considered to be small. In the case of downdrag, the settlement is larger, often times much larger. Downdrag has been reported to have caused extreme movements, differential settlements, and extensive damage to various structures. Garlanger (1974) reported the case of downdrag on a bridge abutment where the abutment rotated, one pile was subjected to four times the design load and several piles were pulled out of the foundation. Brand and Luangdilok (1975) reported the case of a factory building with differential settlements of 100 to 300 mm because of downdrag causing structural damage to beams and panel walls. Bakholdin and Berman (1975) give two cases of failure because of downdrag. An asphalt and concrete plant building in Leningrad had settlements of 600 mm in 3 years. A lightweight storage building in Riga had to be dismantled immediately upon completion because of differential settlements of 250 mm. Fellenius (1969) and Lambe et al. (1974) report several more cases of structural damage because of downdrag, some of which involved the foundation piles being pulled from the floor of the structure. Field measurements and investigations (Bozozuk 1972, Walkinshaw 1984) have shown that the magnitude of the downdrag load may reach 2700 kN.

Downdrag had been originally recognized as a design problem for pile foundations in the case of recent fills (Johannessen and Bjerrum 1965) and with a substantial and continuing lowering of the water table in a soft layer. Mexico City is the most well-known example of this case (Girault 1969). It has been shown that the same phenomenon is of concern in bridge abutments built over soft river embankments (Bozozuk 1972, Garlanger 1974). Many of the early failures resulting from downdrag occurred because the phenomenon was not understood or identified

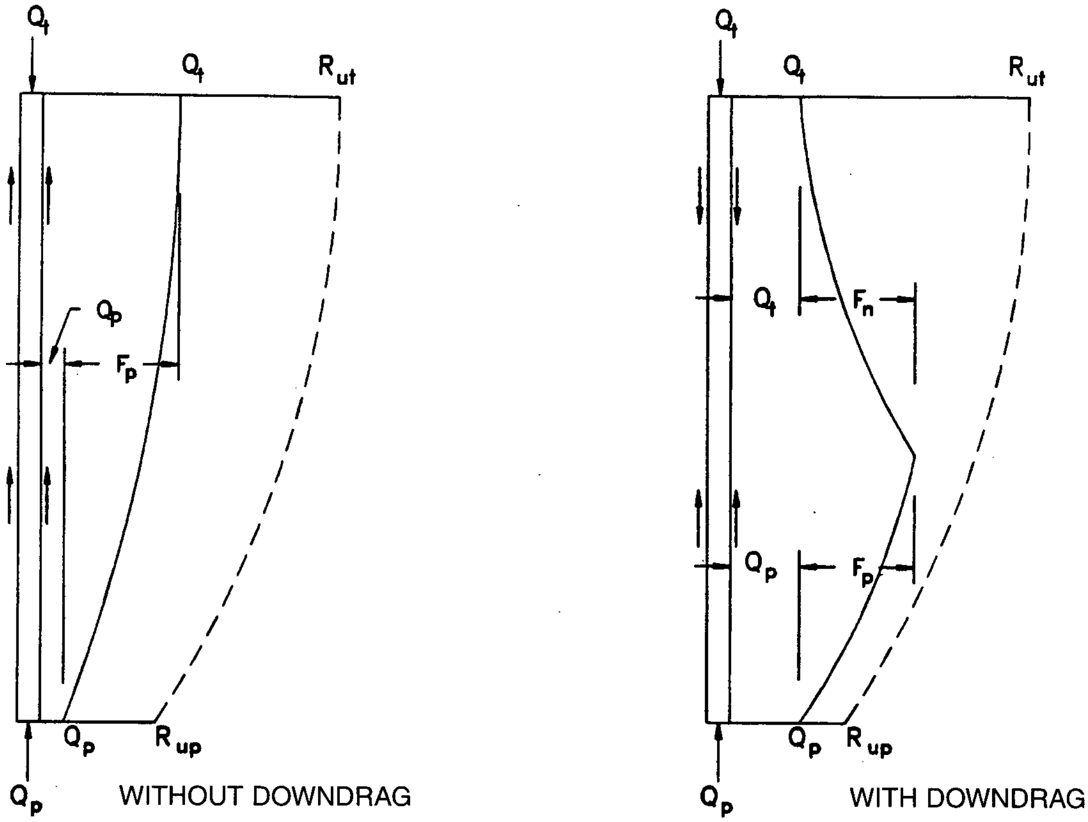


Figure 1.1. Distribution of axial forces in a pile.

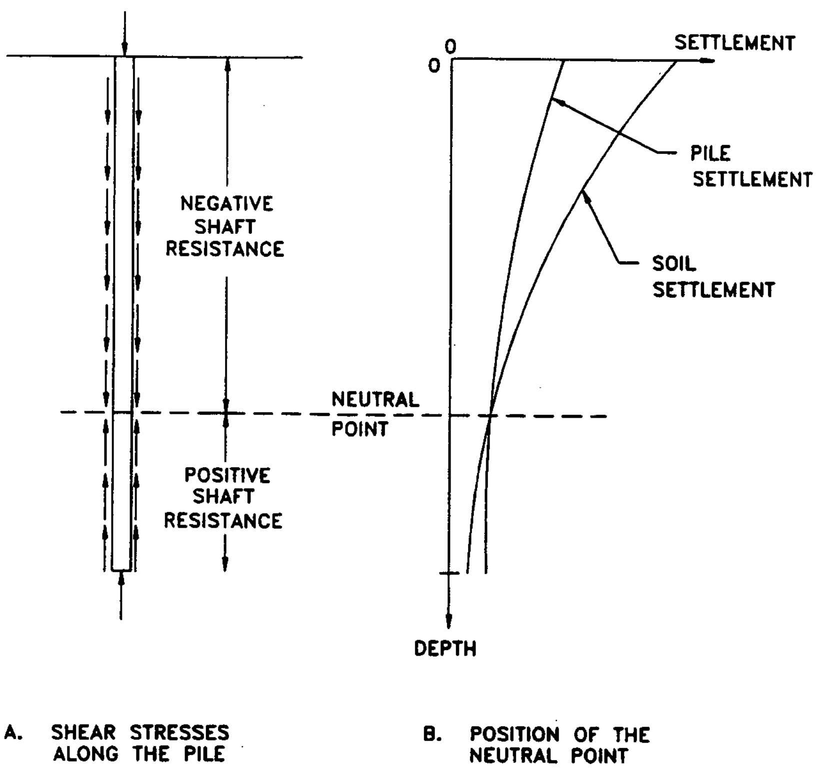


Figure 1.2. Negative skin friction on a pile.

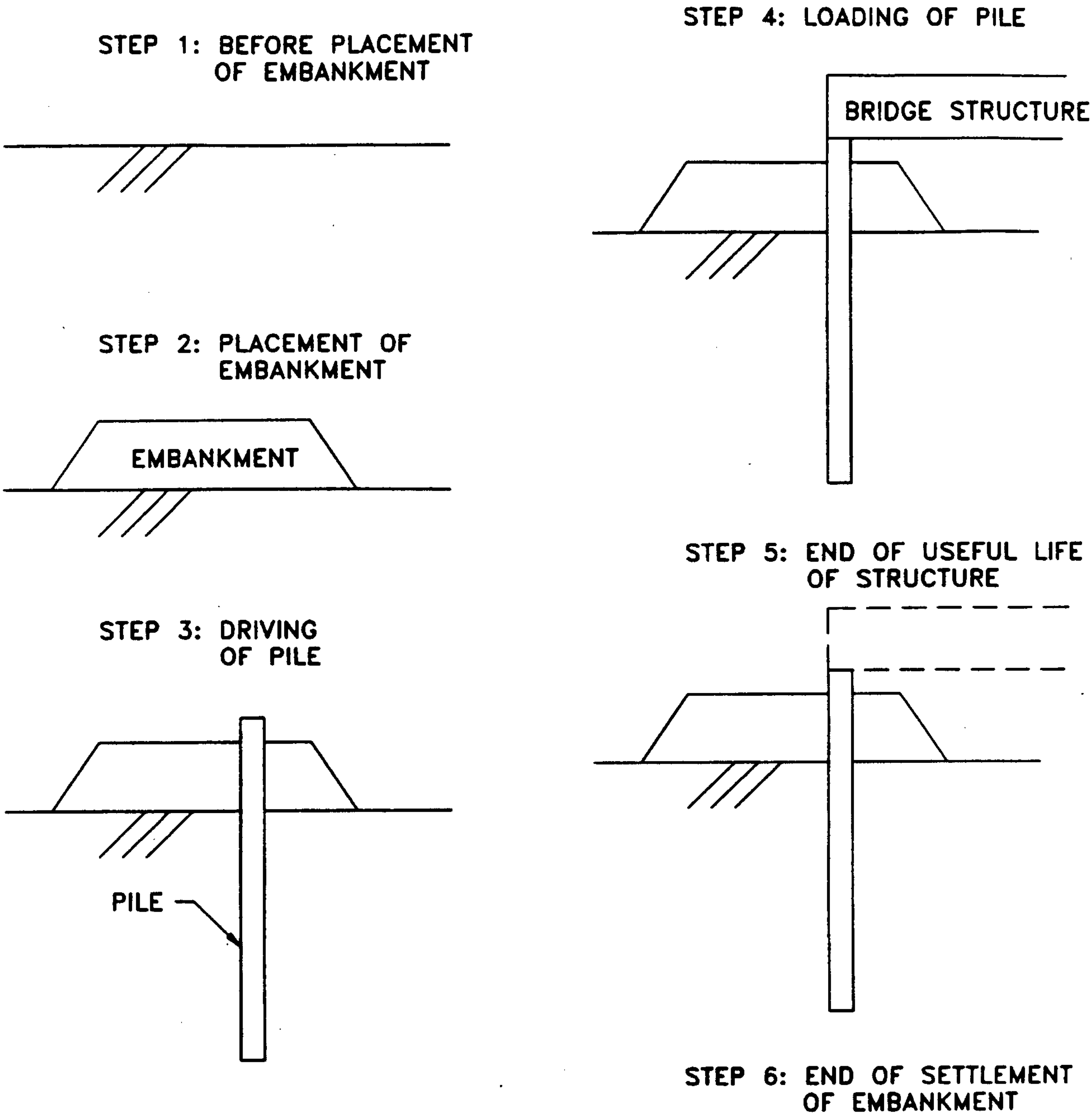


Figure 1.3. Steps in the settlement of a bridge abutment foundation.

as a problem. However, Davisson (1993) used seven case histories (Table 1.1) to show that recent failures have occurred because of downdrag even though geotechnical engineers had identified downdrag as a problem. Davisson identified the following errors made in these cases: (1) failure to anticipate the effect of future dewatering, (2) failure to anticipate the effect of adjacent ground loading, (3) improper analysis of downdrag, and (4) failure to penetrate adequately into the bearing layer.

The following is an example illustrating the settlement process for the pile foundation of a bridge abutment (Figures 1.3 and 1.4). There are 6 basic settlement steps. Step 1 corresponds to the settlement that the soil may experience before the beginning of the placement of the embankment and the driving of the pile. The elevation of the soil surface just before placing the embankment is taken as the origin and corresponds to zero settlement in this example. Step 2 is the settlement of the embankment before driving the pile. Step 3 corresponds to the settlement of the pile because of negative skin friction and settlement of the soil below the pile point from the time of driving to the time when the pile top is subjected to the structural load. Step 4 corresponds to the immediate settlement of the pile because of the loading of the pile by the bridge. Step 5 is the additional settlement of the pile because of the increase in negative skin friction with further consolidation of the soil, pile penetration in the bearing layer, and further downward movement of the soil below the pile point until the end of the useful life of the bridge is reached. Step 6 is the final settlement of the embankment.

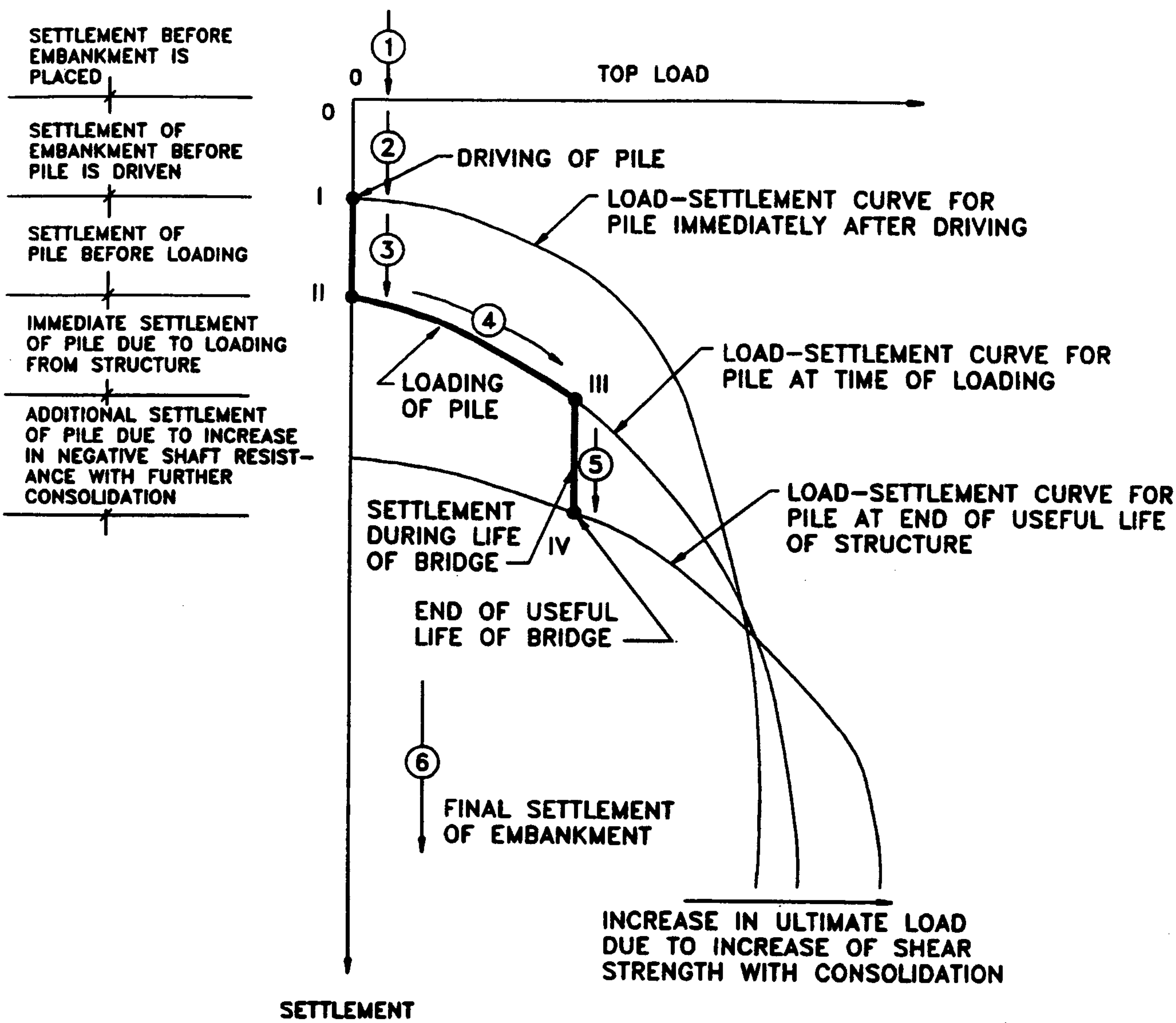


Figure 1.4. Settlement path of the pile foundation of a bridge abutment.

TABLE 1.1 Features of seven case histories of failure (from Davisson 1993)

Case History	Time Period	Type Pile	Fill Placed	Geot. Engineer
1	1950-60	Timber	Yes	No
2	1960-70	Timber	Yes	No
3	1970-80	Timber	No	Yes
4	1980-90	Pipe	Yes	Yes
5	1980-90	Pipe	Yes	Yes
6	1980-90	Timber	Yes	Yes
7	1980-90	PCPS	Yes	Yes

Notes:
- **Large soil settlements in all cases.**
- **Structures all settled excessively.**
- **Structure collapsed in Cases #1, 2.**
- **Pile failed structurally in Cases #1, 2, 6.**
- **PCPS = PreCast Prestressed Concrete.**

The settlement of the bridge abutment corresponds to steps 4 and 5. The load-settlement curve for the pile at any given time depends on the degree of consolidation of the soil at that time. Although three load-settlement curves are shown, only four points are needed to define the pile settlement completely. The settlement of the pile follows the path I, II, III, IV on Figure 1.4. The downdrag problem from the geotechnical point of view is therefore one of excessive settlement of the pile foundation. As a result, settlement calculations for the piles will be extremely important in the case of downdrag and will often control the amount of structural load that can be placed at the pile top. In addition, the proper safety factor must be applied to prevent the possible failure of the pile under the maximum load in the pile (Figure 1.1).

1.3 WHEN TO DESIGN FOR DOWNDRAG?

Because of the variety of conditions under which downdrag can occur, there is not a single factor governing the consideration of downdrag in design. There are, however, some indicators of when downdrag could be a potential problem (Table 1.2). The presence and magnitude of downdrag forces should be checked whenever the total settlement of the ground surface is larger than 100 mm, the settlement of the ground surface after the piles are driven is larger than 10 mm, the height of the embankment to be placed on the ground surface exceeds 2 m, the thickness of the soft compressible layer is larger than 10 m, the water table is drawn down by more than 4 m, or the piles are longer than 25 m. **WARNING: Downdrag can occur even if the above conditions are not present.**

As in the case of no downdrag, the pile foundation must be designed so that the settlement of the top of the pile after the dead load of the structure is placed will be less than can be tolerated by the structure, the stresses in the pile will be lower than the allowable stress for the pile material, and the load placed at the pile top will lead to an acceptable factor of safety against plunging of the pile into the soil (Table 1.3).

1.4 WHEN TO USE BITUMEN AND HOW MUCH DOES IT COST?

If it is concluded that downdrag is a problem, and that it is desirable to reduce downdrag, this reduction can be achieved by

1. Preloading the soil to accelerate the settlement prior to driving the piles, thereby reducing the settlement which will take place after the piles are driven,
2. Using the grouped-pile method, which takes advantage of the fact that the downdrag force on *n* closely spaced piles is much less than *n* times the downdrag on an isolated pile (Endo et al. 1969, Inoue et al. 1975, Okabe 1977, Jeong and Briaud 1992),
3. Using electro-osmosis to increase the water content around the cathode pile, thereby reducing the pile-soil friction (Asakawa 1959, Bjerrum et al. 1969),
4. Using the double-tube pile method whereby the outer pile carries the downdrag load and the inner pile carries the structural load (Okabe 1977),
5. Using tapered piles so that the soil settlement tends to decrease the downdrag (Sawaguchi 1982),
6. Driving piles with an oversize shoe while filling the created annulus with bentonite slurry,
7. Predrilling a hole before lowering the pile in the open hole and filling the created annulus with bentonite slurry,
8. Coating the piles with a friction reducer such as bitumen (Baligh et al. 1978, Bjerrum et al. 1969, Walker et al. 1973, Claessen and Horvat 1974, Saito et al. 1975, Tsutsumi and Nei 1975, Machan and Squier 1983, Nippon Kokan 1977), or
9. Increasing the capacity of the piles by increasing the diameter, length, or number of piles, thus reducing the impact of downdrag on each pile.

Table 1.4 presents an evaluation of the cost and effectiveness of each method. The choice of method is dictated by the site conditions and economic considerations. Several design alternatives should be investigated using uncoated piles, bitumen-coated piles, and any other friction reduction methods which are deemed effective and feasible for

TABLE 1.2 Clues to know when to consider downdrag in design

1.	The total settlement of the ground surface is larger than 100 mm
2.	The settlement of the ground surface after the piles are driven is larger than 10 mm
3.	The height of the embankment to be placed on the ground surface exceeds 2 m
4.	The thickness of the soft compressible layer is larger than 10 m
5.	The water table is drawn down by more than 4 m
6.	The piles are longer than 25 m

WARNING: Downdrag can occur even if the above conditions are not met.

TABLE 1.3 Design criteria for pile foundations

1.	The settlement of the top of the pile after the dead load of the structure is placed will be larger than can be tolerated by the structure
2.	The stresses in the pile will exceed the allowable stress for the pile material
3.	The load placed at the pile top does not lead to an acceptable factor of safety against plunging of the pile into the soil

TABLE 1.4 Evaluation of downdrag reduction alternatives

Method	Cost	Effectiveness in Reducing Downdrag
1. Preloading	medium (time consuming)	medium-high
2. Pile group	medium-high	medium
3. Electro-osmosis	high	medium
4. Double tube	high	high
5. Tapered piles	low	very low
6. Oversized shoe & slurry	low	low
7. Predrilling & slurry	medium	low
8. Bitumen coating	low	high
9. Increase capacity of piles	medium-high	medium-high

WARNING: These ratings are given as guidelines. Individual cases may differ.

the case in question. The costs, the benefits, and the reliability of these alternatives are compared and a decision is then made.

The application of a bitumen coating increases the cost per pile by 15% to 50% over the cost of an uncoated pile. Machan and Squier (1983) reported a 15% increase in cost for bitumen-coated steel pipe piles. Claessen and Horvat (1974) reported a 10% to 20% increase in cost for bitumen-coated precast concrete piles. The field studies conducted in this project on test piles resulted in a 46% increase in cost as a result of the bitumen coating. This large percent increase compared to the reported cases may be due to the fact that a small number of piles were coated for this project and that several different bitumens and coating methods were used. On projects with larger numbers of piles and contractors familiar with the bitumen-coating process, the increase in cost may be expected to be below 20%. Because of this increase in cost, the bitumen coating must allow the load per pile to be at least 15% to 50% more than the load on an uncoated pile in order for bitumen coating to be an economical alternative. Case histories have shown that bitumen coatings reduce the downdrag force as much as 98% (Table 1.5) and can therefore represent a very economical alternative. The bitumen coating also provides protection against acids from the soil, preventing pile corrosion.

1.5 EXAMPLE OF THE EFFECT OF DOWNDRAG AND OF BITUMEN COATING

The following example shows the effect of downdrag and of bitumen coating on the load-settlement curve of a pile. The calculations were made using the data shown in Table 1.6 and Figures 1.5 and 1.6 and the PILENEG microcomputer program (described in Section 2.3). The soil capacities are assumed to be from a quality static prediction method (such as those described in Section 2.1).

Four load-settlement curves for the pile are shown in Figure 1.7 with various assumptions as to the pile being coated or uncoated and whether or not downdrag is present. Each curve is obtained using vertical equilibrium of the pile under the structural and soil loads and using compatibility of pile and soil movement at the neutral point. The details of the procedure are given in Section 2.1. Curve 1 is the predicted load-settlement curve for an uncoated pile if there were no downdrag. Curve 2 is a predicted load-settlement envelope for an uncoated pile with downdrag present; this is not a prediction of load test results. It is assumed that for each point on this curve the top load is applied and then downdrag develops, resulting in the predicted settlement. Note that the ultimate capacity of the pile is unchanged, but the settlement required to achieve that capacity has increased significantly.

Curve 3 is a predicted load-settlement envelope for a coated pile with downdrag present. The bitumen coating depth is assumed to extend to the neutral point. Because the neutral point changes location depending on the applied load, this curve is an envelope of load-settlement coordinates with the bitumen coating length changing for each value of top load. For zero load, the neutral point is at its lowest position and at ultimate load the neutral point is at the ground surface.

Curve 4 is a predicted load-settlement envelope for a coated pile with downdrag. The coating length is held constant (27.5 m) and is equal to the length of coating which

TABLE 1.5 Case histories on bitumen-coated piles

- Bjerrum, Johannessen, Eide (1969)	Downdrag reduced by 95%
- Hutchinson, Jensen (1968)	Friction reduced by 30 to 80%
- Brons et al. (1969); Van Weele (1968)	Downdrag reduced by 90%
- Claessen, Horvat (1974); Claessen, Gelok (1971)	Downdrag reduced by 90%
- Walker, Darval, Le (1973)	Downdrag reduced by 98%
- Bozozuk, Keenan, Pheeney (1979)	Bitumen not very useful
- Clemente (1984)	Downdrag reduced by 90%
- Machan, Squier (1983)	Downdrag reduced by 85% (estimated)
- Board (1975)	Not measured

TABLE 1.6 Data for downdrag example

1. Pile Data	
420 mm Square Precast Concrete Pile	
Cross-sectional area	0.176 m^2
Pile perimeter	1.68 m
Pile elastic modulus, E_c	24 E6 kN/m^2
Embedded pile length	42 m
2. Soil Data	
Friction profile	see Fig. 1.5
Soil settlement profile	see Fig. 1.6
Soil properties in bearing layer (below pile point):	
Elastic modulus	31,100 kN/m^2
Poisson's ratio	0.3
Ultimate bearing pressure	7,100 kN/m^2
3. Bitumen Data	
Shear strength during downdrag	3 kN/m^2

resulted in a 25-mm settlement on Curve 3. Note that the ultimate load for this pile is reduced significantly because of the bitumen coating.

Three safety criteria were considered for the pile and the resulting allowable loads from each criterion for each curve on Figure 1.7 are shown in Table 1.7.

First, the pile must be safe against structural failure. The maximum force allowed in the pile for this case is 3050 kN. This force is independent of whether or not the pile is coated or if downdrag is present; however, the location of the maximum force in the pile depends on coating and downdrag. With no downdrag present (Curve 1) the maximum force is located at the pile top and is equal to the applied top load. With downdrag present the maximum force is located at the neutral point and is equal to the applied top load plus the negative skin friction. This results in an allowable top load of 2,975 kN for Curve 2, the allowable top load being reduced by the downdrag. For Curve 4, the pile capacity is reduced because of the bitumen coating so that, if the structural failure load could be reached, there would be no downdrag occurring at that load and the allowable top load is 3,050 kN.

Second, the pile must be safe against soil failure. This leads to an allowable applied top load of 1,005 kN for Curve 1 with no downdrag present. For Curve 2, if the allowable load is checked at the top of the pile, a load of 1,005 kN would be allowed. However, checking at the neutral point shows that even with no applied top load the pile is not safe against soil failure because of the magnitude of the downdrag. For Curve 4, checking at the pile top leads to an allowable top load of 770 kN, while checking at the neutral point leads to an allowable top load of 940 kN.

Third, the load for an allowable settlement of 25 mm is obtained. The allowable applied top load for this criterion is 1,760 kN for Curve 1,451 kN for Curve 2, and 890 kN for Curve 4.

An examination of the allowable loads in Table 1.7 indicates some important aspects of the downdrag problem and of bitumen coating.

1. For an uncoated pile the ultimate plunging load is the same whether there is downdrag or not (compare Curves 1 and 2 on Figure 1.7).
2. The possibility of structural failure must always be checked both at the pile top and at the neutral point (see Figure 1.1, Curve 2 on Figure 1.7 and Table 1.7).
3. The allowable top load in all cases of this example is controlled by the soil failure criterion, either at the top or at the neutral point. This is not true in all cases and all three safety criteria must be checked. For very hard bearing layers, the structural failure criterion may control, whereas for friction piles the settlement criterion may control.

TABLE 1.7 Comparison of allowable applied top loads

Curve	Safety Criterion	Safe Against Concrete Crushing (kN)	Safe Against Soil Failure: At Top (kN)	Safe Against Soil Failure: At N.P. (kN)	Safe Against Excessive Settlement (kN)
		Allowable Applied Top Load (kN)			
1	Uncoated No Downdrag	3050	1005	—	1760
2	Uncoated Downdrag	2975	1005	0	451
4	Coated Downdrag	3050	770	940	890

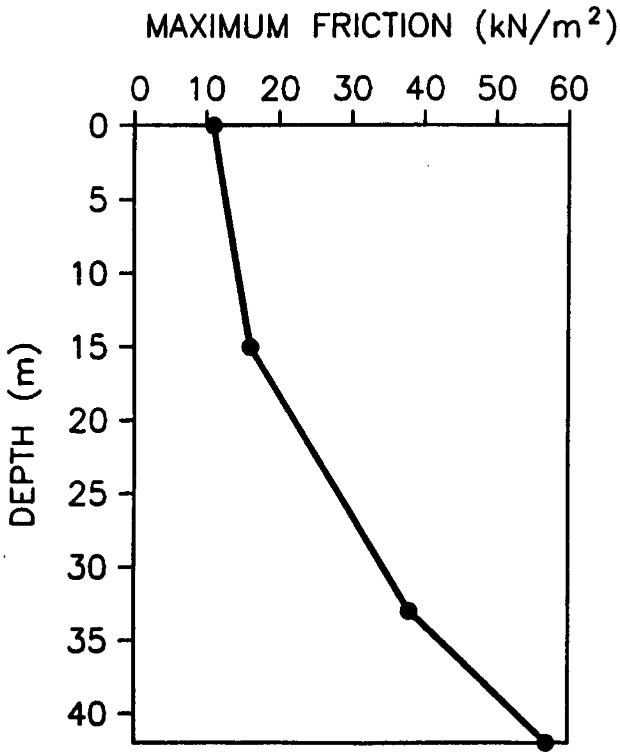


Figure 1.5. Friction profile for example problem.

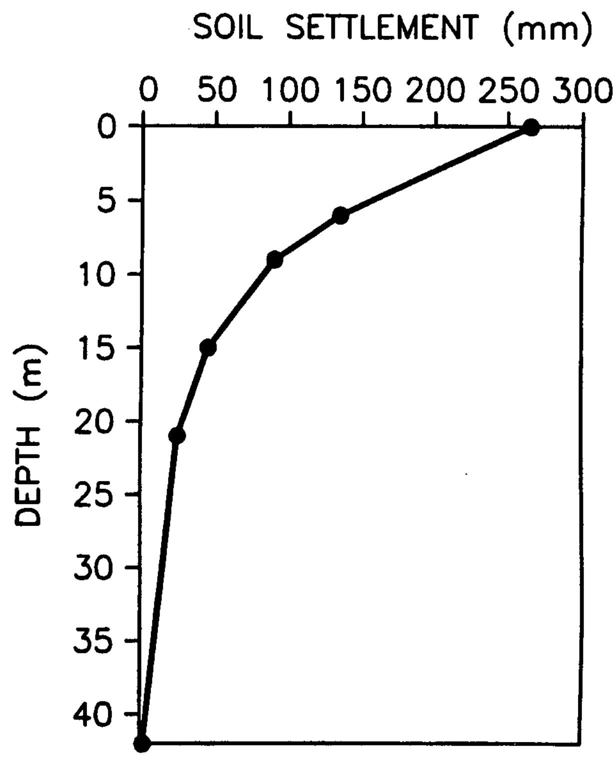


Figure 1.6. Soil settlement profile for example problem.

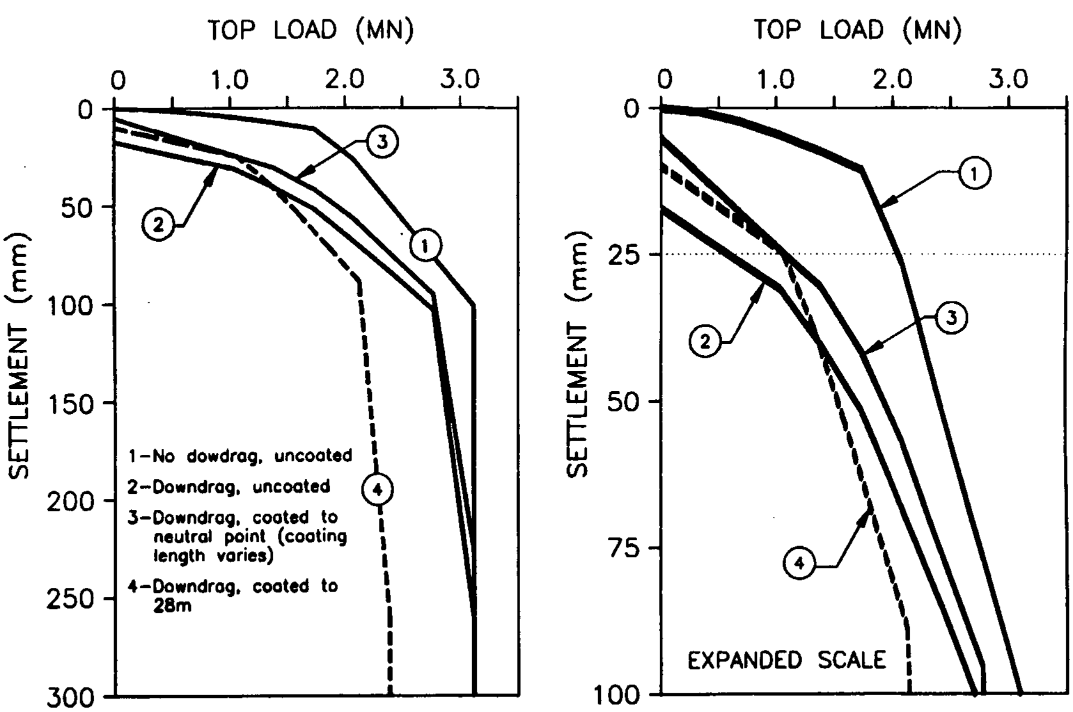


Figure 1.7. Load-settlement curves for a pile with and without downdrag.

CHAPTER 2

PREDICTING THE BEHAVIOR OF PILES SUBJECTED TO DOWNDRAG

2.1 UNCOATED SINGLE PILES

The design of piles subjected to downdrag requires the knowledge of several items. First, the settlement of the pile due to the downdrag plus the structural load must be known. Second, the maximum load in the pile due to the downdrag plus the structural load is required. Lastly, the ultimate capacity of the pile is needed. Several methods are available for analyzing downdrag problems, which have been reviewed by Davisson (1993), Lambe and Baligh (1978), Cambarieu (1974), Fellenius (1969) and Sultan (1969). The method of analysis recommended in this manual is based on the static equilibrium of the pile and on the compatibility of the relative pile-soil movement. It is described in the following sections.

This analysis is limited to vertical piles. Inclined piles are not recommended in the case of downdrag because of the severe bending problem that would be created in the piles.

2.1.1 Static Equilibrium of the Pile

For the static equilibrium of the pile only the vertical forces are considered; horizontal forces and buckling behavior are not taken into account. The forces acting downward on the pile are the structural load, Q_t, applied to the top of the pile, and the negative skin friction, F_n, mobilized from the top of the embedding layer down to the neutral point. The resisting forces are the positive skin friction mobilized below the neutral point down to the pile point, F_p, and the point resistance, Q_p (Figure 2. 1). The resulting equilibrium equation is

$$Q_t + F_n = F_p + Q_p \tag{2.1}$$

Very little relative pile-soil movement is necessary to mobilize the full negative skin friction (Bjerrum, Johannessen and Eide 1969, Broms 1969, Bakholdin and Berman 1975). It has therefore been assumed that the max-

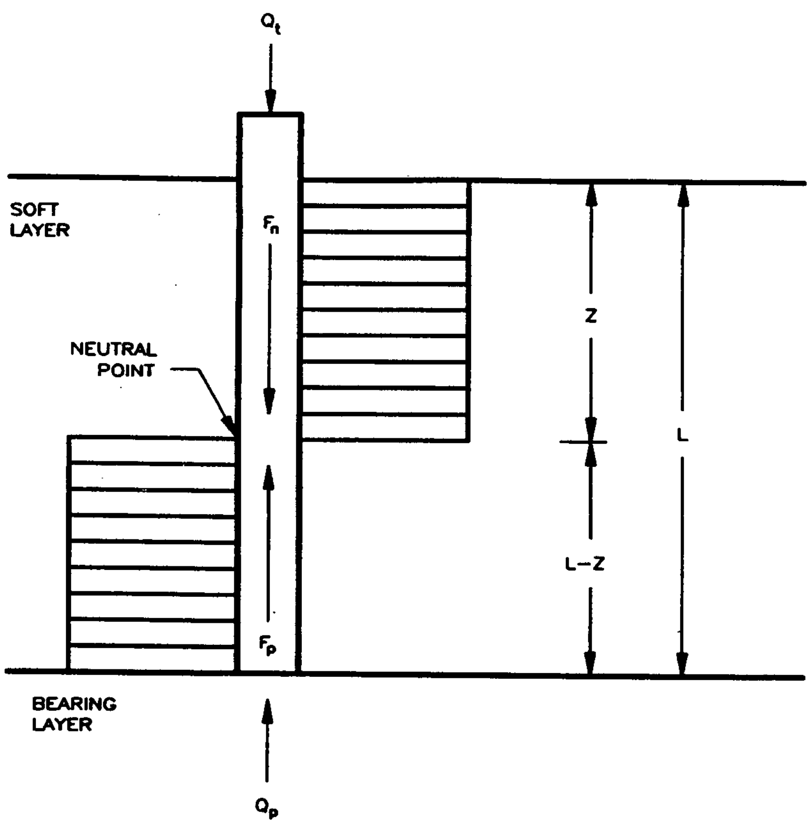


Figure 2.1. Static equilibrium of the pile.

imum shaft resistance is fully mobilized as negative skin friction along the pile above the neutral point and as positive skin friction below that point. This assumption may not be true close to the neutral point where the relative pile-soil movement is small, and the analysis will thus give an upper limit of the negative skin friction; however, the error is small.

An elasto-plastic model has been used for the point resistance. Randolph and Wroth (1978) have shown that "the base of the pile acts like a rigid punch on the surface of a half space—not like a buried plate." The elastic movement of the pile point can thus be calculated using

$$\omega_{punch} = \frac{\pi}{4}(1-\nu^2)\frac{Q_p D}{A\,E_s} \tag{2.2}$$

where ω_{punch} is the pile point movement in the soil bearing layer, A is the area of the pile point, D is the diameter of the pile point, E_s is the Young's modulus of the bearing soil layer, and ν its Poisson's ratio.

The point resistance of the pile has a limiting value, Q_{max}. Once this value is reached, Eq. 2.2 no longer holds and the pile point moves without an increase of the point resistance (Figure 2.2).

2.1.2 Relative Pile-Soil Movement Compatibility

In order to solve Eq. 2.1 it is necessary to know the depth of the neutral point, Z (see Figure 2.1). The neutral point is the point along the pile where the pile movement, ω_p, is equal to the settlement of the surrounding soil, ω_s.

$$\omega_p @ z = Z = \omega_s @ z = Z \tag{2.3}$$

The location of the neutral point can be found by comparing the soil settlement profile with the pile settlement profile. The soil settlement profile can be calculated by appropriate methods such as consolidation theory (Terzaghi and Peck 1967, Lambe and Whitman 1969). The soil settlement profile is assumed to be given. The settlement profile for the pile must be calculated and depends on the location of the neutral point. In order to solve this problem, an envelope of points is developed which gives the settlement of the point along the pile located at a depth z if the neutral point is also located at the same depth z. The location of the neutral point may then be found by comparing the soil settlement profile with this pile movement envelope (Figure 2.3). This process is further explained in the following section.

The pile movement envelope can be determined using Eq. 2.1 and assuming a purely elastic behavior of the pile and of the soil under the pile point (elastic compression of the pile and elastic punch in the bearing soil layer neglecting the maximum bearing value). Then the movement of a point along the pile, assuming that this point is the neutral point, is given by

$$\omega_p @ z = Z = \omega_s @ z = L + \omega_{punch} + \omega_{elastic} \tag{2.4}$$

where ω_s @ $z = L$, is obtained by reading the soil settlement profile at $z = L$; ω_{punch} is obtained by using Eq. 2.2 after determining Q_p; Q_p is calculated from Eq. 2.1 where Q_t is part of the input to the problem, F_n is the negative friction force calculated using the ultimate friction at the

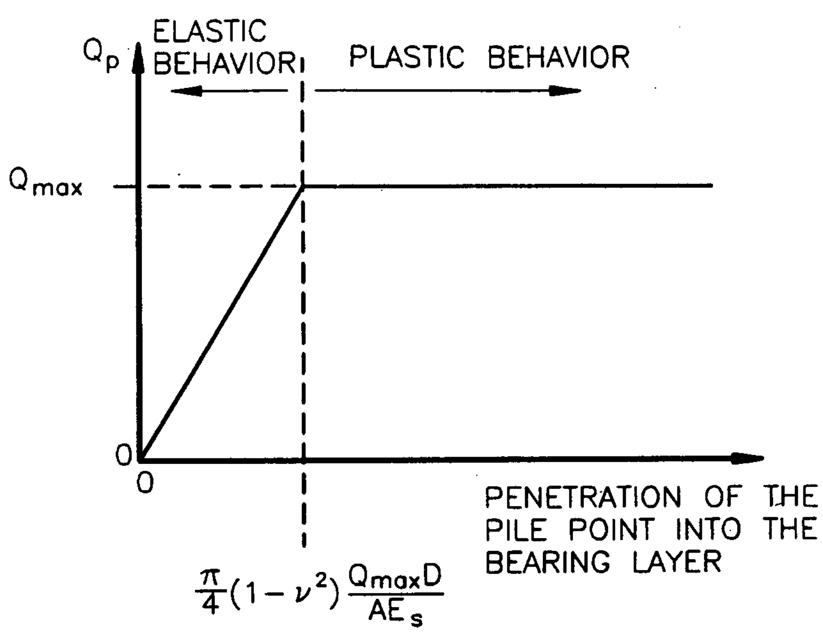


Figure 2.2. Pile point behavior.

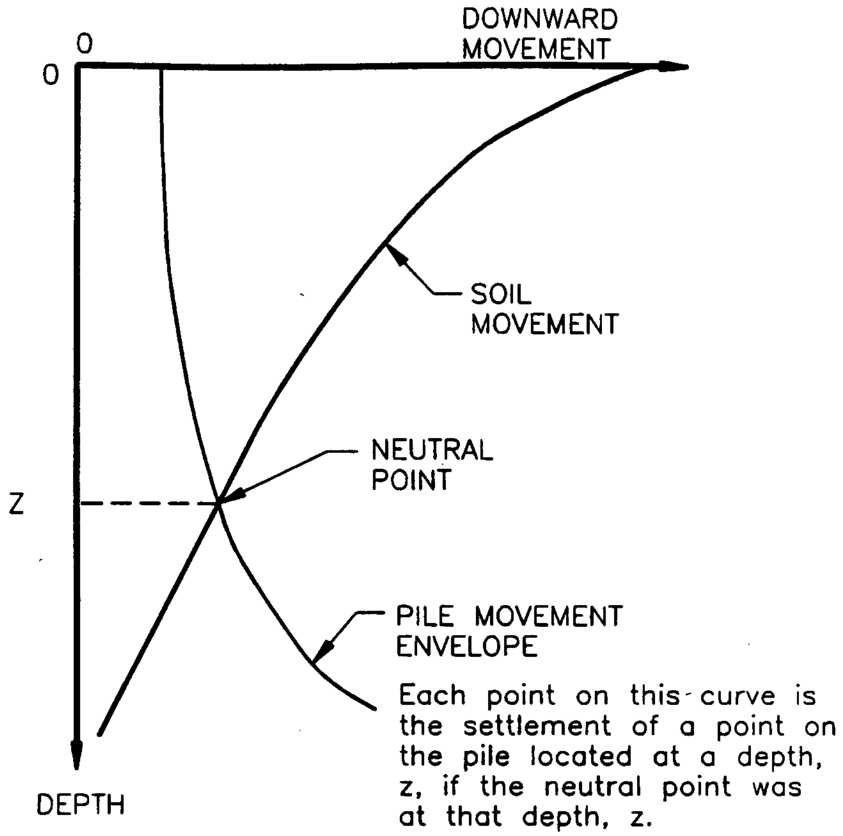


Figure 2.3. Determination of the neutral point.

soil-pile interface from the ground surface down to the assumed neutral point location at $z = Z$; F_p is the positive friction force calculated using the ultimate friction at the soil-pile interface from the assumed neutral point at $z = Z$ down to the pile point; $\omega_{elastic}$ is the elastic compression of the pile between the assumed neutral point at $z = Z$ and the pile point and is given by

$$\omega_{elastic} = \left(Q_p + \frac{1}{2} F_p\right) \frac{L - Z}{AE} \tag{2.5}$$

The position of the neutral point is given by the intersection of the pile movement envelope and the soil-settlement curve (Figure 2.3). However, the pile movement envelope was developed assuming purely elastic behavior of the pile point. This means that some of the points on this envelope may have required a point resistance larger than Q_{max}, to achieve static equilibrium. Therefore, a check must be made to determine if this is the case, and to adjust the neutral point if necessary. This check and adjustment are made based on the following reasoning.

Each point on the pile movement envelope is the movement of the pile at that depth assuming the neutral point is also at that same depth. Each one of those points has a corresponding pile point movement. If this point movement is larger than the movement necessary to mobilize Q_{max} (Figure 2.2), the pile point begins to exhibit plastic behavior. The depth for which the pile point movement equals the movement necessary to mobilize Q_{max}, is the maximum depth to the neutral point, Z_{max}, (Figure 2.4). If the soil-settlement curve and the pile movement envelope intersect below this point it means that a point resistance larger than Q_{max} is necessary for static equilibrium. Since this is not possible, the pile starts to move downward until enough positive skin friction is mobilized to achieve static equilibrium. This corresponds to a horizontal translation of the pile movement envelope until the two curves intersect at Z_{max} (Figure 2.4). If the soil-settlement curve and the pile movement envelope intersect above Z_{max}, this intersection point is the neutral point, and the pile point is still in the elastic range of behavior.

It is assumed that the distribution of the axial force along the pile, negative and positive skin friction, is not affected by the plastic behavior of the pile point. The plastic movement of the pile point translates into a corresponding additional pile settlement. The pile fails when the maximum point resistance, the maximum positive skin friction, and no negative skin friction are mobilized.

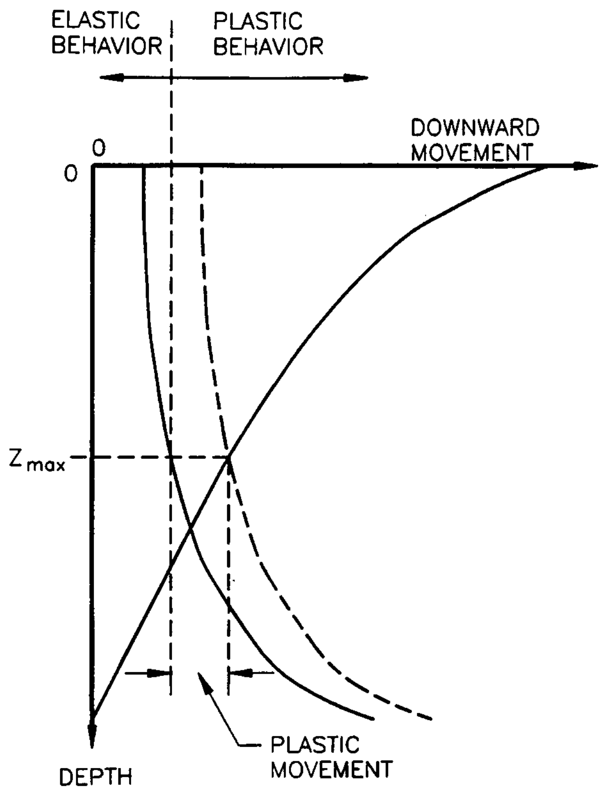


Figure 2.4. Determination of the plastic movement of the pile point.

2.1.3 Determination of Maximum Friction

In order to carry out the calculations detailed in the previous sections, it is necessary to know the maximum friction between the pile and the soil. Since very little movement is necessary to mobilize the full friction and in most downdrag cases the settlement begins rapidly after installation of the piles, the undrained shear strength in clays should be used to design the foundation for the short term. However, the settlement generally continues for long periods of time allowing the soil surrounding the pile to consolidate, so that the drained shear strength parameters should then be used to design the foundation for the long term. Therefore, the pile must be designed for both short-term and long-term cases, with the worst case governing the design. Note, however, that the rates of loading in downdrag problems are such that the maximum friction in sand should be calculated using drained parameters only. Some methods are recommended in the following paragraphs; however, the engineer could use any appropriate method (paying attention to short- and long-term cases), especially if local experience is available.

Short-term analysis in clay. For short-term analysis in clays the maximum friction, f_{max}, can be obtained from the undrained shear strength, s_u, as shown in Figure 2.5 for driven and bored piles. Figure 2.5 shows average values of f_{max} as well as upper and lower ranges. Note that in the case of positive friction, using low values of f_{max} is conservative while in the case of negative friction, using high values of f_{max} is conservative. The undrained friction could also be obtained from cone penetrometer data (Briaud and Miran 1992) or pressuremeter data (Briaud 1992).

Long-term analysis in clay. For long-term analysis in clays, an effective stress method is used. Since the in situ horizontal effective stress is difficult to determine, this method assumes that the maximum friction is a function of the vertical effective stress, σ'_{ov}, at the depth considered.

$$f_{max} = \beta \cdot \sigma'_{ov} \tag{2.6}$$

where β is the ratio of the maximum friction over the vertical effective stress and has been determined from a limited number of observations of downdrag on full-scale piles (Baligh and Vivatrat 1976, Lambe et al. 1974, Johannessen and Bjerrum 1965, Bjerrum et al. 1969). Recommended values of β are shown in Figure 2.6. It can be seen that in clay the value of β is independent of depth.

Short-term and long-term analysis in sand. For short-term and long-term analysis in sands, the maximum friction can also be obtained from an effective stress method (Eq. 2.6). Using load test data for piles entirely in sand, Hossain and Briaud (1992) showed that the average maximum friction along the entire length of the pile correlated better with relative embedment (embedded pile length divided by pile diameter) than with embedded pile length alone. Their findings have been adapted in Figure 2.7 to obtain the value of β at any depth, Z as a function of the relative depth (depth, Z, divided by pile radius, R) for varying friction angle, ϕ.

The maximum friction could also be obtained from standard penetration test data by (Ng et al. 1988)

$$f_{max} = 5\,N^{0.7}\,(\text{kPa}) \tag{2.7}$$

where N is the uncorrected standard penetration test blowcount (blows per 300 mm). Cone penetrometer data (Briaud and Miran 1992) or pressuremeter data (Briaud 1992) may also be used to determine the maximum friction.

2.1.4 Determination of Maximum Point Resistance

The maximum point resistance must also be determined in order to analyze the pile for downdrag. The maximum point resistance must be calculated for short-term and long-term cases with the worst case governing the design. Some methods are recommended in the following paragraphs; however, the engineer could use any appropriate method

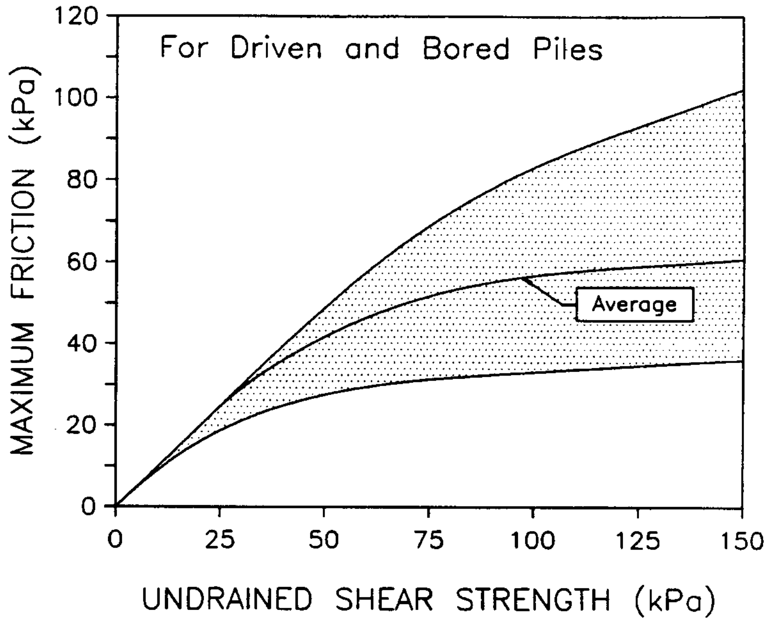
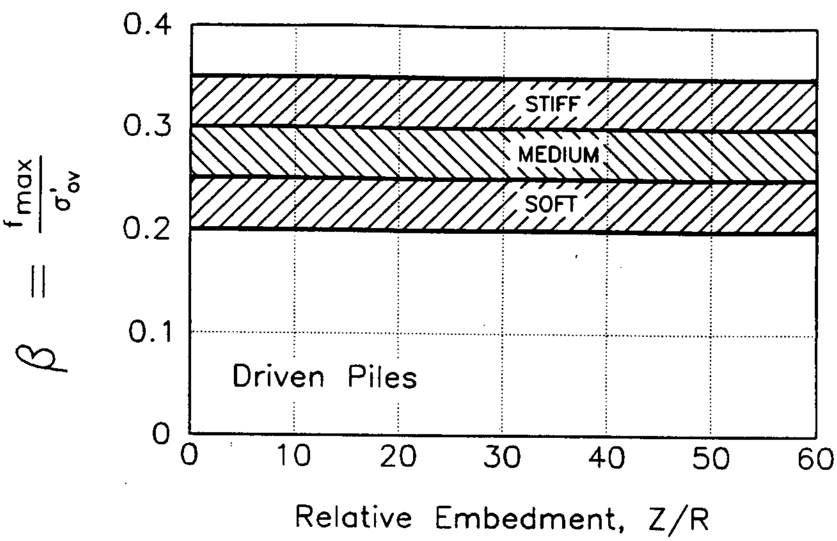


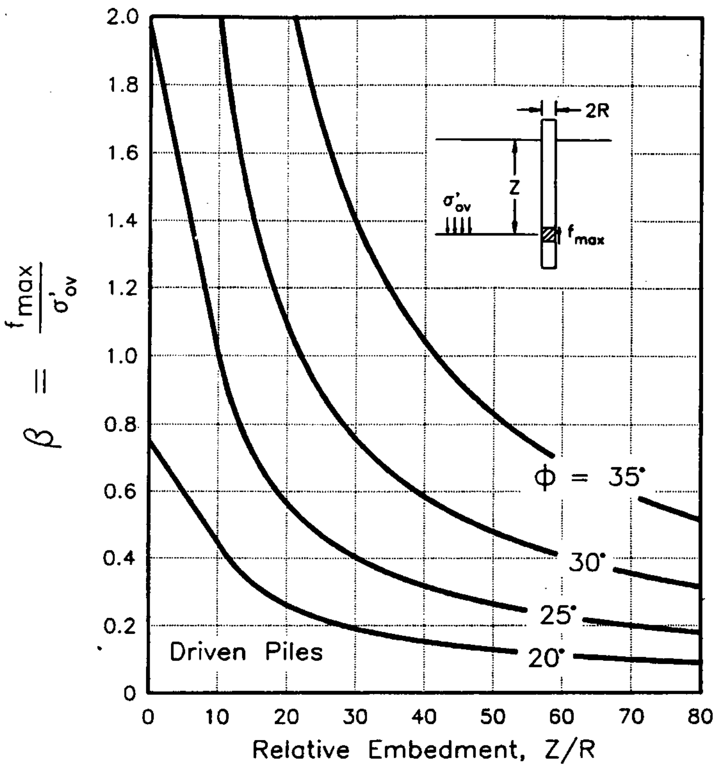
Figure 2.5. Maximum friction: short-term (undrained) friction analysis in clay.



For Bored Piles Use:

f_{max} (Bored) $= 0.75\ f_{max}$ (Driven)

Figure 2.6. Maximum friction: long-term (drained) friction analysis in clay and silt.



For Bored Piles Use:

f_{max} (Bored) = 0.75 f_{max} (Driven)

Figure 2.7. Maximum friction (drained) in sand.

(paying attention to short- and long-term cases), especially if local experience is available.

Short-term analysis in clay. For short-term analysis in clays, the maximum point resistance, Q_{max}, may be taken as

$$Q_{max} = 9s_u \tag{2.8}$$

where s_u is the undrained shear strength near the pile point.

The maximum point resistance may also be obtained from pressuremeter data (Briaud 1992) by

$$Q_{max} = kp_L \tag{2.9}$$

where k is the pressuremeter bearing capacity factor obtained from Table 2.1, and p_L is the net equivalent pressuremeter limit pressure near the pile point.

Cone penetrometer data may also be used to calculate the maximum point resistance (Briaud and Miran 1992) by

$$Q_{max} = K_c q_c \tag{2.10}$$

where K_c is the cone bearing capacity factor obtained from Table 2.2, and q_c is the average cone tip resistance below the pile point.

Long-term analysis in clay. There are very little data available for the long-term maximum point resistance in clay. It can be assumed that the general bearing capacity equation is applicable:

$$Q_{max} = 1.2cN_c + \gamma DN_q + 0.5\gamma BN_\gamma \le 2 \times \text{Short-term capacity} \tag{2.11}$$

where c is the drained cohesion, N_c, N_q, and N_γ are bearing capacity factors from Figure 2.8, γ is the effective soil unit weight, D is the depth of embedment of the pile point, and B is the pile diameter. As shown in Eq. 2.11, the long-term point resistance in clay should be limited to twice the short-term point resistance.

Short-term and long-term analysis in sand. The maximum point resistance in sand may be calculated by

$$Q_{max} = 1{,}000\sqrt{N} \text{ (in kPa)} \tag{2.12}$$

TABLE 2.1 Pressuremeter bearing capacity factor, *k* (after LCPC-SETRA 1985)

Soil	Piles with no soil displacement	Piles with full soil displacement
Clay-silt	1.2	1.8
Sand-gravel	1.1	3.2 to 4.2 [a]
a Use 3.2 for dense sand or gravel (p_L > 3MPa) and 4.2 for loose sand or gravel (p_L < 1MPa). Interpolate in between.		

TABLE 2.2 Cone penetrometer bearing capacity factor, K_c (after Bustamante and Gianeselli 1983)

Soil	Piles with no soil displacement	Piles with full soil displacement
Clay-silt	0.375	0.6
Sand-gravel	0.15	0.375

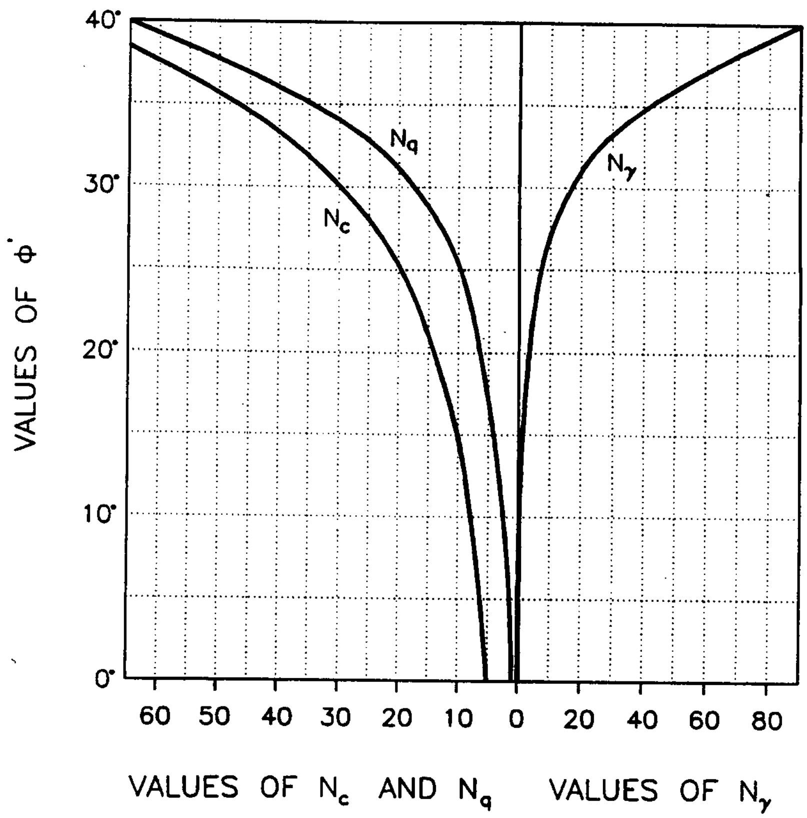


Figure 2.8. Bearing capacity factors for long-term analysis in clay. (after Terzaghi and Peck 1967)

where N is the uncorrected standard penetration test blowcount (blows per 300 mm). The maximum point resistance in sand may also be calculated from pressuremeter test data using Eq. 2.9 and Table 2.1, and from cone penetrometer test data using Eq. 2.1 and Table 2.2.

2.1.5 Recommendation for Displacement to Reach Maximum Point Resistance

As stated in Section 2.1.1, an elasto-plastic model has been assumed for the soil below the pile point. By substituting the maximum unit point resistance, q_{max}, into Eq. 2.2, the displacement required to mobilize this maximum resistance, ω_{max}, can be calculated

$$\omega_{punch} = \frac{\pi}{4}(1 - \nu^2)\frac{q_{max} D}{E_s} \qquad (2.13)$$

The only unknown in this equation is the elastic modulus, E_s, of the soil below the pile point. The initial pressuremeter modulus, E_0, works well, as checked on spread footing tests (Briaud 1992). Where no pressuremeter data are available, the following correlations to standard penetration test blowcount, N, and undrained shear strength, s_u, can be used with caution:

$$E_s \text{ (in kPa)} = 800\,N$$

or $\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad$ (2.14)

$$E_s = 100\,s_u$$

Note that there is considerable scatter in these correlations, and they should therefore be used only for preliminary rough estimates.

2.1.6 Settlement Profile for the Soil

In order to predict the behavior of a pile subjected to downdrag, the behavior of the soil must be known. In particular, the final soil-settlement profile is needed. This profile may be calculated as shown in Figure 2.9 for the common case of downdrag due to embankment loading or lowering of the water table. The consolidating layer must first be broken into a number of smaller layers of thickness H_i. The settlement at the top of each layer, ω_i, may then be calculated as

$$\omega_i = \sum_{1}^{i} (\varepsilon_{vi} - \varepsilon_{vi0})\,H_i \qquad (2.15)$$

where $(\varepsilon_{vi} - \varepsilon_{vi0})$ is the increase in strain in layer i caused by the increase in stress in that layer and can be found from the results of a consolidation test on a sample from the middle of that layer (Figure 2.9). The increase in stress in the middle of layer i, $\Delta\sigma_{vi}$, can be calculated by various methods, such as the chart in Figure 2.10, or Newmark's chart (Newmark 1942).

There are other sources of downdrag besides consolidation beneath an embankment, including densification of granular soils during seismic events, natural settlement of fill, collapse of collapsing soils and recompression of heaved soils. In these cases, special calculations will be required.

2.2 COATED SINGLE PILES

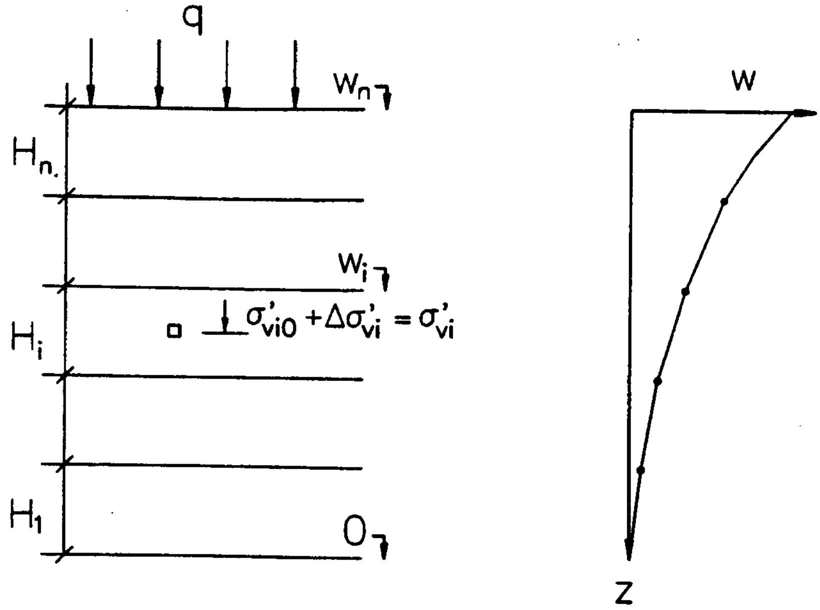
The analysis of coated single piles follows the same principles as that of uncoated single piles with one exception. Whereas the shear strength of the soil is time dependent, the shear strength of the bitumen coating is rate-of-loading dependent. For the typical case where downdrag is caused by the consolidation of a soft soil layer, the downdrag load will normally be largest on an uncoated pile in the long term since the shear strength increases with time. However, since the shear strength of bitumen is dependent on the rate of loading and since that rate decreases with time, it is not obvious when the maximum downdrag occurs on a bitumen-coated pile. Therefore, a bitumen-coated pile must be analyzed at several different times. The recommended times for analysis are (1) immediately after pile driving, (2) immediately after loading the piles, and (3) at the end of the life of the structure. If the sequence of construction operations indicates that the maximum rate of settlement will occur at some time other than one of these recommended times, then the pile should also be analyzed at that time. All rates of settlement should be the average rate of settlement over a 1-month period at the ground surface, rather than at an instantaneous rate of settlement.

2.3 PILENEG COMPUTER PROGRAM

2.3.1 The Program

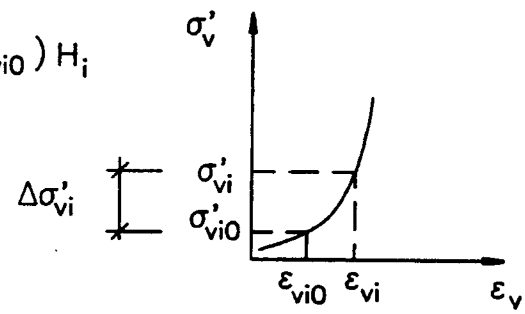
The PILENEG computer program was developed at Texas A&M University (Appendix) to run on MS-DOS based computers. The PILENEG software is available on the World Wide Web at http://civilgrads.tamu.edu/briaud. The program analyzes axially loaded single piles under negative skin friction based on the static equilibrium of the pile and on the compatibility of the relative pile-soil movement as explained in Section 2.1. For comparison purposes, it will also analyze piles assuming positive friction only.

The program assumes that the full pile-soil friction is mobilized if any relative movement occurs between the pile and the soil. Above the neutral point, the friction acts downward (negative) to add more load to the pile; below the neutral point, the friction acts upward (positive) to support the pile. The point resistance is assumed to follow an



Result of consolidation test on sample from layer i.

$$w_i = \sum_{1}^{i} (\varepsilon_{vi} - \varepsilon_{vi0}) H_i$$



$\Delta\sigma'_{vi}$ is the increase in stress in the middle of layer i due to the pressure q at the ground surface. It can be calculated by various methods, one of which is the chart in Fig. 2.10.

Figure 2.9. Calculation of soil settlement profile.

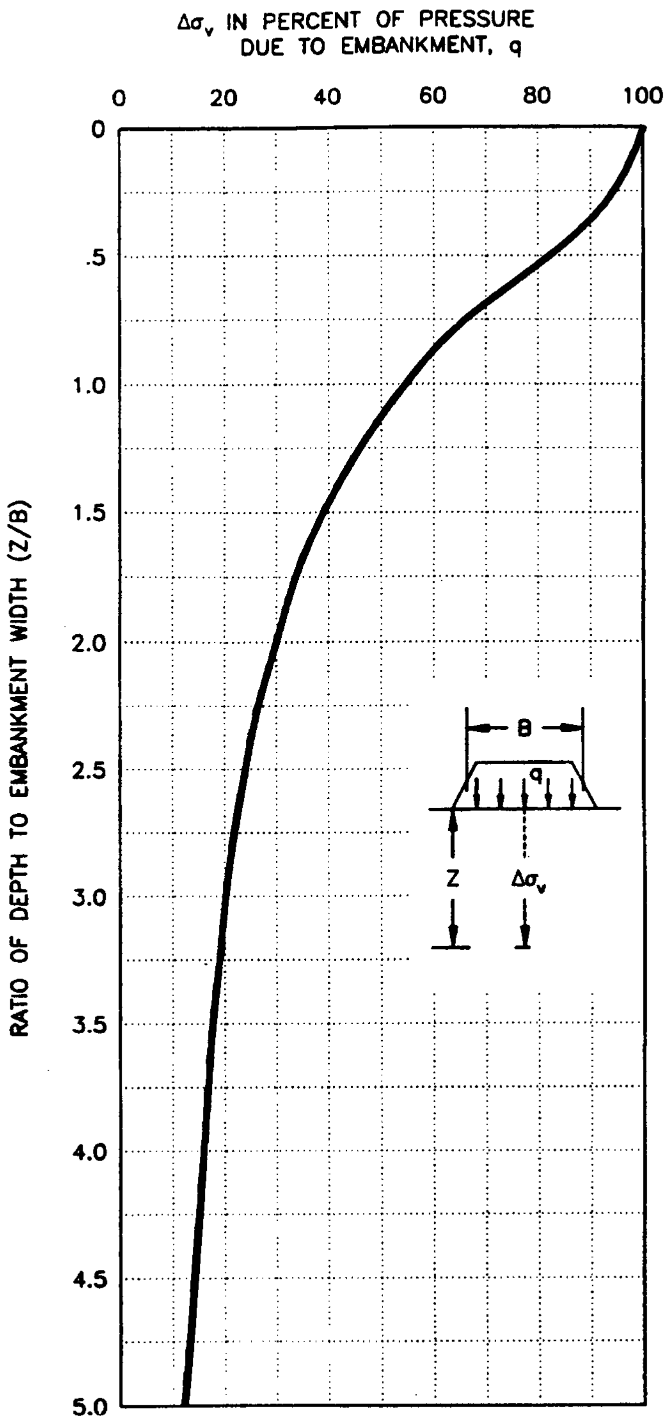


Figure 2.10. Variation of vertical stress beneath the center of an embankment, Boussinesq analysis. (after McCarthy 1982)

elasto-plastic model as stated in Section 2.1. The elastic portion of the curve can be calculated using Eq. 2.2.

Two options for loading are included in the program: (1) applying one top load with the value defined by the user or (2) calculating the entire load-settlement curve. There are five options for specifying the type of friction and bitumen-coating depth. First, the option of negative skin friction with no bitumen-coating is considered. Second, the option of negative skin friction with the bitumen-coating depth set equal to the depth of the neutral point is considered. When calculating the entire load-settlement curve with this option, the user must be aware that for each top load the depth to the neutral point changes, and therefore, the depth of the bitumen-coating changes. This option is used to determine the desired depth of coating by finding the load that corresponds to an acceptable settlement. The third option considers negative skin friction, but keeps the depth of the bitumen-coating constant This option is used to analyze the pile once the depth of bitumen-coating has been decided. The fourth and fifth options are included for comparison purposes. Option four considers positive friction only, with no bitumen-coating; option five considers positive friction only, with a constant bitumen-coating depth.

Depending on the options chosen, the following input will be needed: number of increments to divide the pile, top load on the pile, number of points on the load-settlement curve, shear strength of the bitumen, maximum number of iterations to find the depth of the bitumen coating, tolerance for convergence of the neutral point, depth of the bitumen coating, cross-sectiorial area of the pile, bearing area of the pile point, pile perimeter, embedded pile length, pile elastic modulus, profile of maximum friction values between the soil and the pile, soil settlement profile and the elastic modulus, and Poisson's ratio and ultimate bearing capacity of the soil below the pile point.

The output from the program includes top load, top settlement, depth of bitumen coating, depth to the neutral point, maximum load in the pile, maximum stress in the pile, and pile point load. There is also a table of axial force, axial stress, soil settlement, and pile settlement with depth. Several plots can be viewed or sent to a plotter.

The full users' manual with example problems is included in the Appendix.

2.3.2 Residual Driving Stresses

During driving, a pile is compressed under each hammer blow and moves downward into the soil. During the downward movement of the pile, the pile-soil friction is acting upward to resist the penetration of the pile; the point soil resistance is also acting upward. As the driving force decreases, the soil under the point pushes the pile back up and the pile decompresses elastically. These two components of the rebound create enough upward movement to reverse the direction of the pile-soil friction in the upper portion of the pile. The rebound continues until equilibrium is reached; that is, when enough of the friction stresses have been reversed in order to keep the bottom of the pile stressed against the soil.

The phenomena of residual stresses and downdrag lead to similar load versus depth profiles. The magnitude of downdrag stresses is large compared with residual stresses (Figure 2.11). This is due in part to the fact that residual stresses come from the friction developed by a remolded unconsolidated soil (driving), while downdrag stresses come from the friction developed by the same soil after consolidation. This is also because downdrag depends on the maximum friction profile and the soil-settlement profile whereas residual stresses depend simply on equilibrium of forces upon unloading after driving. In order to have significant residual stresses, large point load must be overcome during driving. This is the case only in very hard end bearing layers.

Measurements on piles in sand have shown that the residual point load is about 35% of the ultimate point load (Briaud et al. 1983). Piles with the tip in clay have very small residual stresses.

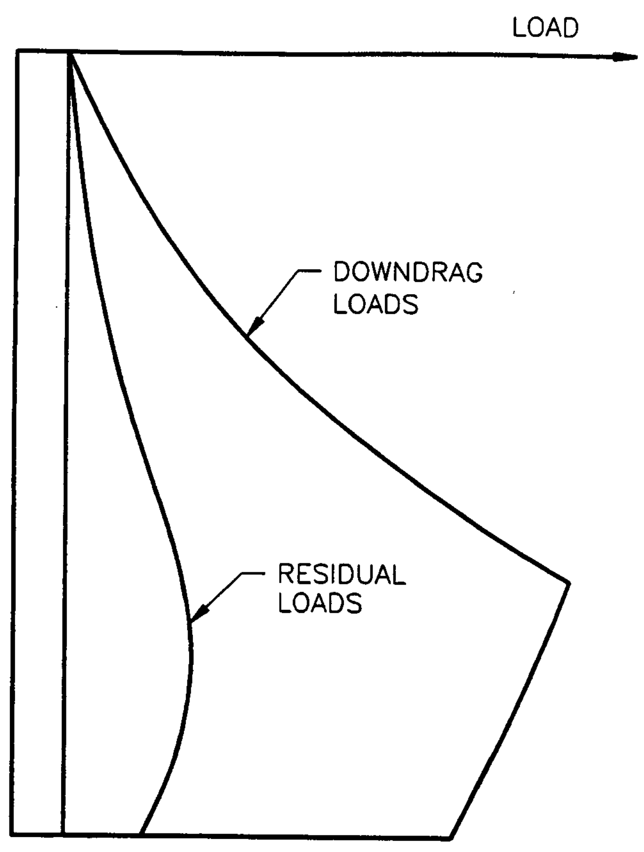


Figure 2.11. Comparison of downdrag and residual loads.

2.3.3 Evolution of the Load Distribution

The distribution of load in a pile evolves from the residual loads existing in the pile immediately following driving to the loads existing in the pile a significant period of time after the structural loads have been applied. This evolution follows different paths depending on whether downdrag is present or not. Figure 2.12 shows the steps of this evolution with and without downdrag present. The history of the pile has been broken up into four steps as follows: (1) immediately after driving, (2) a significant time after driving before the structural load is placed, but long enough for consolidation of the soil to produce downdrag, (3) immediately after placement of the structural load, and (4) a significant amount of time after placement of the structural load, long enough for consolidation of the soil to produce downdrag. It can be seen in Figure 2.12a that the loads in the pile do not change with time if downdrag is not present, but only change when the load at the pile top changes. In Figure 2.12b, however, it can be seen that when downdrag is present the load distribution in the pile changes with time due to consolidation of the soil. It can also be seen from Figure 2.12b that the residual loads are absorbed into the downdrag profile. Therefore, the PILENEG program ignores residual stresses.

A conventional pile load test is a series of step 3 loadings with the top load increasing until failure or termination of the test. However, the design of the pile should be governed by a series of step 4 loadings, where a load is applied and then downdrag develops. This will lead to much more settlement than a load test would indicate. This is what the PILENEG program predicts.

If the structural load is applied to the pile soon enough after driving to preclude significant consolidation of the soil, then the PILENEG program can be run one time with the long-term soil-settlement profile for input. However, if there is significant time between driving and loading of the pile, then the pile will experience some settlement before the structural load is applied to the pile. This settlement is due to the consolidation of the soil beneath the pile point plus the compression of the pile due to the downdrag load. This settlement should not be considered when checking the allowable settlement of the pile. Therefore, in this case the PILENEG program must be run twice. First, the settlement corresponding to zero load at the pile top is obtained using the estimated soil-settlement profile at the time of pile loading. This settlement occurs before the pile is loaded. Second, the load-settlement envelope is calculated using the long-term soil-settlement profile. The amount of settlement that occurs before the pile is loaded must be sub-

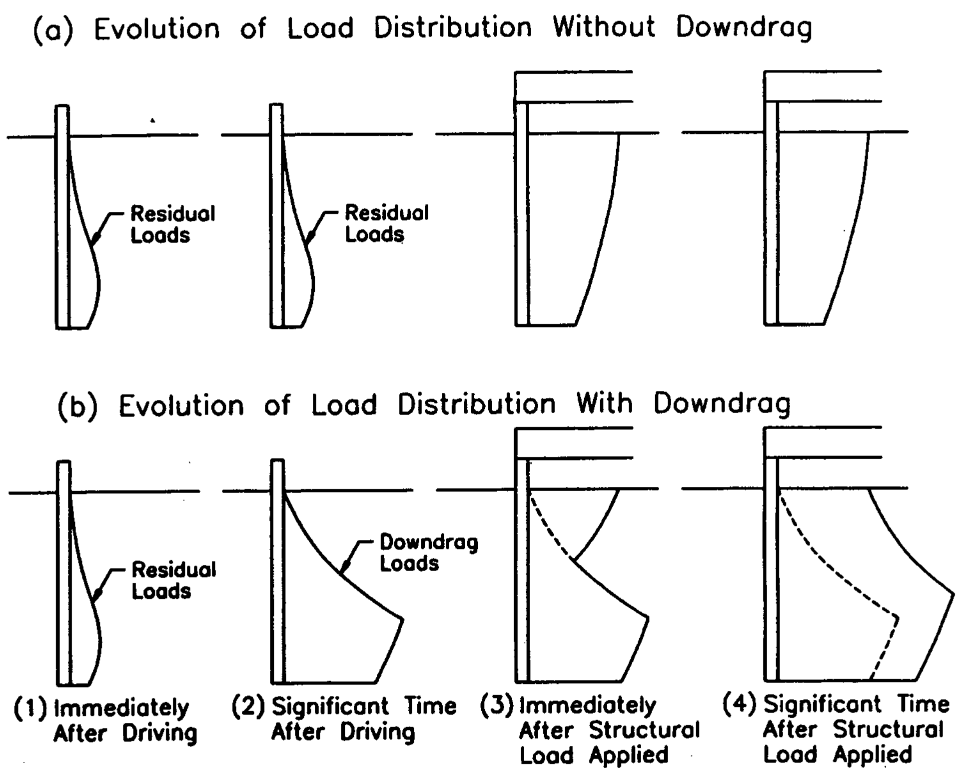


Figure 2.12. Evolution of the load distribution in a pile.

tracted from each point on this curve. This curve can then be used to obtain an allowable load corresponding to the allowable settlement.

2.3.4 Load-Settlement Envelopes

The results of PILENEG resemble a load-settlement curve that would be obtained from a pile load test. As stated in Section 2.3.3, however, a load test is a series of loadings corresponding to step 3 on Figure 2.12, whereas PILENEG predicts the equilibrium conditions of the pile under a series of loadings corresponding to step 4 on Figure 2.12. In order to avoid confusion, the PILENEG results are called a "load-settlement envelope" rather than a "load-settlement curve." Depending on the soil strength profile and the soil-settlement profile, the PILENEG results may have an irregular shape which would not be expected from a load-settlement curve from a load test. This is because this is a load-settlement "envelope" resulting from equilibrium conditions under rapid top-load application plus long-term downdrag rather than a load-settlement "curve" resulting from rapid top-load application only.

2.4 FACTORS OF SAFETY FOR SINGLE PILES

The design of a pile foundation is controlled by the driveability of the pile, the allowable settlement of the pile, the safety against structural failure of the pile material, and the safety against soil failure. The driveability of piles is not affected by downdrag and is therefore not treated in this manual. An exception to this is the driving of bitumen-coated piles, which appears to be sometimes more difficult than the driving of uncoated piles. The allowable settlement of the pile can be checked if a method, such as the PILENEG computer program, is used which calculates settlement. The last two criteria, safety against structural failure of the pile material and safety against soil failure, are generally accounted for by the use of load and resistance factors.

There are three sets of recommended load factors in use: American Concrete Institute (ACI), American Institute of Steel Construction (AISC), and American Association of State Highway and Transportation Officials (AASHTO). ACI (1989) uses 1.4D + 1.7L, AISC (1986) uses 1.2D + 1.6L, and AASHTO (1989) uses 1.3D + 2.17L. At this time, the ACI factors are favored because of their long-accepted record (Davisson 1993). The chosen load factors are 1.4 for dead load, 1.7 for permanent live load, 1.7 for transient live load, and 1.7 for downdrag load. The factor of 1.7 is used for the downdrag load because the uncertainties on the downdrag load are similar to the uncertainties on the live load and corresponds to ACI 318 for earth load.

In the case of downdrag, the maximum load in the pile may not be at the top of the pile, but rather at the neutral point (Figure 1.1). Therefore, the structural safety of the pile must be considered at the pile top and at the neutral point. The load factors to be applied at the pile top and at the neutral point for safety against structural failure are shown in Table 2.3. The resistance factors for the nominal axial structural capacity, Q_{nom}, of piles are shown in Table 2.4 for various pile types. Expressions for the value of Q_{nom} are shown in Tables 2.5, 2.6, and 2.7 for various pile types.

The proposed load and resistance factors for safety against soil failure are shown in Table 2.8. The resistance factors vary depending on the method of obtaining the pile capacity: pile load tests or a static prediction method. The resistance factor is higher if load tests are available. Two load tests must be run in order to use these higher factors. First, to obtain the ultimate total soil capacity in compression, Q_u, a compression load test must be conducted. Second, to obtain the negative friction load, F_n, a tension test must be conducted on a pile whose embedded length is equal to the estimated depth of the neutral point.

The resistance factors are also different for checking safety at the pile top or at the neutral point. The amount of reserve capacity when checking the safety at the pile top is depicted in Figure 2.13a. The amount of reserve capacity when checking the safety at the neutral point is depicted in Figure 2.13b. It can be seen that, at the neutral point, a larger amount of reserve capacity is available; therefore, the resistance factor in this case may be higher than at the pile top.

Note that at the top of the pile dead loads, permanent live loads and transient live loads are considered, whereas at the neutral point dead loads, permanent live loads, and downdrag loads are considered. Transient live loads are not considered at the neutral point because, being transient, they only reverse temporarily the negative skin friction caused by downdrag.

2.5 PILE GROUPS

Piles are usually driven in groups for the support of structures. There is evidence that the downdrag force on a group of n closely spaced piles is less than n times the downdrag force on an isolated single pile. This statement is based on one well-instrumented full-scale case history (Okabe 1977), on two laboratory scale studies (Koerner and Mukhopadhyay 1972, Ito and Matsu 1976), and on an extensive numerical anaysis with a three-dimensional finite element computer program using a nonlinear soil model (Jeong and Briaud 1992).

The full-scale case history was reported by Okabe (1977). Figure 2.14 shows the pile group configuration together with the load distribution for different piles in the group compared with a single pile. The center-to-center spacing for the group piles is approximately 2.1 diameters. It can be seen that the single-pile experiences a large downdrag load (7,000 kN), that the outer piles in the group carry

TABLE 2.3 Proposed load and resistance factors for structural safety of piles subjected to downdrag

SAFETY AGAINST STRUCTURAL FAILURE	
Top	$1.4D+1.7PL+1.7TL < \phi Q_{nom}$
Neutral Point[1]	$1.4D+1.7PL+1.7F_n < \phi Q_{nom}$

D = Dead load

PL = Permanent live load

Q_{nom} = Nominal axial structural capacity (see Table 2.5)

F_n = Negative friction load

TL = Transient live load

ϕ = Resistance factor (see Table 2.4)

1 The depth of the neutral point is determined by a settlement analysis with the top load equal to D+PL, the dead load plus the permanent live load.

TABLE 2.4 Resistance factors for the nominal axial structural capacity of piles (after Barker et al. 1991)

Pile Type	Resistance Factor, ϕ
Prestressed Concrete Piles	0.75 for spiral columns 0.70 for tied columns
Precast Concrete Piles	0.75 for spiral columns 0.70 for tied columns
Steel H-Piles	0.85
Steel Pipe Piles	0.85
Timber Piles	1.20[a]

a Davisson et al. (1983) stated that the minimum factor of safety for the structural capacity of timber piles in compression is 1.25. The resistance factor is greater than unity since the average load factor for vertical loads (dead and live loads) is greater than the factor of safety itself.

TABLE 2.5 Expressions for the nominal axial structural capacity of piles, Q_{nom}, in the absence of bending moments (after Barker et al. 1991)

Pile Type	Q_{nom}
Prestressed Concrete Piles	$(0.85f_c' - 0.6f_{pre})A_c$
Precast Concrete Piles	$0.85f_c'A_c + f_yA_y$
Steel H-Piles	f_yA_y
Steel Pipe Piles	f_yA_y
Timber Piles	$k_cs_cA_t$

f_c' = 28-day concrete cylinder strength
f_y = yield stress of steel
f_{pre} = effective prestress in the concrete
A_c = cross-sectional area of concrete
A_y = cross-sectional area of steel
A_t = cross-sectional area of timber
s_c = 5% exclusion limit in compression parallel to grain for green, small, clear wood specimens (see Table 2.6)
k_c = factor to account for the treatment condition of the timber pile and where along the pile the ultimate axial load is desired (see Table 2.7)

TABLE 2.6 5% exclusion values for compression parallel to grain, s_c (after Barker et al. 1991)

		s_c (kPa)
Douglas Fir	Coast	17756
	Interior West	17625
	Interior North	17080
	Interior South	15902
Southern Pine	Loblolly	17253
	Longleaf	21759
	Shortleaf	17907
	Slash	20139

TABLE 2.7 k_c factor to account for the treatment condition of timber piles when calculating the axial compressive strength parallel to grain (after Barker et al. 1991)

		Treatment Condition			
Location	Pile Length	Untreated or Air-Seasoned	Kiln Dried	Boulton Process	Steamed
Pile Butt	All Lengths	0.534	0.473	0.457	0.396
Pile Tip	≤ 15m	0.473	0.427	0.412	0.366
	> 15m	0.442	0.396	0.366	0.335

TABLE 2.8 Proposed load and resistance factors for soil capacity for piles subjected to downdrag

SAFETY AGAINST SOIL FAILURE		
Top	$1.4D+1.7PL+1.7TL < 0.75\ Q_u$	Where Q_u is from a load test
Top	$1.4D+1.7PL+1.7TL < 0.5\ Q_u$	Where Q_u is from a high quality static method
Neutral Point[1]	$1.4D+1.7PL+1.7F_n < 0.9\ (Q_u - F_n)$	Where Q_u and F_n are measured from load tests
Neutral Point[1]	$1.4D+1.7PL+1.7F_n < 0.75\ (Q_{pu} + F_{pu})$	Where Q_{pu} and F_{pu} are from a high quality static method

D = Dead load

PL = Permanent live load

TL = Transient live load

Q_u = Ultimate total soil capacity

Q_{pu} = Ultimate point bearing capacity

F_{pu} = Ultimate positive friction capacity

F_n = Negative friction load

1 The depth of the neutral point is determined by a settlement analysis with the top load equal to D+PL, the dead load plus the permanent live load.

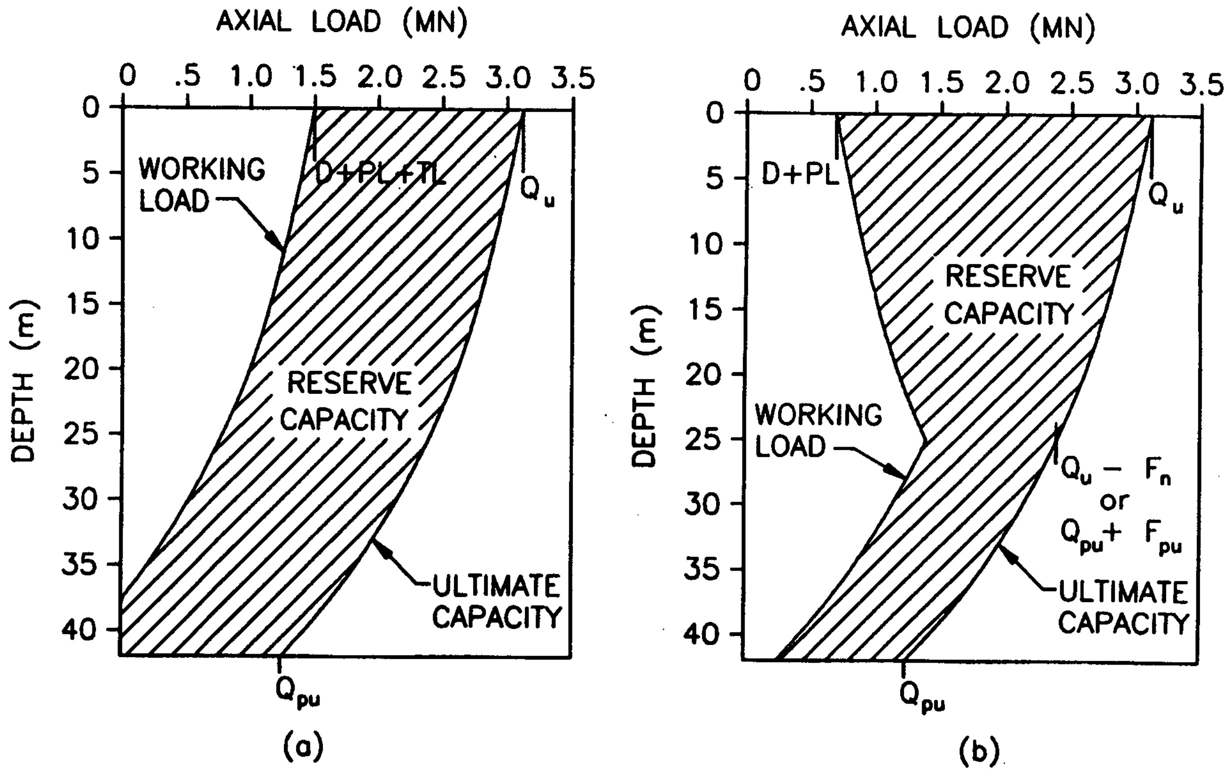


Figure 2.13. Amount of reserve capacity when checking safety at pile top and neutral point.

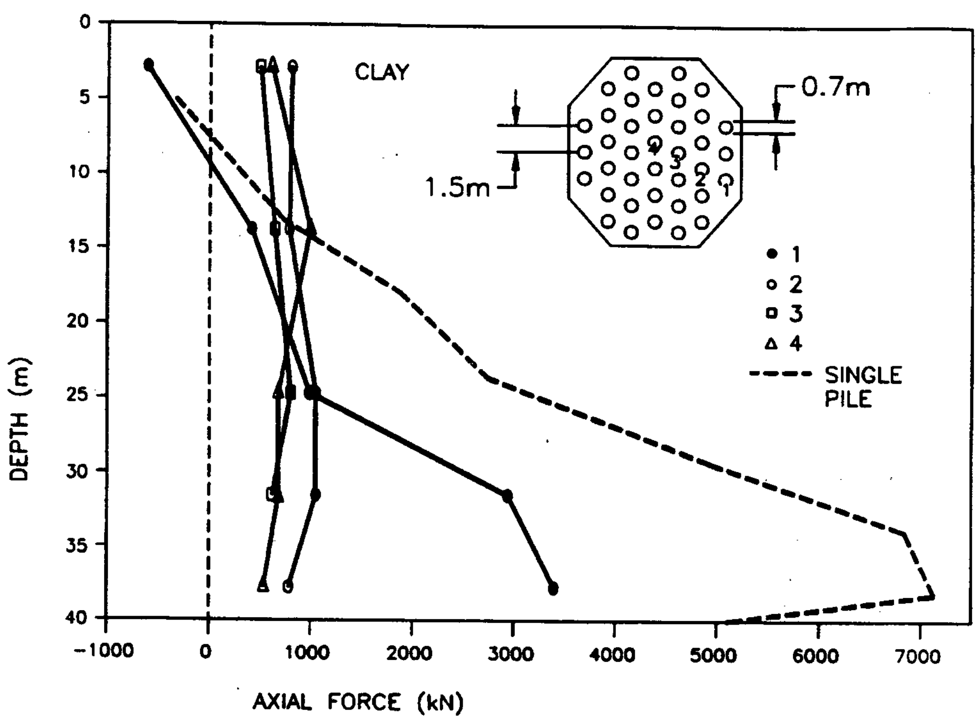


Figure 2.14. Axial forces in a pile group and in a single pile. (after Okabe 1977)

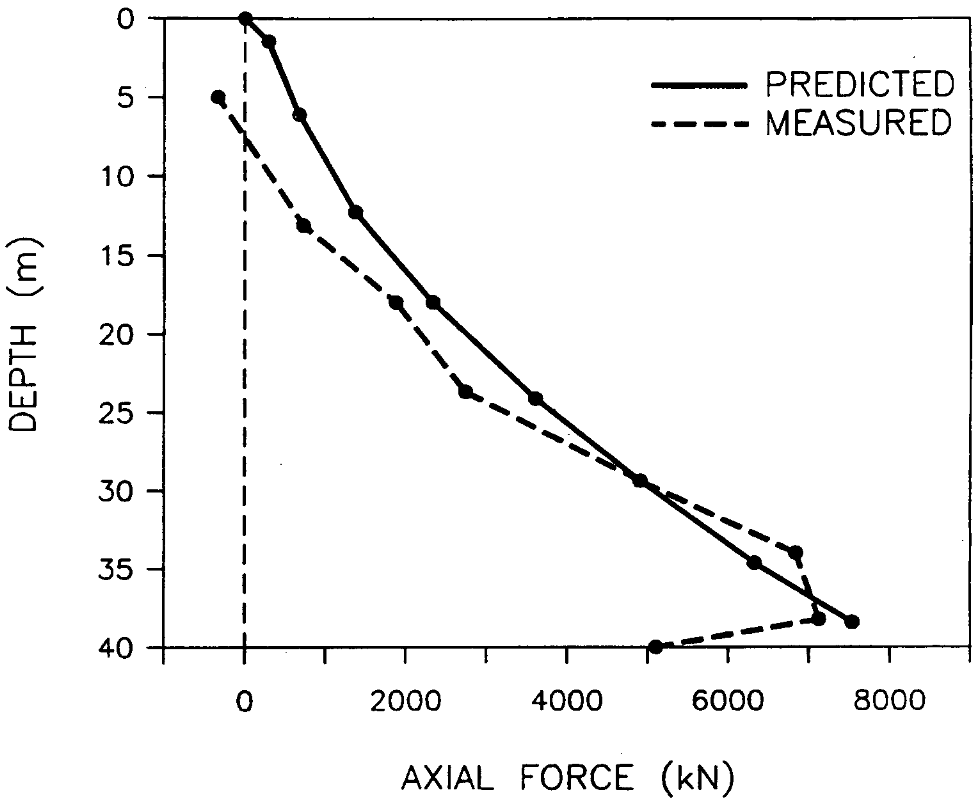


Figure 2.15. Comparison of predicted and measured axial forces in an endbearing single pile.

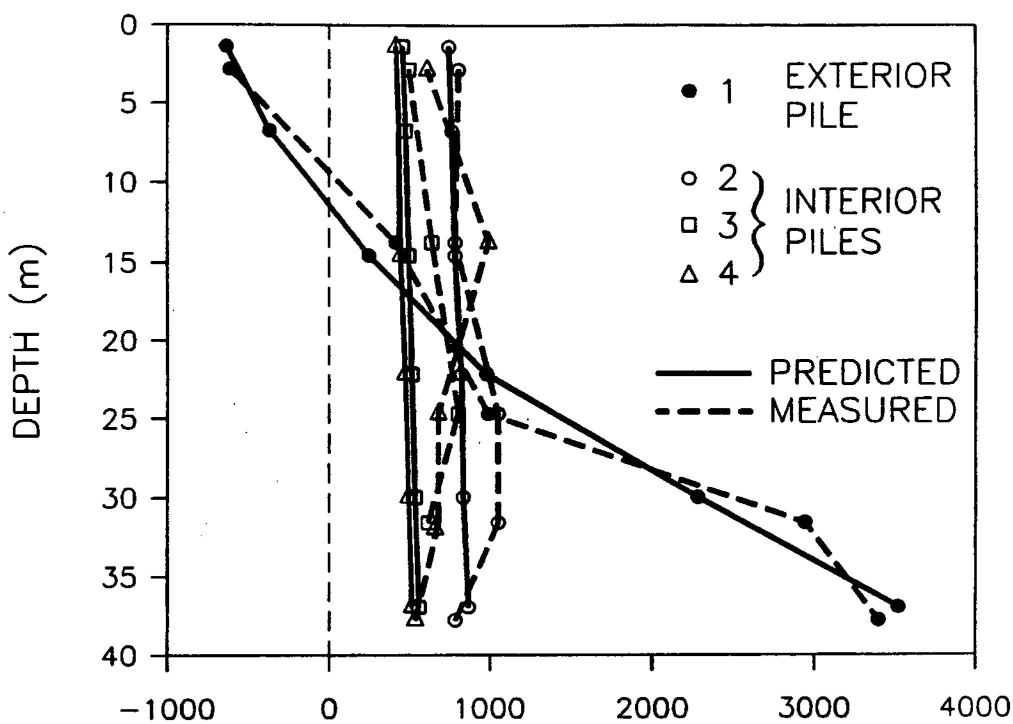


Figure 2.16. Comparison of predicted and measured axial forces in endbearing group piles.

TABLE 2.9 Proposed design factors for pile groups

S/D = 5

$F_{n(corner)} = 0.9\, F_{n(single)}$

$F_{n(side)} = 0.8\, F_{n(single)}$

$F_{n(interior)} = 0.5\, F_{n(single)}$

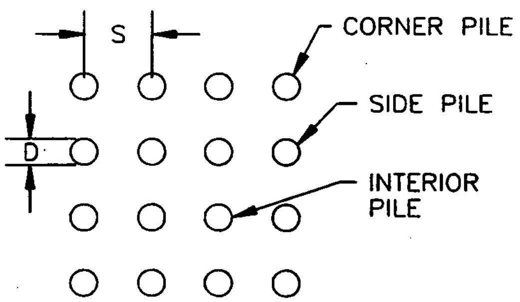
S/D = 2.5

$F_{n(corner)} = 0.5\, F_{n(single)}$

$F_{n(side)} = 0.4\, F_{n(single)}$

$F_{n(interior)} = 0.15\, F_{n(single)}$

S	=	center-to-center spacing
D	=	pile diameter
Fn(single)	=	downdrag force on the single pile
Fn(corner)	=	downdrag force on a corner pile in the group
Fn(side)	=	downdrag force on a side pile in the group
Fn(interior)	=	downdrag force on an interior pile in the group



about one-half of the single-pile downdrag load (3,500 kN), and that the interior piles in the group are subjected to only about 500 kN to 1,000 kN of downdrag or 7% to 14% of the single-pile downdrag load. The reduction of downdrag on the interior pile in the group is dramatic.

A parametric study was performed using a three-dimensional finite element analysis with a nonlinear soil model to determine the effect of pile spacing and of friction versus endbearing piles on the distribution of downdrag loads in pile groups (Jeong and Briaud 1992). An analysis was also performed for Okabe's pile group to verify the accuracy of the program. The results of the analysis of Okabe's piles are shown in Figure 2.15 for the single pile and Figure 2.16 for the pile group. The predicted loads correspond quite well with the measured data. The computer analysis revealed that the reduction of downdrag loads in group piles is due to the soil-settlement pattern shown in Figure 2.17. Within the pile group, the soil settlement is significantly reduced, thus reducing the development of downdrag.

On the basis of these findings, the reduction factors for the downdrag force, F_n, are shown in Table 2.9 and are recommended for pile group design. Note that this assumes the same point resistance for each pile in the group. Therefore, all piles should be driven to the same criteria regardless of position in the group. For a center-to-center spacing greater than 5 pile diameters, the piles act as single piles.

It is important to note that a significant tension load develops between the pile cap and the exterior piles (Figures 2.14 and 2.16). The reason for this is that the pile cap is relatively rigid, and that the exterior piles are subjected to significant downdrag while the interior piles are not. The soil drags the exterior piles down while the interior piles hold the pile cap. Therefore, the exterior piles are being pulled out of the pile cap, and the pile cap/exterior pile connections should be designed to resist such tension load. In order to estimate this tensile load, a three-dimensional nonlinear finite element analysis needs to be performed. In the case of Okabe's test, the magnitude of this tensile load was of the order of 15% of the downdrag on the single pile ($F_{n(single)}$). The center-to-center spacing for Okabe's test was 2.1 diameters. In the case of the finite element analysis, the magnitude of the tensile load was of the order of 10% of $F_{n(single)}$ for a spacing of 5 diameters.

Note that the pile cap, especially if it is buried, can develop a significant additional downdrag load.

Note also that another way to calculate the downdrag on the interior piles is to evaluate the weight of fill attributed to each pile.

If the downdrag loads calculated with the above procedures are too high for the uncoated piles, then bitumen can be used. However, bitumen should be used on all the piles in the group. Coating only the outer piles would simply transfer high downdrag loads to the inner piles.

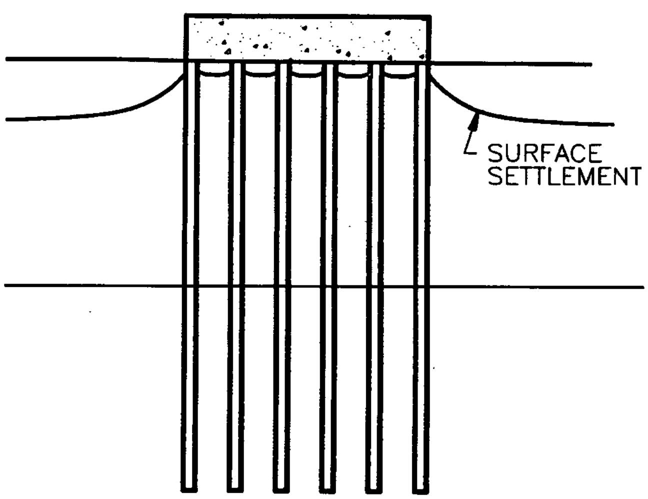


Figure 2.17. Settlement pattern under a pile group.

CHAPTER 3

CHARACTERISTICS OF BITUMEN

3.1 WHAT IS BITUMEN?

An engineer who is not an expert on bitumen can read this chapter to get a summary on bitumens. This will not make this engineer an expert on the topic. It will be necessary for this engineer to refer to an asphalt technologist for help in selecting the appropriate product based on the design considerations that are elaborated on in the report.

Several references will help the reader get a more detailed information on bitumens: Asphalt Institute (1981, 1987, 1990), Puzinauskas and Corbett (1978), Petersen et al. (1994a), Anderson et al. (1994), Petersen et al. (1994b), and Roberts et al. (1991).

If, after analyzing an uncoated single pile, it is concluded that downdrag is a problem that needs to be included in the design, one solution is to coat the piles with bitumen. In order to select the proper bitumen, it is necessary to understand the characteristics and behavior of bitumen.

The words "bitumen" and "asphalt" have been used interchangeably in the past. The definitions of bitumen and asphalt according to ASTM D 1079-87a are as follows:

Bitumen—(1) A class of amorphous, black or dark-colored (solid, semi-solid, or viscous) cementitious substances, natural or manufactured, composed principally of high molecular weight hydrocarbons, soluble in carbon disulfide, and found in asphalts, tars, pitches, and asphaltites; (2) A generic term used to denote any material composed principally of bitumen.

Asphalt—A dark brown to black cementitious material in which the predominating constituents are bitumens which occur in nature or are obtained in petroleum processing.

Asphalt, tar, wax, pitch, and resin all contain various amounts of bitumen that constitute the binder for those materials. Certain types of asphalts work best for downdrag reduction and are therefore discussed in this report. Because the word bitumen, instead of asphalt, has been used for years, in downdrag projects, therefore, in this report, the word bitumen is used to mean asphalt.

Asphalts are found in nature (natural asphalt) or are obtained by refining petroleum (manufactured asphalt). In both cases, asphalt is obtained by fractional distillation of petroleum, whether over short periods of time as in the refining process or over longer periods of time as in the natural process. Figure 3.1 is a schematic representation of the various fractions that are obtained during the distillation of crude oil.

3.2 HOW DOES BITUMEN BEHAVE?

Bitumen is a nonlinear viscous material. The shearing response of bitumen can be modeled by

$$\tau = \eta \dot{\gamma} \tag{3.1}$$

where τ is the shear stress, $\dot{\gamma}$ is the shear strain rate, and η is the secant viscosity hereafter referred to as viscosity. The units of viscosity are, therefore, those of shear stress divided by shear strain rate. The SI unit would be the Pascal second (Pa·s). However, viscosity has been traditionally reported in the CGS system of units in poise (P) or centipoise (cP). The conversion factors between these are as follows:

$$1\ \mathrm{P} = 1\ \frac{\mathrm{g}}{\mathrm{cm \cdot sec}} = 100\ \mathrm{cP}$$

$$1\ \mathrm{Pa \cdot s} = 1\left(\frac{\mathrm{N}}{\mathrm{m^2}}\right)\mathrm{s} = 1\ \frac{\mathrm{kg \cdot m \cdot s}}{\mathrm{s^2 \cdot m^2}} = 10\ \frac{\mathrm{g}}{\mathrm{cm{\cdot}s}} = 10P = 1{,}000\ \mathrm{cP}$$

Viscosity, η, is a measure of the resistance to flow of the fluid. A very viscous material has a high viscosity and, therefore, is a material which develops a high shear stress for a given strain rate. Note that viscosity is not a constant for a given bitumen, but depends on the shear strain rate and also on the temperature. Figure 3.2 shows that, for a given temperature, viscosity decreases with increasing strain rate. Of course, as the strain rate, $\dot{\gamma}$, increases, the shear stress τ increases, but the ratio $\tau/\dot{\gamma}$ decreases because bitumen is a nonlinear viscous material.

Figure 3.3 shows that, for a given shear strain rate, viscosity increases with decreasing temperature. For a given bitumen, the variation of η as a function of temperature, T, and shear strain rate, $\dot{\gamma}$, is described by the master curve (Figure 3.4). This master curve is obtained by performing a rheometer test on a sample of the bitumen (Section 3.3.2). Figure 3.4 is merely an example.

Baligh et al. (1981) and Baligh and Vivatrat (1976) conducted a series of experiments on bitumen. Their experiments revealed the following:

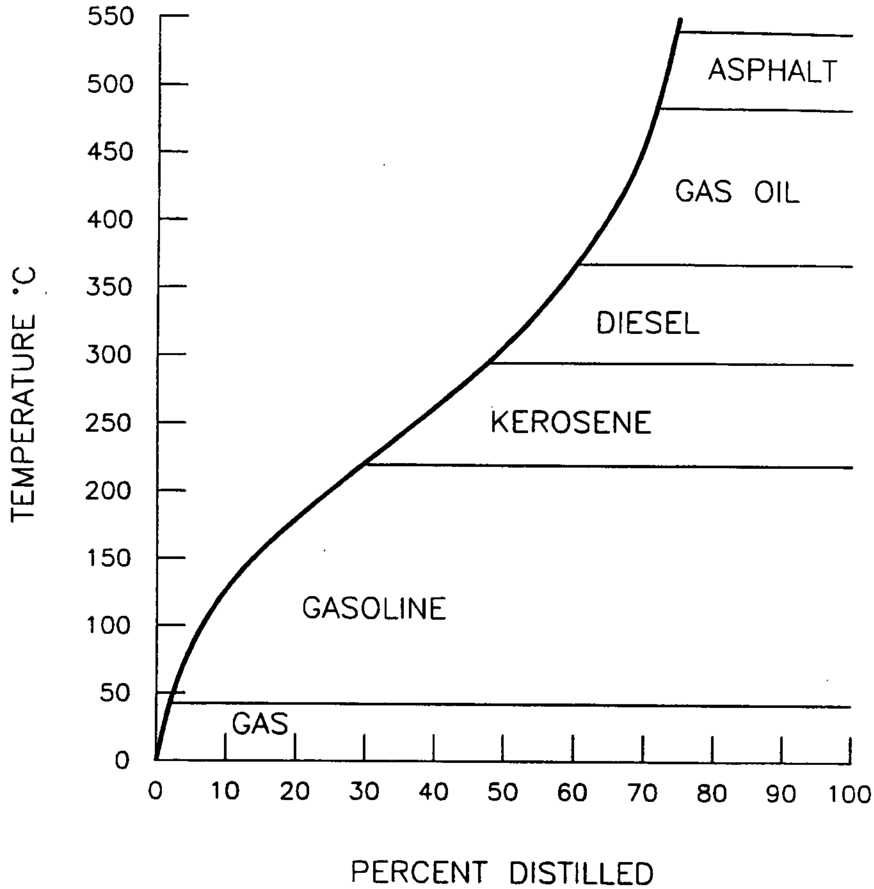


Figure 3.1. Typical distillation temperatures and products. (after the Asphalt Institute MS-22 1983)

1. The shear strength of bitumens is extremely sensitive to temperature. For example, an increase in temperature from 10°C to 20°C (factor of 2) was found to decrease the shear strength of bitumen by a factor larger than 10.
2. The shear strength of bitumen is very senstive to the shear strain rate. An increase in strain rate by a factor of 2 was found to increase the shear strength by a factor approaching 2. That is to say, the shear strength is almost linearly related to the shear strain rate.
3. The shear strength of the bitumen and its viscosity are independent of the coating thickness, the normal stress, the shearing direction, and the sliding distance.
4. Comments 1, 2, and 3 above hold true only if the bitumen layer has not been penetrated by soil particles. If the bitumen layer is contaminated with penetrating coarse soil particles, the shear strength of the bitumen can be increased by a factor of 10 and may lose its strain-rate dependent properties.

From these experimental observations, it is clear that it is essential to know the temperature conditions and the shear strain rate with accuracy and to ensure that the penetration of sand particles into the coating is minimized if not prevented.

3.3 LABORATORY TESTS FOR BITUMEN

Bitumens are characterized by their physical and chemical properties. The physical properties of importance include viscosity, penetration, softening point, and flash point. Of these properties, viscosity is the most fundamental and is recommended here for use in specifications. The other properties are also discussed because of their frequent use in practice.

3.3.1 Viscometer Test for Viscosity

Viscometers have traditionally been used to measure the viscosity of asphalts over a wide temperature range. The most commonly used viscometers are the gravity-flow viscometer and the vacuum-capillary viscometer. The procedures to be followed in making viscosity determinations are described in ASTM D 2170, ASTM D 2171, and

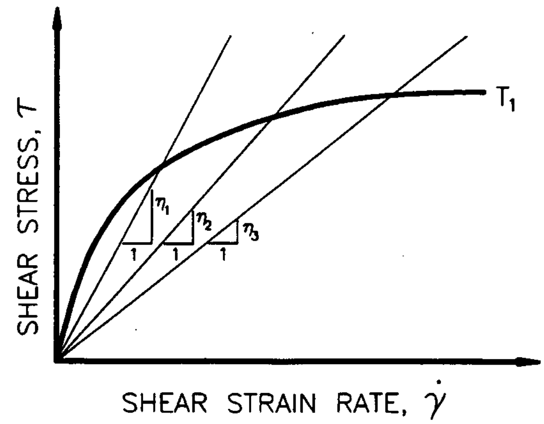


Figure 3.2. Influence of shear strain rate on viscosity.

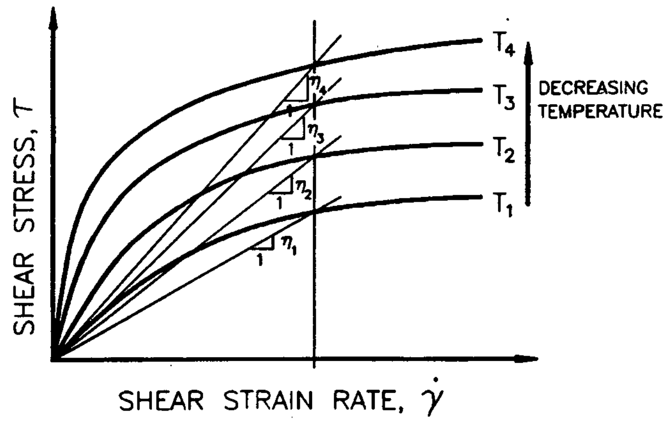


Figure 3.3. Influence of temperature on viscosity.

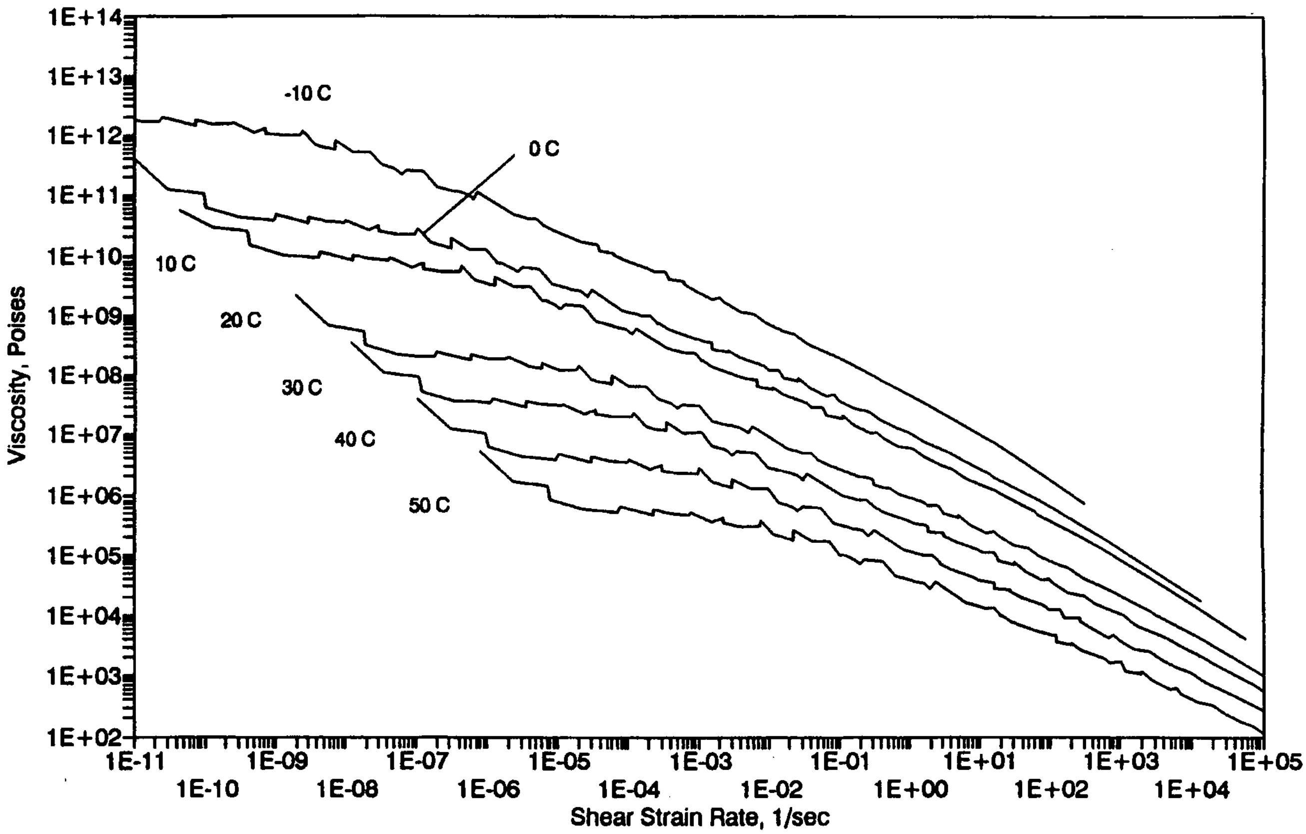


Figure 3.4. Example of viscosity master curves for softbearing pile lubricant obtained from rheometer tests.

ASTM D 3205. The procedure to be followed will depend on the magnitude of the viscosity to be measured and the temperature at which this measurement is to be made. Normally, at temperatures higher than 135°C (275°F), gravity-flow viscometers are used (ASTM D 2170). For temperatures between 60°C (140°F) and 135°C (275°F), vacuum-capillary viscometers are used (ASTM D 2171), and at temperatures lower than 60°C (140°F) cone-plate viscometers are used (ASTM D 3205).

Figure 3.5 shows generalized viscosity ranges for different asphalts. It can be seen that, in general, roofing asphalts have higher viscosity values than paving asphalts. For downdrag problems, paving asphalts are generally too soft. Roofing asphalts, culvert compounds, and canal liners have been used with success in reducing downdrag.

The limitations of these traditional viscometers are that, for each test, only one viscosity value at one temperature is found, and the shear strain rate is unknown. Therefore, the bitumen behavior cannot be adequately described and the master curve which is needed for design cannot be obtained with these viscometers.

3.3.2 Rheometer Test for Viscosity

The rheometer is an instrument that was originally designed for polymer research, but is now being used to make accurate measurements of the viscoelastic properties of asphalt. By this method, master curves of viscosity, η, versus shear strain rate, $\dot{\gamma}$, can be generated for various temperatures (see Figure 3.4) in much less time than the limited information that can be obtained with the three traditional viscometers. These master curves are used in the bitumen selection process described in Chapter 4.

The test procedure is as follows. The asphalt sample (8 mm to 25 mm in diameter, 1 mm thick) is placed between two parallel circular plates (Figure 3.6). The preselected strain rates (generally ranging from 0.01 rad/sec to 100 rad/sec) are applied to the sample by a cyclic torsional shear actuator. A transducer measures the actual strain, deformation, and force (torque), and the data are recorded and analyzed by a computerized data acquisition system. The temperature of the asphalt is controlled with an environmental chamber—using liquid nitrogen to cool the sample or an oven heater to heat the sample—and usually ranges from -65°C to 100°C. The Cox-Merz rule (Cox and Merz 1958) shows that the viscosity obtained from the rheometer test can be used in Eq. 3.1.

The shear strain rates needed for design are generally out of the range of the rheometer. However, the curves obtained at different temperatures can be shifted along the shear strain rate axis (using the time-temperature superposition principle) to extend the range of shear strain rate

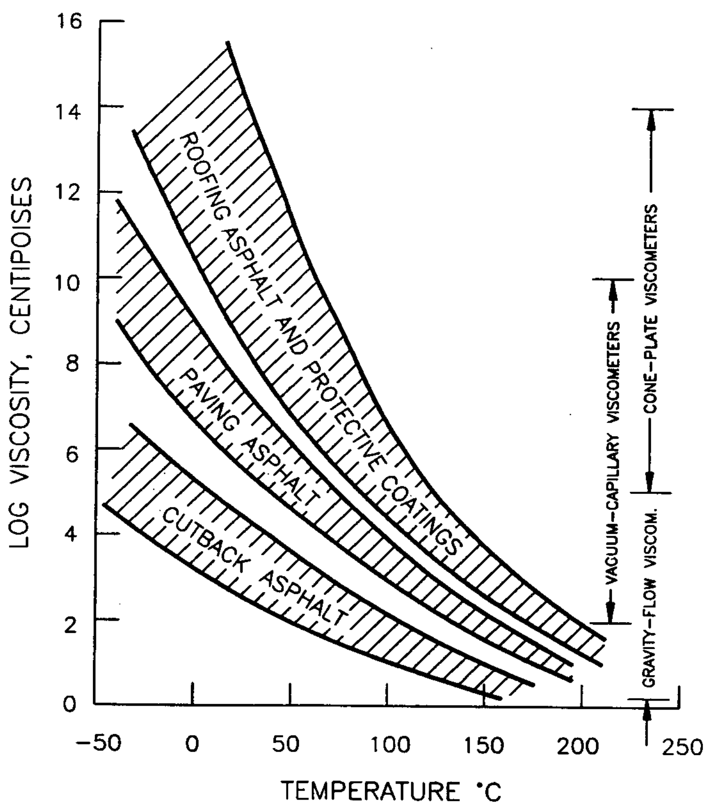


Figure 3.5. Viscosity ranges for different asphalts. (after Puzinauskas 1982)

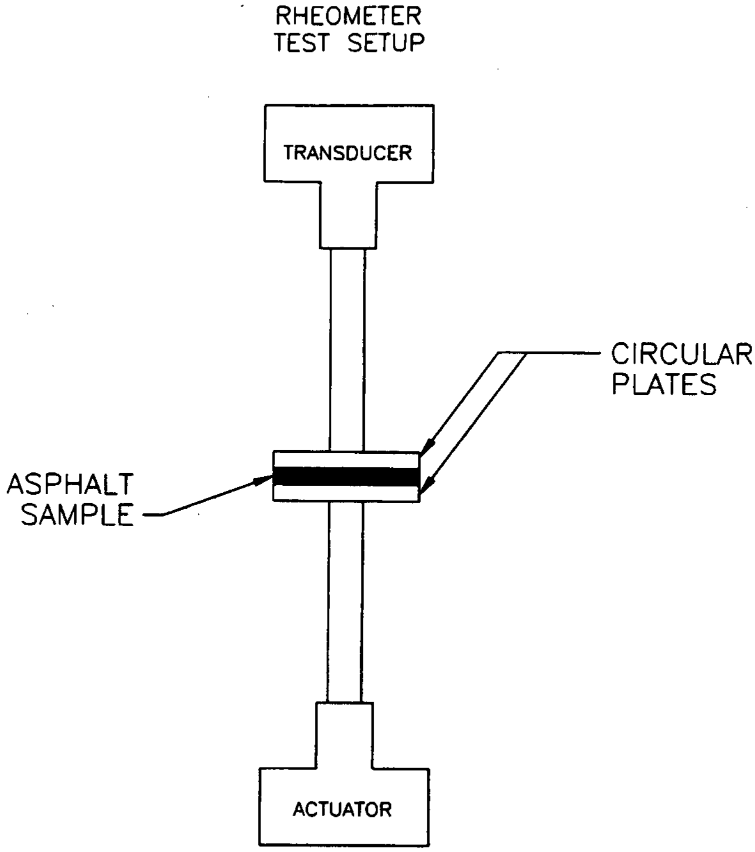


Figure 3.6. Rheometer test parallel plate geometry.

characterization beyond that which is experimentally practical (Ferry 1980).

3.3.3 Penetration Test

The penetration test is an empirical test used to measure asphalt consistency. The procedure to be followed is described in ASTM D 5-86. According to ASTM, penetration is defined as the distance in tenths of a millimeter that a standard needle (1 mm in diameter with tapered point) penetrates vertically into a sample of the material under a load of 100g at 25°C in a time of 5 seconds. Other load, temperature, and time conditions may be used if specified. The higher the value of penetration is, the softer is the asphalt. As an example, paving asphalts have penetrations at 25°C which vary from 4 to 220 (AC40 to AC2.5), roofing asphalts from 12 to 60 (type IV to type I). Bitumens to reduce the downdrag on piles often have standard penetration values higher than 50.

3.3.4 Softening Point Test (Ring-and-Ball Method)

Asphalts become softer and less viscous at higher temperatures. Because asphalts do not have a sharply defined melting point, it is important to determine the softening point, which is an arbitrarily but consistently defined point during the softening process. This softening point is used as one of the parameters in the classification process. The softening point also indicates the tendency of an asphalt to flow at elevated temperatures during its life.

The test procedure for the Ring-and-Ball method is described in ASTM D 36-86. It involves the heating of two horizontal disks of asphalt at a controlled rate in a liquid bath (Figure 3.7). Each disk of asphalt supports a steel ball. The temperature is raised at a uniform rate of 5°C/min. The temperature at which the two asphalt disks soften sufficiently so as to allow the enveloped steel balls to fall through a distance of 25mm (1.0 in.) is considered to be the softening point of that asphalt. As an example, roofing asphalts have softening points which vary from 57°C to 107°C (type I to type IV) and paving asphalts from 35°C to 60°C.

3.3.5 Flash and Fire Points

The flash point is a temperature which indicates the tendency of an asphalt to form a flammable mixture with air under controlled laboratory conditions. The flash point is

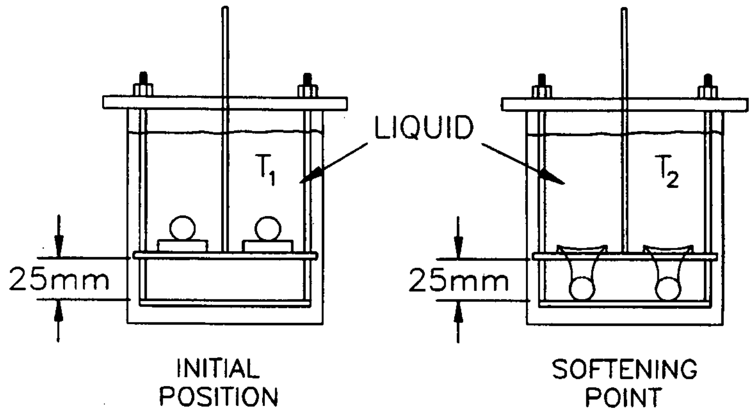


Figure 3.7. Ring-and-Ball test setup.

one of the properties that must be considered while assessing the overall flammability of an asphalt. In general, asphalts should not be heated to or above their flash point temperature. The test procedure is described in ASTM D 92-85. It involves heating an asphalt sample in a test cup—rapidly at first, then at a slow constant rate as the flash point is approached—and passing a small test flame across the cup at specified intervals. The flash point is the lowest temperature, corrected to a barometric pressure of 101.3 kPa (760 mm Hg), at which the application of the test flame causes the vapor of the asphalt specimen to ignite. The test is continued, and the lowest temperature at which the test flame ignites and burns the sample for at least 5 seconds is recorded as the fire point. For example, the flash point of paving asphalts varies from 162°C to 232°C (AC2.5 to AC40) while the flash point of roofing asphalts is greater than 246°C.

3.4 CURRENT METHODS OF BITUMEN CLASSIFICATION

Asphalts possess diverse properties and are used for many purposes. As a result, asphalts are classified by the various methods described below. These classifications are designed to aid the user in selecting an asphalt that meets his or her requirements.

3.4.1 Classification According to Use

This is the primary basis of asphalt classification. For convenience, asphalts have been categorized into three broad groups, namely paving asphalts, roofing asphalts, and protective coatings.

Paving asphalts. These asphalts are widely used in road construction. Asphalt cements, liquid asphalts, and emulsified asphalts are included in this category. These asphalts tend to be too soft for use as bituminous coatings on piles, especially in hot climate areas.

Roofing asphalts. This category includes mopping grades of asphalts used primarily in built-up roofing, produced by the air-blowing process, and asphalts used in prepared roofing, such as shingles and roll roofing. Air blowing is a process in which air is blown through molten asphalt at an elevated temperature to raise its softening point and modify other properties.

Protective coatings. This category includes asphalts to be used in pipe coatings, reservoir and canal linings, and other applications where an impervious or protective coating is required. These asphalts are also generally air blown and often contain finely divided mineral fillers, in which case they are referred to as "filled" asphalts. Catalytically blown asphalts, asphalts that are blown in the presence of special catalysts such as $P_2 0_5$, are also included in this category.

The physical properties of air-blown asphalts seem to be best suited for pile coating, because these asphalts have higher softening points, lower temperature susceptibility, and are less brittle in comparison with paving asphalts.

3.4.2 Classification According to Viscosity

This method of classification has been used for paving asphalts (Table 3.1), but is not currently used for roofing asphalts. Viscosity is a fundamental parameter that can be used to quantify an extremely wide range of consistencies for various temperatures and shearing rates, whereas empirical measurements, such as softening point or penetration, cannot. Therefore, it is recommended that viscosity be used as the primary parameter for the specification of asphalt for bitumen coatings on piles.

TABLE 3.1 Requirements for asphalt cement graded by viscosity at 60°C (AASHTO M226)

TEST	VISCOSITY GRADE					
	AC-2.5	AC-5	AC-10	AC-20	AC-30	AC-40
Viscosity, 60° C(140° F), poises	250±50	500±100	1000±200	2000±400	3000±600	4000±800
Viscosity, 135° C(275° F),Cs-minimum	125	175	250	300	350	400
Penetration, 25° C(77° F), 100g, 5 sec.-minimum	220	140	80	60	50	40
Flash point, COC, ° C° F minimum	162(325)	177(350)	219(425)	232(450)	232(450)	232(450)
Solubility in trichloroethylene, percent-minimum	99.0	99.0	99.0	99.0	99.0	99.0
Tests on residue from Thin-Film Oven Test:						
Loss on heating, percent-maximum (optional)[3]	...	1.0	0.5	0.5	0.5	0.5
Viscosity, 60° C(140° F), poises-maximum	1000	2000	4000	8000	12000	16000
Ductility, 25° C(77° F), 5cm per minute, cm-minimum	100[1]	100	75	50	40	25
Spot test (when and as specified)[2] with:						
Standard naphtha solvent	Negative for all grades					
Naphtha-Xylene, % Xylene	Negative for all grades					
Heptane-Xylene, % Xylene	Negative for all grades					

1 If ductility is less than 100, material will be accepted if ductility at 15.6° C(60° F) is 100 minimum.

2 The use of the spot test is optional. When it is specified, the Engineer shall indicate whether the standard naphtha solvent, the naphtha-xylene solvent, or the heptane-xylene solvent will be used in determining compliance with the requirement, and also, in the case of xylene solvents, the percentage of xylene to be used.

3 The use of loss on heating requirement is optional.

3.4.3 Classification According to Penetration

This method of classification is based on the values of asphalt penetration (Table 3.2).The asphalts are classified as Grade 40-50, Grade 60-70, and so forth. The numbers indicate the minimum and maximum values of penetration from the penetration test (Section 3.3.3).

3.4.4 Classification According to Penetration Index

This method uses the Penetration Index (P.I.), a measure of temperature susceptibility, to classify asphalts. Knowing the penetration at 25°C (77°F) and the Ring-and-Ball softening point for an asphalt, its penetration index can be determined from the nomograph shown in Figure 3.8.

The suggested classifications are

1. High temperature susceptibility P.I. < –2 (pitch type)
2. Medium temperature susceptibility –2 < P.I. < +2 (normal asphalts)
3. Low temperature susceptibility P.I. > +2 (air blown asphalts and polymer modified)

A bitumen with a penetration independent of temperature has a P.I. of +20 while, at the other extreme, a bitumen with infinite temperature susceptibility should have a P.I. of -10. Certain waxy bitumens exhibit a false softening point; therefore, Figure 3.8 cannot be used for these types of bitumens.

3.4.5 Classification According to Softening Point

This method of classification has been used to classify roofing asphalts, asphalts used in canal, ditch, and pond linings, and asphalts used in dampproofing and waterproofing. On the basis of the softening point range, roofing asphalts have been classified into four types (ASTM D

TABLE 3.2 Requirements for asphalt cement graded by penetration (AASHTO M20)

	Penetration Grade									
	40-50		60-70		85-100		120-150		200-300	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
Penetration at 25° C(77° F) 10g, 5 sec	40	50	60	70	85	100	120	150	200	300
Flash point, Cleveland Open Cup	450	...	450	...	450	...	425	...	350	...
Ductility at 25° C(77° F) 5cm. per min., cm	100	...	100	...	100	...	100	...	...	...
Solubility in trichloroethylene, percent	99	...	99	...	99	...	99	...	99	...
Thin-film oven test, 3.2mm (1/8 in.), 163° C(325° F) 5 hour										
Loss on heating, percent	...	0.8	...	0.8	...	1.0	...	1.3	...	1.5
Penetration, of residue, percent of original	58	...	54	...	50	...	46	...	40	...
Ductility of residue at 25° C (77° F) 5cm. per min., cm	...	...	50	...	75	...	100	...	100	...
Spot test (when and as specified (see note)) with:										
Standard naphtha solvent	Negative for all grades									
Naphtha-Xylene, % Xylene	Negative for all grades									
Heptane-Xylene, % Xylene	Negative for all grades									

NOTE: The use of the spot test is optional. When it is specified, the Engineer shall indicate whether the standard naphtha solvent, the naphtha-xylene solvent, or the heptane-xylene solvent will be used in determining compliance with the requirement, and also, in the case of xylene solvents, the percentage of xylene to be used.

312-84) (Table 3.3), asphalts used in dampproofing and waterproofing have been classified into three types (ASTM D 449-79[83]) (Table 3.4), and asphalts for use in waterproof membranes have been specified (ASTM D 2521-76[81]) (Table 3.5). These types of asphalts have been used successfully for coating piles subjected to downdrag.

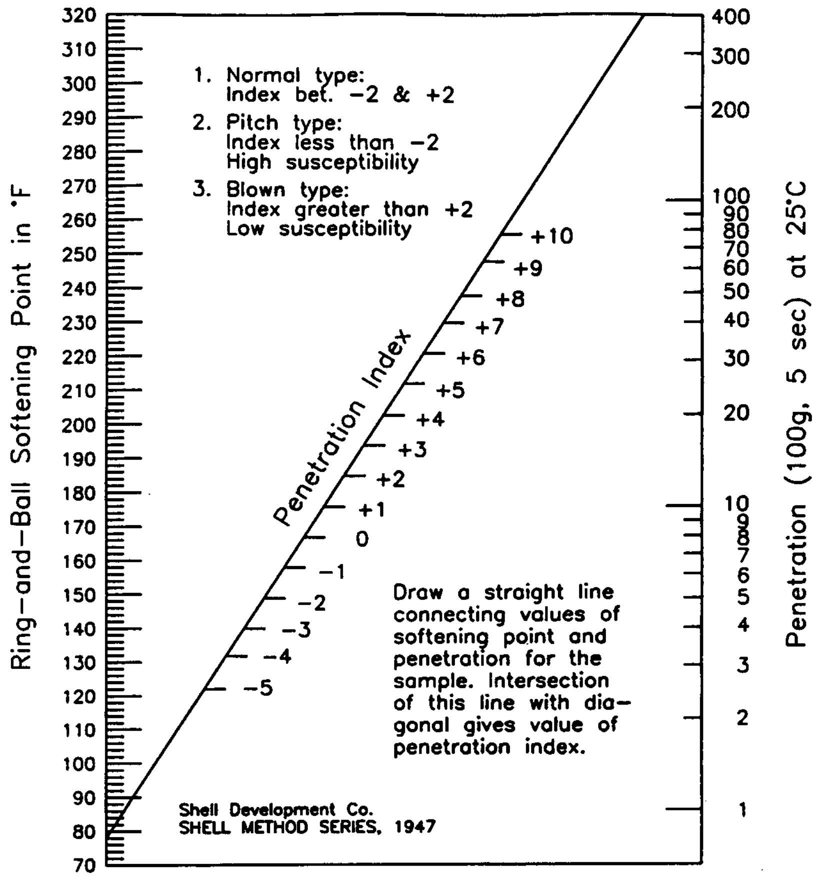


Figure 3.8. Nomograph for determining penetration index of asphalts from values of softening point and penetration. (from Shell Method Series, Shell Development Co.)

TABLE 3.3 Physical requirements of asphalt in roofing (ASTM D312-84)

Property	Type I		Type II		Type III		Type IV	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
Softening Point, ° C (° F)	57(135)	66(151)	70(158)	80(176)	85(185)	96(205)	99(210)	107(225)
Flash Point, ° C(° F)	246(475)	...	246(475)	...	246(475)	...	246(475)	...
Penetration, units								
at 0° C(32° F)	3	...	6	...	6	...	6	...
at 25° C(77° F)	18	60	18	40	15	35	12	25
at 46° C(115° F)	90	180	...	100	...	90	...	75
Ductility at 25° C(77° F), cm	10.0	...	3.0	...	2.5	...	1.5	...
Solubility in trichloroethylene, %	99	...	99	...	99	...	99	...

TABLE 3.4 Physical requirements of asphalt used in dampproofing and waterproofing (ASTM D449-79[83])

Property	Type I		Type II		Type III	
	Min.	Max.	Min.	Max.	Min.	Max.
Softening Point (ring-and-ball), ° C (° F)	46(115)	60(140)	63(145)	77(170)	82(180)	93(200)
Flash Point (Cleveland open cup), ° C(° F)	175(347)	...	200(392)	...	205(401)	...
Penetration, 0.1mm:						
at 0° C(32° F), 200g, 60 s	5	...	10	...	10	...
at 25° C(77° F), 100g, 5 s	50	100	25	50	20	40
at 46° C(115° F), 50g, 5 s	100	...	...	130	...	100
Ductility at 25° C(77° F), cm	30	...	10	...	2	...
Solubility in trichloroethylene, %	99	...	99	...	99	...

TABLE 3.5 Requirements for asphalt for use in waterproof membrane construction (ASTM D2521-76[81])

Property	Min.	Max.
Softening Point (ring-and-ball), ° C (° F)	79(175)	93(200)
Penetration of original sample, 0.1mm:		
at 0° C(32° F), 200g, 60 s	30	...
at 25° C(77° F), 100g, 5 s	50	60
at 46° C(115° F), 50g, 5 s	...	120
Ductility at 25° C(77° F), mm	35	...
Flash Point (Cleveland open cup), ° C(° F)	218(425)	...
Solubility in trichloroethylene, %	97.0	...
Loss on heating, %	...	1.0
Penetration after loss on heating, % of original at 25° C(77° F), 100g, 5 s	60	...

CHAPTER 4

DESIGN AND SELECTION OF THE BITUMEN

4.1 DESIGN APPROACH

Once downdrag has been determined to exist, and coating with bitumen has been shown to be an economical alternative, the appropriate bitumen must be selected. The selected bitumen must be available for use, it must be able to be applied to the pile, and it must remain on the pile before, during, and after installation to be effective in reducing downdrag. There are several parameters that the designer must know as a function of depth in order to design and select a bitumen:

1. Soil shear strength. This is obtained by soil testing in shear.
2. Soil type (particle size). This is obtained by sieve analysis.
3. Soil unit weight. This is obtained by soil sampling.
4. Soil temperature. This can be taken as the mean yearly air temperature.
5. Soil settlement rate as a function of time. This can be obtained by consolidation tests and the consolidation theory and may be significantly influenced by the use of wick drains.
6. Air temperature during storage and driving. This can be obtained by contacting the closest weather service station (National Weather Service, National Oceanic and Atmospheric Administration [NOAA], radio, or television).

These parameters are used in the design process for selecting the bitumen. The bitumen must be designed so that (1) it does not deform excessively under the gravity stresses during the storage period (design for storage), (2) it does not deform excessively under the dynamic stresses present during driving (design for driving), (3) it offers little shearing resistance so as to reduce downdrag during the soil-settlement process (design for downdrag reduction), and (4) it allows the soil particles to penetrate only an allowable amount into the coating (design for particle penetration). Each of these design aspects is described in the following sections and illustrated by examples in Chapter 6.

4.2 GENERAL PROCEDURE

The design criteria are

1. Criterion for storage,
2. Criterion for driving,
3. Criterion for downdrag, and
4. Criterion for particle penetration.

The design approach for each of these criteria is similar and consists of the following. First, the shear stress, τ, applied to the bitumen is obtained. Second, the shear strain rate, $\dot{\gamma}$, to which the bitumen is subjected is calculated. Third, the required viscosity, $\eta_{req} = \tau/\dot{\gamma}$, is calculated. Fourth, the viscosity of the bitumen must be larger or smaller than η_{req} depending on the design criterion; for example, storage requires that $\eta > \eta_{req(storage)}$ downdrag requires $\eta < \eta_{req(downdrag)}$. Fifth, curves of η versus $\dot{\gamma}$ as a function of temperature, T, (master curves) are obtained from rheometer tests on a trial bitumen. The η_{req} are plotted at the corresponding $\dot{\gamma}$ and T° on the η versus $\dot{\gamma}$ plot where the master curves are located (Figure 4. 1). Then the η_{req} is compared with the η for this bitumen at the same $\dot{\gamma}$ and T. This bitumen is acceptable if it satisfies the requirements on η for criteria 1 through 4 above.

Note that criteria 1, 2, and 4 require that η be larger than the corresponding η_{req} for that criterion while criterion 3 requires that η be smaller than η_{req} . Therefore, a window of acceptable η values may exist that satisfies the four criteria (Figure 4.2a). It may also be that no window exists that satisfies the four criteria (Figure 4.2b). In this case, it is necessary to impose restrictions on the construction of the piles, such as coating, storing, and driving during the cooler months of the year, shading the piles during storage, or using other means to open the window of acceptable η.

Note also that because the bitumen softens when the temperature increases, a satisfactory bitumen is more likely to be found if the air temperature (storage and driving) is lower than the soil temperature (downdrag); this is the case in winter months. Indeed in the summer, it is required to find a bitumen which has a high viscosity (hard) at hot temperatures (air T°) and a low viscosity (soft) at low temperatures (soil T°). That is not the natural tendency for bitu-

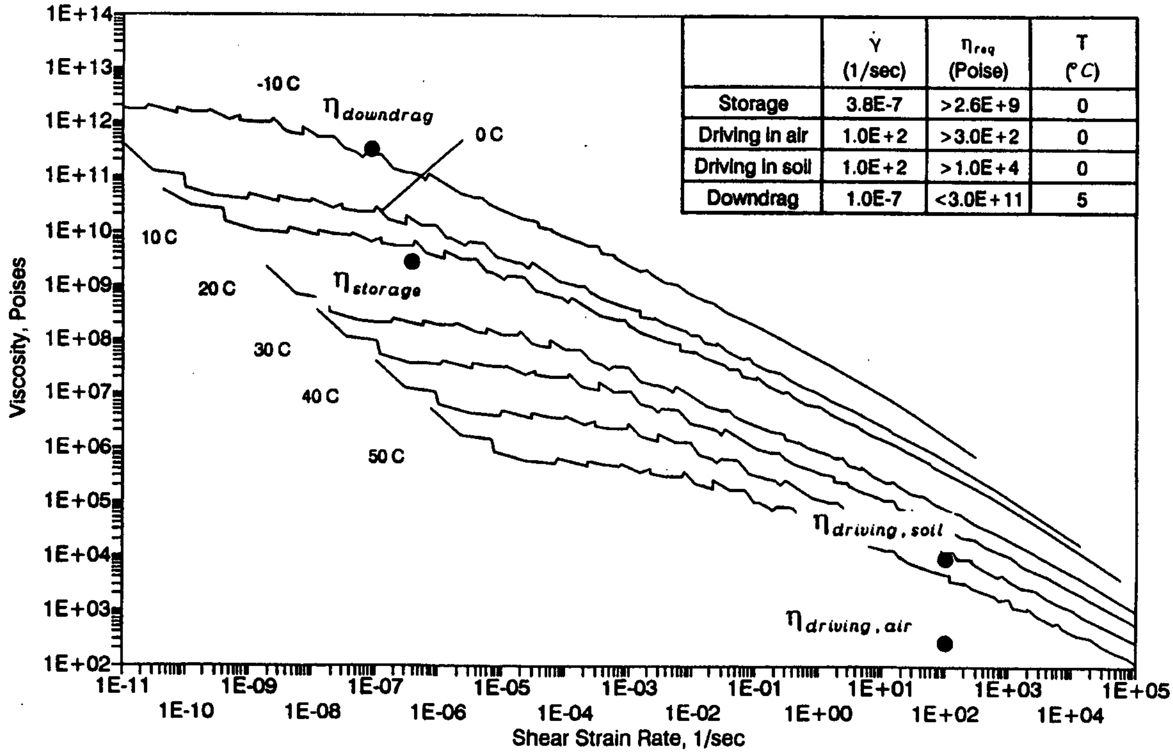


Figure 4.1 Plotting of required viscosity on master curve for softbearing pile lubricants.

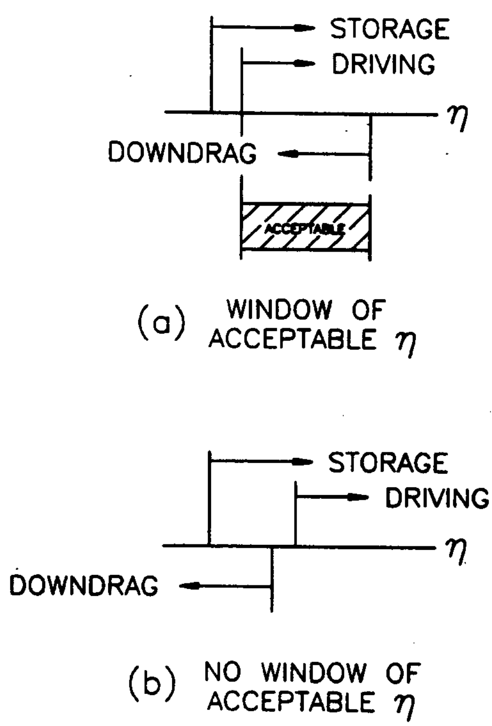


Figure 4.2. Required viscosities forming a window of acceptable viscosity.

mens. It is still possible to find an adequate bitumen if the shear strain rates are sufficiently different.

4.3 CRITERION FOR STORAGE

Storage is the period of time between the pile coating process and pile driving. During this time period the bitumen deforms, and if this period is too long, the bitumen may deform excessively or even drip off the pile. This is more critical in warm or hot weather. The shear strain rate to which the bitumen is subjected during the storage period is given by (Figure 4.3)

$$\dot{\gamma} = \frac{\gamma}{t} = \frac{h/d}{t} \tag{4.1}$$

where:

- $\dot{\gamma}$ = shear strain rate,
- γ = shear strain in the bitumen,
- d = bitumen-coating thickness (Section 4.6),
- h = allowable bitumen flow distance (usually taken as equal to d), and
- t = maximum anticipated storage period. Note that a longer anticipated storage period leads to a more conservative estimate of the required viscosity.

Figure 4.3 illustrates this relationship. The shear stress, τ, in the bitumen is due to gravity forces and can be determined as

$$\tau = \frac{F}{A} = \frac{m \cdot g}{A} = \frac{\rho \cdot V \cdot g}{A} \tag{4.2}$$

$$= \frac{\rho \cdot d \cdot P \cdot dz \cdot g}{P \cdot dz} = \rho g \cdot d$$

where:

- F = weight of the bitumen in element considered,
- A = pile contact area of element considered,
- m = mass of bitumen in element considered,
- ρg = unit weight of the bitumen (approximately 10 kN/m^3),
- ρ = mass density of the bitumen (approximately 1,000 kg/m^3),
- g = acceleration due to gravity (9.81 m/s^2),
- V = volume of the bitumen in element considered,
- P = perimeter of the pile, and
- dz = length of pile element.

Knowing the shear strain rate and the shear stress, the required viscosity can be determined by using the viscous model for the bitumen ($\tau = \eta\dot{\gamma}$):

$$\eta_{req} = \frac{\tau}{\dot{\gamma}} = \frac{\rho \cdot g \cdot t \cdot d^2}{h} \tag{4.3}$$

This viscosity is the viscosity required for the bitumen to flow down a distance h (Figure 4.3) in a time t at a temperature equal to the storage temperature and at the calculated $\dot{\gamma}$. If the coated piles are stored in direct sunlight, the storage temperature can be taken as the air temperature plus 10°C. If the piles are shaded, the storage temperature is equal to the air temperature. The air temperature can be obtained by calling the closest weather station (National Weather Service, NOAA, radio, or television). If the bitumen must not, by design, flow down more than this allowable distance h (usually taken as equal to the bitumen thickness d), then $\eta_{storage}$ for the design bitumen (at the storage temperature and the calculated shear strain rate $\dot{\gamma}$) must be larger than η_{req} above.

$$\eta_{storage} \geq \frac{\rho \cdot g \cdot t \cdot d^2}{h} \tag{4.4}$$

4.4 CRITERION FOR DRIVING

During driving the bitumen must remain on the pile. Initially, the pile is positioned straight up in the air. During the hammer blow, the bitumen must resist the inertia force generated by its own mass and the deceleration during the blow. After the bitumen-coated length of the pile is in the soil, the bitumen must resist the shearing forces of the soil during the remaining part of the driving.

Another consideration is cold weather driving when the bitumen can become brittle and spall off the pile. Therefore there are three considerations:

1. Pile in the air,
2. Pile in the soil, and
3. Cold weather driving.

The first case is when the driving has just begun and the bitumen-coated region is still in the air (Figure 4.4a). The shear stress in the bitumen while in the air, τ_a, can be calculated as

$$\tau_a = \frac{F_a}{A} = \frac{m \cdot a}{A} = \frac{\rho \cdot d \cdot P \cdot dz \cdot a}{P \cdot dz} \tag{4.5}$$

$$= \rho \cdot d \cdot a = \rho \cdot d \frac{v_a}{t_v}$$

where:

- F_a = inertial force on the bitumen,
- a = acceleration of the bitumen,
- v_a = velocity of the bitumen in the air (taken as the velocity of the hammer at the time of impact; this velocity can vary from 0.3 to 6 m/sec), and

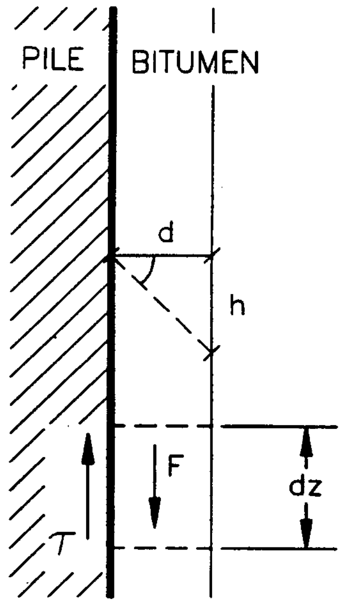


Figure 4.3 Definition of terms for shear strain calculation.

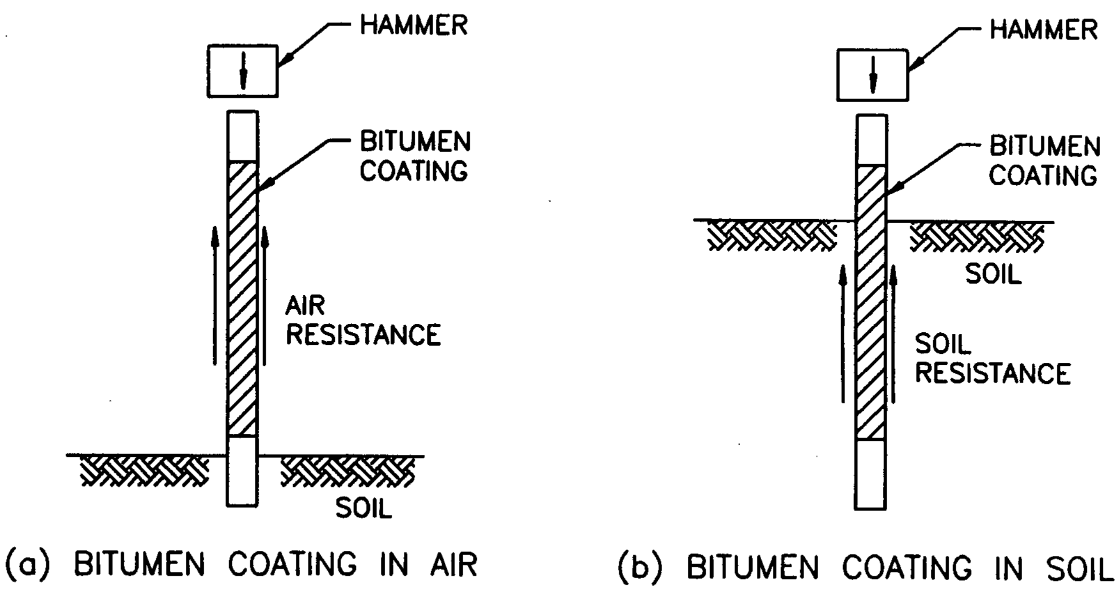


Figure 4.4. Two cases for design during driving.

t_v = time of hammer impact (this time can vary from 1 to 20 milliseconds).

The shear strain rate in the air is calculated as

$$\dot{\gamma}_a = \frac{h/d}{t_v} \tag{4.6}$$

Again it is generally accepted that h can be as large as d. Therefore, to prevent excessive shearing (sagging) during driving, the viscosity required of the bitumen is

$$\eta \geq \frac{\tau_a}{\dot{\gamma}_a} = \frac{\rho \cdot v_a \cdot d^2}{h} \tag{4.7}$$

If the bitumen must not, by design, flow more than an allowable distance h (usually taken as equal to the bitumen thickness d) then $\eta_{driving}$ for the design bitumen must be larger than η_{req} above:

$$\eta_{driving} \geq \frac{\rho \cdot v_a \cdot d^2}{h} \tag{4.8}$$

This requirement on η is associated with the storage temperature (Section 4.3) and the calculated shear strain rate $\dot{\gamma}$.

The second case considered during driving is when the bitumen-coated region has penetrated the soil (Figure 4.4b). The bitumen must resist being sheared (scraped off) by the soil. Using the same type of calculations, the required viscosity is

$$\eta_{req} = \frac{\tau_s}{\dot{\gamma}} = \frac{\tau_s \cdot t \cdot d}{h} \tag{4.9}$$

where: τ_s = maximum friction f_{max} obtained from Figure 2.5 for clays (undrained behavior) and from Figure 2.7 for sands, and
t = time of hammer impact (this time can vary from 1 to 20 milliseconds).

If the bitumen must not, by design, flow more than an allowable distance h (usually taken as equal to the bitumen thickness d) then $\eta_{driving}$ for the design bitumen must be larger than η_{req} above:

$$\eta_{driving} = \frac{\tau_s \cdot t \cdot d}{h} \tag{4.10}$$

This requirement on η is associated with the storage temperature (Section 4.3) and the calculated shear strain rate $\dot{\gamma}$.

The velocity of the hammer can vary from 0.3 to 4.0 m/sec. As can be seen from the equations, the higher velocity will be critical when the bitumen is in the air (the first case), while the lower velocity is more critical when the bitumen is in the soil (the second case).

In the case of cold weather driving, the possibility of the bitumen becoming brittle and spalling off the pile should be considered. In general, the stiffer the bitumen, the greater the chance for spalling. Spalling is the worst when the bitumen is struck directly, such as in rough handling in the driving leads, or when the bitumen is not applied to the pile properly with a primer. This will be covered in more detail in Chapter 8. To minimize the chances for spalling, the softest bitumen that meets all the other requirements should be used.

4.5 CRITERION FOR DOWNDRAG REDUCTION

The bitumen must not only stay on the pile during storage and driving, but it must also be able to reduce the anticipated downdrag adequately during the settlement period. Settlement rates generally vary from about 0.01 to 10 m per year. The actual settlement rate (after driving) for each case can be obtained from consolidation theory as follows.

4.5.1 Estimating the Settlement Rate

The following recommendations apply to the common case of an embankment creating consolidation of a soft clay layer. The problem of secondary compression may need to be addressed, especially in the case of peats and organic soils. The following recommendations also do not apply to the cases of seismic densification and collapsing soils where bitumen may not work because of the high settlement rate.

The settlement rate $\dot{s}$ is defined as

$$\dot{s} = \frac{s_2 - s_1}{t_2 - t_1} \tag{4.11}$$

where s_2 and s_1 are the settlements of the ground surface at time t_2 and t_1, respectively, after the placement of the embankment. The maximum settlement rate averaged over 1 month should be used. Because the settlement rate for an embankment is very high at first and then tapers off, the settlement rate should be calculated as the average over the first month after pile driving. Therefore, t_1 is the time elapsed between completion of the embankment and driving of the piles, and t_2 is t_1 plus 1 month. The settlement s_1, is the settlement of the ground surface at the time of pile driving and s_2 is the settlement of the ground surface 1 month later. At the time t_1, the consolidation equation gives

$$t_1 = T_1 \frac{H^2}{c_v} \tag{4.12}$$

where T_I is the time factor, H is the length of the drainage path (Figure 4.5), and c_v is the coefficient of consolidation obtained in consolidation tests. Equation 4.12 allows one to

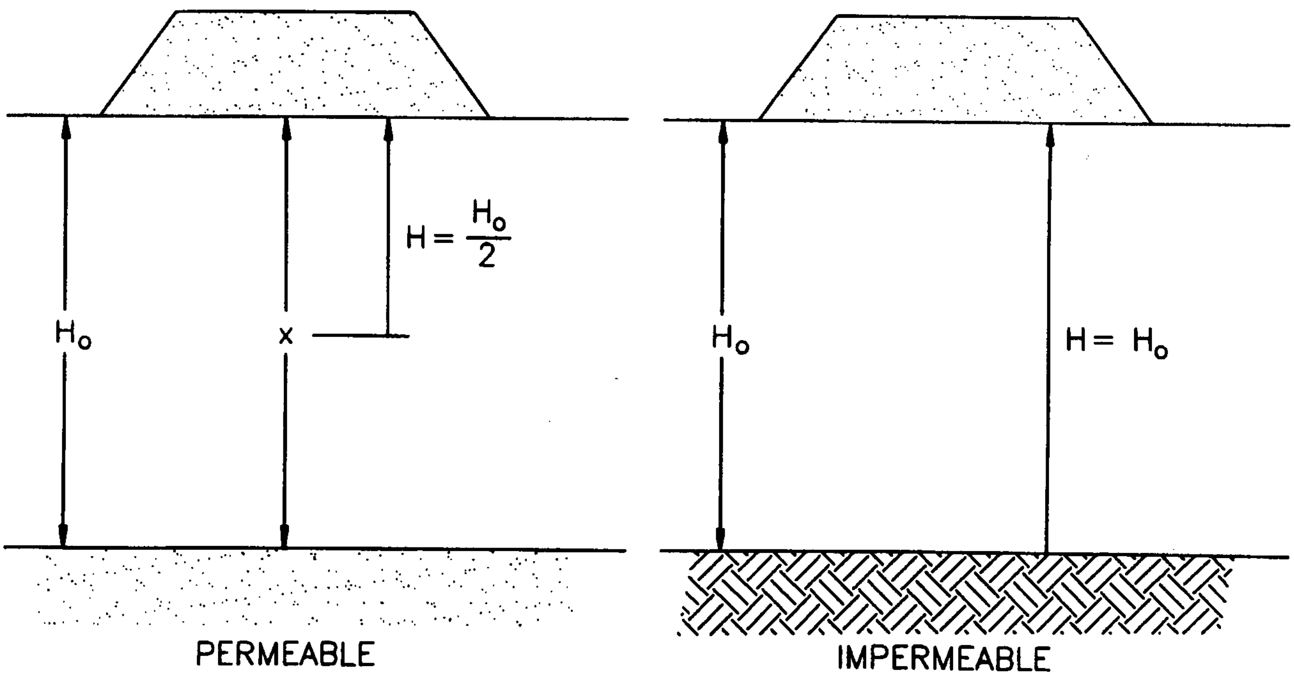


Figure 4.5. Length of drainage path for settlement calculations.

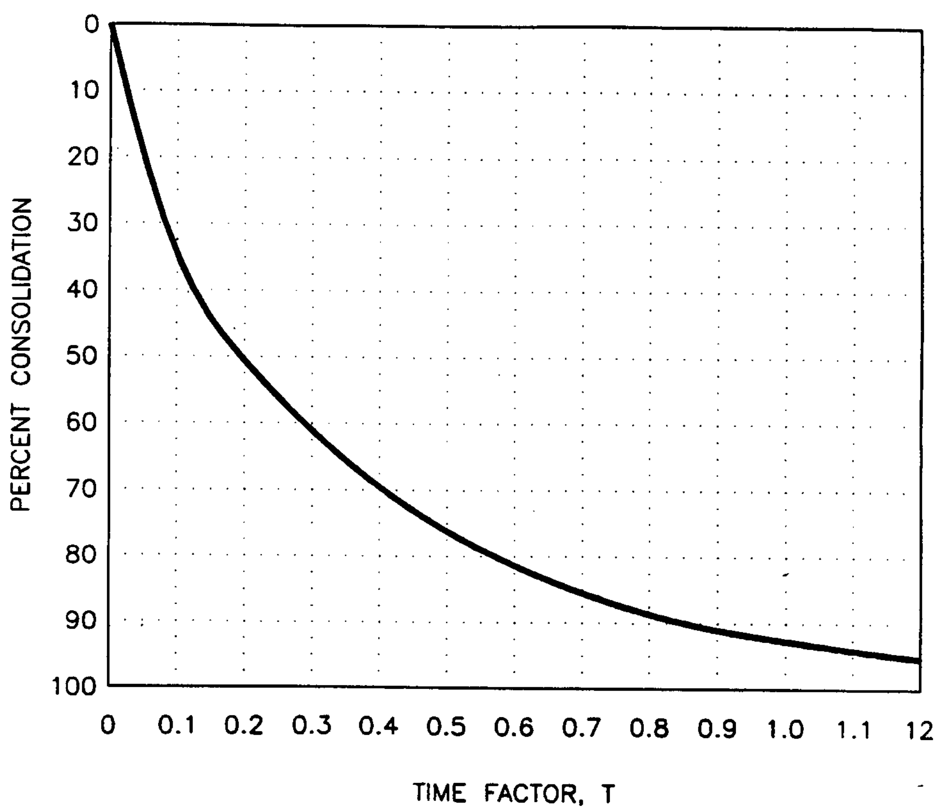


Figure 4.6. Relationship between time factor and percent consolidation.

obtain T_1. Then, this value of T_1 is used to obtain the corresponding percent consolidation U_1, from Figure 4.6. The settlement s_1 is given by

$$s_1 = U \cdot s_{max} \quad (4.13)$$

The settlement s_{max} is calculated using the consolidation theory described in Section 2.1.6.

This process is repeated for the chosen time t_2 and leads to the settlement s_2. The settlement rate is calculated using Eq. 4.11.

In the case of more complex conditions such as wick drains and multilayered profiles, a conservative (high) estimate of the maximum settlement rate averaged over 1 month should be obtained. In obtaining the maximum settlement rate, future dewatering and adjacent ground loading should be anticipated.

4.5.2 Determining the Viscosity

The shear strain rate for settlement, $\dot{\gamma}_s$, is calculated as

$$\dot{\gamma}_s = \frac{\dot{s}}{d} \quad (4.14)$$

Because the settlement rate for an embankment is very high at first and then tapers off, the settlement rate should be calculated as the average over the first month after pile driving as described in Section 4.5. 1. For other cases, such as the lowering of the water table, the maximum rate of settlement averaged over a period of 1 month should be used.

The allowable shear stress, τ_{bit}, is governed by how much negative skin friction the designer is willing to accept. The value of τ_{bit} can be determined by taking a certain percentage of the soil shear strength averaged over the length to be coated or by limiting the maximum load in the pile. Because case histories have shown that bitumen can reduce the friction to about 5% to 30% of the soil shear strength, a value of 10% may be used. In other words

$$\tau_{bit} = 0.1\, \tau_{s,\, aver.} \quad (4.15)$$

Once the settlement rate and allowable shear stress, τ_{bit}, are known, the required viscosity can be calculated:

$$\eta_{req} = \frac{\tau_{bit}}{\dot{\gamma}_s} = \frac{d \cdot \tau_{bit}}{\dot{s}} \quad (4.16)$$

In this case the viscosity of the design bitumen must be smaller than or equal to $\eta_{req.}$

$$\eta_{downdrag} \leq \frac{d\, \tau_{bit}}{\dot{s}} \quad (4.17)$$

This requirement on η is associated with the soil temperature T and the calculated shear strain rate $\dot{\gamma}$.

4.6 CRITERION FOR PARTICLE PENETRATION AND THICKNESS REQUIREMENTS

The problem to be addressed is the penetration of particles into the bitumen coating. These particles are pressed against the coating by the effective horizontal stress σ'_{OH} in the soil. This stress is calculated as follows:

$$\sigma'_{OH} = K \cdot \sigma'_{OV} \quad (4.18)$$

where σ'_{OV} is the effective vertical stress at the depth considered and K is a coefficient of horizontal earth pressure. It is prudent to take a value of K equal to 1 for driven piles and the value of K_O for bored piles as given by

$$K_O = (1 - \sin\phi)\, OCR^{0.5} \quad (4.19)$$

where OCR is the overconsolidation ratio and ϕ is the effective stress friction angle.

The critical time for the thickness of the bitumen coating is at the time of maximum rate of settlement. After primary compression, the rate of settlement is that of secondary compression, which continues to decrease steadily with time. At low rates of settlement, a very thin layer of bitumen is enough to reduce downdrag. Therefore, the criterion is to allow full penetration of the bitumen-coating at the end of life of the structure. Figure 4.7 depicts an acceptable bitumen for a particle penetration of 10-mm in 50 years at a temperature of 20°C. If the chosen bitumen falls in the questionable area between the two curves, a better prediction of the acceptability may be obtained from the following simple test called the particle penetration test.

Heat the bitumen sample to approximately 135°C and pour it into two direct shear molds to form a layer approximately 10-mm thick in each mold. Allow the bitumen to cure for 1 day. All tests after that are performed at room temperature (~20°C). In a third mold, prepare a 10-mm-thick layer of the soil encountered at the site concerned. Also form a 10-mm-thick layer of this soil in one of the direct shear molds containing a bitumen sample (Figure 4.8). Apply a normal stress to all three samples equal to the maximum horizontal effective stress expected on the pile coating and monitor the vertical displacement for 1 week. The net particle penetration is obtained by subtracting the values of displacement for the reference samples (soil and bitumen alone) from the values of displacement for the composite sample. The test is run at room temperature (~20°C) unless the maximum soil temperature expected in the field is higher than room temperature. If the soil temperature is lower than room temperature, the penetration obtained will be a conservative estimate.

This procedure leads to a penetration versus time curve at a given temperature over a period of 1 week (Figure 4.8). Then it is necessary to extrapolate that curve to the end of the life of the structure in order to check the criterion. This is done by fitting the following model to the 1 week curve:

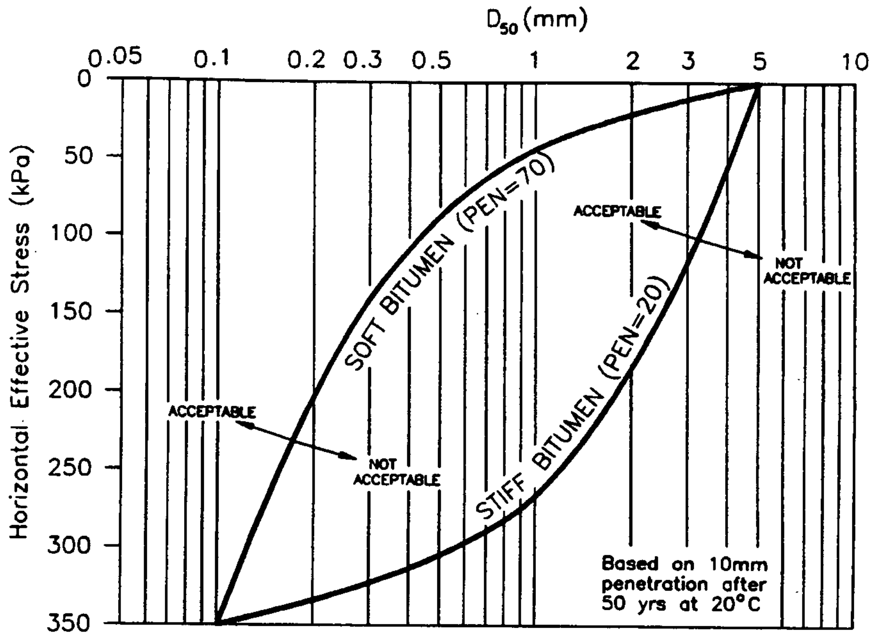


Figure 4.7. Guidelines for bitumen acceptability on the basis of particle penetration.

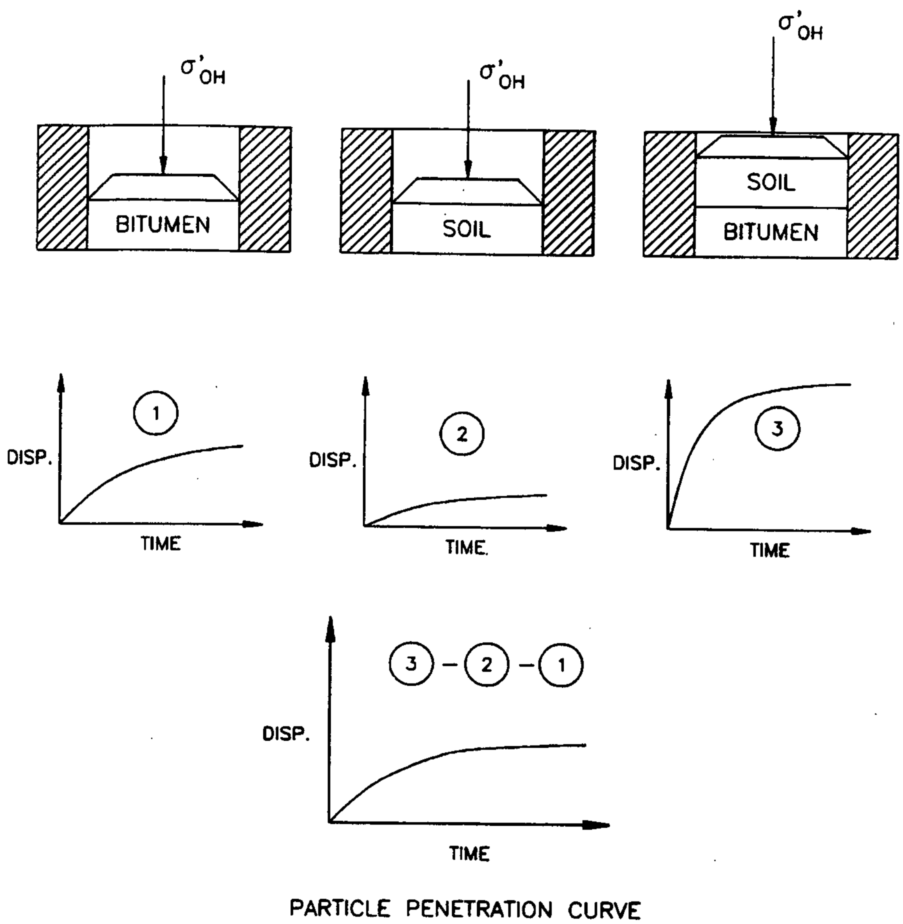


Figure 4.8. Particle penetration test.

$$D_2 = D_1 \left(\frac{t_2}{t_1}\right)^n \quad (4.20)$$

where D_2 and D_1 are the penetrations at times t_2 and t_1, respectively, on the particle penetration curve. The exponent n is back calculated by

$$n = \frac{\log (D_2 / D_1)}{\log (t_2 / t_1)} \quad (4.21)$$

Select $t_2 = 10{,}000$ min and $t_1 = 1{,}000$ min, then n is simply equal to $\log (D_2 / D_1)$. Assuming a life of 50 years, the final penetration can be calculated by

$$D_{50\ \mathrm{years}} = D_{10{,}000\ \mathrm{min}} \left(\frac{50\ \mathrm{years}}{10{,}000\ \mathrm{min}}\right)^n \quad (4.22)$$

$$\text{or} \quad D_{50\ \mathrm{years}} = D_{10{,}000\ \mathrm{min}} \times 2628^n$$

The values of n range widely but are typically between 0.05 and 1.0.

The bitumen is acceptable if the penetration at the end of the life of the structure is less than the bitumen-coating thickness. The bitumen thickness is normally taken to be 10 mm; however, thicknesses as small as 2 mm have been used successfully when particle penetration is not a problem.

Note that particle penetration is not a problem in clays with a maximum particle size less than 0.1 mm. For gravels, it is unlikely that any bitumen can resist penetration to an acceptable degree. In this case, it becomes necessary to predrill and case through the gravel layer to ensure no contact between the gravel and the bitumen. For intermediate soil particles (sands and silts), it is necessary to perform the tests described above and find a bitumen which satisfies the criterion.

4.7 HOW TO SELECT A BITUMEN

Sections 4.1 through 4.6 are design steps used to determine what is required of the bitumen. The engineer must select a bitumen and determine if this bitumen satisfies the design criteria established for storage (Section 4.3), for driving (Section 4.4), for downdrag (Section 4.5), and for particle penetration (Section 4.6).

If previous experience exists, then the previously used bitumen can be checked against the required viscosities. In order to select a bitumen in areas or climates where there is no previous experience, the first step is to calculate the required viscosities for storage, driving, and settlement as outlined previously. The second step is to establish the temperatures corresponding to each required viscosity. The soil temperature below a depth of about 2 m can be taken as equal to the mean air temperature during the year. The upper 2 m are influenced by the daily air temperature. Approximate soil temperatures can be obtained from Figure 4.9. The air temperature can also be obtained from a local weather station (National Weather Service, NOAA, radio, or television).

Then, the engineer should call a bitumen manufacturer or supplier and describe the problem, mentioning that roofing asphalts, culvert compounds, and canal liners with penetration values greater than 50 have been used successfully. Some indication of the bitumen type can be obtained from Figure 3.5 and Tables 3.3 through 3.5. The engineer can give the required viscosities (with the corresponding temperatures and strain rates) to the manufacturer and ask for the viscosity-strain rate master curves for two or three possible bitumens. It is probable that these will not be available and will have to be generated by either the manufacturer or the engineer. In either case, a rheometer will be necessary (Section 3.3.2). This process has to be followed only once because the chosen bitumen will be applicable in that area at all times. The engineer's organization may wish to find a bitumen for summer work and for winter work because of the variation in air temperature. Note that the soil temperature is relatively constant and equal to the mean air temperature.

The procedure can be summarized in the following steps:

1. Estimate the required viscosities for storage, driving, and downdrag.
2. Establish the temperatures for storage, driving, and downdrag.
3. Establish the shear strain rate for storage, driving, and downdrag.
4. Obtain a candidate bitumen.
5. Obtain master curves for this candidate bitumen.
6. Plot the various viscosities, shear strain rates, and temperatures on the master curves (Figure 4.1).
7. Check that the viscosity for the candidate bitumen at each shear strain rate and temperature satisfies the required viscosity criterion.
8. If step 7 is not satisfied, go back to step 4.
9. If step 7 is satisfied, perform particle penetration test.
10. If step 9 is not satisfied, go back to step 4.
11. If step 9 is satisfied, the candidate bitumen is acceptable.

It is possible, especially in hot climates, that a bitumen cannot be found that satisfies all criteria. In this case, it may be necessary to change the job timing from summer to winter or, in some other way, reduce the temperatures encountered during storage and driving. This should resolve the conflicting criteria.

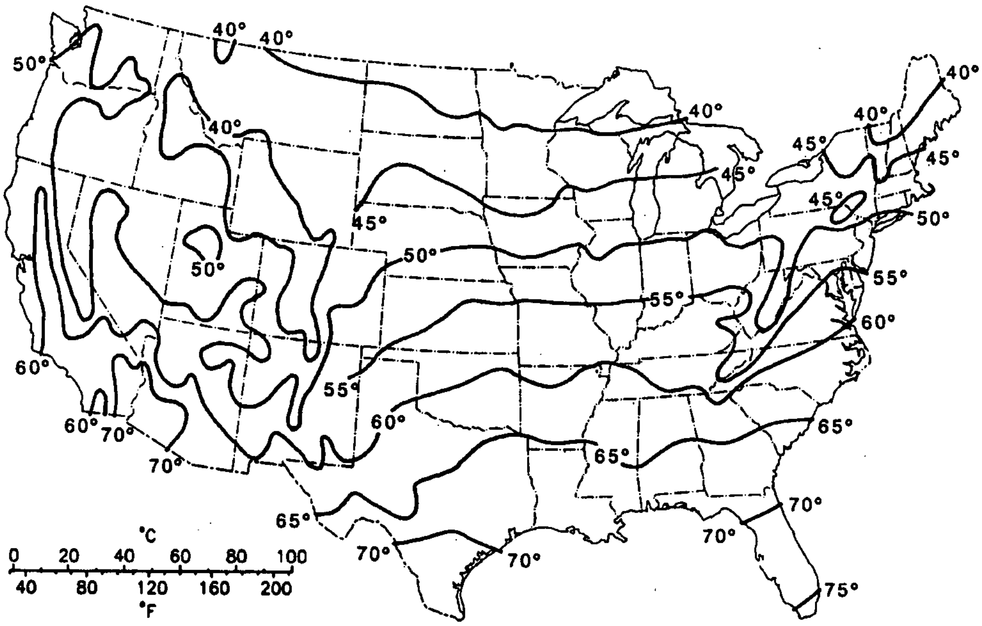


Figure 4.9. Average annual surface air temperature (Fahrenheit). (after Visher 1954)

CHAPTER 5

OVERALL STEP-BY-STEP PROCEDURE

5.1 IS DOWNDRAG A PROBLEM?

This chapter presents a step-by-step design procedure for determining whether bitumen-coated piles should be selected, and how to select the bitumen.

1. Establish soil profile, estimate pile length, estimate pile load.
2. Estimate total ground surface settlement and ground surface settlement after the piles are driven.
   Data Required: Loading, increase in stress, soil deformation properties. See Sections 2.1.6 and 4.5.1
3. Is downdrag a problem?
   Data Required: See Tables 1.2 and 1.3.

5.2 ESTABLISH SOIL AND PILE DATA

4. Establish the settlement profiles down to a depth where the settlement is negligible. These profiles are calculated at the following times:

   a. pile driving,
   b. 1 month after driving,
   c. load application on the piles, and
   d. end of the life of the structure.

   Data Required: Loading, stress increase, soil deformation properties. See Sections 2.1.6 and 4.5.1.
5. Obtain the maximum friction profile.
   Data Required: Soil strength. See Section 2.1.3.
6. Obtain the point bearing capacity, the soil modulus, and and Poisson's ratio under the pile point.
   Data Required: Soil strength, soild deformation test results. See Sections 2.14 and 2.1.5.
7. List the pile properties.
   Data Required: Pile properties.

5.3 ANALYZE THE UNCOATED PILE

8. Analyze the uncoated single pile. Obtain the load-settlement envelope by running PILENEG at the following times:
   a. load application on the piles, and
   b. end of the life of the structure.
   Data Required: Soil settlement profile, maximum friction profile, point load transfer curve. See Section 2.3 and the Appendix.
9. Obtain the settlement s_o at zero load on the curve from step 8a.
10. Establish the allowable load as the load corresponding to a settlement equal to the allowable settlement plus the settlement s_o of step 9 on the curve from 8b. If that load is unacceptably low, increase the embedded pile length and go back to step 4 or coat the pile with bitumen and go to step 14.
11. For the allowable load from step 10, obtain the load distribution in the pile by running PILENEG for that top load.
    Data Required: See Section 2.3 and the Appendix.
12. Check that the safety criteria are satisfied.
    Data Required: See Section 2.4.
13. If the factors of safety are not satisfied, decrease the top load and perform steps 11 and 12 until the factors of safety are satisfied.

Now the uncoated single pile is designed.

5.4 ANALYZE THE COATED PILE

14. Analyze the coated single pile by choosing the percent reduction of f_{max} to be achieved and then inputing the corresponding bitumen shear strength.
    Data Required: See Section 1.4 and Table 1.5.
15. Perform steps 8 through 13 for the coated pile.

5.5 SELECT THE BITUMEN

16. Select the bitumen by establishing the viscosity, temperature, and shear strain rate requirements for the following:

    a. storage,
    Data Required: See Section 4.3.
    b. driving,
    Data Required: See Section 4.4.
    c. downdrag reduction.

Data Required: See Section 4.5.

17. Find the bitumen which satisfies the requirements of step 16.
    Data Required: See Section 4.7.
18. Check the selected bitumen for particle penetration.
    Data Required: See Section 4.6.
19. Consider group effect.
    Data Required: See Section 2.5.

5.6 PERFORM THE ECONOMIC ANALYSIS

20. Perform a cost comparison analysis between the uncoated piles, the coated piles, and any other option to reduce downdrag.
    Data Required: See Section 8.2.
21. Prepare specifications if bitumen is selected.
    Data Required: See Section 8.3.

CHAPTER **6**

EXAMPLE OF SINGLE PILE DESIGN

6.1 EXAMPLE 1: HAND CALCULATION

Given the pile and soil data in Figure 6.1, find an allowable top load for a top settlement of less than 14 mm for both a coated and an uncoated pile.

Uncoated Pile

1. Find the ultimate pile capacity

$$Q_u = (25\ \text{kN/m}^2 \times 1.2\ \text{m} \times 30\ \text{m}) + 1{,}000\ \text{kN}$$
$$Q_u = 1{,}900\ \text{kN}$$

2. Try a top load $Q_t = 500$ kN

If neutral point (*NP*) is at 20 m,
$w_{NP(soil)}$ = 50 mm from settlement profile.

$$Q_p = 500\ \text{kN} + (20\ \text{m})(1.2\ \text{m})(25\ \text{kN/m}^2) - (10\ \text{m})(1.2\ \text{m})(25\ \text{kN/m}^2)$$
$$= 500\ \text{kN} + 600\ \text{kN} - 300\ \text{kN} = 800\ \text{kN}$$

w_p = 4 mm from point load transfer curve

$$w_{NP(pile)} = 4\ \text{mm} + \frac{950\ \text{kN} \times 10\ \text{mm}}{0.09\ \text{m}^2 \times 2E7\ \text{kN/m}^2} \quad \frac{1{,}000\ \text{mm}}{1\ \text{m}}$$
$$= 9.3\ \text{mm}$$

$$w_{NP(pile)} \neq w_{NP(pile)}$$

If neutral point (NP) is at 29 m,
$w_{NP(soil)}$ = 5 mm from settlement profile.

$$Q_p = 500\ \text{kN} + 870\ \text{kN} - 30\ \text{kN} = 1{,}340\ \text{kN}$$

This is not possible. Maximum point load is 1,000 kN.

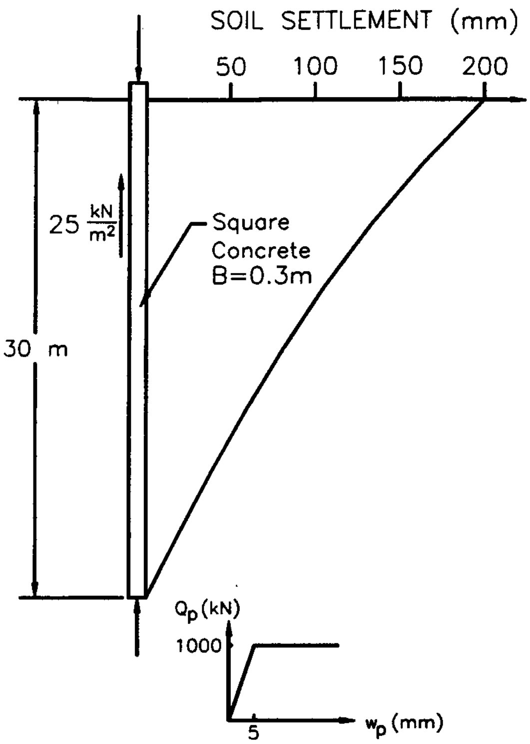
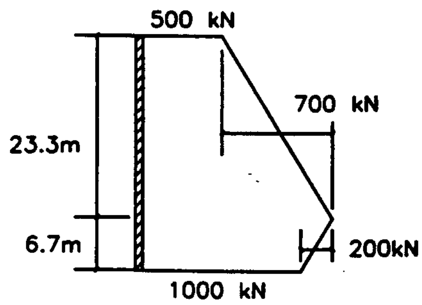


Figure 6.1. Hand calculation example.

Find the maximum depth to NP (Q_p = 1,000 kN) that satisfies equilibrium:

If Q_p = 1,000 kN, $500 + X = 1{,}000 + (900 - X)$

X = 700 kN or 23.3 m of friction



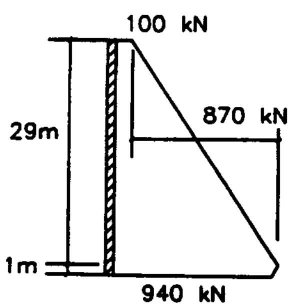
If neutral point (*NP*) is at 23.3 m,
$w_{NP(soil)}$ = 35 mm from settlement profile.

$$w_{top} = 35\text{ mm} + \frac{850\text{ kN} \times 23.3\text{ m}}{0.09\text{ m}^2 \times 2E7\text{ kN/m}^2}\,\frac{1{,}000\text{ mm}}{1\text{ m}}$$
$$= 46\text{ mm}$$

This is more than the allowable settlement.

3. Try a top load Q_t = 100 kN

If neutral point (*NP*) is at 25m,
$w_{NP(soil)}$ = 25 mm from settlement profile. This is already larger than the allowable top settlement.



If neutral point (*NP*) is at 29 m,
$w_{NP(soil)}$ = 5 mm

Q_p = 100 kN + 870 kN – 30 kN = 940 kN
w_p = 4.7 mm

$$w_{NP(pile)} = 4.7\text{ mm} + \frac{955\text{ kN} \times 1\text{ m}}{0.09\text{ m}^2 \times 2E7\text{ kN/m}^2}\,\frac{1{,}000\text{ mm}}{1\text{ m}}$$
$$= 5.2\text{ mm} \approx w_{NP(soil)}$$

$$w_{top} = 5\text{ mm} + \frac{535\text{ kN} \times 29\text{ m}}{0.09\text{ m}^2 \times 2E7\text{ kN/m}^2}\,\frac{1{,}000\text{ mm}}{1\text{ m}}$$
$$= 13.6\text{ mm} \qquad \textbf{OK.}$$

This pile passes the settlement criterion. However, checking the soil failure criterion (Table 2.8), assuming that the top load is half dead load and half live load and that the soil capacities are from load tests, leads to

1.4(50 kN) + 1.7(50 kN) + 1.7(870 kN) < 0.9 (1,900 – 870)
1,634 kN < 927 kN **Not true.**

Therefore, this pile is not safe against soil failure.

Coated Pile

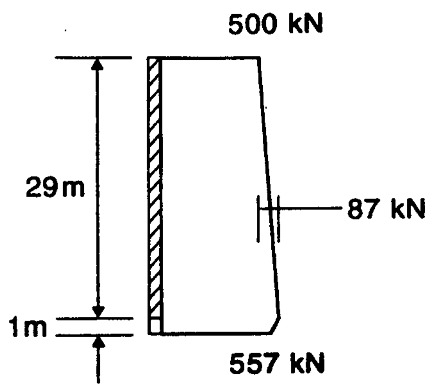
1. Use bitumen coating with shear strength of 2.5 kN/m_2.
2. Try a top load Q_t = 500 kN

If neutral point (*NP*) is at 29 m.

$w_{NP(soil)}$ = 5 mm from settlement profile.
Q_p = 500 kN + 87 kN – 30 kN
= 557 kN
w_p = 2.8 mm

$$w_{top} = 5\text{ mm} + \frac{543.5\text{ kN} \times 29\text{ m}}{0.09\text{ m}^2 \times 2E7\text{ kN/m}^2}\,\frac{1{,}000\text{ mm}}{1\text{ m}}$$
$$= 13.8\text{ mm} \qquad \textbf{OK.}$$

Checking this pile for safety against soil failure (Table 2.8) shows that if the soil capacities are obtained from load tests the pile is safe. However, if the capacities are from static prediction methods, the pile does not meet the safety criterion.



Load Test
1.4(250 kN) + 1.7(25 kN)
+ 1.7(87 kN) < 0.9 (1,117 – 87)
923 kN < 927 kN **OK.**

Static Method
1.4(250 kN) + 1.7(250 kN)

$$+ 1.7(87 \text{ kN}) < 0.75\ (1{,}000 + 30)$$
$$923 \text{ kN} < 773 \text{ kN}$$

Not true.

6.2 EXAMPLE 2: USE OF COMPUTER PROGRAM

The following example will follow the overall step-by-step procedure outlined in Chapter 6, including the numbering of the steps. Example calculations of soil settlement, soil friction, and bearing capacity are shown. Calculations of downdrag are performed using the computer program, PILENEG, from the Appendix.

6.2.1 Is Downdrag a Problem?

Step 1. Establish soil profile. estimate pile load, and estimate pile length.

The soil profile is shown in Figure 6.2. The pile should carry a dead load of 400 kN, a permanent live load of 100 kN, and a transient live load of 250 kN. Because the soft clay layer that may cause downdrag extends to a depth of 22 m, the pile will need to bear in the dense sand layer. The pile will be a 0.4-m-square prestressed, precast concrete pile and will probably need to be at least 30 m long.

Step 2. Estimate total ground surface settlement and around surface settlement after the piles are driven.

The results of a representative consolidation test on a sample from the clay layer are shown in Figure 6.3.

The stress induced at the original ground surface.by the embankment will be

$$6 \text{ m} \times 16.8 \text{ kN/m}^3 = 100.8 \text{ kN/m}^2$$

Knowing this increase in stress at the ground surface and that the embankment is 20 m wide, the average increase in vertical stress in the clay layer under the center of the embankment can be calculated using Figure 2.10. The ratio of depth, Z, at the middle of the clay layer (8 m below the bottom of the embankment) to embankment width, B, is

$$\frac{Z}{B} = \frac{8 \text{ m}}{20 \text{ m}} = 0.4$$

Entering this value into Figure 2.10 gives an increase in stress of about 85% of the stress increase at the ground surface or

$$0.85 \times 100.8 \text{ kN/m}^2 = 85.7 \text{ kN/m}^2$$

The vertical effective stress at the middle of the clay layer before placement of the embankment, can be calculated knowing the soil unit weights and the location of the water table (Figure 6.2).

$$\sigma'_{ov} = (8 \text{ m} \cdot 14.8 \text{ kN/m}^2) - (4 \text{ m} \cdot 9.8 \text{ kN/m}^2)$$
$$= 79.2 \text{ kN/m}^2$$

Entering these values of stress into Figure 6.3, the corresponding strains can be obtained:

Stress (kN/m^2)	Strain (%)
79.2	7.0
164.9	8.6

From this change in strain the total ground surface settlement, s_t, can be calculated:

$$s_t = \varepsilon \cdot H = .016 \times 16 \text{ m} = 0.256 \text{ m}$$

Pile driving is assumed to take place 1 month after placement of the embankment. The percent settlement that takes place before pile driving may be calculated using Eqs. 4.12 and 4.13 with Figure 4.6. The time factor for 1 month is

$$T_{1 \text{ month}} = \frac{t \cdot c_v}{H^2}$$
$$= \frac{2.6E6 \text{ sec} \times 2.0E-2 \text{ cm}^2/\text{sec}}{(800 \text{ cm})^2}$$
$$= .08$$

Entering this time factor into Figure 4.6 gives 30% consolidation at the end of 1 month. This means that about 180 mm (0.7 × 256 mm) of ground surface settlement will occur after pile driving.

Step 3. Is downdrag a problem?

Table 1.2 presents six clues to indicate when downdrag may be a problem. Five of these six are present at this site. First, the total settlement of the ground surface is larger than 100 mm. Second, the settlement of the ground surface after the piles are driven is larger than 10 mm. Third, the height of the embankment exceeds 2 m. Fourth, the thickness of the soft compressible layer is larger than 10 m. Fifth, the piles are estimated to be longer than 30 m. On the basis of these clues, it is evident that downdrag needs to be considered in the pile design.

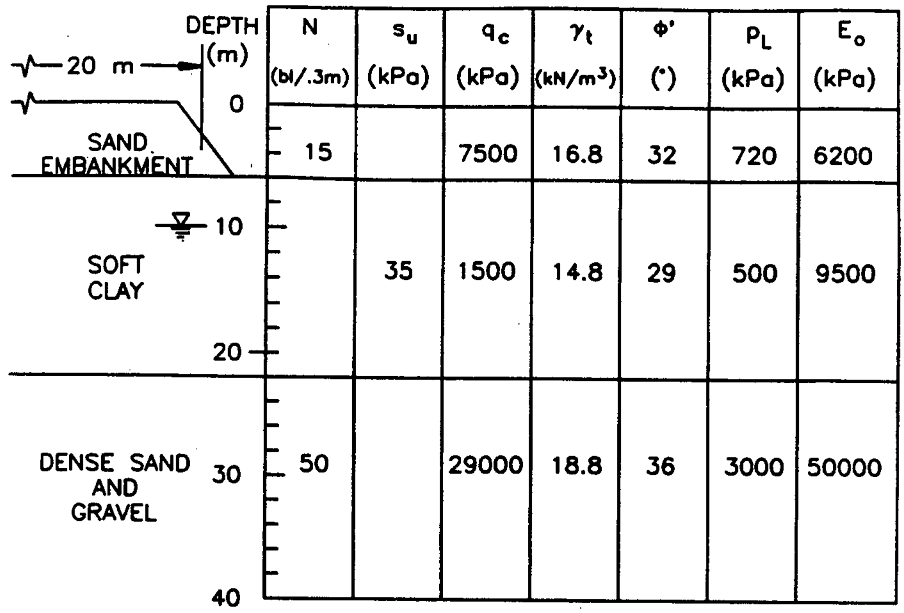


Figure 6.2. Soil profile for example problem.

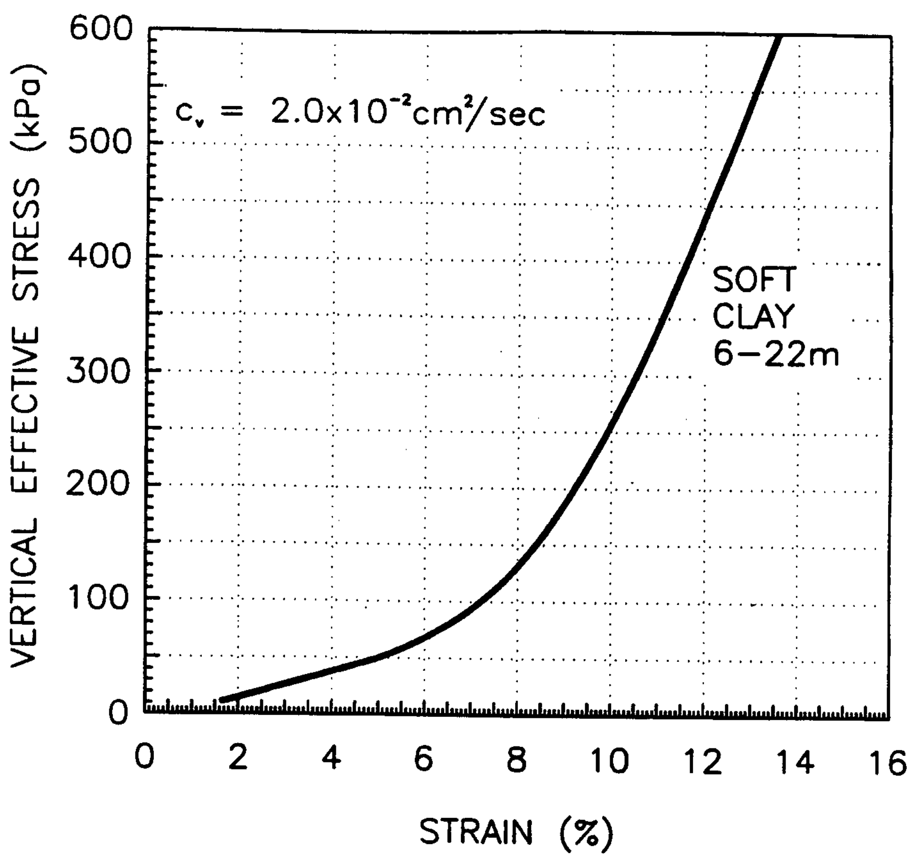


Figure 6.3. Consolidation test results for clay layer.

6.2.2 Establish Soil and Pile Data

Step 4. Establish the settlement profiles down to a depth where the settlement is negligible. These profiles are calculated at the times of

a. pile driving,
b. 1 month after driving,
c. load application on the piles, and
d. end of the life of the structure.

Once it has been determined that downdrag may be a problem, the soil settlement must be calculated in a more detailed way not only to obtain ground surface settlement, but also to obtain the soil-settlement profile at various times. The increase in vertical stress can be calculated as in Section 6.2.1 for several depths. This calculation gives a profile of increase in vertical stress as shown in Figure 6.4. Assuming negligible compression of the sand layers, Table 6.1 shows the calculation of the final settlement profile. The clay layer is divided into a number of thinner sublayers (2 m thick in this case) and the effective vertical stress before the placement of the embankment, σ'_{ov}, at the middle of each sublayer is calculated (Table 6.1, column 3). The increase in vertical stress, $\Delta\sigma_v$, at the middle of each sublayer is obtained from Figure 6.4 (Table 6. 1, column 4). The strain, $\Delta\varepsilon$, is obtained by entering the results of the consolidation tests (Figure 6.3) with the stresses in Table 6.1, columns 3 and 4. The consolidation settlement of each sublayer, Δs, is obtained by multiplying the layer thickness by the average strain in that layer (column 2 × column 5). The total settlement at any level is obtained by summing the settlement of the layers beneath that level.

The total settlement at the ground surface at any given time after placement of the embankment can be obtained by calculating the time factor and entering it into Figure 4.6 to obtain the percent consolidation as done in Section 6.2.1.

TABLE 6.2 Time factors and percent consolidation for various settlement profiles

Time	Time Factor	Percent Consolidation
1 month	.08	30
2 months	.16	46
6 months	.49	75
7 months	.57	80
50 years	48.75	100

The necessary times assumed for this example are the time of pile driving, which is 1 month after placement of the embankment in this example; 1 month after pile driving, that is, at 2 months; the time of load application to the pile, at 6 months in this example; 1 month after load application, 7 months; and end of the life of the structure at, that is, 50 years. The corresponding time factors and percent consolidation are given in Table 6.2. These values apply to the total settlement of the ground surface. Local (depthwise) time factors are very difficult to obtain and are probably an unnecessary refinement of the problem. For this reason, the shape of the intermediate settlement profiles (at 1, 2, and 6 months) are assumed to have the same shape as the final settlement profile.

The amount of settlement occurring between the time of placement of the embankment and the time of pile driving is not experienced by the piles and therefore should be subtracted from all succeeding calculations. The resulting settlement profiles are shown in Figure 6.5.

Step 5. Obtain the maximum friction profile.

The maximum friction can be calculated by the methods presented in Section 2.1.3. The long-term (drained) friction

TABLE 6.1 Calculation of final soil settlement profile

(1) Depth (m)	(2) Layer Thickness (m)	(3) σ'_{ov} (kPa)	(4) $\Delta\sigma_v$ (kPa)	(5) $\Delta\epsilon$ (m/m)	(6) Δs (mm)	(7) s (mm)
0-6	6	—	—	—	0	419
6-8	2	14.8	100.3	0.0550	110	419
8-10	2	44.4	97.5	0.0370	74	309
10-12	2	64.2	95.4	0.0270	54	235
12-14	2	74.2	90.9	0.0235	47	181
14-16	2	84.2	86.0	0.0210	42	134
16-18	2	84.2	78.9	0.0175	35	92
18-20	2	104.2	72.3	0.0150	30	57
20-22	2	114.2	66.3	0.0135	27	27

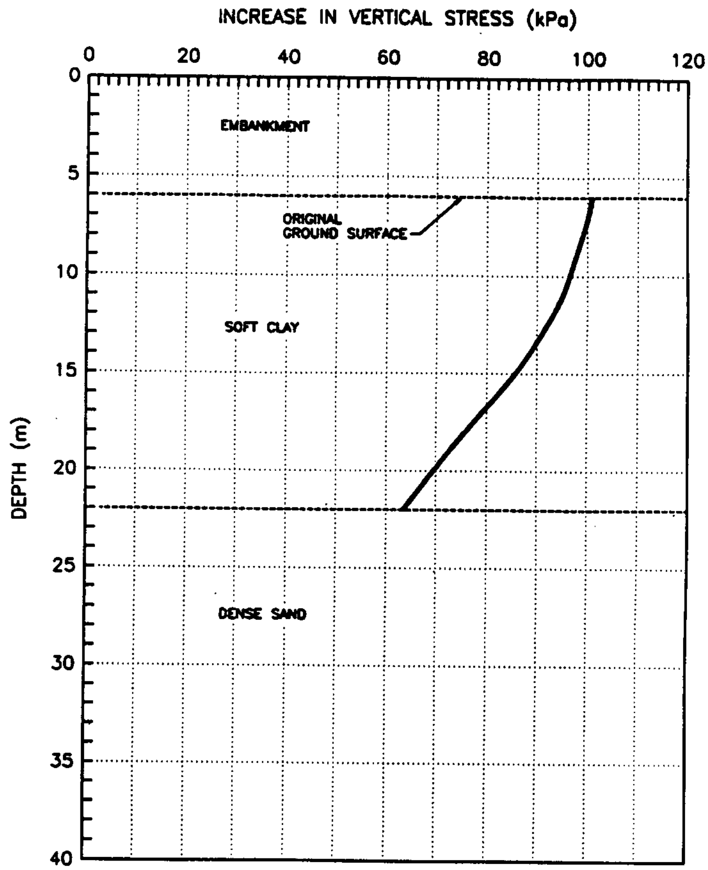


Figure 6.4. Increase in vertical stress in clay layer because of embankment loading.

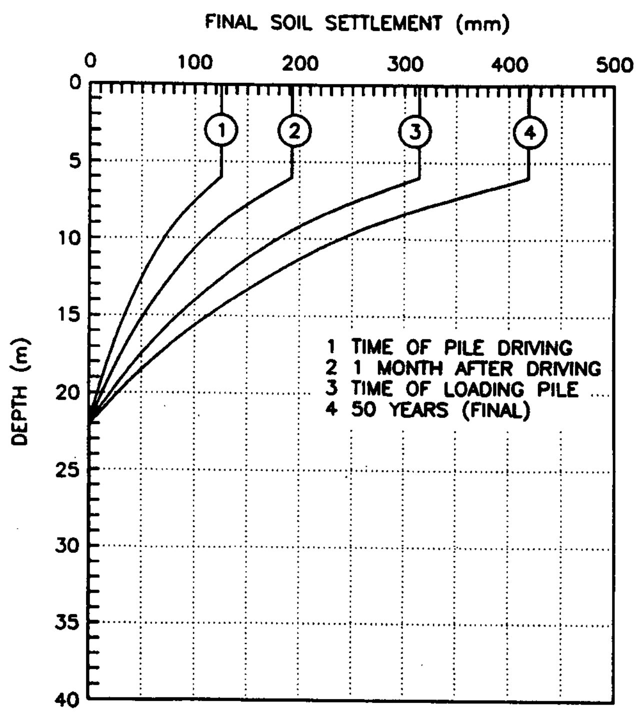


Figure 6.5. Soil settlement profiles at various times.

TABLE 6.3 Maximum friction

Depth (m)	σ'_{ov} (kPa)	β (from Figs. 2.6 and 2.7)	Long Term $f_{max} = \beta \cdot \sigma'_{ov}$ (kPa)	Short Term (from Fig. 2.5) (kPa)
0	0	-	0	0
2	33.6	2	67	67
6	100.8	1	101	101
6.01	100.8	0.25	25	33
22	220	0.25	55	33
22.01	220	0.40	88	88
40	382	0.35	134	134

can be obtained using Eq. 2.6 and Figure 2.6 for clay and Figure 2.7 for sand. In the sand layers, the short-term friction is also drained and is therefore the same as the long-term friction. The short-term friction in the clay layer must be obtained using an undrained method such as Figure 2.5. Entering the undrained shear strength of the clay (35 kPa) into Figure 2.5 gives a maximum friction of 33 kPa. The results are shown in Table 6.3.

Step 6. Obtain the bearing capacity, the soil modulus, and Poisson's ratio under the pile point.

In order to meet the allowable settlement criteria, the pile must be driven into the lower sand layer. The bearing capacity may be calculated using the Standard Penetration Test (SPT) data, the pressuremeter (PMT) data, the cone penetrometer (CPT) data (Section 2.1.4), or any quality method the engineer is confident with in the particular design situation.

SPT: from Eq. 2.12:

$$q_{max} = 1{,}000\sqrt{50} = 7{,}071 \text{ kPa}$$

PMT: from Table 2.1: $k = 3.2$
from Eq. 2.9:

$$q_{max} = (3.2 \times 3{,}000) = 9{,}600 \text{ kPa}$$

CPT: from Table 2.2: $K_c = 0.375$
from Eq. 2.10:

$$q_{max} = 0.375 \times 29{,}000 = 10{,}875 \text{ kPa}$$

For the purpose of this example, an intermediate value of 9,250 kPa is selected for the final bearing capacity. The modulus of the soil at the pile top can be taken as the initial pressuremeter modulus, 50,000 kPa. Poisson's ratio may be taken as 0.33.

Step 7. List the pile properties.

Embedded pile length = 30 m
Pile perimeter = 1.6 m
Cross-sectional area = 0.16 m^2
Pile modulus = 2×10^6 kN/m^2

6.2.3 Analyze the Uncoated Pile

Step 8. Analyze the uncoated single pile. Obtain the load-settlement envelope by running PILENEG at the following times:

a. load application on the piles, and
b. end of the life of the structure.

The results of the PILENEG analysis for steps 8a and 8b for the uncoated pile are shown on Figure 6.6.

Step 9. Obtain the settlement s_o, at zero load on the curve from step 8a.

The settlement for zero load at the time of applying the load to the piles is 13.2 mm.

Step 10. Establish the allowable load as the load corresponding to a settlement equal to the allowable settlement plus the settlement s_o of step 9 on the curve from 8b. If that load is unacceptably low, increase the embedded pile length and go back to step 4 or coat the pile with bitumen and go to step 14.

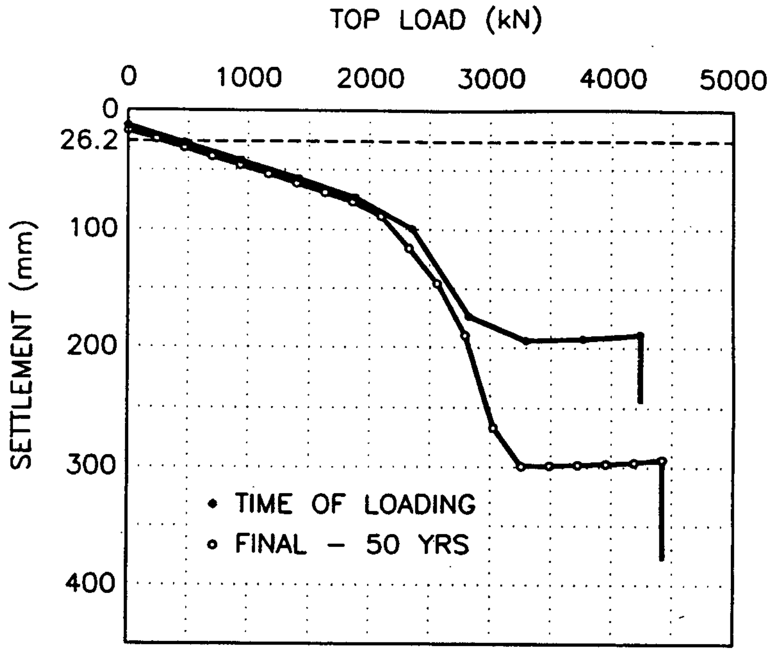
The allowable settlement after application of the load to the piles is 13 mm. Because 13.2 mm of settlement occurs before application of the load to the piles, the allowable load corresponds to a settlement of 26.2 mm on the pile-settlement envelope developed for the end of the life of the structure. This load is 265 kN (Figure 6.6). This load is unacceptably low: coat the pile with bitumen and go to step 14.

6.2.4 Analyze the Coated Pile

Step l4. Analyze the coated single pile by first choosing the percent reduction of f_{max} to be achieved and then inputting the corresponding bitumen shear strength.

The bitumen coating should be able to reduce the shear strength to at least 10 kN/m^2. Try this as a first step.

Step 15. Perform steps 8 through 13 for the coated pile.
Step 8. Analyze the coated single pile. Obtain the load-settlement envelope by running PILENEG at the following times:



Note: These are not load-settlement curves as would be obtained from a load test on the pile. They are envelopes of long-term equilibrium points; each point represents the long-term equilibrium with downdrag occurring after the top load has been placed; whereas in a load test, the downdrag occurs before the top load is placed (see Section 2.3.3). This is the reason for the strange shape of the curve at high loads where the top load finally generates enough settlement to mobilize positive friction in the upper strong fill. This shape would not have occurred if the piles were cased through the fill or coated with bitumen.

Figure 6.6. PILENEG results for uncoated pile.

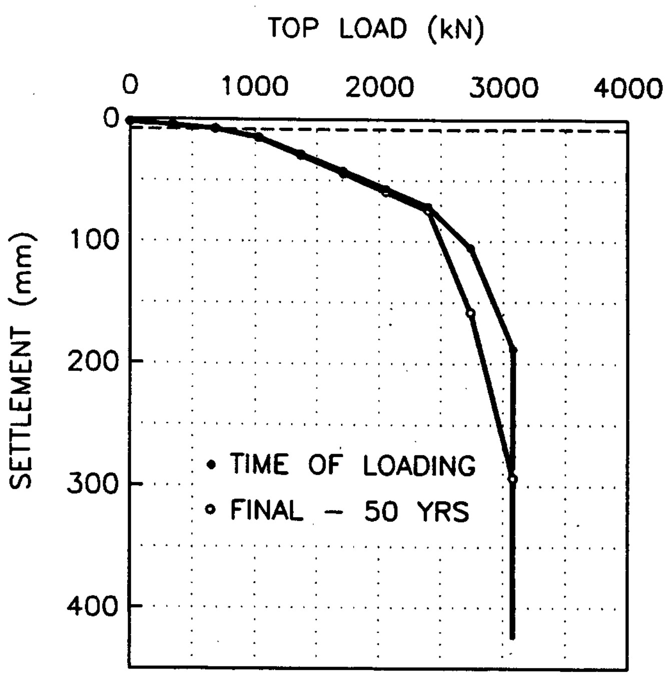


Figure 6.7. PILENEG results for coated pile.

a. load application on the piles and
b. end of the life of the structure.

The results of the PILENEG analysis for steps 8a and 8b for the coated pile are shown in Figure 6.7.

Step 9. Obtain the settlement s_o at zero load on the curve from step 8a. The settlement for zero load at the time of applying the load to the piles is 2.3 mm.

Step 10. Establish the allowable load as the load corresponding to a settlement equal to the allowable settlement plus the settlement s_o of step 9 on the curve from 8b. If that load is unacceptably low, increase the pile length and go back to step 4 or coat the pile with bitumen and go to step 14.

The allowable settlement after application of the load to the piles is 13 mm. Because 2.3 mm of settlement occurs before application of load to the piles, the allowable load corresponds to a settlement of 15.3 mm on the settlement envelope developed for the end of the life of the structure. The load corresponding to this settlement is 1,050 kN (Figure 6.7). This load is acceptable because from step 1 the dead load plus permanent live load equals 500 kN and the transient live load equals 250 kN.

Step 11. For the allowable load from step 10, obtain the load distribution in the pile by running PILENEG for that top load.

The load distributions from a PILENEG analysis for top loads of 500 kN and 750 kN are shown in Figure 6.8.

Step 12. Check that the safety criteria are satisfied.

The safety criteria against structural failure for load and resistance factor design are as follows (Table 2.3):

At the pile top:

$1.4D + 1.7(PL + TL) < 0.75\ Q_{nom}$

$(1.4 \times 400\ kN) + (1.7 \times 350\ kN)$

$< 0.75 \times (0.85 \times 2 \times 10^4\ kPa \times 0.16\ m^2)$

$1{,}155\ kN < 2{,}040\ kN$ **OK.**

At the neutral point:

$1.4D + 1.7(PL + F_n) < 0.75\ Q_{nom}$

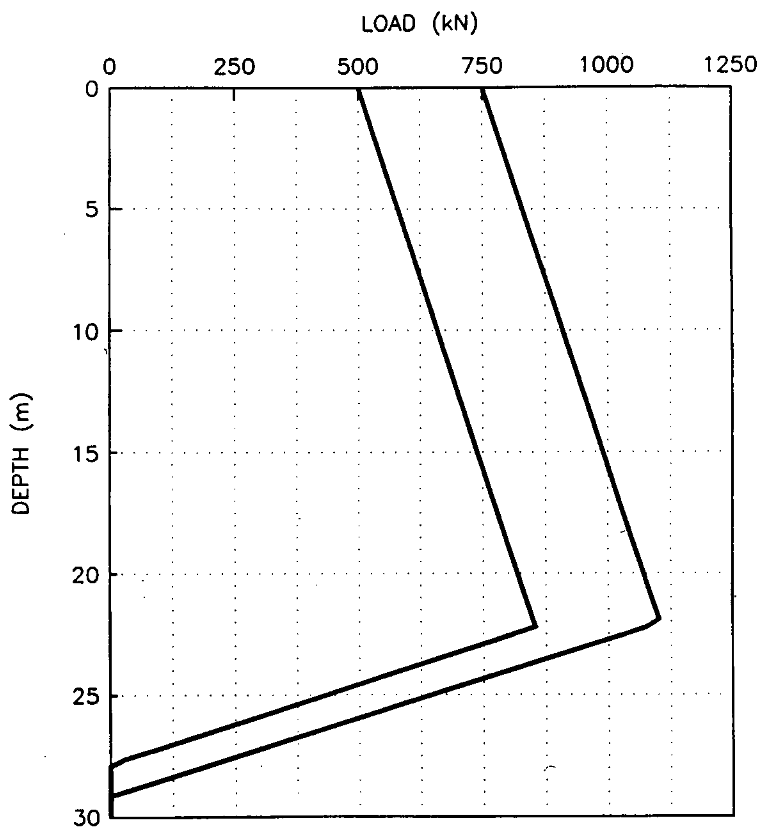


Figure 6.8. Load distribution from PILENEG analysis.

$$(1.4 \times 400 \text{ kN}) + (1.7 \times 452 \text{ kN}) < 0.75 \times (0.85 \times 2 \times 10^4 \text{ kPa} \times 0.16 \text{ m}^2)$$
$$1{,}328 \text{ kN} < 2{,}040 \text{ kN} \qquad \textbf{OK.}$$

The safety criteria against structural failure are satisfied.

The safety criteria against soil failure for load and resistance factor design using a quality static method are as follows (Table 2.8):

At the pile top:
$$1.4D + 1.7(PL + TL) < 0.5\ Q_u$$

At the neutral point:
$$1.4D + 1.7(PL + F_n) < 0.75(Q_{pu} + F_{pu})$$

Using the plunging load of 3,080 kN, the safety criterion at the pile top is as follows:
$$[1.4 \times 400 \text{ kN} + 1.7 \times (100 \text{ kN} + 250 \text{ kN})] < (0.5 \times 3{,}080 \text{ kN})$$
$$1{,}155 \text{ kN} < 1{,}540 \text{ kN} \qquad \textbf{OK.}$$

The criterion for the pile top is satisfied.
At the neutral point, the safety criterion is as follows:

$$[1.4 \times 400 \text{ kN} + 1.7 \times (100 \text{ kN} + 352 \text{ kN})] < .75 \times (1{,}480 \text{ kN} + 1{,}257 \text{ kN})$$
$$1{,}328 \text{ kN} < 2{,}053 \text{ kN} \qquad \textbf{OK.}$$

The criterion for the neutral point is satisfied.

Step 13. If the safety criteria are not satisfied, decrease the top load and perform steps 11 and 12 until the safety criteria are satisfied.

The safety criteria against soil failure and structural failure are satisfied. The allowable settlement criterion is satisfied. Therefore, use a 30-m-long pile coated with bitumen for the top 22 m. Go on to step 16 to select the bitumen.

6.2.5 Select the Bitumen

Step 16. Select the bitumen by first establishing the viscosity, temperature, and shear strain rate requirements for

a. storage,
b. driving,
c. downdrag reduction, and
d. particle penetration.

The viscosities, shear strain rates, and temperatures are to be obtained in this step. Two extreme cases of temperature will be considered. Case 1 will be the extreme low temperatures encountered in the winter in the northern United States. Case 2 will be the extreme high temperatures encountered in the summer in the southern United States.

a. Design for storage (Section 4.3)

The piles are assumed to be stored for 1 month; the coating thickness is assumed to be 10 mm; and the allowable flow distance is assumed to be equal to the coating thickness. This leads to a storage viscosity requirement of Eq. 4.4:

$$\eta_{storage} \geq \frac{(10 \text{ kN/m}^3)(2.6 \times 10^6 \text{sec})(0.01 \text{ m})^2}{0.01\text{m}} = 2.6 \times 10^5 \frac{\text{kN} \cdot \text{sec}}{\text{m}^2}$$
$$\geq 2.6 \times 10^9\ P$$

The associated strain rate is

$$\dot{\gamma} = \frac{1}{2.6 \times 10^6 \text{ sec}} = 3.85 \times 10^{-7} \text{ sec}^{-1}$$

The storage requirement is $\eta_{storage} \geq 2.6 \times 10^9\ P$ for $\dot{\gamma} = 3.85 \times 10^{-7}$ sec^{-1} and for a temperature taken as 0°C for the cold climate of Case 1 and 30°C for the hot climate of Case 2.

b. Design for driving (Section 4.4)

The shear strain rate for driving is a function of the velocity of the hammer Eq. 4.6. Assuming an impact time of 10 milliseconds, the shear strain rate is

$$\dot{\gamma} = \frac{1}{0.01 \text{ sec}} = 100 \text{ sec}^{-1}$$

The required viscosity must be calculated for two cases: one while the pile is in the air and another when the coated portion of the pile is in the soil. While the pile is still in the air, the required viscosity is given by Eq. 4.8:

$$\eta_{driving} \geq \frac{(1{,}000\text{kg/m}^3)(3 \text{ m/sec})(0.01 \text{ m})^2}{0.01 \text{ m}} = 30 \frac{N \cdot \text{sec}}{\text{m}^2}$$
$$\geq 300\ P$$

When the coated portion of the pile is in the soil, the required viscosity is given by Eq. 4.10:

$$\eta_{driving} \geq \frac{(100\text{kg/m}^2)(0.01 \text{ sec})(0.01 \text{ m})^2}{0.01 \text{ m}} = 1 \frac{\text{kN} \cdot \text{sec}}{\text{m}^2}$$
$$\geq 10{,}000\ P \qquad \text{This case governs driving.}$$

The driving requirement is: $\eta_{driving} \geq 1.0 \times 10^4\ P$ for $\dot{\gamma} = 1.0 \times 10^2$ sec^{-1} and for a temperature taken as 0°C for the cold climate of Case 1 and 30°C for the hot climate of Case 2.

c. Design for downdrag reduction (Section 4.5)

The settlement rate during the first month after driving can be determined from the settlement profiles in Figure

6.5. The settlement of the ground surface during this time is 67 mm (from Tables 6.2 and 6.1: (46% – 30%) × 419 = 67 mm). The resulting shear strain rate is Eq. 4.14:

$$\dot{\gamma} = \frac{67\ \text{mm}/2.6 \times 10^{6}\,\text{sec}}{10\ \text{mm}} = 2.58 \times 10^{-6}\ \text{sec}^{-1}$$

The assumed bitumen shear stress for the PILENEG analysis was 10 kN/m². This results in a required viscosity of Eq. 4.17:

$$\eta_{downdrag} \geq \frac{(10\ \text{mm})(10\ \text{kN/m}^2)}{2.58 \times 10^{-5}\ \text{mm/sec}} = 3.88 \times 10^{6}\ \frac{\text{kN} \cdot \text{sec}}{\text{m}^2}$$
$$\leq 3.88 \times 10^{10}\ P$$

The downdrag reduction requirement is $\eta_{downdrag} \leq 3.88 \times 10^{10}\ P$ *for* $\dot{\gamma} = 2.58 \times 10^{-6}\ \text{sec}^{-1}$ and for a soil temperature taken as 5°C for the cold climate of Case 1 and 20°C for the hot climate of Case 2.

Step 17. Find the bitumen which satisfies the requirements of step 16.

The viscosities and shear strain rates obtained in step 16 are independent of temperature. Table 6.4 summarizes the results of step 16 with temperatures from two extreme cases. One extreme would be temperatures encountered in the winter in the northern United States. The other extreme would be temperatures encountered in the summer in the southern United States.

The shear strain rates and viscosities for the four criteria are plotted on the master curves for seven commercially available bitumens in Figures 6.9 through 6.15. By comparing the temperatures for the two cases listed in Table 6.4 with the plotted points on Figures 6.9 through 6.15, it can be seen which bitumens would be applicable to each situation. The results are summarized in Table 6.5. Only bitumens A and B would work in the cold climate in the winter, with bitumen C being borderline on reducing the downdrag. In the hot climate, only bitumens E and F are even borderline for working in the summer. If the work in the hot climate were performed when the storage temperatures were somewhat lower (fall, winter, or spring), bitumens C and D are possible candidates.

These predicted results are consistent with the results obtained in the field. In Edmonton (cold), bitumens A and B reduced the downdrag by 76% and 100%, respectively, while bitumens F and G reduced it only by 46% and 26%, respectively. In New Orleans (hot), bitumen G reduced the downdrag by 0%.

Step 18. Check the selected bitumen for particle penetration.

The particle penetration will be considered for Case 1 (cold climate) using the soft bearing pile lubricant. A particle penetration test as described in Section 4.6 is performed using a sample of the sand from the embankment. Using a *K* value equal to 1, the effective horizontal stress at a depth of 4 m is 67 kPa. This stress is applied in the particle penetration test and the results are shown in Figure 6.16.

The particle penetration at 50 years can be calculated using Eq. 4.22.

$$D_{50\ \text{years}} = D_{10{,}000\ \text{min}} \times 2{,}628^{n}$$

The exponent *n* is obtained from the particle penetration test results by

$$\begin{aligned} n &= \log\left(D_{10{,}000\ \text{min}}/D_{1{,}000\ \text{min}}\right) \\ &= \log\left(0.45\ \text{mm}/0.4\ \text{mm}\right) \\ &= 0.05 \end{aligned}$$

Therefore, the penetration at 50 years is estimated to be

$$\begin{aligned} D_{50\ \text{years}} &= 0.45\ \text{mm} \times 2{,}628^{0.05} \\ &= 0.67\ \text{mm} \end{aligned}$$

Because the estimated penetration at 50 years is less than the design coating thickness (10 mm), the bitumen is acceptable.

Step 19. Consider group effect.

See the example in Chapter 7.

6.2.6 Perform the Economic Analysis

Step 20. Perform a cost comparison analysis between the uncoated piles, the coated piles, and any other option to reduce downdrag.

The cost of the bitumen-coated piles may now be estimated using the design length of pile and coating (see Section 8-2). Other options such as uncoated piles of greater length or casing through the embankment to reduce the downdrag load should also be considered. The most economical solution with the highest probability of success should be chosen.

Step 21. Prepare specifications if bitumen is selected.

If bitumen coating is selected as the most economical option, specifications may be prepared according to the guidelines in Section 8.3.

TABLE 6.4 Summary of coating requirements

	Shear strain rate $\dot{\gamma}$ (1/sec)	Viscosity η_{req} (Poise)	Temperature (°C)	
			Cold	Hot
Storage	3.85E-7	>2.6E+9	0	30
Driving in air	1E+2	>3E+2	0	30
Driving in soil	1E+2	>1E+4	0	30
Downdrag	2.58E-6	<3.88E+10	5	20

TABLE 6.5 Results of bitumen selection process

Bitumen	Cold	Hot
A Soft Bearing Pile Lubricant	OK	No (storage)
B Bearing Pile Lubricant	OK	No (storage)
C Husky Oil type 1	Borderline (Downdrag)	No (storage)
D Husky Oil type 2	No (downdrag)	Borderline (storage)
E Trumbull type 1	No (downdrag)	Borderline (storage)
F Culvert compound	No (downdrag)	Borderline (downdrag)
G Intec Blue (Poly mod)	No (downdrag)	No (downdrag)

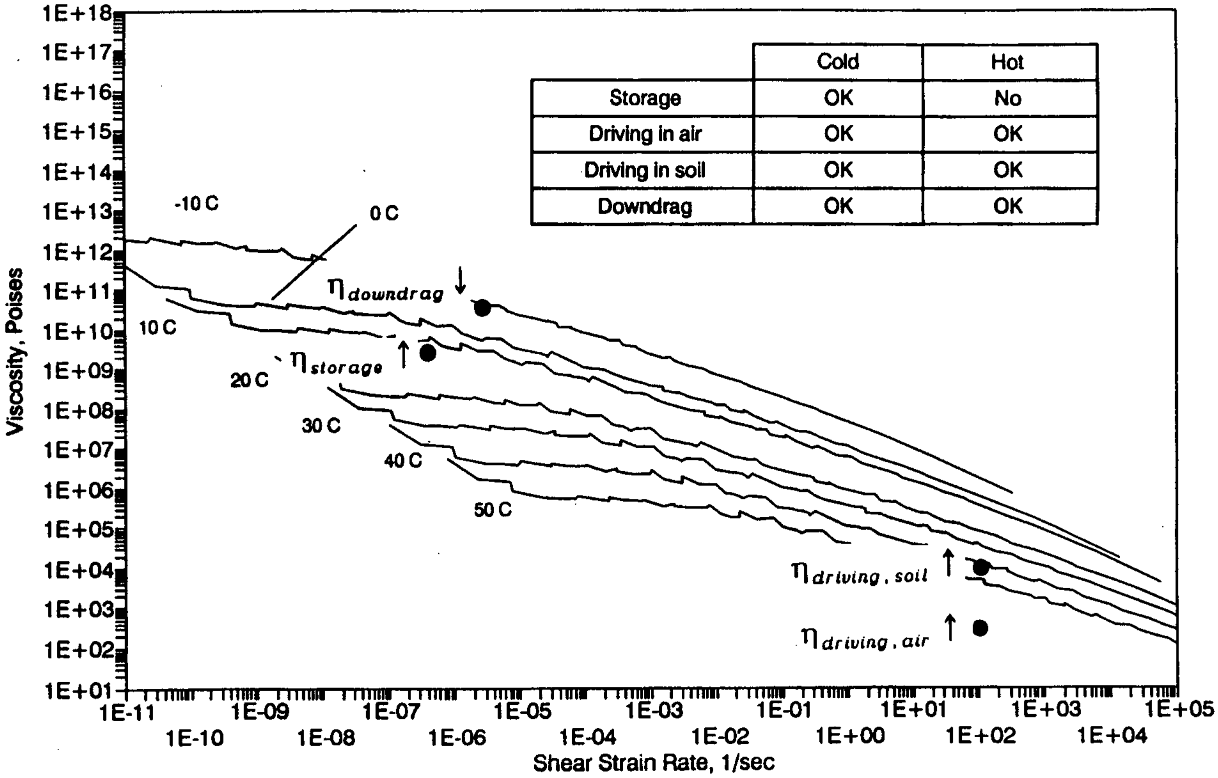


Figure 6.9. Bitumen viscosity and shear strain rate requirements plotted on master curve for softbearing pile lubricant.

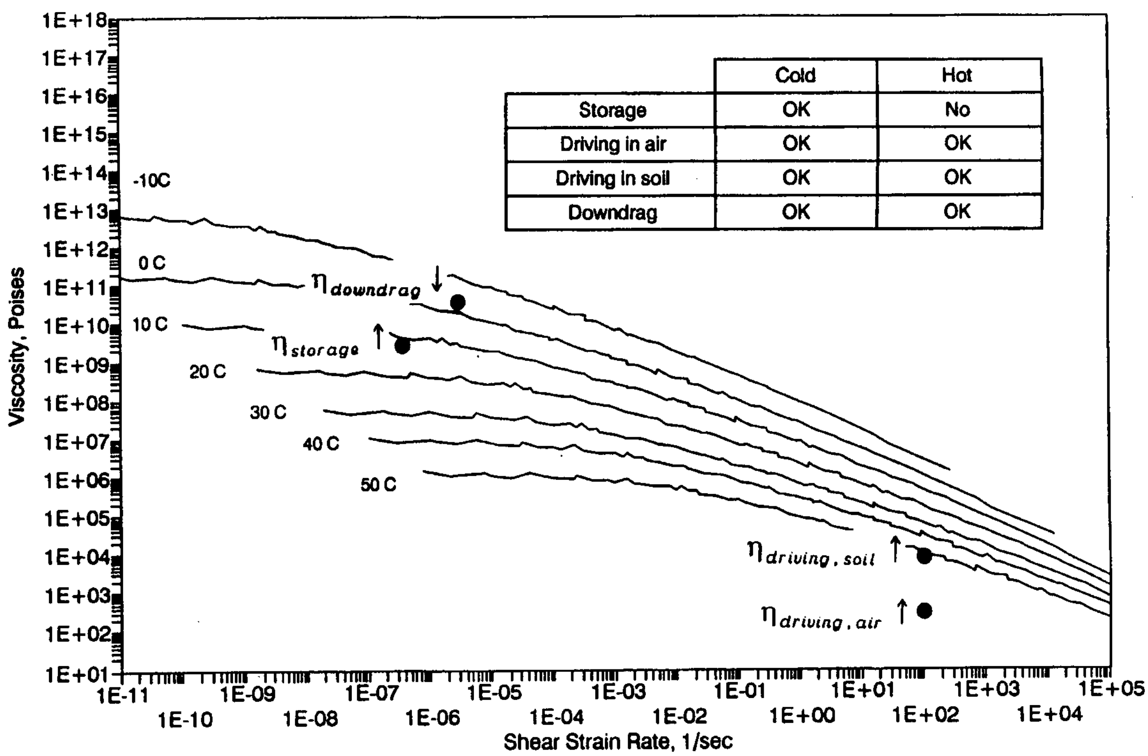


Figure 6.10. Bitumen viscosity and shear strain rate requirements plotted on master curve for bearing pile lubricant.

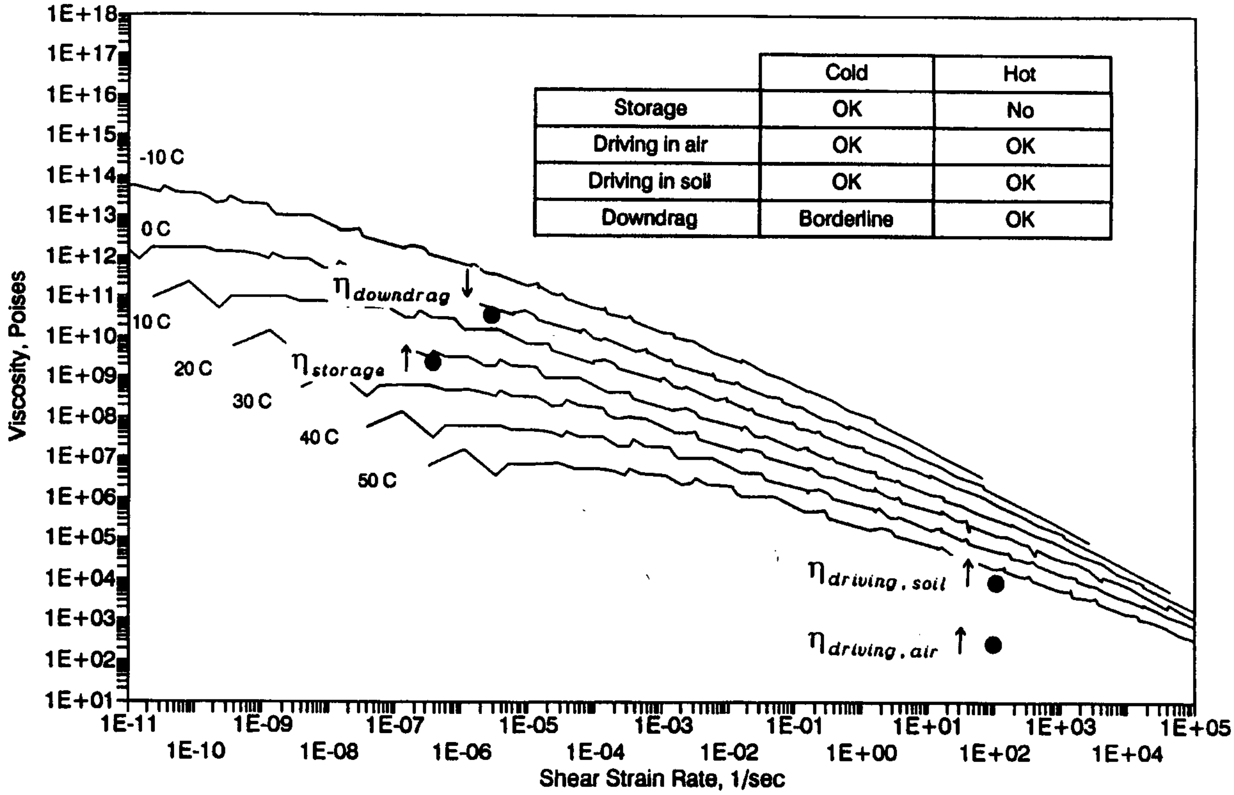


Figure 6.11. Bitumen viscosity and shear strain rate requirements plotted on master curve for Husky Oil type 2.

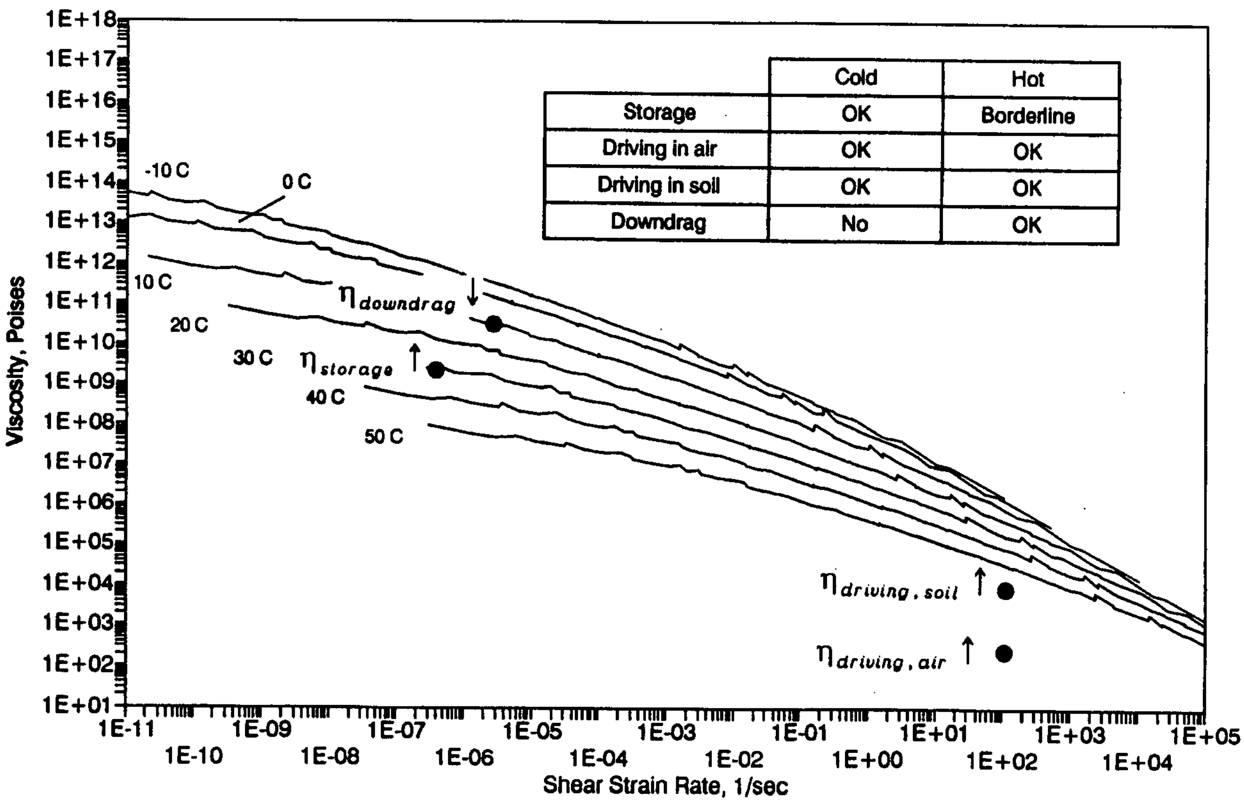


Figure 6.12. Bitumen viscosity and shear strain rate requirements plotted on master curve for Husky Oil type 2.

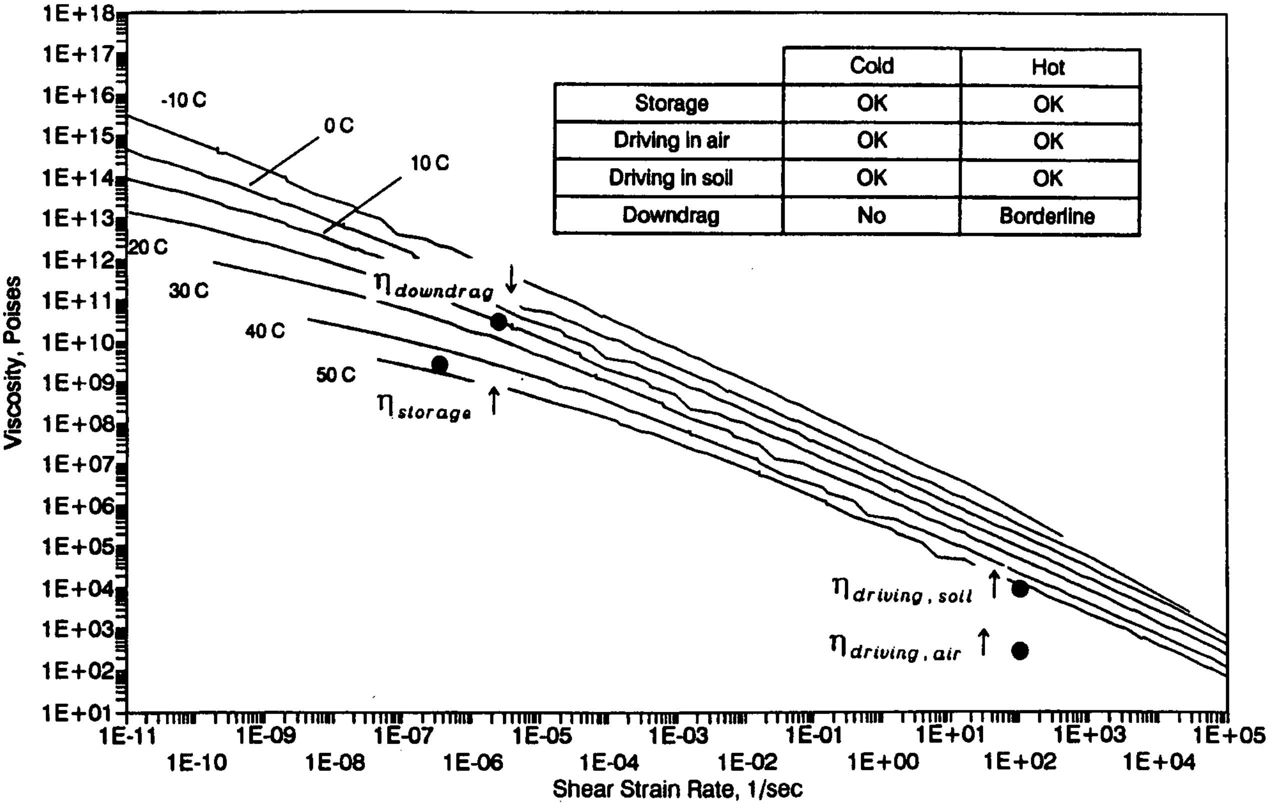


Figure 6.13. Bitumen viscosity and shear strain rate requirements plotted on master curve for Trumbull type 1.

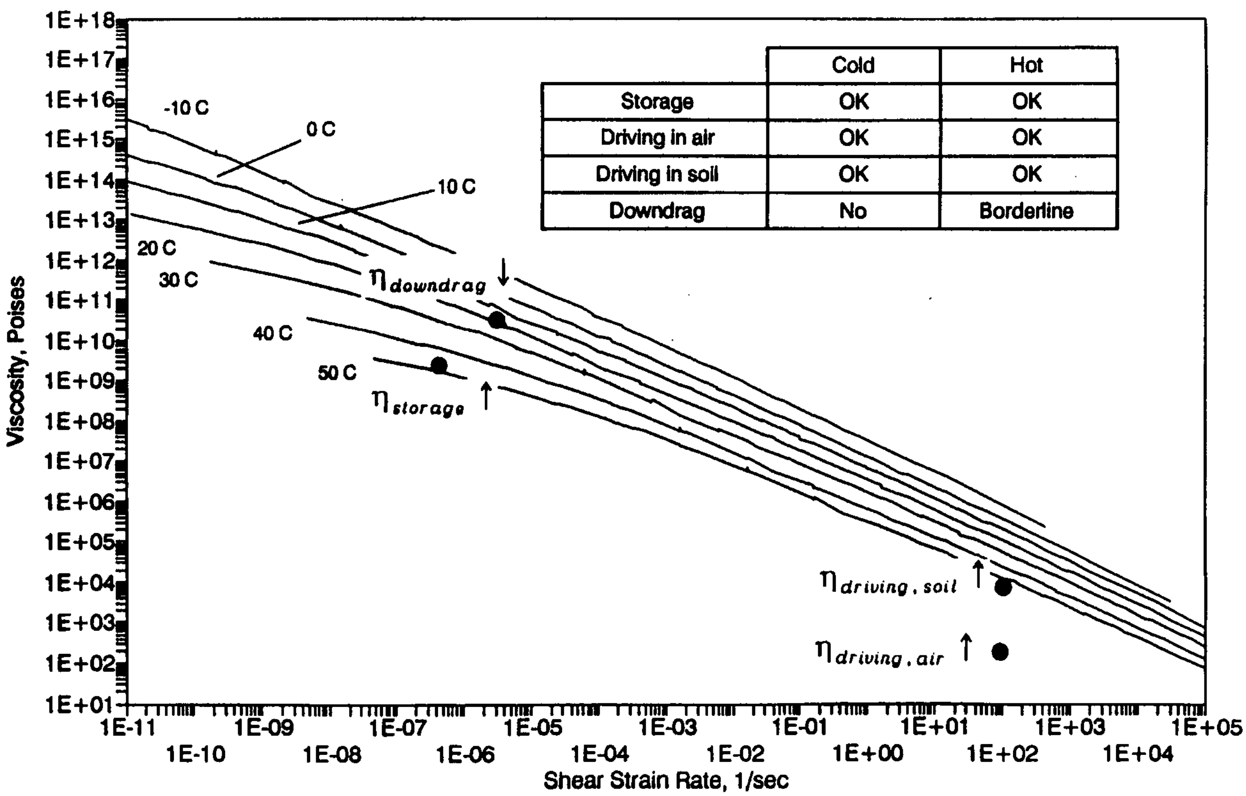


Figure 6.14. Bitumen viscosity and shear strain rate requirements plotted on master curve for Culvert compound.

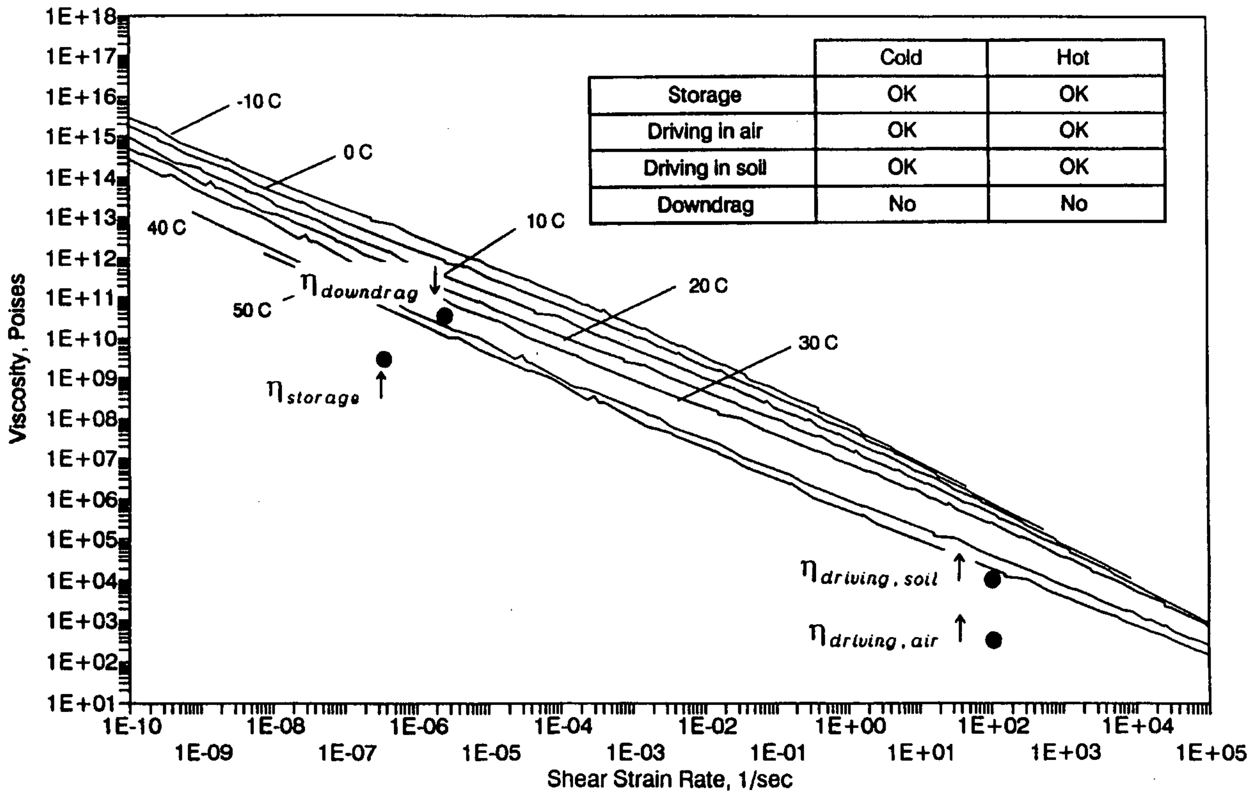


Figure 6.15. Bitumen viscosity and shear strain rate requirements plotted on master curve for Intec Blue (polymer modified).

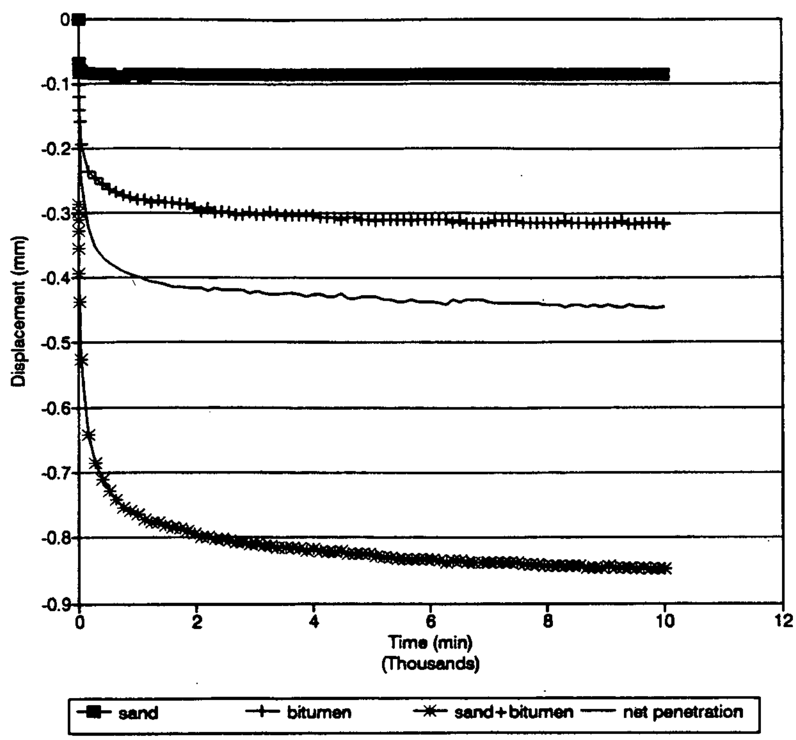


Figure 6.16. Particle penetration test results.

CHAPTER 7

EXAMPLE OF PILE GROUP DESIGN

A group of 12 uncoated, 0.4 m-square, 30 m-long, concrete piles is placed in the same soil profile used in Chapter 6 for the single pile example (Figure 7. 1). An uncoated single pile is analyzed using PILENEG for no top load to determine the downdrag on a single pile. The results are shown in Figure 7.2 and show a downdrag load on a single pile of 1,572 kN. The downdrag loads on the group piles can be obtained using the reduction factors given in Table 2.9 for a pile group with a center-to-center pile spacing of 2.5 pile diameters:

$F_{n(comer)}$ = 0.5 $F_{n(single)}$ = 0.5 × 1,572 kN = 786 kN
$F_{n(side)}$ = 0.4 $F_{n(single)}$ = 0.4 × 1,572 kN = 629 kN
$F_{n(interior)}$ = 0.15 $F_{n(single)}$ = 0.15 × 1,572 kN = 236 kN

For the 12-pile group shown in Figure 7.1 there are 4 corner piles, 6 side piles, and 2 interior piles. The total downdrag load on the group is 7,390 kN, which is only 39% of the downdrag load estimated on the basis of a single pile (12 × 1,572 = 18,864 kN). If this downdrag load is too high for the pile group, then bitumen coating can be used.

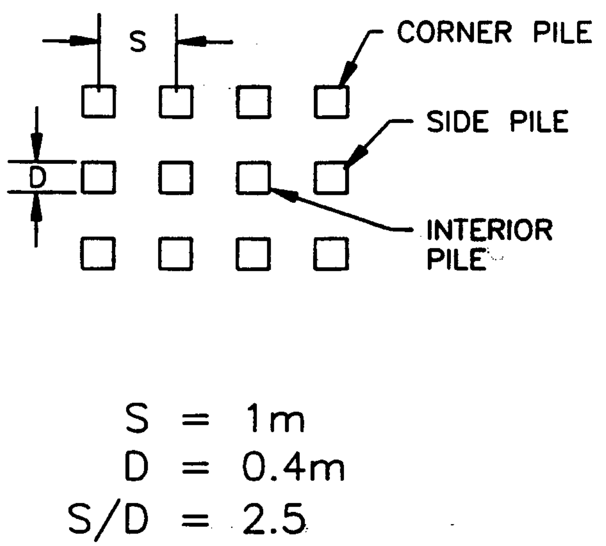


Figure 7.1. Example pile group.

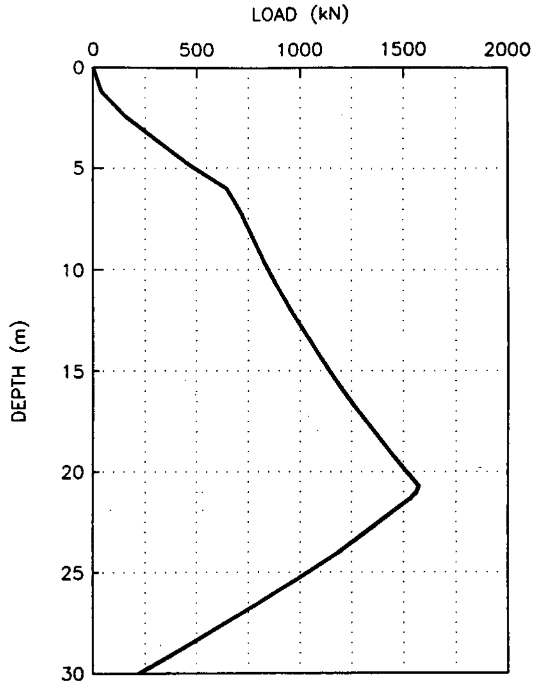


Figure 7.2. Load distribution on a single pile.

CHAPTER 8

MATERIALS, COSTS, AND CONSTRUCTION SPECIFICATIONS

8.1 EXAMPLES OF BITUMENS

A field testing program was conducted at two sites to test the effectiveness of various bitumens under a wide range of soil, temperature, and construction conditions (Briaud et al. 1989, Bush et al. 1991). The first site was in Edmonton, Alberta, Canada, where a construction project took place in the winter of 1989-90. This provided a cold temperature extreme. The second site was in New Orleans, Louisiana, where a project took place in the summer of 1990. This provided a hot temperature extreme.

Nine commercially available bitumen were evaluated in this study. Some properties of these bitumen are listed in Table 8.1.

8.2 EXAMPLES OF COSTS

The material costs and person-hours reported in Table 8.2 were achieved on the above mentioned projects. The labor was performed by persons without previous experience in applying bitumen coating to piles and therefore should represent an upper limit on cost. The material costs are average costs for the several bitumens used in the research project. Lower costs would be realized for larger quantities used in production piles.

The average increase in cost as a result of bitumen coating was 46%. Others have reported cost increases on production piles in the 15%-20% range (see Section 1.4).

8.3 GUIDELINES FOR CONSTRUCTION SPECIFICATIONS

The construction specifications for bitumen coating of piles should generally contain the following sections:

1. Description,
2. Materials,
3. Construction requirements,
4. Method of measurement, and
5. Basis of payment.

TABLE 8.1 Properties of example bitumens

Bitumen	Viscosity (Poise)		Penetration at 25°C		Softening Point (°C)
	Manf.	TAMU	Manf.	TAMU	Manf.
Intec Yellow[a]	5.76×10^5	NA	23-26	30	153-156
Intec Blue[a]	6.52×10^5	3.54×10^7	27-35	30	153-156
Culvert Compound[b]	6.50×10^6	7.17×10^6	25-50	34	63-77
Husky Oil Type 2[c]	6.00×10^5	2.0×10^6	20-30	20	75-83
Husky Oil Type 1[c]	2.30×10^5	4.47×10^5	30-45	25	60-68
Bearing Pile Lubricant[c]	NA	1.26×10^5	50-60	38	60-70
Soft Bearing Pile Lubricant[c]	NA	6.1×10^4	70-75	55	45-50
Trumbull Type 1[b]	1.50×10^4	3.56×10^4	18-60	NA	57-66
Trumbull Type 3[b]	2.00×10^4	NA	15-35	NA	85-96

Note: TAMU viscosity measurements were determined by the rheometer at a temperature of 60°C and a shear strain rate of 1.59×10^{-3} sec^{-1}. The temperature and test methods vary in the viscosity measurements made by the manufacturer.

a U.S. Intec, Inc., P.O. Box 2845, Port Arthur, TX 77643, Mr. Joey Bruns, (409)724-7024
Polymer-Modified roofing compounds (also available in membrane sheets)

b Husky Oil Marketing Company, P.O.Box 6525, Station "D", Calgary, Alberta, CANADA T2P 3G7, Mr. John Berti, (403)488-8143
Standard roofing compounds and air-blown asphalt products

c Trumbull Asphalt Division Owens-Corning Fiberglas Corporation, 3750 NW Yeon, Portland, OR 97210, Mr. Frank Burg, (503)220-2457
Standard roofing, paving and waterproofing asphalts

TABLE 8.2 Example material and labor costs for bitumen coating (1990)

		Material	Labor & Equipment
PILES	350mm Precast Concrete	$30/m	$19/m
	Timber	$13/m	$8/m
	300mm Steel Pipe	$26/m	$13/m
	Total(average)	$36.30/m	

		Meter of pile coated per man-hour*	Material*	Labor & Equipment*
BITUMEN	Primer	18.3	$0.12/m	$1.50/m
	Bitumen	3.3	$7.50/m	$7.50/m
	Total(average)		$16.62/m	

*These costs and times are based on a pile perimeter of 1.2 m and an hourly wage of $25/hr charged to the client.

AVERAGE % INCREASE = $16.62/$36.30 = 46%

8.3.1 Description

This item is simply a concise description of the work to be done such as

> This work shall consist of furnishing and applying bituminous coating and primer to (*pile type*) pile surfaces as required in the plans and as specified herein.

The description should indicate the pile type (e.g., steel, concrete, timber).

8.3.2 Materials

The bitumen to be used in coating the piles should be specified by the engineer on the basis of his determination of the viscosity, shear strain rate, and temperature requirements for storage, driving, downdrag reduction, and particle penetration (Chapter 4).

A rapid-cure type primer, such as RC 30, conforming to ASTM D41, should be specified unless the bitumen manufacturer recommends otherwise.

8.3.3 Construction Requirements

Preparation of the pile surface. The surface of the piles should be clean, dry, and free of grease or other poorly adhering substances. In the case of steel piles, the surface should also be free of excessive rust and loose scale; use of a wire brush may be required for minor cleaning, while sand blasting may be necessary in more severe cases. Concrete piles should also be dry and fully cured. They may simply need to be wiped with a cloth or swept with a broom.

It should be noted that creosote-treated timber piles should not be coated with bitumen. Because creosote is a coal tar derivative, it is incompatible with petroleum derived asphalts. The creosote would eventually degrade the bitumen causing it to delaminate from the pile or to become ineffective in reducing downdrag. CCA (trivalent Chromium, Copper oxide, Arsenic pentoxide)-treated timber piles or untreated timber piles can be coated with bitumen. These piles should also be clean, dry, and free of any poorly adhered substances.

Application of the primer. The primer should be applied to the piles at an air and pile temperature equal to or greater than 15°C. The primer should be applied uniformly to the pile surface, making sure that the entire surface is thoroughly coated. Brushing, rolling, or mopping is preferable to spraying, because these manual techniques tend to work the primer into the surface more thoroughly (Figure 8.1). The amount of primer needed is between 0.1 to 0.5 L/m^2 of pile surface. After application the primer should be allowed to cure for at least 24 hours or until dry to the touch.

Application of the bitumen. The bitumen should be applied to the piles at an air and pile temperature equal to or greater than 15°C. The bitumen should be heated to the minimum temperature necessary for application, but in no case above its flash point. The heating unit should have a temperature control and be capable of uniform heat distribution to prevent any localized burning of the bitumen. During the heating and application process breathing apparatus should be worn to avoid breathing the fumes given off by the bitumen.

Application should be by pouring the bitumen on the pile and brushing it to even out the coverage. Heating the bitumen to extremely high temperatures is not required

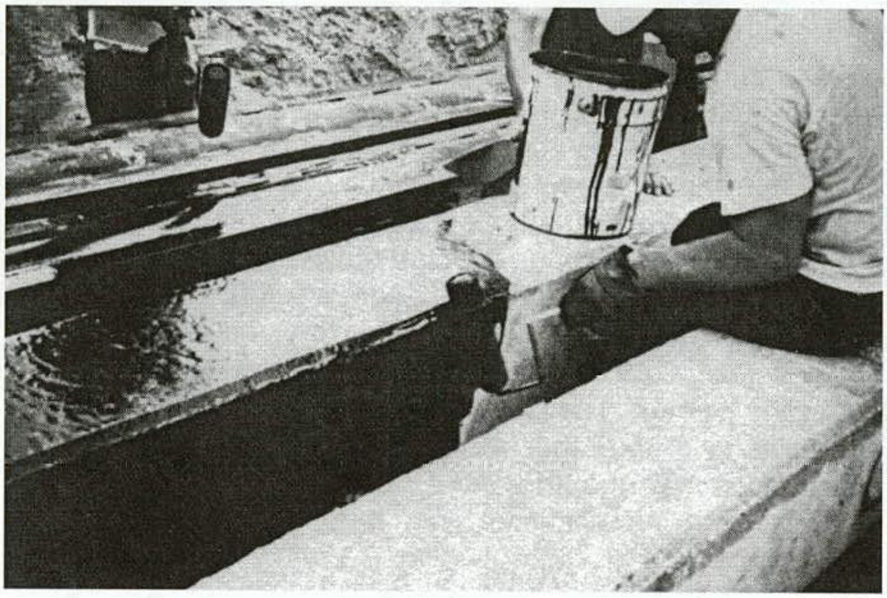


Figure 8.1. Application of primer with a roller.

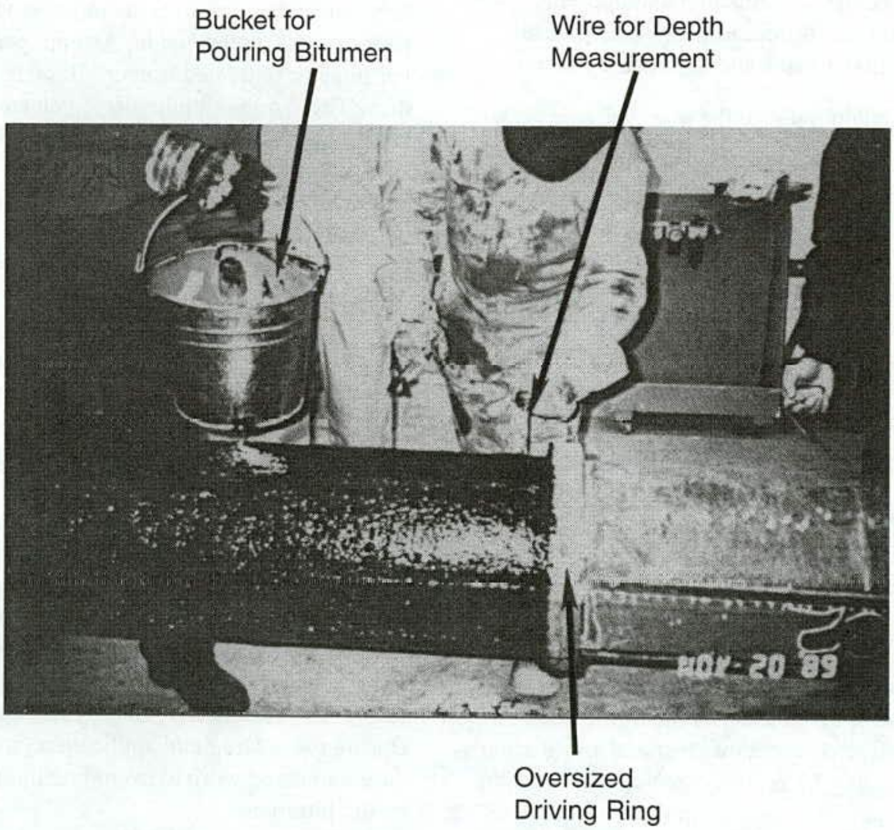


Figure 8.2. Application of bitumen by pouring.

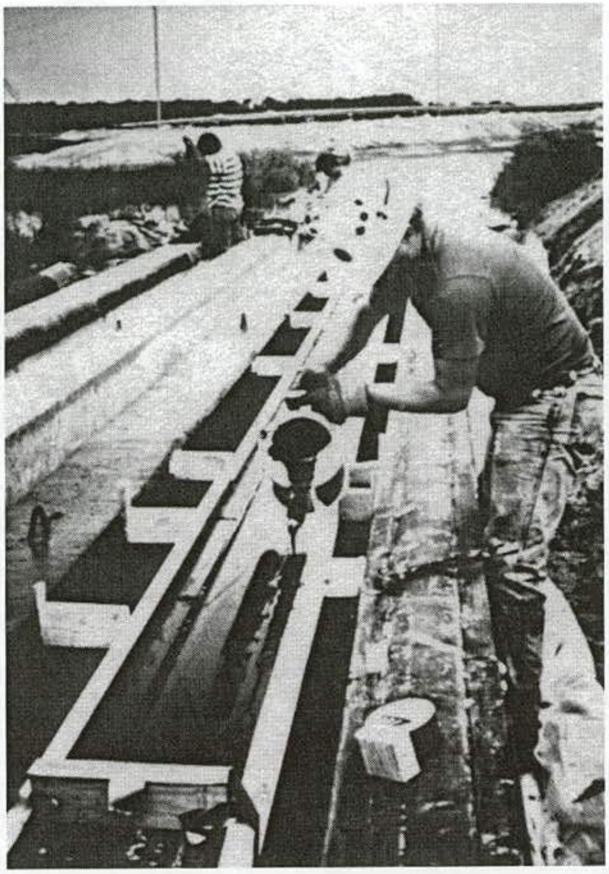


Figure 8.3. Use of temporary forms in warm weather.

with this method. A temperature which brings the bitumen to a honey or molasses consistency is ideal. Buckets or water cans with spouts seem to work best (Figure 8.2). In warmer weather, temporary forms may be necessary to contain the bitumen until it hardens sufficiently (Figure 8.3). In extremely cold weather, the piles must be coated indoors (Figure 8.4) to prevent the bitumen from cooling too rapidly and cracking (Figure 8.5). The piles should be coated at a temperature equal to or greater than 15°C. It should be noted that storage of the coated piles outside in the cold after the bitumen has slowly cooled down is not detrimental to the coating (Figure 8.6). However, this storage period should not exceed 30 days. More than one layer of bitumen may be necessary to obtain the thickness required. This method has been used with piles mounted on a lathe and rotated (manually) while the bitumen was poured on (Machan and Squier 1983); it has also been used with square precast concrete piles set in forms where up to 15 piles were coated in one batch (Claessen and Horvat 1974).

The thickness of the bitumen-coating should be 10 mm (tolerance: –2 mm, +4 mm). The thickness may be adjusted if it is determined that a different thickness is justified. The thickness should be checked at several locations on each pile to ensure proper coverage. This is done by simply probing with a steel wire (Figure 8.2).

It should be noted that the bitumen coating only extends to the depth of the neutral point, because below that depth positive friction is developed. This length should be included in the plans or specifications.

If steel piles are to be spliced, approximately 0.3 m of the pile should be left uncoated on either side at the spliced ends to prevent igniting the volatiles in the bitumen during welding. Water may be used to cool the piles during welding. The spliced ends should be primed and coated with bitumen as soon as practical after welding and prior to continued driving. Note that there is generally no time allowed for the primer to cure on the splice. If concrete piles are to be spliced no portion of the pile needs to be left uncoated.

Handling and storage. After the bitumen coating has hardened sufficiently, the piles may be moved to the storage area. The use of pad eyes (lifting hooks) on all piles is highly recommended to minimize damage to the coatings from chokers and slings. Chokers and slings may also slip on the bitumen when the pile is lifted, causing a safety hazard. Bitumen-coated piles should be handled with spreader bars (Figure 8.7). During storage, the piles should be separated by planks to prevent the bitumens from sticking to one another. The piles should be moved and handled as little as possible to minimize damage to the coatings.

During warm weather, the piles should be protected from direct sunlight during storage. They should also be monitored periodically to ensure that the bitumen is not slumping or dripping off the piles. In cold weather, the prime consideration is avoiding sharp impacts to the coatings. Rough handling of the piles can cause the bitumens to shatter, delaminate, and spall off the pile.

Driving. The bitumen coating should be protected from scraping or shearing from the soil during driving. This can be accomplished by predrilling an oversized hole through heavily compacted or granular layers and by using an oversized driving ring around the pile at the bottom of the bitumen coating (Figure 8.2). After driving, the annulus

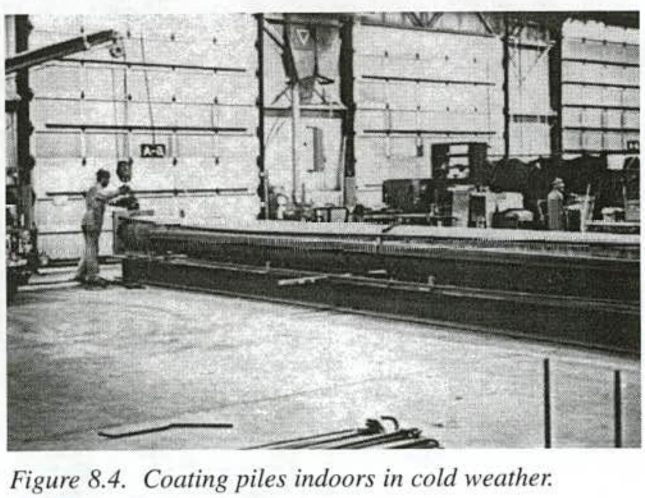


Figure 8.4. Coating piles indoors in cold weather.

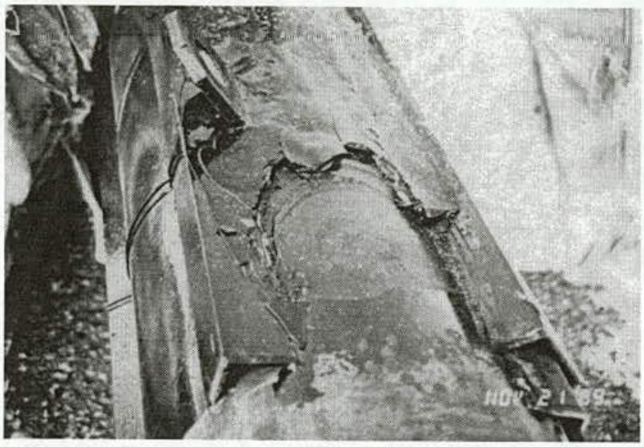


Figure 8.5. Bitumen cracking because of rapid cooling. (Note the concoidal fracture faces indicating an amorphous material.)

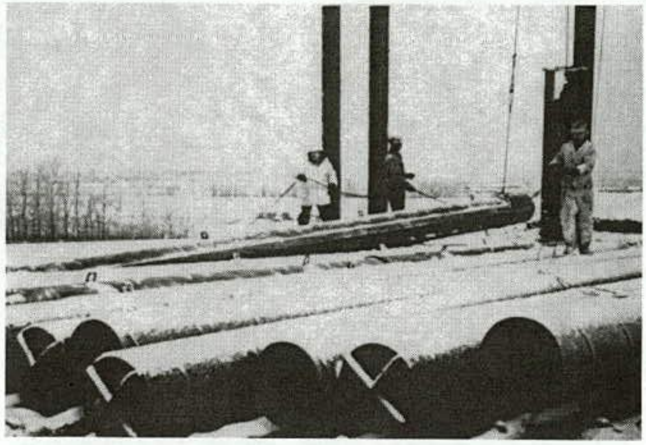


Figure 8.6. Storing piles outdoors in cold weather.

between the pile and the predrilled hole should be backfilled with fine sand. The pile should be properly aligned in the driving leads to prevent the bitumen from being scraped off on the leads during driving. If a significant portion of the coating (more than 5% of the coated area) has been damaged or lost, it should be repaired or replaced before driving is continued.

8.3.4 Method of Measurement

Measurement may be made by length or by area measurement of coating in place on the pile surface. Generally no separate payment is made for the primer or coating of spliced areas.

8.3.5 Basis of Payment

The payment is made at a contracted unit price according to the method of measurement. The payment is full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in applying the bituminous coating and primer, as shown in the plans, specified in the specification, and directed by the engineer.

8.4 ENVIRONMENTAL ASPECTS

The potential contamination of the groundwater by the bitumen coating was investigated at the two research sites during this study. The results point in the direction of no contamination by the bitumen 2 to 3 years after installation. Further studies are ongoing; an evaluation of the impact of construction materials is being performed as part of NCHRP Project 25-9, *Environmental Impact of Construction and Repair Materials on Surface and Ground Waters*. Bitumen-coated piles are being considered as part of that study.

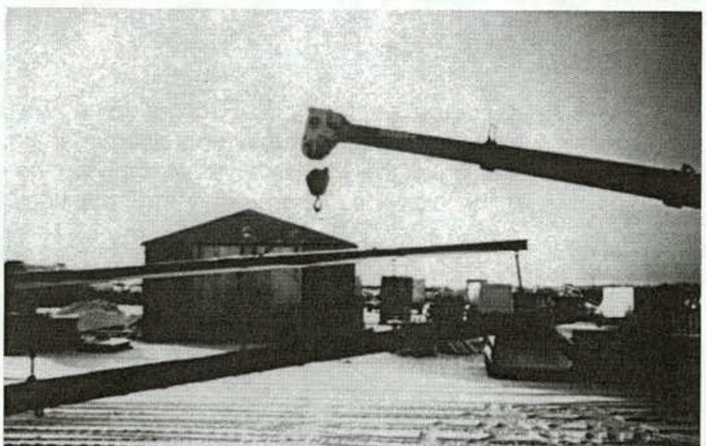


Figure 8.7. Use of spreader bars for pile handling.

BIBLIOGRAPHY

American Association of State Highway and Transportation Officials, "Standard Specifications for Highway Bridges," 14th ed. Washington, DC (1989).

American Concrete Institute, "Metric Building Code Requirements for Reinforced Concrete and Commentary." *ACI 318M-92/318/RM-92*, Detroit, Michigan (1992).

American Institute of Steel Construction, "Load and Resistance Factor Design Specification for Structural Steel Buildings." Chicago, Illinois (1986).

American Society for Testing and Materials, *Annual Book of Standards*, Vol. 04.04 Roofing, Waterproofing, and Bituminous Materials, Philadelphia, Pennsylvania (1989).

Anderson, D.A., D.W. Christensen, H.U. Bahia, R. Dongre, M.G. Sharma, C.E. Antle, and J. Button, "Binder Characterization and Evaluation—Volume 3: Physical Characterization." *Report No. SHRP-A-369*, Strategic Highway Research Program, Washington, DC (1994).

Asakawa, M., "Reduction of Skin Friction Between Soil and Metal Wall by the Electro-Osmosis Method." *Trans,* of Japan Society of Civil Engineers, No. 64 (1959) pp. 55-65.

Asphalt Institute, "Introduction to Asphalt." *Report No. MS-5*, Lexington, Kentucky (1990).

Asphalt Institute, "Principles of Construction of Hot-Mix Asphalt Pavements." *Manual Series 22*, College Park, Maryland (1983).

Asphalt Institute, "Safe Storage and Handling of Hot Asphalt." *Report No. IS-180*, Lexington, Kentucky (1981).

Asphalt Institute, "Specifications for Paving and Industrial Asphalts." *Report No. SS-2*, Lexington, Kentucky (1987).

Bakholdin, B.V. and V.I. Berman, "Investigation of Negative Skin Friction on Piles and Suggestions on Its Calculations." *Journal of Soil Mechanics and Foundation Engineering*, ASCE, Vol. 2, No. 4 (1975) pp. 238-244.

Baligh, M.M., H. Figi, V. Vivatrat, and A.M. Azzouz, "Design of Bitumen Coatings to Reduce Downdrag on Piles." *Research Report R80-42*, Geotechnical Order No. 686, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge (1981).

Baligh, M.M. and V. Vivatrat, "Prediction of Downdrag Loads on Bitumen-Coated Piles." *Report No. MIT-CE-R77-7*, Massachusetts Institute of Technology, Cambridge (1976).

Baligh, M.M., V. Vivatrat, and H. Figi, "Downdrag on Bitumen-Coated Piles." *Journal of Geo. Eng. Div.*, ASCE, Vol. 104, No. 11 (1978) pp. 1355-1370.

Barker, R.M., J.M. Duncan, K.B. Rojiani, P.S.K. Ooi, C.K. Tan, and S.G. Kim, "Manuals for the Design of Bridge Foundations." *NCHRP Report 343*, Transportation Research Board, National Research Council, Washington, DC (1991).

Bjerrum, L., I.J. Johannessen, and O. Eide, "Reduction of Negative Skin Friction on Steel Piles to Rock." *Proceedings*, 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City, Mexico, Vol. 2 (1969) pp. 27-34.

Board, J.R., "Bituminous Slip Layers for Bearing Piles in South Africa." *Shell Bitum Rev.*, 2nd Quarter, No. 50 (1975) pp. 20-23.

Bozozuk, M. "Downdrag Measurements on a 160 Foot Floating Test Pile in Marine Clay." *Canadian Geotechnical Journal*, Vol. 9, No. 2 (1972) pp. 127-136.

Bozozuk, M., G.H. Keenan, and P.E. Pheeney, "Analysis of Load Tests on Instrumented Steel Test Piles in Compressible Silty Soil." *Behavior of Deep Foundations, ASTM STP 670*, Raymond Lundgren, ed., American Society for Testing and Materials (1979) pp. 153-180.

Brand, E.W. and N. Luangdilok, "A Long-Term Foundation Failure Caused by Downdrag on Piles." *Proceedings*, 4th South-East Asian Conference on Soil Engineering, Kuala Lumpur, Malaysia (1975) pp. 4.15-4.24.

Briaud, J.L., *The Pressuremeter.* A.A. Balkema Publishers, Rotterdam, the Netherlands (1992).

Briaud, J.L. and J. Miran, "The Cone Penetrometer Test." U.S. Department of Transportation, *Publication No. FHWA-SA-91-043* (1992).

Briaud, J.L., L.M. Tucker, R.L. Lytton, H.M. Coyle, "Behavior of Piles and Pile Groups in Cohensionless Soils." Federal Highway Administration, *Report No. FHWA/RD-83-38* (1983).

Briaud, J.L., Z. Al-Gurjia, M. Quraishi, R.K. Bush, and S. Jeong, "Downdrag on Bitumen Coated Piles." Texas Transportation Institute, Texas A&M University System, NCHRP Preliminiary Draft Report Final Report (1989) unpublished.

Broms, B.B. "Design of Pile Groups with Respect to Negative Skin Friction." *Proceedings*, 7th International Conference on Soil Mechanics and Foundation Engineering, Specialty Session 8, Mexico City, Mexico (1969).

Brons, K.F., A.W. Amesz, and J. Rinck, "The Negative Skin Friction Along the Shaft of a Foundation Pile." *Proceedings*, 7th International Conference on Soil Mechanics and Foundation Engineering, Specialty Session 8, Mexico City, Mexico (1969).

Bush, R.K., R. Viswanathan, S. Jeong, and J.L. Briaud, "Downdrag on Bitumen Coated Piles." Texas Transportation Institute, Texas A&M University System, Interim Report to National Cooperative Research Program (1991) unpublished.

Bustamante, M. and L. Gianeselli, "Prevision de la capacite portantne des pieux par la methode penetrometrique." Compte Rendu de Recherche FAER 1.05.022, Laboratoire Centrale des Ponts et Chaussees, in private communication (1983).

Cambarieu, O., "Effet d'accroachage et methode d'evaluation du frottment negatif." *Bulletin de Liaison des Ponts et Chaussees*, No. 71 (1974).

Claessen, A.I.M. and J.J. Gelok, "Reduction of Negative Skin Friction on Bearing Piles by Bitumen Slip Layers." *Publication No. M59-71*, Shell Laboratorium, Amsterdam, the Netherlands (1971) p. 100.

Claessen, A.I.M. and E. Horvat, "Reducing Negative Skin

Friction with Bitumen Slip Layers." *Journal of the Geo. Eng. Div.*, ASCE, Vol. 100, No. GT8, Paper 10764 (1974) pp. 925-944.

Clemente, F.M., Jr., "Downdrag, Negative Skin Friction and Bitumen Coatings on Prestressed Concrete Piles." Ph.D. Dissertation, Tulane University, Department of Civil Engineering (1984).Cox, W.P. and E.H. Merz, "Correlation of Dynamic and Steady Flow Viscosities." *Journal of Polymer Science*, Vol. 28, No. 118 (April 1958) pp. 619-622.

Davisson, M.T., "Negative Skin Friction in Piles and Design Decisions." 3rd International Conference on Case Histories in Geo. Eng., Rolla, Missouri, Vol 3. (1993) pp. 1694-1702.

Davisson, M.T., F.S. Manuel, and R.M. Armstrong, "Allowable Stresses in Piles." *FHWA-RD-83-059*, Federal Highway Administration, Washington, DC (1983).

Endo, M. et al. "Negative Skin Friction Acting on Steel Pipe Pile in Clay." *Proceedings*, 7th International Conference on Soil Mechanics and Foundation Engineering. Mexico City, Mexico, Vol 2 (1969) pp. 85-92.

Fellenius, B.H., "Negative Skin Friction on Piles in Clay—A Literature Survey." *Proceedings*, 7th International Conference on Soil Mechanics and Foundation Engineering. Mexico City, Mexico (1969) pp. 93-98.

Fellenius, B.H., "Discussion of Downdrag on Bitumen Coated Piles." *Journal of Geo. Eng. Div.*, ASCE, Vol. 105, No. GT10 (1979).

Ferry, J.D., *Viscoelastic Properties of Polymers*, 3rd ed. John Wiley & Sons, New York (1980).

Garlanger, J.E., "Measurements of Pile Downdrag Beneath a Bridge Abutment." *Transportation Research Record 517*, Transportation Research Board, National Research Council, Washington, DC (1974) pp. 61-69.

Girault, P., "Positive Skin Friction and Settlement of Piled Foundations." 7th International Conference on Soil Mechanics and Foundation Engineering. Mexico City, Mexico, Specialty Session 8 (1969).

Hossain, M.K. and J.L. Briaud, "Pipe Piles in Sand: An Improvement for the API RP2A." Offshore Technology Research Center, Texas A&M University (1992).

Hutchinson, J.N. and E.V. Jensen, "Loading Tests on Piles Driven into Estuarine Clays at Port of Khorramshar, and Observation on the Effect of Bitumen Coatings on Shaft Bearing Capacity." *Publication No. 78*, Norwegian Geotechnical Institute, Oslo, Norway (1968) pp. 1-12.

Inoue, Y., Y. Kobayashi, and K. Tamaoki, "Characteristics of Negative Friction Acting on Piles." *Report of Shimizu Tech. Research Institute*, No. 24 (1975) pp. 21-35.

Ito, T. and T. Matsui, "Negative Skin Friction Acting on Piles." *Proceedings*, 5th Budapest Conference on Soil Mechanics and Foundation Engineering, Akademiai Kiado, Budapest, Hungary (1976).

Jeong, S., and J.-L. Briaud, "Nonlinear Three Dimensional Analysis of Downdrag on Pile Groups." Research Report, Department of Civil Engineering, Texas A&M University, College Station, Texas (1992).

Johannessen, I.J. and L. Bjerrum, "Measurements of the Compression of a Steel Pile to Rock Due to Settlement of the Surrounding Clay." *Proceedings*, 6th International Conference on Soil Mechanics and Foundation Engineering, Montreal, Canada (1965) pp. 261-264.

Koerner, M.K. and C. Mukhopadhyay, "Behavior of Negative Skin Friction on Model Piles in Medium-Plastic Silt." *Highway Research Record 405* (1972) pp. 34-50.

Lambe, T.W. and M.M. Baligh, "Negative Skin Friction Downdrag on a Pile." Massachusetts Institute of Technology, *Report No. FHWA/TS-73/210*, Cambridge (1978) p. 89.

Lambe, T.W., J.E. Garlanger, and S.A. Leifer, "Prediction and Field Evaluation of Downdrag Forces on a Single Pile." *Research Record No. R74-27*, Soil Publication No. 339, Massachusetts Institute of Technology, Cambridge (1974).

Lambe, T.W. and R.V. Whitman, *Soil Mechanics.* John Wiley & Sons, New York (1969).

LCPC-SETRA, "Regies de justification des fondations sur pieux a partir des resultats des essais pressiometriques." Labortoire Central des Ponts et Chaussees, Service d'Etudes Techniques des Routes et Autoroutes, Paris, France (1985).

Machan, G. and L.R. Squier, "Bitumen Coating on Piles to Reduce Downdrag." *Public Roads.* Vol. 46, No. 4 (1983) pp. 142-151.

McCarthy, D.F., *Essentials of Soil Mechanics and Foundations*, 2nd ed. Reston Publishing Co., Reston, Virginia (1982).

Meyerhof, G.G., "Influence of Roughness of Base and Ground-Water Conditions on the Ultimate Bearing Capacity of Foundations." *Geotechnique*, 5 (1955) pp. 227-242.

Newmark, N.M., "Influence Charts for Computation of Stresses in Elastic Foundations." *University of Illinois Engineering Exp. Sta. Bulletin 338* (1942).

Ng, E.S., J.L. Briaud, and L.M. Tucker, "Field Study of Pile Group Action in Sand." *Research Report FHWA-RD-88-QB2* (1988).

Nippon Kokan, K.K., "NF Pile, Non-Negative Friction Pile." *Technical Bulletin*, Japan (April 1977).

Okabe, T., "Large Negative Friction and Friction-Free Pile Methods." *Proceedings*, 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Japan, Vol. 1 (1977) pp. 672-679.

Petersen, J.C., R.E. Robertson, J.F. Branthaver, P.M. Harnsberger, J.J. Duvall, S.S. Kim, D.A. Anderson, D.W. Christiansen, and M.U. Bahia, "Binder Characterization and Evaluation—Volume 1." *Report No. SHRP-A-367*, Strategic Highway Research Program, Washington, DC (1994a).

Petersen, J.C., R.E. Robertson, J.F. Branthaver, P.M. Harnsberger, J.J. Duvall, S.S. Kim, D.A. Anderson, D.W. Christiansen, M.U. Bahia, R. Dongre, C.E. Antle, M.G. Sharma, J.W. Button, and C.J. Glover, "Binder Characterization and Evaluation—Volume 4: Test Methods." *Report No. SHRP-A-370*, Strategic Highway Research Program, Washington, DC (1994b).

Puzinauskas, V.P., "Properties of Roofing Asphalts." *Research Report 82-1*, The Asphalt Institute, College Park, Maryland (1982).

Puzinauskas, V.P. and L.W. Corbett, "Differences Between Petroleum Asphalt, Coal-Tar Pitch and Road Tar." *Report No. RR-78-1*, The Asphalt Institute, College Park, Maryland (1978).

Randolph, M.F. and C.P. Wroth, "Analysis of Deformation on Vertically Loaded Piles." *Journal of Soil Mechanics and Foundation Engineering*, ASCE, Vol. 2, No. GT12 (1978) pp. 1465-1488.

Roberts, F.L., P.S. Kandhal, E.R. Brown, D.Y. Lee, and T.W. Kennedy, *Hot Mix Asphalt Materials, Mixture Design, and Construction.* NAPA Research and Education Foundation, Lanham, Maryland (1991).

Saito, A., K. Ishigami, T. Kamei, and T. Fukuya, "Bearing Capacity of Steel Pile Driven in Soft Ground." *Journal of the Japan Society of Soil Mechanics and Foundation Engineering*, No. 209 (1975) pp. 35-42.

Sawaguchi, M., "Model Test in Relation to a Method to Reduce Negative Skin Friction by Tapering a Pile." *Soil and Foundations,* Vol. 22, No. 3, Tech. Note (1982) pp. 130-133.

Sultan, H.A., "Downdrag on Piles, State-of-the-Art." *Proceedings*, 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City, Mexico, Specialty Session 8 (1969).

Terzaghi, K. and R.B. Peck, *Soil Mechanics in Engineering Practice*, 2nd ed. John Wiley & Sons, New York (1967).

Tsutsumi, K. and M. Nei, "Observations of Negative Friction." *Journal of the Japan Society of Soil Mechanics and Foundation Engineering*, No. 209 (1975) pp. 49-55.

VanWeele, A.F., "Negative Skin Friction Along Foundation Piles." Review, *Home Magazine of Nederhost United*, Gouda, the Netherlands (1968).

Visher, S.S., *Climatic Atlas of the United States*. Harvard University Press, Cambridge, Massachusetts (1954).

Walker, L.K., L. Darvall, and P. Le, "Dragdown on Coated and Uncoated Piles." *Proceedings*, 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, Russia (1973) pp. 257-262.

Walkinshaw, J.L., Personal communication (1984).

APPENDIX

USERS' MANUAL FOR PILENEG

This appendix contains the users' manual for the PILENEG program. PILENEG is a program for the analysis of vertical axially loaded single piles subjected to downdrag forces. The program was written by Larry M. Tucker and Jean-Louis Briaud in 1995.

A1 SYSTEM REQUIREMENTS

The program requires three files to be in the default directory in order to run properly. These files are PILENEG.EXE—the main program file, PILENEG.STP—the setup file, and PILENEG.LBR—a screen library file. The program is written in Microsoft QuickBASIC 4.0 and is distributed in compiled form. The screen files were created using HI-SCREEN XL Professional and the screen and plotter routines are from Quinn-Curtis Science and Engineering Tools. Because the screen and plotting routines are copyrighted, the source code for this program is only available with proof of purchase of HI-SCREEN XL Professional and Quinn-Curtis Science and Engineering Tools.

The program should run on any MS-DOS based computer with CGA/EGA/VGA graphics. Tabular form of the output is saved in an ASCII file and may be viewed on screen or dumped to a printer if available. Plots of the output may be viewed on screen or plotted on an HPGL compatible plotter/printer. The screen plots may be dumped to a printer or the ASCII file may be imported into a spreadsheet and plots created there to match the user's own format.

A2 ASSUMPTIONS AND LIMITATIONS

This program analyzes vertical axially loaded piles under negative skin friction based on the static equilibrium of the pile and on the compatibility of the relative pile-soil movement as explained in Chapter 2. It will also analyze piles with positive shaft resistance only.

The first assumption is that the full shaft resistance is mobilized if any relative movement occurs between the pile and the soil. In other words, the friction model is a rigid plastic model. The error due to this assumption is small because very little movement is required to mobilize the full pile-soil friction. Above the neutral point, the shaft resistance acts downward (negative) to add more load to the pile; below the neutral point the shaft resistance acts upward (positive) to support the pile.

The point resistance curve is assumed to be elasto-plastic as stated in Section 2.1.1 (Figure 2.2). The elastic portion of the curve can be calculated using Eq. 2.2.

The program is written to use any set of consistent units. The only units necessary to be chosen are force and length. All other units can be derived from these.

The pile is divided into a number of increment lengths specified by the user. When inputting a known depth to the bitumen coating, the program will match this as closely as possible with the nearest increment, and print out the depth used for calculation. This means that the depth may be off by as much as one-half the increment length. If the depth chosen is not acceptable, a different number of increments may be chosen.

There are two options for loading: inputting one top load or calculating the entire load-settlement curve. For the load-settlement curve option, the maximum load is calculated and then divided into a number of loads as specified by the user.

There are five options for specifying the type of shaft resistance and bitumen coating depth:

1. Considering negative skin friction, with no bitumen coating.
2. Considering negative skin friction and setting the bitumen-coating depth equal to the neutral point. When calculating the entire load-settlement curve with this option the user must be aware that for each top load the depth to the neutral point changes and therefore the depth of the bitumen coating changes.
3. Considering negative skin friction and keeping the depth of the bitumen coating constant.
4. Considering positive shaft resistance only, with no bitumen coating.
5. Considering positive shaft resistance only, with a constant bitumen-coating depth.

In analyzing piles subjected to downdrag, it is conservative to overestimate the negative shaft resistance and underestimate the positive shaft resistance. In order to accomplish this, the program prompts the user to input two multipliers: one for negative shaft resistance and another for positive shaft resistance. The default value for these multipliers is 1, but they may be changed by the user.

The cross-sectional area, perimeter, and pile modulus are considered to be constant along the entire length of the pile. Therefore, this program cannot currently be used to analyze tapered piles, step-tapered piles, or any pile where these parameters vary with depth.

The maximum shaft resistance profile is not the soil shear strength profile, but rather the shaft resistance profile as calculated by any method deemed reasonable by the user. **The shaft resistance at the ground surface should not be set to zero; it may be set to a very small value relative to the rest of the profile.**

The maximum shaft resistance profile and the soil-settlement profile are input as discrete points at various depths. The program interpolates linearly to obtain values at depths between these points. Each of these profiles should start at the ground surface and extend below the pile point. **The program cannot handle two values at the same depth. If a stepped profile is desired, the user must input two values at very slightly different depths.**

A3 SETUP FILE

The setup file PILENEG.STP is an ASCII file that may be edited using any line or text editor, being careful not to change the line numbers. The file contains four parameters. The first line contains the maximum dimension for all arrays dealing with the number of increments that the pile may be divided into, including the maximum shaft resistance data and the soil-settlement data. The second line contains the maximum dimension for all arrays dealing with the number of points on the load-settlement curve. The third line is a switch for the screen plotting routine. All output plots may be sent to a Hewlett-Packard compatible plotter. If such a plotter is not available, the user may want to dump the screen plots to a printer. If this is the case, line 3 must contain the number 0 so that the background color for the plots is black. Otherwise line 3 should be set to 1, and the plots will have a background color that makes them easier to see on the screen. Line 4 of the setup file contains the setup for the plotter port in some form such as COM2:9600,S,7,1,RS,CS65535,DS,CD. Check your computer and plotter reference manuals for more information on port configuration. The dimension variables on lines 1 and 2 may be increased, if necessary, until the array size reaches 64 k (about 16,000 values), which should not be a limiting problem.

A4 INPUT

The program includes an interactive input routine which allows the user to input data from the keyboard and then examine or modify the input before continuing. The input routine also allows data that has been previously saved on disk to be reused and modified if necessary. The input is as follows:

Title	Problem title,
Units	Abbreviations for force and length units,
Loading option	(1) One top load, or (2) load-settlement curve,
Friction and coating option	(1) Negative friction, no bitumen coating,
	(2) Negative friction, varying bitumen-coating length,
	(3) Negative friction, constant bitumen-coating length,
	(4) Positive friction only, no bitumen coating, and
	(5) Positive friction only, constant bitumen-coating length.

Depending on the options chosen above some of the following items will be needed:

Number of increments on the pile,
Top load on the pile,
Number of points on the load-settlement curve,
Shear strength of the bitumen,
Maximum number of iterations to find the depth of bitumen coating,
Tolerance for convergence of neutral point, and
Depth of bitumen coating.

The following pile data are needed:

Cross-sectional area,
Area of the pile point,
Perimeter,
Embedded length, and
Modulus.

The maximum shaft resistance profile and the soil-settlement profile are needed. The following bearing soil layer data are needed:

Young's modulus,
Poisson's ratio, and
Ultimate bearing capacity.

For help in compiling the necessary data some blank input forms are included in Section A7.

A5 OUTPUT

The output from the program is automatically written to a disk file specified by the user. As an option, the results may be printed or plotted, or viewed on the screen. The disk file results and the printed results are exactly the same in content. These include

- Echo of all input data,
- Table of top load, top settlement, depth of bitumen coating, depth to neutral point, maximum load in pile, maximum stress in the pile, and point load, and
- Table of axial force, axial stress, soil settlement, and pile settlement with depth.

The plotting routine will plot settlement versus depth, load versus depth, or the top load-settlement curve. These plots may be viewed on the screen or sent to a plotter.

A6 EXAMPLE PROBLEMS

The following ten example problems show the input needed for each of the shaft resistances and coating options. In example problem 1, all the input screens are shown and comments are made. For the following nine problems, only the input data and the output results are shown. The pile data, maximum shaft resistance and the soil-settlement profiles, and the bearing soil data are the same for all ten problems and are therefore shown only in example problem 1 input.

A6.1 One Top-Load, Negative Friction, No Bitumen Coating

The following three pages show the input data sheets for example problem 1. After this, each input screen is shown with comments regarding the input data on this screen. Finally, all the output results for this problem are shown. The pile used in this problem is a 419 mm octagonal concrete pile.

Data file name: EXAMPLE1.DAT	
Result file name: EXAMPLE1.RES	

Problem title: EXAMPLE PROBLEM 1	
Force units: kN	Length units: m

Choose one of the following:

✓	1. One top load only
	Top load on the pile: 2225 kN
	2. Load-settlement curve
	Number of points on load-settlement curve:

Choose one of the following:

✓	1. Negative friction, no bitumen coating
	Number of increments on the pile: 50
	2. Negative friction, varying bitumen coating length
	Number of increments on the pile:
	Shear strength of bitumen:
	Maximum number of iterations to find the depth of bitumen coating:
	Tolerance for convergence of neutral point:
	3. Negative friction, constant bitumen coating length
	Number of increments on the pile:
	Shear strength of bitumen:
	Depth of bitumen coating:
	4. Positive friction, no bitumen coating
	Number of increments on the pile:
	5. Positive friction, constant bitumen coating length
	Number of increments on the pile:
	Shear strength of bitumen:
	Depth of bitumen coating:

Multiplier for negative friction: 1
Multiplier for positive friction: 1

PILE DATA

Cross sectional area:	0.145 m^2
Area of pile point:	0.145 m^2
Perimeter:	1.39 m
Embedded length:	41.76 m
Modulus:	2.41×10^7 kN/m^2

MAXIMUM SHAFT RESISTANCE DATA

No.	Depth (m)	Maximum Shaft Resist. (kN/m^2)	No.	Depth	Maximum Shaft Resist.	No.	Depth	Maximum Shaft Resist.
1	0	12.92	16			31		
2	22.86	30.80	17			32		
3	41.76	94.19	18			33		
4			19			34		
5			20			35		
6			21			36		
7			22			37		
8			23			38		
9			24			39		
10			25			40		
11			26			41		
12			27			42		
13			28			43		
14			29			44		
15			30			45		

SOIL SETTLEMENT DATA

No.	Depth (m)	Soil Settle-ment (m)	No.	Depth	Soil Settle-ment	No.	Depth	Soil Settle-ment
1	0	0.335	11			21		
2	6.10	0.165	12			22		
3	9.14	0.119	13			23		
4	12.19	0.088	14			24		
5	15.24	0.058	15			25		
6	21.34	0.034	16			26		
7	41.76	0.015	17			27		
8			18			28		
9			19			29		
10			20			30		

BEARING SOIL DATA

Young's modulus of bearing soil:	2153 kN/m^2
Poisson's ratio of bearing soil:	0.3
Ultimate bearing capacity of bearing soil:	7097 kN/m^2

```
                     --------------------------------
                     | Civil Engineering Department |
---------------------|    Texas A&M University      |----------------------------
|                    |    Ver. 1.00/August 1994     |          |                |
                     --------------------------------          v
|                                                             ---               |
     ANALYSIS OF AXIALLY LOADED SINGLE PILES               ----| |----
|                                                              v| |v            |
          UNDER NEGATIVE SKIN FRICTION                         v| |v
|                                                              v| |v            |
                                                               ^| |^
|   Written by: H.P.Porwoll and J.L.Briaud  1984               ^| |^            |
    Revised by: L.M.Tucker 1994                                ^| |^
|                                                              ^---^            |
                                                                 ^
|-------------------------------------------------------------------------------|
|This program analyzes axially loaded single piles under donwdrag load          |
|based on the static equilibrium of the pile and on the compatibility of the    |
|relative pile-soil movement. The program will compute the neutral point, the   |
|maximum force and stress in the pile, and the entire load-settlement curve     |
|for a pile under negative skin friction, as well as for piles under positive   |
|friction only.  See the user's manual for assumptions and limitations of the   |
|program, details of the analysis method and application to pile groups.       |
---------------------------------------------------------------------------------
                     Press function key <F10> to continue.
```

This is the title screen of the program. All subsequent screens will be superimposed on this screen. In order to continue, press the function key <Fl0>. This key will be used on all screens (except menus) to validate the data on the screen and continue the program.

```
                     --------------------------------
                     | Civil Engineering Department |
---------------------|    Texas A&M University      |----------------------------
|                    |    Ver. 1.00/August 1994     |          |                |
                     --------------------------------          v
|               ***************************************       ---               |
     ANALYSIS OF A*-----------------------------------*    ----| |----
|               *| > Create new data file         < |*       v| |v            |
          UNDER NE*|   Retrieve existing data file    |*       v| |v
|               *|   Quit                           |*       v| |v            |
                *-----------------------------------*          ^| |^
|   Written by: H.P*************************************       ^| |^            |
    Revised by: L.M ************************************       ^| |^
|                                                              ^---^            |
                                                                 ^
|-------------------------------------------------------------------------------|
|This program analyzes axially loaded single piles under donwdrag load          |
|based on the static equilibrium of the pile and on the compatibility of the    |
|relative pile-soil movement. The program will compute the neutral point, the   |
|maximum force and stress in the pile, and the entire load-settlement curve     |
|for a pile under negative skin friction, as well as for piles under positive   |
|friction only.  See the user's manual for assumptions and limitations of the   |
|program, details of the analysis method and application to pile groups.       |
---------------------------------------------------------------------------------
```

Notice that a window has been superimposed on the title screen. This window is a menu of three choices. To select an option, either locate the reverse video bar on the option desired using the arrow keys (or the space bar) and press the < Enter > or < Return > key, or simply type the first letter of the option desired.

Option 1 allows the user to create a new data file. Option 2 allows an existing file to be rerun with or without modifications. Option 3 exits the program.

For this example, choose option 1: create a new data file.

```
                           ------------------------------
                          | Civil Engineering Department |
 -------------------------|    Texas A&M University      |---------------------------
|                         |    Ver. 1.00/August 1994     |                  |        |
|       ##########################################################################
|       #------------------------------------------------------------------#     |
|  ANALYSIS#| FILE TO SAVE DATA TO:                                      |# --- |
|         #|   EXAMPLE1.DAT                                             |#     |
|      UND#|------------------------------------------------------------|#     |
|         #| FILE TO SAVE RESULTS TO:                                   |#     |
|         #|   EXAMPLE1.RES                                             |#     |
| Written by#------------------------------------------------------------#     |
| Revised by##############################################################     |
|          ##############################################################       |
|                                                                    ^          |
 --------------------------------------------------------------------------------
|This program analyzes axially loaded single piles under donwdrag load          |
|based on the static equilibrium of the pile and on the compatibility of the    |
|relative pile-soil movement. The program will compute the neutral point, the   |
|maximum force and stress in the pile, and the entir-----------------------------
|for a pile under negative skin friction, as well as|##########################|
|friction only.  See the user's manual for assumptio|#Press <F10> to validate#|
|program, details of the analysis method and applica|##########################|
 --------------------------------------------------------------------------------
        Input name of file to save results to: <x:><\path\>filename.ext
```

This window allows the user to input the file names in which to save the data and the results. The file names should be complete, including drive, paths, file name and extension. If no drive and path are input, the file will be saved on the default drive and directory.

If the user selected the option of the previous screen to retrieve an existing data file, this screen would require the input of three file names: (1) the file to retrieve the data from, (2) the file to save the modified data to, and (3) the file to save the results to. The file name in which to save the modified data defaults to the file name the data is retrieved from, but may be changed to something else if desired.

After the names have been input, press the function key < Fl0 > to validate the data.

```
                           ------------------------------
                          | Civil Engineering Department |
 -------------------------|    Texas A&M University      |---------------------------
|                         |    Ver. 1.00/August 1994     |                  |        |
|                          ------------------------------                   v        |
|                                                                         ---       |
|   ANALYSIS OF AXIALLY LOADED SINGLE PILES                          ----| |----   |
####################################################################################
|-----------------------------------------------------------------------------------
|Input title of run:                                                                |
|                                                                                   |
|EXAMPLE PROBLEM 1                                                                  |
|-----------------------------------------------------------------------------------
####################################################################################
|                                                                    ^              |
 -----------------------------------------------------------------------------------
|This program analyzes axially loaded single piles under donwdrag load             |
|based on the static equilibrium of the pile and on the compatibility of the       |
|relative pile-soil movement. The program will compute the neutral point, the      |
|maximum force and stress in the pile, and the entir--------------------------------
|for a pile under negative skin friction, as well as|############################|
|friction only.  See the user's manual for assumptio|#Press <F10> to validate#|
|program, details of the analysis method and applica|############################|
 -----------------------------------------------------------------------------------
```

This screen asks for the title of the run. After the title has been input, press the function key < Fl0 > to validate.

```
                      ------------------------------
                      | Civil Engineering Department|
 ---------------------|     Texas A&M University    |--------------------------
|                     |     Ver. 1.00/August 1994   |            |             |
|                     ------------------------------             v             |
|                    ########################################    ---            |
|   ANALYSIS OF A#------------------------------------#     ----| |----       |
|                #| Enter units abbreviations for    |#         v| |v          |
|        UNDER NE#| Force: kN                        |#         v| |v          |
|                #| Length: m                        |#         v| |v          |
|                #------------------------------------#         ^| |^          |
| Written by: H.P#####################################          ^| |^          |
| Revised by: L.M #####################################         ^| |^          |
|                                                               ^---^          |
|                                                                 ^            |
 ------------------------------------------------------------------------------
| This program analyzes axially loaded single piles under donwdrag load        |
| based on the static equilibrium of the pile and on the compatibility of the  |
| relative pile-soil movement. The program will compute the neutral point, the |
| maximum force and stress in the pile, and the entir--------------------------
| for a pile under negative skin friction, as well as|#########################|
| friction only.  See the user's manual for assumptio|#Press <F10> to validate#|
| program, details of the analysis method and applica|#########################|
 ------------------------------------------------------------------------------
          Enter abbreviation for force units (3 characters max.)
```

This screen asks for abbreviations for the force units and length units. Force units must be three characters or less; length units must be two characters. For this example use kN and m. Press function key < Fl0 > to validate.

```
                      ------------------------------
                      | Civil Engineering Department|                          |
 ---------------------|     Texas A&M University    |--------------------------
|                     |     Ver. 1.00/August 1994   |            |             |
|                     ------------------------------             v             |
|                                                               ---            |
|   ANALYSIS OF A########################################    ----| |----       |
|                ########## Loading Option ###########          v| |v          |
|        UNDER NE#------------------------------------#         v| |v          |
|                #| > One top load only           < |#         v| |v          |
|                #|   Load-settlement curve          |#         ^| |^          |
| Written by: H.P#------------------------------------#         ^| |^          |
| Revised by: L.M######################################         ^| |^          |
|                ######################################          ^---^          |
|                                                                 ^            |
 ------------------------------------------------------------------------------
| This program analyzes axially loaded single piles under donwdrag load        |
| based on the static equilibrium of the pile and on the compatibility of the  |
| relative pile-soil movement. The program will compute the neutral point, the |
| maximum force and stress in the pile, and the entire load-settlement curve   |
| for a pile under negative skin friction, as well as for piles under positive |
| friction only.  See the user's manual for assumptions and limitations of the |
| program, details of the analysis method and application to pile groups.      |
 ------------------------------------------------------------------------------
```

This screen is a menu for the selection of the loading option. The option for one top-load only will run a specific top load. The option for the load-settlement curve will generate an entire load-settlement curve with as many points as the user specifies. Press the first letter of the desired option or use the arrow keys to choose the desired option and press < Enter >. For this example choose one top load.

```
                             --------------------------------
                            | Civil Engineering Department |
---------------------------|     Texas A&M University     |----------------------------
|                           |     Ver. 1.00/August 1994    |              |                    |
                             --------------------------------               v
                                                                           ---
     ■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■
     ■■■■■■■■■■■■■■■■■ Friction and Coating Option ■■■■■■■■■■■■■■■■■■■■■
     ■-----------------------------------------------------------------■
     ■| > 1. Negative friction, no bitumen coating                 < |■
     ■|   2. Negative friction, varying bitumen coating length       |■
   Wr■|   3. Negative friction, constant bitumen coating length      |■
   Re■|   4. Positive friction only, no bitumen coating              |■
     ■|   5. Positive friction only, constant bitumen coating length |■
     ■-----------------------------------------------------------------■
-----■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■ -----
| This ■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■           |
| based on the static equilibrium of the pile and on the compatibility of the |
| relative pile-soil movement. The program will compute the neutral point, the |
| maximum force and stress in the pile, and the entire load-settlement curve  |
| for a pile under negative skin friction, as well as for piles under positive |
| friction only.  See the user's manual for assumptions and limitations of the |
| program, details of the analysis method and application to pile groups.     |
---------------------------------------------------------------------------------
```

This screen is a menu for the selection of the shaft resistance and coating option. The options were explained in section A2. The user is again reminded that option 2 with a varying bitumen-coating length means that the depth of the bitumen coating is set to the depth of the neutral point. For the load-settlement option this means that each point on the load-settlement curve corresponds to a different depth of bitumen coating.

For this example choose option 1: negative friction, no bitumen coating.

```
                       ------------------------------
                       | Civil Engineering Department|
-----------------------|    Texas A&M University     |---------------------------
|                      |    Ver. 1.00/August 1994    |          |                |
                       ------------------------------           v                
                                                               ---               
    ANALYSIS OF AXIALLY LOADED SINGLE PILES                ----| |----           
            ######################################################              
         UND#---------------------------------------------------#               
            #| Number of increments on the pile . . . . . . 50   |#               
            #| Top load on the pile (kN ) . . . . .  2.225 E +3  |#               
  Written by#| Multiplier for negative friction . . . .   1.000  |#               
  Revised by#| Multiplier for positive friction . . . .   1.000  |#               
            #---------------------------------------------------#               
            ######################################################              
------------- ######################################################----------
 This program analyzes axially loaded single piles under donwdrag load         
 based on the static equilibrium of the pile and on the compatibility of the   
 relative pile-soil movement. The program will compute the neutral point, the  
 maximum force and stress in the pile, and the entir--------------------------
 for a pile under negative skin friction, as well as|#########################|
 friction only.  See the user's manual for assumptio|#Press <F10> to validate#|
 program, details of the analysis method and applica|#########################|
-------------------------------------------------------------------------------
                 Minimum of 20 increments. Recommend about 50.
```

The input requested in this screen varies depending on the previous two choices of loading option and shaft resistance and coating option. See the data input sheets for the data requested in each case.

For this example, the requested items are the number of increments on the pile and the top load on the pile. The number of increments on the pile must be 20 or more, with a recommended value of 50. For very long piles more increments may be necessary, but for piles up to 150 ft long more increments simply increase the run time (logarithmically) without much increase in accuracy.

Also notice that the top load must be input in exponential notation. This is true of all lengths, forces and stresses in the input and was done in order to use consistent units. The mantissa may contain one digit before the decimal and three digits after the decimal. The exponent may range from -99 to +99.

For this example use the above data and press < F10 > to validate.

```
                               --------------------------------
                              | Civil Engineering Department |
-----------------------------|    Texas A&M University      |--------------------------
|                  ■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■      |          |
|                  ■■■■■■■■■■■■■■■■■ PILE DATA ■■■■■■■■■■■■■■■■■      v          |
|                  ■------------------------------------------------■    ---         |
|     ANALYSIS OF■| Cross sectional area (m2 )  1.450 E -1   |■ -|  |----            |
|                 ■|                                          |■ v|  |v               |
|           UNDER ■| Area of pile point (m2 )    1.450 E -1   |■ v|  |v               |
|                 ■|                                          |■ v|  |v               |
|                 ■| Perimeter (m )              1.390 E +0   |■ ^|  |^               |
|    Written by: H■|                                          |■ ^|  |^               |
|    Revised by: L■| Embedded length (m )        4.176 E +1   |■ ^|  |^               |
|                 ■|                                          |■ ^---^               |
|                 ■| Modulus (kN /m2 )           2.410 E +7   |■   ^                 |
|-----------------■|                                          |■ --------------------|
| This program an■--------------------------------------------■ drag load            |
| based on the st■■■■■■■■■ Use exponential notation. ■■■■■■■■■ tibility of the       |
| relative pile-s ■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■utral point, the     |
| maximum force and stress in the pile, and the entir---------------------------------|
| for a pile under negative skin friction, as well as|■■■■■■■■■■■■■■■■■■■■■■■■■■■■■|
| friction only.  See the user's manual for assumptio|■Press <F10> to validate■     |
| program, details of the analysis method and applica|■■■■■■■■■■■■■■■■■■■■■■■■■■■■■|
--------------------------------------------------------------------------------------
         Input pile cross sectional area. Use exponential notation.
```

This screen asks for pile data. Each of the items must be input in exponential notation. The cross sectional area, perimeter and modulus must be constant with depth.

For this example use the above data and press < F10 > to validate.

```
                   ■■■■■■■ SOIL-PILE FRICTION DATA ■■■■■■
                   ■-------------------------------------■
-------------------■|      |               |   MAXIMUM    |■ ----------------------
|                  ■|      |     DEPTH     |   FRICTION   |■        |
|                  ■| NO.  |     (m )      |  (kN /m2 )   |■        v
|                  ■|------+---------------+--------------|■       ---
|    ANALYSIS OF A■|  1   |  0.000 E +0   |  1.292 E +1  |■    ----|  |----
|                  ■|  2   |  2.286 E +1   |  3.080 E +1  |■       v|  |v
|         UNDER NE■|  3   |  4.176 E +1   |  9.418 E +1  |■       v|  |v
|                  ■|      |        E      |        E     |■       v|  |v
|                  ■|      |        E      |        E     |■       ^|  |^
|   Written by: H.P■|      |        E      |        E     |■       ^|  |^
|   Revised by: L.M■|      |        E      |        E     |■       ^|  |^
|                  ■|      |        E      |        E  -------------------------
|                  ■|      |        E      |        E  |■■■■■■■■Press■■■■■■■■■■■■|
|------------------■|      |        E      |        E  |■  <F7> to delete row   ■|
| This program anal■|      |        E      |        E  |■  <F8> to insert row   ■|
| based on the stat■|      |        E      |        E  |■■■■■■■■■■■■■■■■■■■■■■■■|
| relative pile-soi■|      |        E      |        E  -------------------------
| maximum force and■|      |        E      |        E  -------------------------
| for a pile under ■|      |        E      |        E  |■■■■■■■■■■■■■■■■■■■■■■■■|
| friction only.  S■-------------------------------------|■Press <F10> to validate■|
| program, details ■■■■■ Use exponential notation. ■■|■■■■■■■■■■■■■■■■■■■■■■■■|
------------------- ■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■-------------------------
      Input depth to soil-pile friction data. Use exponential notation.
```

This screen allows the maximum shaft resistance to be entered. The data must start at the ground surface (depth= 0) and continue at least to the depth of the pile point. The profile may go below the pile point. Again the data is entered in exponential notation.

The function key < F7 > will delete a row of data; the function key < F8 > will insert a row of data at the cursor position. The arrow keys may be used to go up or down. The data will scroll up or down if more than 15 data points are input.

After all the data is input correctly, press the function key < F10 > to validate the data.

```
                    ###### SOIL SETTLEMENT PROFILE #######
                    #-----------------------------------#
------------------#|     |             |    Soil      |# -------------------
                  #|     |    DEPTH    |  Settlement  |#         |
                  #| NO. |    (m )     |    (m )      |#         v
                  #|-----+-------------+--------------|#        ---
   ANALYSIS OF A#  |  1  | 0.000 E +0  | 3.350 E -1   |#    ----| |----
                  #|  2  | 6.100 E +0  | 1.650 E -1   |#       v| |v
        UNDER NE#  |  3  | 9.140 E +0  | 1.190 E -1   |#       v| |v
                  #|  4  | 1.219 E +1  | 8.800 E -2   |#       v| |v
                  #|  5  | 1.524 E +1  | 5.800 E -2   |#       ^| |^
 Written by: H.P#  |  6  | 2.134 E +1  | 3.400 E -2   |#       ^| |^
 Revised by: L.M#  |  7  | 4.176 E +1  | 1.500 E -2   |#       ^| |^
                  #|     |         E   |         E    ---------------------
                  #|     |         E   |         E    |########Press#########
------------------#|     |         E   |         E    |#  <F7> to delete row #
This program anal# |     |         E   |         E    |#  <F8> to insert row #
based on the stat# |     |         E   |         E    |######################
relative pile-soi# |     |         E   |         E    ----------------------
maximum force and# |     |         E   |         E    ----------------------
for a pile under # |     |         E   |         E    |######################
friction only.  S#-------------------------------------|#Press <F10> to validate#
program, details ##### Use exponential notation. ##|########################
------------------ #################################----------------------
           Input depth to friction measurement.
```

This screen allows the soil-seftlement profile to be entered. The data must start at the ground surface (depth = 0) and continue at least to the depth of the pile point. The profile may go below the pile point. Again the data is entered in exponential notation.

The function key < F7 > will delete a row of data; the function key < F8 > will insert a row of data at the cursor position. The arrow keys may be used to go up or down. The data will scroll up or down if more than 15 data points are input.

After all the data is input correctly, press the function key < Fl0 > to validate the data.

```
                          ------------------------------
                         | Civil Engineering Department|
------------------------|     Texas A&M University     |--------------------------
                 ############################################     |
                 ########### BEARING SOIL DATA ##############     v
                 #------------------------------------------#    ---
   ANALYSIS OF# | Young's modulus of                        |# -| |----
                #|   bearing soil (kN /m^2 )    2.153 E +4  |# v| |v
      UNDER #   |                                           |# v| |v
                #| Poisson's ratio of                       |# v| |v
                #|   bearing soil                     0.30  |# ^| |^
  Written by: H# |                                          |# ^| |^
  Revised by: L# | Ultimate bearing capacity of             |# ^| |^
                #|   bearing soil (kN /m^2 )    7.097 E +3  |# ^---^
                 #------------------------------------------#     ^
----------------############################################# ----------------
This program an ##############################################drag load
based on the static equilibrium of the pile and on the compatibility of the
relative pile-soil movement. The program will compute the neutral point, the
maximum force and stress in the pile, and the entir-------------------------
for a pile under negative skin friction, as well as|#########################
friction only.  See the user's manual for assumptio|#Press <F10> to validate#
program, details of the analysis method and applica|#########################
-------------------------------------------------------------------------------
            Input soil modulus. Use exponential notation.
```

This screen asks for the bearing soil layer data. After the data is entered correctly, press function key < Fl0 > to validate the data.

EXAMPLE PROBLEM 1

Abbreviation for force units: kN
Abbreviation for length units: m

Loading option: One top load only
Friction and
coating option: Negative friction, no bitumen coating

Number of increments on the pile: 50
Top load on the pile (kN): 2.225 E +3
Multiplier for negative friction: 1.000
Multiplier for positive friction: 1.000

Review input data. Press <F10> to continue.

This screen allows the user to review the input data. If any data is incorrect the user will have a chance to modify the data. Press function key < F10 > to continue.

PILE DATA

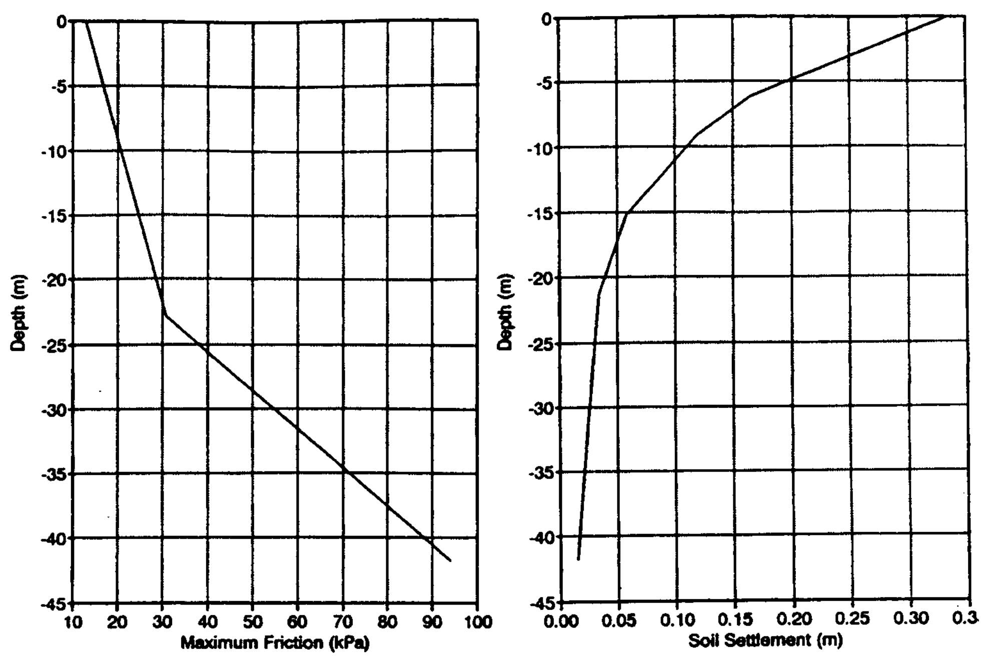
Cross sectional area (m^2)	1.450 E -1
Area of pile point (m^2)	1.450 E -1
Perimeter (m)	1.390 E +0
Embedded length (m)	4.176 E +1
Modulus (kN /m^2)	2.410 E +7

BEARING SOIL DATA

Young's modulus of bearing soil (kN /m^2)	2.153 E +4
Poisson's ratio of bearing soil	0.30
Ultimate bearing capacity of bearing soil (kN /m^2)	7.097 E +3

Review input data. Press <F10> to continue.

This screen allows the user to review the pile and bearing soil data. If any data is incorrect the user will have a chance to modify the data. Press function key < F10 > to continue.



This screen allows a graphical review of the soil-pile friction and soil-settlement profiles. If any data is incorrect the user will have a chance to modify the data. Press function key <Fl0> to continue.

```
                          ---------------------------------
------------------------| Civil Engineering Department |-------------------------
|                       |    Texas A&M University      |              |
|                       |    Ver. 1.00/August 1994     |              v
|                        ------------------------------              ---
|                 ###################################            ----| |----
|   ANALYSIS OF A#-----------------------------------#              v| |v
|                #| >  Continue to run problem   <  |#              v| |v
|       UNDER NE#|     Review and modify data        |#              v| |v
|                #-----------------------------------#              ^| |^
|                 ###################################                ^| |^
|  Written by: H.P ###################################              ^| |^
|  Revised by: L.M.Tucker 1994                                       ^---^
|                                                                      ^
----------------------------------------------------------------------------------
| This program analyzes axially loaded single piles under donwdrag load
| based on the static equilibrium of the pile and on the compatibility of the
| relative pile-soil movement. The program will compute the neutral point, the
| maximum force and stress in the pile, and the entire load-settlement curve
| for a pile under negative skin friction, as well as for piles under positive
| friction only.  See the user's manual for assumptions and limitations of the
| program, details of the analysis method and application to pile groups.
----------------------------------------------------------------------------------
```

After reviewing the data, this menu gives the user the choice of continuing to run the program or of modifying the data. Press the first letter of the desired option or use the arrow keys to choose the desired option and press < Enter>.

```
                                ------------------------------
                               | Civil Engineering Department |
 ---------------------------- |    Texas A&M University      |-----------------------------
|                  ■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■                |
|                  ■------------------------------------------------■                |
|                  ■|              SCREEN OUTPUT                   |■                |
|    ANALYSIS OF A■| >  1. Settlement vs Depth Plot          <   |■ ---            |
|                  ■|    2. Load vs Depth Plot                    |■                |
|        UNDER NE■ |    3. Load-Settlement Curve                 |■                |
|                  ■|    4. View Tabulated Results                |■                |
|                  ■|                                             |■                |
|  Written by: H.P■|         PRINTER/PLOTTER OUTPUT              |■                |
|  Revised by: L.M■|    5. Settlement vs Depth Plot              |■                |
|                  ■|    6. Load vs Depth Plot                    |■                |
|                  ■|    7. Load-Settlement Curve                 |■                |
|------------------■|    8. Print Tabulated Results               |■ -------------  |
| This program anal■|                                             |■ load           |
| based on the stat■|    Run Another Problem                      |■ ity of the     |
| relative pile-soi■|    Quit                                     |■  point, the    |
| maximum force and■------------------------------------------------■ ent curve      |
| for a pile under ■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■ er positive    |
| friction only.  S ■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■■ions of the    |
| program, details of the analysis method and application to pile groups.          |
 -----------------------------------------------------------------------------------
```

This screen allows the user to route the output. The tabulated results may be viewed on the screen or sent to a printer. Also the results may be plotted on the screen or sent to an HP7470 (or compatible) plotter.

After choosing the desired output another problem may be run or the program may be exited. Press the first letter (or number) of the desired option or use the arrow keys to choose the desired option and press < Enter>.

The tabulated and plotted results of this example are shown on the following pages.

EXAMPLE PROBLEM 1

Force units: kN
Length units: m

Loading option:One top load only
Friction and coating option:Negative friction, no bitumen coating

Number of increments on the pile: 50
Top load on the pile (kN): 2225
Multiplier for positive friction: 1
Multiplier for negative friction: 1

PILE DATA

Cross sectional area (sq m): .145
Area of pile point (sq m): .145
Perimeter (m): 1.39
Embedded length (m): 41.76
Modulus (kN/sq m): 2.41E+07

BEARING SOIL DATA

Young's modulus (kN/sq m): 21530
Poisson's ratio: .3
Ultimate bearing capacity (kN/sq m): 7097

SOIL FRICTION AND SETTLEMENT DATA

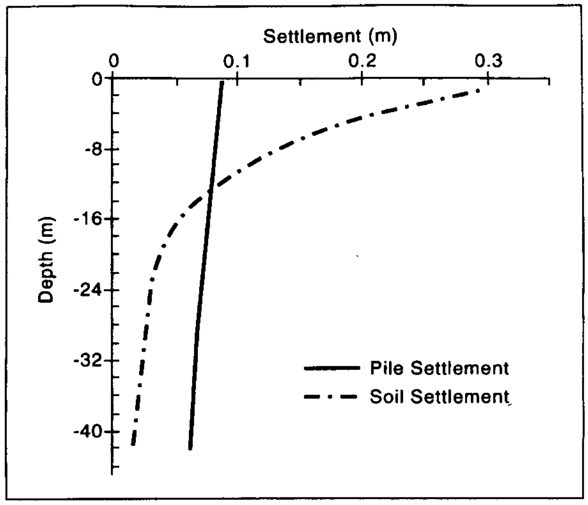
Depth (m)	Friction (kN/sq m)	Depth (m)	Soil Settlement (m)
0.000E+00	0.129E+02	0.000E+00	0.335E+00
0.229E+02	0.308E+02	0.610E+01	0.165E+00
0.418E+02	0.942E+02	0.914E+01	0.119E+00
		0.122E+02	0.880E-01
		0.152E+02	0.580E-01
		0.213E+02	0.340E-01
		0.418E+02	0.150E-01

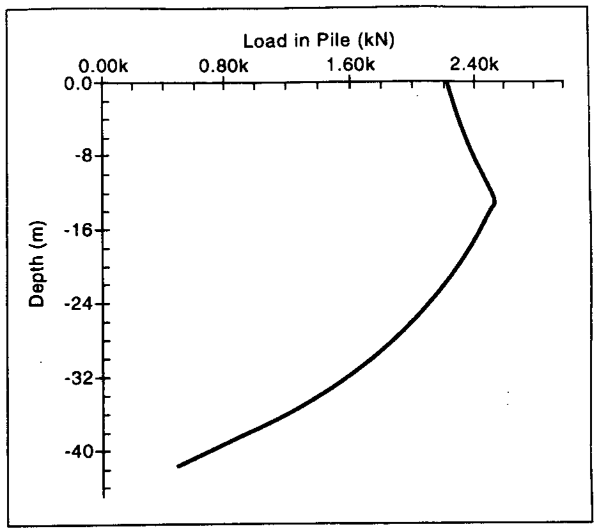
RESULTS OF THE ANALYSIS

Top Load (kN)	Top Settlement (m)	Depth to Bitumen Coating (m)	Depth to Neutral Point (m)	Maximum Load in Pile (kN)	Maximum Stress in Pile (kN/sq m)	Point Load (kN)
2.225E+03	9.047E-02	0.000E+00	1.282E+01	2.545E+03	1.755E+04	5.279E+02

Top Load = 2225 kN

Depth (m)	Axial Force (kN)	Axial Stress (kN/sq m)	Soil Settlement (m)	Pile Settlement (m)
0.000E+00	2.225E+03	1.534E+04	3.200E-01	9.047E-02
8.352E-01	2.240E+03	1.545E+04	2.967E-01	8.994E-02
1.670E+00	2.257E+03	1.556E+04	2.734E-01	8.940E-02
2.506E+00	2.273E+03	1.568E+04	2.502E-01	8.886E-02
3.341E+00	2.291E+03	1.580E+04	2.269E-01	8.831E-02
4.176E+00	2.309E+03	1.593E+04	2.036E-01	8.776E-02
5.011E+00	2.329E+03	1.606E+04	1.803E-01	8.721E-02
5.846E+00	2.349E+03	1.620E+04	1.571E-01	8.665E-02
6.682E+00	2.369E+03	1.634E+04	1.412E-01	8.609E-02
7.517E+00	2.391E+03	1.649E+04	1.286E-01	8.552E-02
8.352E+00	2.413E+03	1.664E+04	1.159E-01	8.494E-02
9.187E+00	2.436E+03	1.680E+04	1.035E-01	8.436E-02
1.002E+01	2.460E+03	1.696E+04	9.503E-02	8.378E-02
1.086E+01	2.484E+03	1.713E+04	8.654E-02	8.319E-02
1.169E+01	2.509E+03	1.731E+04	7.805E-02	8.259E-02
1.253E+01	2.535E+03	1.748E+04	6.968E-02	8.199E-02
1.336E+01	2.527E+03	1.743E+04	6.146E-02	8.138E-02
1.420E+01	2.500E+03	1.724E+04	5.325E-02	8.078E-02
1.503E+01	2.472E+03	1.705E+04	4.503E-02	8.019E-02
1.587E+01	2.443E+03	1.685E+04	4.053E-02	7.960E-02
1.670E+01	2.413E+03	1.664E+04	3.724E-02	7.902E-02
1.754E+01	2.382E+03	1.643E+04	3.395E-02	7.845E-02
1.837E+01	2.351E+03	1.621E+04	3.067E-02	7.788E-02
1.921E+01	2.319E+03	1.599E+04	2.738E-02	7.732E-02
2.004E+01	2.286E+03	1.577E+04	2.410E-02	7.677E-02
2.088E+01	2.253E+03	1.553E+04	2.081E-02	7.623E-02
2.172E+01	2.218E+03	1.530E+04	1.865E-02	7.570E-02
2.255E+01	2.183E+03	1.506E+04	1.787E-02	7.517E-02
2.339E+01	2.146E+03	1.480E+04	1.710E-02	7.465E-02
2.422E+01	2.107E+03	1.453E+04	1.632E-02	7.414E-02
2.506E+01	2.064E+03	1.424E+04	1.554E-02	7.365E-02
2.589E+01	2.018E+03	1.392E+04	1.477E-02	7.316E-02
2.673E+01	1.969E+03	1.358E+04	1.399E-02	7.268E-02
2.756E+01	1.917E+03	1.322E+04	1.321E-02	7.222E-02
2.840E+01	1.861E+03	1.284E+04	1.243E-02	7.177E-02
2.923E+01	1.802E+03	1.243E+04	1.166E-02	7.133E-02
3.007E+01	1.740E+03	1.200E+04	1.088E-02	7.090E-02
3.090E+01	1.675E+03	1.155E+04	1.010E-02	7.050E-02
3.174E+01	1.606E+03	1.107E+04	9.325E-03	7.010E-02
3.257E+01	1.534E+03	1.058E+04	8.548E-03	6.973E-02
3.341E+01	1.459E+03	1.006E+04	7.771E-03	6.937E-02
3.424E+01	1.380E+03	9.519E+03	6.994E-03	6.903E-02
3.508E+01	1.299E+03	8.956E+03	6.217E-03	6.871E-02
3.591E+01	1.214E+03	8.370E+03	5.440E-03	6.841E-02
3.675E+01	1.125E+03	7.762E+03	4.663E-03	6.813E-02
3.758E+01	1.034E+03	7.131E+03	3.886E-03	6.787E-02
3.842E+01	9.393E+02	6.478E+03	3.108E-03	6.764E-02
3.925E+01	8.413E+02	5.802E+03	2.331E-03	6.743E-02
4.009E+01	7.401E+02	5.104E+03	1.554E-03	6.724E-02
4.092E+01	6.357E+02	4.384E+03	7.771E-04	6.707E-02
4.176E+01	5.279E+02	3.641E+03	0.000E+00	6.693E-02


Settlement (m)
0
0.1
0.2
0.3
0
-8
-16
-24
-32
-40
Depth (m)
Pile Settlement
Soil Settlement


Load in Pile (kN)
0.00k
0.80k
1.60k
2.40k
0.0
-8
-16
-24
-32
-40
Depth (m)

A6.2 One Top-Load, Negative Friction, Varying Bitumen Coating Length

```
EXAMPLE PROBLEM 2

Force units:  kN
Length units: m

Loading option:One top load only
Friction and coating option:Negative friction, varying bitumen coating length

Number of increments on the pile:  50
Top load on the pile (kN):  2225
Shear strength of bitumen (kN/sq m):  2
Maximum number of iterations to find
    the depth of the bitumen coating:  10
Tolerance for convergence of neutral point (m):  .15
Multiplier for positive friction:  1
Multiplier for negative friction:  1

            PILE DATA

Cross sectional area (sq m):  .145
Area of pile point (sq m):  .145
Perimeter (m):  1.39
Embedded length (m):  41.76
Modulus (kN/sq m):  2.41E+07

        BEARING SOIL DATA

Young's modulus (kN/sq m):  21530
Poisson's ratio:  .3
Ultimate bearing capacity (kN/sq m):  7097
```

SOIL FRICTION AND SETTLEMENT DATA

Depth (m)	Friction (kN/sq m)	Depth (m)	Soil Settlement (m)
0.000E+00	0.129E+02	0.000E+00	0.335E+00
0.229E+02	0.308E+02	0.610E+01	0.165E+00
0.418E+02	0.942E+02	0.914E+01	0.119E+00
		0.122E+02	0.880E-01
		0.152E+02	0.580E-01
		0.213E+02	0.340E-01
		0.418E+02	0.150E-01

RESULTS OF THE ANALYSIS

Top Load (kN)	Top Settlement (m)	Depth to Bitumen Coating (m)	Depth to Neutral Point (m)	Maximum Load in Pile (kN)	Maximum Stress in Pile (kN/sq m)	Point Load (kN)
2.225E+03	6.959E-02	1.503E+01	1.505E+01	2.267E+03	1.564E+04	3.369E+02

Top Load = 2225 kN

Depth (m)	Axial Force (kN)	Axial Stress (kN/sq m)	Soil Settlement (m)	Pile Settlement (m)
0.000E+00	2.225E+03	1.534E+04	3.200E-01	6.959E-02
8.352E-01	2.227E+03	1.536E+04	2.967E-01	6.906E-02
1.670E+00	2.230E+03	1.538E+04	2.734E-01	6.853E-02
2.506E+00	2.232E+03	1.539E+04	2.502E-01	6.800E-02
3.341E+00	2.234E+03	1.541E+04	2.269E-01	6.746E-02
4.176E+00	2.237E+03	1.542E+04	2.036E-01	6.693E-02
5.011E+00	2.239E+03	1.544E+04	1.803E-01	6.639E-02
5.846E+00	2.241E+03	1.546E+04	1.571E-01	6.586E-02
6.682E+00	2.244E+03	1.547E+04	1.412E-01	6.532E-02
7.517E+00	2.246E+03	1.549E+04	1.286E-01	6.478E-02
8.352E+00	2.248E+03	1.550E+04	1.159E-01	6.425E-02
9.187E+00	2.251E+03	1.552E+04	1.035E-01	6.371E-02
1.002E+01	2.253E+03	1.554E+04	9.503E-02	6.317E-02
1.086E+01	2.255E+03	1.555E+04	8.654E-02	6.263E-02
1.169E+01	2.258E+03	1.557E+04	7.805E-02	6.209E-02
1.253E+01	2.260E+03	1.559E+04	6.968E-02	6.155E-02
1.336E+01	2.262E+03	1.560E+04	6.146E-02	6.101E-02
1.420E+01	2.264E+03	1.562E+04	5.325E-02	6.047E-02
1.503E+01	2.267E+03	1.563E+04	4.503E-02	5.993E-02
1.587E+01	2.252E+03	1.553E+04	4.053E-02	5.939E-02
1.670E+01	2.222E+03	1.532E+04	3.724E-02	5.886E-02
1.754E+01	2.191E+03	1.511E+04	3.395E-02	5.833E-02
1.837E+01	2.160E+03	1.490E+04	3.067E-02	5.781E-02
1.921E+01	2.128E+03	1.468E+04	2.738E-02	5.730E-02
2.004E+01	2.095E+03	1.445E+04	2.410E-02	5.679E-02
2.088E+01	2.062E+03	1.422E+04	2.081E-02	5.630E-02
2.172E+01	2.027E+03	1.398E+04	1.865E-02	5.581E-02
2.255E+01	1.992E+03	1.374E+04	1.787E-02	5.533E-02
2.339E+01	1.956E+03	1.349E+04	1.710E-02	5.485E-02
2.422E+01	1.916E+03	1.321E+04	1.632E-02	5.439E-02
2.506E+01	1.873E+03	1.292E+04	1.554E-02	5.394E-02
2.589E+01	1.828E+03	1.260E+04	1.477E-02	5.350E-02
2.673E+01	1.778E+03	1.226E+04	1.399E-02	5.307E-02
2.756E+01	1.726E+03	1.190E+04	1.321E-02	5.265E-02
2.840E+01	1.670E+03	1.152E+04	1.243E-02	5.224E-02
2.923E+01	1.611E+03	1.111E+04	1.166E-02	5.185E-02
3.007E+01	1.549E+03	1.068E+04	1.088E-02	5.147E-02
3.090E+01	1.484E+03	1.023E+04	1.010E-02	5.111E-02
3.174E+01	1.415E+03	9.758E+03	9.325E-03	5.076E-02
3.257E+01	1.343E+03	9.262E+03	8.548E-03	5.043E-02
3.341E+01	1.268E+03	8.743E+03	7.771E-03	5.012E-02
3.424E+01	1.189E+03	8.202E+03	6.994E-03	4.983E-02
3.508E+01	1.108E+03	7.639E+03	6.217E-03	4.955E-02
3.591E+01	1.023E+03	7.053E+03	5.440E-03	4.930E-02
3.675E+01	9.344E+02	6.444E+03	4.663E-03	4.906E-02
3.758E+01	8.430E+02	5.814E+03	3.886E-03	4.885E-02
3.842E+01	7.483E+02	5.160E+03	3.108E-03	4.866E-02
3.925E+01	6.503E+02	4.485E+03	2.331E-03	4.849E-02
4.009E+01	5.491E+02	3.787E+03	1.554E-03	4.835E-02
4.092E+01	4.446E+02	3.066E+03	7.771E-04	4.823E-02
4.176E+01	3.369E+02	2.323E+03	0.000E+00	4.814E-02

A6.3 One Top-Load, Negative Friction, Constant Bitumen Coating Length

EXAMPLE PROBLEM 3

Force units: kN
Length units: m

Loading option:One top load only
Friction and coating option:Negative friction, constant bitumen coating length

Number of increments on the pile: 50
Top load on the pile (kN): 2225
Shear strength of bitumen (kN/sq m): 2
Depth of bitumen (m): 23
Multiplier for positive friction: 1
Multiplier for negative friction: 1

PILE DATA

Cross sectional area (sq m): .145
Area of pile point (sq m): .145
Perimeter (m): 1.39
Embedded length (m): 41.76
Modulus (kN/sq m): 2.41E+07

BEARING SOIL DATA

Young's modulus (kN/sq m): 21530
Poisson's ratio: .3
Ultimate bearing capacity (kN/sq m): 7097

SOIL FRICTION AND SETTLEMENT DATA

Depth (m)	Friction (kN/sq m)	Depth (m)	Soil Settlement (m)
0.000E+00	0.129E+02	0.000E+00	0.335E+00
0.229E+02	0.308E+02	0.610E+01	0.165E+00
0.418E+02	0.942E+02	0.914E+01	0.119E+00
		0.122E+02	0.880E-01
		0.152E+02	0.580E-01
		0.213E+02	0.340E-01
		0.418E+02	0.150E-01

RESULTS OF THE ANALYSIS

Top Load (kN)	Top Settlement (m)	Depth to Bitumen Coating (m)	Depth to Neutral Point (m)	Maximum Load in Pile (kN)	Maximum Stress in Pile (kN/sq m)	Point Load (kN)
2.225E+03	9.967E-02	2.339E+01	1.179E+01	2.258E+03	1.557E+04	6.246E+02

Top Load = 2225 kN

Depth (m)	Axial Force (kN)	Axial Stress (kN/sq m)	Soil Settlement (m)	Pile Settlement (m)
0.000E+00	2.225E+03	1.534E+04	3.200E-01	9.967E-02
8.352E-01	2.227E+03	1.536E+04	2.967E-01	9.914E-02
1.670E+00	2.230E+03	1.538E+04	2.734E-01	9.860E-02
2.506E+00	2.232E+03	1.539E+04	2.502E-01	9.807E-02
3.341E+00	2.234E+03	1.541E+04	2.269E-01	9.754E-02
4.176E+00	2.237E+03	1.542E+04	2.036E-01	9.700E-02
5.011E+00	2.239E+03	1.544E+04	1.803E-01	9.647E-02
5.846E+00	2.241E+03	1.546E+04	1.571E-01	9.593E-02
6.682E+00	2.244E+03	1.547E+04	1.412E-01	9.540E-02
7.517E+00	2.246E+03	1.549E+04	1.286E-01	9.486E-02
8.352E+00	2.248E+03	1.550E+04	1.159E-01	9.432E-02
9.187E+00	2.251E+03	1.552E+04	1.035E-01	9.378E-02
1.002E+01	2.253E+03	1.554E+04	9.503E-02	9.325E-02
1.086E+01	2.255E+03	1.555E+04	8.654E-02	9.271E-02
1.169E+01	2.258E+03	1.557E+04	7.805E-02	9.217E-02
1.253E+01	2.256E+03	1.556E+04	6.968E-02	9.163E-02
1.336E+01	2.253E+03	1.554E+04	6.146E-02	9.109E-02
1.420E+01	2.251E+03	1.552E+04	5.325E-02	9.055E-02
1.503E+01	2.249E+03	1.551E+04	4.503E-02	9.001E-02
1.587E+01	2.246E+03	1.549E+04	4.053E-02	8.948E-02
1.670E+01	2.244E+03	1.548E+04	3.724E-02	8.894E-02
1.754E+01	2.242E+03	1.546E+04	3.395E-02	8.840E-02
1.837E+01	2.239E+03	1.544E+04	3.067E-02	8.787E-02
1.921E+01	2.237E+03	1.543E+04	2.738E-02	8.733E-02
2.004E+01	2.235E+03	1.541E+04	2.410E-02	8.680E-02
2.088E+01	2.232E+03	1.540E+04	2.081E-02	8.627E-02
2.172E+01	2.230E+03	1.538E+04	1.865E-02	8.573E-02
2.255E+01	2.228E+03	1.536E+04	1.787E-02	8.520E-02
2.339E+01	2.226E+03	1.535E+04	1.710E-02	8.467E-02
2.422E+01	2.204E+03	1.520E+04	1.632E-02	8.414E-02
2.506E+01	2.161E+03	1.490E+04	1.554E-02	8.362E-02
2.589E+01	2.115E+03	1.459E+04	1.477E-02	8.311E-02
2.673E+01	2.066E+03	1.425E+04	1.399E-02	8.261E-02
2.756E+01	2.014E+03	1.389E+04	1.321E-02	8.212E-02
2.840E+01	1.958E+03	1.350E+04	1.243E-02	8.164E-02
2.923E+01	1.899E+03	1.310E+04	1.166E-02	8.118E-02
3.007E+01	1.837E+03	1.267E+04	1.088E-02	8.074E-02
3.090E+01	1.771E+03	1.222E+04	1.010E-02	8.031E-02
3.174E+01	1.703E+03	1.174E+04	9.325E-03	7.989E-02
3.257E+01	1.631E+03	1.125E+04	8.548E-03	7.949E-02
3.341E+01	1.555E+03	1.073E+04	7.771E-03	7.911E-02
3.424E+01	1.477E+03	1.019E+04	6.994E-03	7.875E-02
3.508E+01	1.395E+03	9.623E+03	6.217E-03	7.841E-02
3.591E+01	1.310E+03	9.037E+03	5.440E-03	7.808E-02
3.675E+01	1.222E+03	8.429E+03	4.663E-03	7.778E-02
3.758E+01	1.131E+03	7.798E+03	3.886E-03	7.750E-02
3.842E+01	1.036E+03	7.145E+03	3.108E-03	7.724E-02
3.925E+01	9.380E+02	6.469E+03	2.331E-03	7.700E-02
4.009E+01	8.368E+02	5.771E+03	1.554E-03	7.679E-02
4.092E+01	7.323E+02	5.051E+03	7.771E-04	7.660E-02
4.176E+01	6.246E+02	4.308E+03	0.000E+00	7.644E-02

A6.4 One Top-Load, Positive Friction, No Bitumen Coating

EXAMPLE PROBLEM 4

Force units: kN
Length units: m

Loading option:One top load only
Friction and coating option:Positive friction only, no bitumen coating

Number of increments on the pile: 50
Top load on the pile (kN): 2225
Multiplier for positive friction: 1
Multiplier for negative friction: 1

PILE DATA

Cross sectional area (sq m): .145
Area of pile point (sq m): .145
Perimeter (m): 1.39
Embedded length (m): 41.76
Modulus (kN/sq m): 2.41E+07

BEARING SOIL DATA

Young's modulus (kN/sq m): 21530
Poisson's ratio: .3
Ultimate bearing capacity (kN/sq m): 7097

SOIL FRICTION AND SETTLEMENT DATA

Depth (m)	Friction (kN/sq m)	Depth (m)	Soil Settlement (m)
0.000E+00	0.129E+02	0.000E+00	0.335E+00
0.229E+02	0.308E+02	0.610E+01	0.165E+00
0.418E+02	0.942E+02	0.914E+01	0.119E+00
		0.122E+02	0.880E-01
		0.152E+02	0.580E-01
		0.213E+02	0.340E-01
		0.418E+02	0.150E-01

RESULTS OF THE ANALYSIS

Top Load (kN)	Top Settlement (m)	Depth to Bitumen Coating (m)	Depth to Neutral Point (m)	Maximum Load in Pile (kN)	Maximum Stress in Pile (kN/sq m)	Point Load (kN)
2.225E+03	1.719E-02	0.000E+00	0.000E+00	2.225E+03	1.534E+04	0.000E+00

Top Load = 2225 kN

Depth (m)	Axial Force (kN)	Axial Stress (kN/sq m)	Soil Settlement (m)	Pile Settlement (m)
0.000E+00	2.225E+03	1.534E+04	3.200E-01	1.719E-02
8.352E-01	2.210E+03	1.524E+04	2.967E-01	1.666E-02
1.670E+00	2.193E+03	1.513E+04	2.734E-01	1.614E-02
2.506E+00	2.177E+03	1.501E+04	2.502E-01	1.561E-02
3.341E+00	2.159E+03	1.489E+04	2.269E-01	1.509E-02
4.176E+00	2.141E+03	1.476E+04	2.036E-01	1.458E-02
5.011E+00	2.121E+03	1.463E+04	1.803E-01	1.407E-02
5.846E+00	2.101E+03	1.449E+04	1.571E-01	1.357E-02
6.682E+00	2.081E+03	1.435E+04	1.412E-01	1.307E-02
7.517E+00	2.059E+03	1.420E+04	1.286E-01	1.257E-02
8.352E+00	2.037E+03	1.405E+04	1.159E-01	1.208E-02
9.187E+00	2.014E+03	1.389E+04	1.035E-01	1.160E-02
1.002E+01	1.990E+03	1.373E+04	9.503E-02	1.112E-02
1.086E+01	1.966E+03	1.356E+04	8.654E-02	1.065E-02
1.169E+01	1.941E+03	1.338E+04	7.805E-02	1.018E-02
1.253E+01	1.915E+03	1.320E+04	6.968E-02	9.720E-03
1.336E+01	1.888E+03	1.302E+04	6.146E-02	9.266E-03
1.420E+01	1.860E+03	1.283E+04	5.325E-02	8.818E-03
1.503E+01	1.832E+03	1.264E+04	4.503E-02	8.376E-03
1.587E+01	1.803E+03	1.244E+04	4.053E-02	7.942E-03
1.670E+01	1.773E+03	1.223E+04	3.724E-02	7.515E-03
1.754E+01	1.743E+03	1.202E+04	3.395E-02	7.094E-03
1.837E+01	1.711E+03	1.180E+04	3.067E-02	6.682E-03
1.921E+01	1.679E+03	1.158E+04	2.738E-02	6.276E-03
2.004E+01	1.647E+03	1.136E+04	2.410E-02	5.879E-03
2.088E+01	1.613E+03	1.112E+04	2.081E-02	5.489E-03
2.172E+01	1.579E+03	1.089E+04	1.865E-02	5.108E-03
2.255E+01	1.544E+03	1.065E+04	1.787E-02	4.735E-03
2.339E+01	1.507E+03	1.039E+04	1.710E-02	4.370E-03
2.422E+01	1.468E+03	1.012E+04	1.632E-02	4.015E-03
2.506E+01	1.425E+03	9.826E+03	1.554E-02	3.669E-03
2.589E+01	1.379E+03	9.510E+03	1.477E-02	3.334E-03
2.673E+01	1.330E+03	9.171E+03	1.399E-02	3.010E-03
2.756E+01	1.277E+03	8.809E+03	1.321E-02	2.699E-03
2.840E+01	1.222E+03	8.425E+03	1.243E-02	2.400E-03
2.923E+01	1.163E+03	8.018E+03	1.166E-02	2.115E-03
3.007E+01	1.100E+03	7.589E+03	1.088E-02	1.845E-03
3.090E+01	1.035E+03	7.138E+03	1.010E-02	1.590E-03
3.174E+01	9.663E+02	6.664E+03	9.325E-03	1.351E-03
3.257E+01	8.944E+02	6.168E+03	8.548E-03	1.128E-03
3.341E+01	8.192E+02	5.649E+03	7.771E-03	9.234E-04
3.424E+01	7.407E+02	5.108E+03	6.994E-03	7.370E-04
3.508E+01	6.590E+02	4.545E+03	6.217E-03	5.697E-04
3.591E+01	5.741E+02	3.959E+03	5.440E-03	4.224E-04
3.675E+01	4.858E+02	3.351E+03	4.663E-03	2.957E-04
3.758E+01	3.944E+02	2.720E+03	3.886E-03	1.905E-04
3.842E+01	2.997E+02	2.067E+03	3.108E-03	1.076E-04
3.925E+01	2.017E+02	1.391E+03	2.331E-03	4.766E-05
4.009E+01	1.005E+02	6.930E+02	1.554E-03	1.155E-05
4.092E+01	0.000E+00	0.000E+00	7.771E-04	0.000E+00
4.176E+01	0.000E+00	0.000E+00	0.000E+00	0.000E+00

A6.5 One Top-Load, Positive Friction, Constant Bitumen Coating Length

EXAMPLE PROBLEM 5

Force units: kN
Length units: m

Loading option:One top load only
Friction and coating option:Positive friction only, constant bitumen coating length

Number of increments on the pile: 50
Top load on the pile (kN): 2225
Shear strength of bitumen (kN/sq m): 2
Depth of bitumen (m): 23
Multiplier for positive friction: 1
Multiplier for negative friction: 1

PILE DATA

Cross sectional area (sq m): .145
Area of pile point (sq m): .145
Perimeter (m): 1.39
Embedded length (m): 41.76
Modulus (kN/sq m): 2.41E+07

BEARING SOIL DATA

Young's modulus (kN/sq m): 21530
Poisson's ratio: .3
Ultimate bearing capacity (kN/sq m): 7097

SOIL FRICTION AND SETTLEMENT DATA

Depth (m)	Friction (kN/sq m)	Depth (m)	Soil Settlement (m)
0.000E+00	0.129E+02	0.000E+00	0.335E+00
0.229E+02	0.308E+02	0.610E+01	0.165E+00
0.418E+02	0.942E+02	0.914E+01	0.119E+00
		0.122E+02	0.880E-01
		0.152E+02	0.580E-01
		0.213E+02	0.340E-01
		0.418E+02	0.150E-01

RESULTS OF THE ANALYSIS

Top Load (kN)	Top Settlement (m)	Depth to Bitumen Coating (m)	Depth to Neutral Point (m)	Maximum Load in Pile (kN)	Maximum Stress in Pile (kN/sq m)	Point Load (kN)
2.225E+03	7.755E-02	2.339E+01	0.000E+00	2.225E+03	1.534E+04	5.591E+02

Top Load = 2225 kN

Depth (m)	Axial Force (kN)	Axial Stress (kN/sq m)	Soil Settlement (m)	Pile Settlement (m)
0.000E+00	2.225E+03	1.534E+04	3.200E-01	7.755E-02
8.352E-01	2.223E+03	1.533E+04	2.967E-01	7.702E-02
1.670E+00	2.220E+03	1.531E+04	2.734E-01	7.649E-02
2.506E+00	2.218E+03	1.530E+04	2.502E-01	7.596E-02
3.341E+00	2.216E+03	1.528E+04	2.269E-01	7.543E-02
4.176E+00	2.213E+03	1.526E+04	2.036E-01	7.490E-02
5.011E+00	2.211E+03	1.525E+04	1.803E-01	7.437E-02
5.846E+00	2.209E+03	1.523E+04	1.571E-01	7.384E-02
6.682E+00	2.206E+03	1.522E+04	1.412E-01	7.331E-02
7.517E+00	2.204E+03	1.520E+04	1.286E-01	7.279E-02
8.352E+00	2.202E+03	1.518E+04	1.159E-01	7.226E-02
9.187E+00	2.199E+03	1.517E+04	1.035E-01	7.173E-02
1.002E+01	2.197E+03	1.515E+04	9.503E-02	7.121E-02
1.086E+01	2.195E+03	1.514E+04	8.654E-02	7.068E-02
1.169E+01	2.192E+03	1.512E+04	7.805E-02	7.016E-02
1.253E+01	2.190E+03	1.510E+04	6.968E-02	6.963E-02
1.336E+01	2.188E+03	1.509E+04	6.146E-02	6.911E-02
1.420E+01	2.186E+03	1.507E+04	5.325E-02	6.859E-02
1.503E+01	2.183E+03	1.506E+04	4.503E-02	6.807E-02
1.587E+01	2.181E+03	1.504E+04	4.053E-02	6.755E-02
1.670E+01	2.179E+03	1.502E+04	3.724E-02	6.702E-02
1.754E+01	2.176E+03	1.501E+04	3.395E-02	6.650E-02
1.837E+01	2.174E+03	1.499E+04	3.067E-02	6.598E-02
1.921E+01	2.172E+03	1.498E+04	2.738E-02	6.546E-02
2.004E+01	2.169E+03	1.496E+04	2.410E-02	6.495E-02
2.088E+01	2.167E+03	1.494E+04	2.081E-02	6.443E-02
2.172E+01	2.165E+03	1.493E+04	1.865E-02	6.391E-02
2.255E+01	2.162E+03	1.491E+04	1.787E-02	6.339E-02
2.339E+01	2.160E+03	1.490E+04	1.710E-02	6.288E-02
2.422E+01	2.138E+03	1.475E+04	1.632E-02	6.236E-02
2.506E+01	2.096E+03	1.445E+04	1.554E-02	6.186E-02
2.589E+01	2.050E+03	1.414E+04	1.477E-02	6.136E-02
2.673E+01	2.001E+03	1.380E+04	1.399E-02	6.088E-02
2.756E+01	1.948E+03	1.343E+04	1.321E-02	6.041E-02
2.840E+01	1.892E+03	1.305E+04	1.243E-02	5.995E-02
2.923E+01	1.833E+03	1.264E+04	1.166E-02	5.950E-02
3.007E+01	1.771E+03	1.222E+04	1.088E-02	5.907E-02
3.090E+01	1.706E+03	1.176E+04	1.010E-02	5.866E-02
3.174E+01	1.637E+03	1.129E+04	9.325E-03	5.826E-02
3.257E+01	1.565E+03	1.079E+04	8.548E-03	5.787E-02
3.341E+01	1.490E+03	1.028E+04	7.771E-03	5.751E-02
3.424E+01	1.411E+03	9.734E+03	6.994E-03	5.716E-02
3.508E+01	1.330E+03	9.171E+03	6.217E-03	5.683E-02
3.591E+01	1.245E+03	8.585E+03	5.440E-03	5.653E-02
3.675E+01	1.157E+03	7.977E+03	4.663E-03	5.624E-02
3.758E+01	1.065E+03	7.346E+03	3.886E-03	5.597E-02
3.842E+01	9.704E+02	6.693E+03	3.108E-03	5.573E-02
3.925E+01	8.725E+02	6.017E+03	2.331E-03	5.551E-02
4.009E+01	7.713E+02	5.319E+03	1.554E-03	5.531E-02
4.092E+01	6.668E+02	4.599E+03	7.771E-04	5.514E-02
4.176E+01	5.591E+02	3.856E+03	0.000E+00	5.500E-02

A6.6 Load-Settlement Curve, Negative Friction, No Bitumen Coating

EXAMPLE PROBLEM 6

Force units: kN
Length units: m

Loading option:Load-settlement curve
Friction and coating option:Negative friction, no bitumen coating

Number of increments on the pile: 50
Number of top loads: 10
Multiplier for positive friction: 1
Multiplier for negative friction: 1

PILE DATA

Cross sectional area (sq m): .145
Area of pile point (sq m): .145
Perimeter (m): 1.39
Embedded length (m): 41.76
Modulus (kN/sq m): 2.41E+07

BEARING SOIL DATA

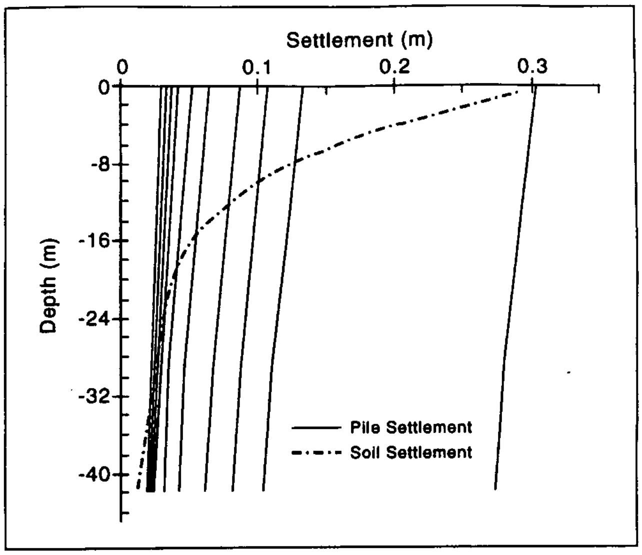
Young's modulus (kN/sq m): 21530
Poisson's ratio: .3
Ultimate bearing capacity (kN/sq m): 7097

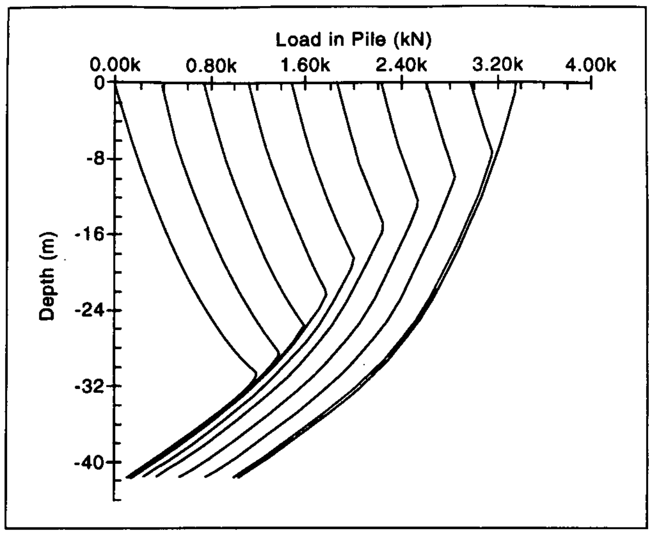
SOIL FRICTION AND SETTLEMENT DATA

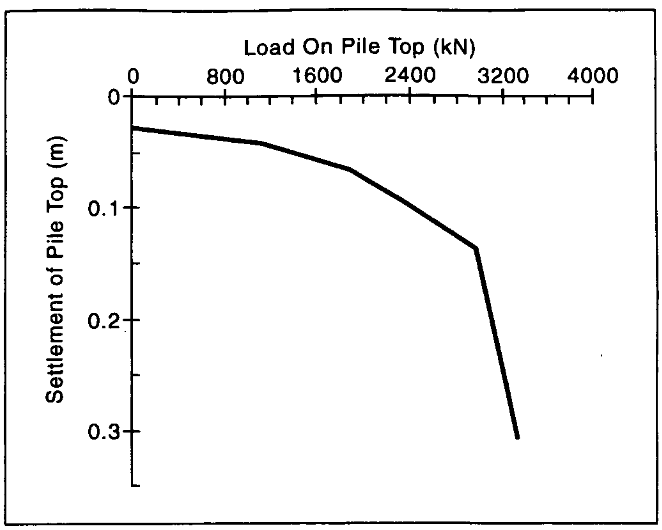
Depth (m)	Friction (kN/sq m)	Depth (m)	Soil Settlement (m)
0.000E+00	0.129E+02	0.000E+00	0.335E+00
0.229E+02	0.308E+02	0.610E+01	0.165E+00
0.418E+02	0.942E+02	0.914E+01	0.119E+00
		0.122E+02	0.880E-01
		0.152E+02	0.580E-01
		0.213E+02	0.340E-01
		0.418E+02	0.150E-01

RESULTS OF THE ANALYSIS

Top Load (kN)	Top Settlement (m)	Depth to Bitumen Coating (m)	Depth to Neutral Point (m)	Maximum Load in Pile (kN)	Maximum Stress in Pile (kN/sq m)	Point Load (kN)
0.000E+00	2.905E-02	0.000E+00	3.112E+01	1.208E+03	8.331E+03	7.940E+01
3.723E+02	3.357E-02	0.000E+00	2.875E+01	1.401E+03	9.659E+03	9.237E+01
7.445E+02	3.789E-02	0.000E+00	2.594E+01	1.594E+03	1.099E+04	1.061E+02
1.117E+03	4.209E-02	0.000E+00	2.230E+01	1.788E+03	1.233E+04	1.218E+02
1.489E+03	5.339E-02	0.000E+00	1.876E+01	2.017E+03	1.391E+04	2.090E+02
1.861E+03	6.766E-02	0.000E+00	1.527E+01	2.262E+03	1.560E+04	3.271E+02
2.234E+03	9.102E-02	0.000E+00	1.277E+01	2.551E+03	1.760E+04	5.328E+02
2.606E+03	1.157E-01	0.000E+00	1.025E+01	2.847E+03	1.963E+04	7.518E+02
2.978E+03	1.432E-01	0.000E+00	8.001E+00	3.157E+03	2.177E+04	9.986E+02
3.350E+03	3.238E-01	0.000E+00	4.176E-01	3.358E+03	2.316E+04	1.029E+03


Settlement (m)
0
0.1
0.2
0.3
0
-8
-16
-24
-32
-40
Depth (m)
Pile Settlement
Soil Settlement


Load in Pile (kN)
0.00k
0.80k
1.60k
2.40k
3.20k
4.00k
0
-8
-16
-24
-32
-40
Depth (m)


Load On Pile Top (kN)
0
800
1600
2400
3200
4000
0
0.1
0.2
0.3
Settlement of Pile Top (m)

A6.7 Load-Settlement Curve, Negative Friction, Varying Bitumen Coating Length

EXAMPLE PROBLEM 7

Force units: kN
Length units: m

Loading option:Load-settlement curve
Friction and coating option:Negative friction, varying bitumen coating length

Number of increments on the pile: 50
Number of top loads: 10
Shear strength of bitumen (kN/sq m): 2
Maximum number of iterations to find the depth of the bitumen coating: 10
Tolerance for convergence of neutral point (m): .15
Multiplier for positive friction: 1
Multiplier for negative friction: 1

PILE DATA

Cross sectional area (sq m): .145
Area of pile point (sq m): .145
Perimeter (m): 1.39
Embedded length (m): 41.76
Modulus (kN/sq m): 2.41E+07

BEARING SOIL DATA

Young's modulus (kN/sq m): 21530
Poisson's ratio: .3
Ultimate bearing capacity (kN/sq m): 7097

SOIL FRICTION AND SETTLEMENT DATA

Depth (m)	Friction (kN/sq m)	Depth (m)	Soil Settlement (m)
0.000E+00	0.129E+02	0.000E+00	0.335E+00
0.229E+02	0.308E+02	0.610E+01	0.165E+00
0.418E+02	0.942E+02	0.914E+01	0.119E+00
		0.122E+02	0.880E-01
		0.152E+02	0.580E-01
		0.213E+02	0.340E-01
		0.418E+02	0.150E-01

RESULTS OF THE ANALYSIS

Top Load (kN)	Top Settlement (m)	Depth to Bitumen Coating (m)	Depth to Neutral Point (m)	Maximum Load in Pile (kN)	Maximum Stress in Pile (kN/sq m)	Point Load (kN)
0.000E+00	1.567E-02	4.009E+01	4.029E+01	1.243E+02	8.569E+02	0.000E+00
3.723E+02	2.333E-02	3.758E+01	3.774E+01	4.861E+02	3.352E+03	3.474E+01
7.445E+02	2.955E-02	3.424E+01	3.455E+01	8.556E+02	5.901E+03	5.797E+01
1.117E+03	3.541E-02	3.007E+01	3.082E+01	1.231E+03	8.493E+03	8.112E+01
1.489E+03	4.097E-02	2.589E+01	2.609E+01	1.567E+03	1.081E+04	1.054E+02
1.861E+03	5.051E-02	2.004E+01	1.987E+01	1.917E+03	1.322E+04	1.732E+02
2.234E+03	7.041E-02	1.503E+01	1.496E+01	2.275E+03	1.569E+04	3.443E+02
2.606E+03	9.968E-02	1.169E+01	1.192E+01	2.642E+03	1.822E+04	6.056E+02
2.978E+03	1.310E-01	9.187E+00	8.860E+00	3.003E+03	2.071E+04	8.865E+02
3.350E+03	3.238E-01	0.000E+00	4.176E-01	3.355E+03	2.314E+04	1.029E+03

A6.8 Load-Settlement Curve, Negative Friction, Constant Bitumen Coating Length

```
EXAMPLE PROBLEM 8

Force units:  kN
Length units: m

Loading option:Load-settlement curve
Friction and coating option:Negative friction, constant bitumen
  coating length

Number of increments on the pile:  50
Number of top loads:  10
Shear strength of bitumen (kN/sq m):  2
Depth of bitumen (m):  23
Multiplier for positive friction:  1
Multiplier for negative friction:  1

             PILE DATA

Cross sectional area (sq m):  .145
Area of pile point (sq m):  .145
Perimeter (m):  1.39
Embedded length (m):  41.76
Modulus (kN/sq m):  2.41E+07

         BEARING SOIL DATA

Young's modulus (kN/sq m):  21530
Poisson's ratio:  .3
Ultimate bearing capacity (kN/sq m):  7097
```

SOIL FRICTION AND SETTLEMENT DATA

Depth (m)	Friction (kN/sq m)	Depth (m)	Soil Settlement (m)
0.000E+00	0.129E+02	0.000E+00	0.335E+00
0.229E+02	0.308E+02	0.610E+01	0.165E+00
0.418E+02	0.942E+02	0.914E+01	0.119E+00
		0.122E+02	0.880E-01
		0.152E+02	0.580E-01
		0.213E+02	0.340E-01
		0.418E+02	0.150E-01

RESULTS OF THE ANALYSIS

Top Load (kN)	Top Settlement (m)	Depth to Bitumen Coating (m)	Depth to Neutral Point (m)	Maximum Load in Pile (kN)	Maximum Stress in Pile (kN/sq m)	Point Load (kN)
0.000E+00	2.110E-02	2.339E+01	3.463E+01	8.515E+02	5.872E+03	3.705E+01
2.992E+02	2.546E-02	2.339E+01	3.312E+01	1.008E+03	6.954E+03	5.152E+01
5.984E+02	3.000E-02	2.339E+01	3.150E+01	1.167E+03	8.046E+03	6.914E+01
8.976E+02	3.446E-02	2.339E+01	2.972E+01	1.325E+03	9.140E+03	8.724E+01
1.197E+03	3.801E-02	2.339E+01	2.766E+01	1.480E+03	1.021E+04	9.786E+01
1.496E+03	4.145E-02	2.339E+01	2.524E+01	1.636E+03	1.128E+04	1.093E+02
1.795E+03	5.600E-02	2.339E+01	1.815E+01	1.846E+03	1.273E+04	2.301E+02
2.094E+03	8.609E-02	2.339E+01	1.320E+01	2.131E+03	1.470E+04	5.017E+02
2.393E+03	1.172E-01	2.339E+01	9.996E+00	2.421E+03	1.670E+04	7.831E+02
2.693E+03	3.237E-01	2.339E+01	4.172E-01	2.694E+03	1.858E+04	1.029E+03

A6.9 Load-Settlement Curve, Positive Friction, No Bitumen Coating

```
EXAMPLE PROBLEM 9

Force units:  kN
Length units: m

Loading option:Load-settlement curve
Friction and coating option:Positive friction only, no bitumen coating

Number of increments on the pile:  50
Number of top loads:  10
Multiplier for positive friction:  1
Multiplier for negative friction:  1

              PILE DATA

Cross sectional area (sq m):  .145
Area of pile point (sq m):  .145
Perimeter (m):  1.39
Embedded length (m):  41.76
Modulus (kN/sq m):  2.41E+07

          BEARING SOIL DATA

Young's modulus (kN/sq m):  21530
Poisson's ratio:  .3
Ultimate bearing capacity (kN/sq m):  7097
```

SOIL FRICTION AND SETTLEMENT DATA

Depth (m)	Friction (kN/sq m)	Depth (m)	Soil Settlement (m)
0.000E+00	0.129E+02	0.000E+00	0.335E+00
0.229E+02	0.308E+02	0.610E+01	0.165E+00
0.418E+02	0.942E+02	0.914E+01	0.119E+00
		0.122E+02	0.880E-01
		0.152E+02	0.580E-01
		0.213E+02	0.340E-01
		0.418E+02	0.150E-01

RESULTS OF THE ANALYSIS

Top Load (kN)	Top Settlement (m)	Depth to Bitumen Coating (m)	Depth to Neutral Point (m)	Maximum Load in Pile (kN)	Maximum Stress in Pile (kN/sq m)	Point Load (kN)
0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
3.723E+02	8.461E-04	0.000E+00	0.000E+00	3.723E+02	2.567E+03	0.000E+00
7.445E+02	2.916E-03	0.000E+00	0.000E+00	7.445E+02	5.135E+03	0.000E+00
1.117E+03	5.811E-03	0.000E+00	0.000E+00	1.117E+03	7.702E+03	0.000E+00
1.489E+03	9.243E-03	0.000E+00	0.000E+00	1.489E+03	1.027E+04	0.000E+00
1.861E+03	1.309E-02	0.000E+00	0.000E+00	1.861E+03	1.284E+04	0.000E+00
2.234E+03	1.729E-02	0.000E+00	0.000E+00	2.234E+03	1.540E+04	0.000E+00
2.606E+03	4.821E-02	0.000E+00	0.000E+00	2.606E+03	1.797E+04	2.692E+02
2.978E+03	8.927E-02	0.000E+00	0.000E+00	2.978E+03	2.054E+04	6.414E+02
3.350E+03	1.303E-01	0.000E+00	0.000E+00	3.350E+03	2.311E+04	1.014E+03

A6.10 Load-Settlement Curve, Positive Friction, Constant Bitumen Coating Length

```
EXAMPLE PROBLEM 10

Force units:  kN
Length units: m

Loading option:Load-settlement curve
Friction and coating option:Positive friction only, constant bitumen
   coating length

Number of increments on the pile:  50
Number of top loads:  10
Shear strength of bitumen (kN/sq m):  2
Depth of bitumen (m):  23
Multiplier for positive friction:  1
Multiplier for negative friction:  1

             PILE DATA

Cross sectional area (sq m):  .145
Area of pile point (sq m):  .145
Perimeter (m):  1.39
Embedded length (m):  41.76
Modulus (kN/sq m):  2.41E+07

         BEARING SOIL DATA

Young's modulus (kN/sq m):  21530
Poisson's ratio:  .3
Ultimate bearing capacity (kN/sq m):  7097
```

SOIL FRICTION AND SETTLEMENT DATA

Depth (m)	Friction (kN/sq m)	Depth (m)	Soil Settlement (m)
0.000E+00	0.129E+02	0.000E+00	0.335E+00
0.229E+02	0.308E+02	0.610E+01	0.165E+00
0.418E+02	0.942E+02	0.914E+01	0.119E+00
		0.122E+02	0.880E-01
		0.152E+02	0.580E-01
		0.213E+02	0.340E-01
		0.418E+02	0.150E-01

RESULTS OF THE ANALYSIS

Top Load (kN)	Top Settlement (m)	Depth to Bitumen Coating (m)	Depth to Neutral Point (m)	Maximum Load in Pile (kN)	Maximum Stress in Pile (kN/sq m)	Point Load (kN)
0.000E+00	0.000E+00	2.339E+01	0.000E+00	0.000E+00	0.000E+00	0.000E+00
2.992E+02	1.955E-03	2.339E+01	0.000E+00	2.992E+02	2.063E+03	0.000E+00
5.984E+02	4.520E-03	2.339E+01	0.000E+00	5.984E+02	4.127E+03	0.000E+00
8.976E+02	7.390E-03	2.339E+01	0.000E+00	8.976E+02	6.190E+03	0.000E+00
1.197E+03	1.052E-02	2.339E+01	0.000E+00	1.197E+03	8.253E+03	0.000E+00
1.496E+03	1.387E-02	2.339E+01	0.000E+00	1.496E+03	1.032E+04	0.000E+00
1.795E+03	3.012E-02	2.339E+01	0.000E+00	1.795E+03	1.238E+04	1.292E+02
2.094E+03	6.313E-02	2.339E+01	0.000E+00	2.094E+03	1.444E+04	4.284E+02
2.393E+03	9.614E-02	2.339E+01	0.000E+00	2.393E+03	1.651E+04	7.276E+02
2.693E+03	1.291E-01	2.339E+01	0.000E+00	2.693E+03	1.857E+04	1.027E+03

A7 DATA INPUT FORMS

Data file name:	
Result file name:	

Problem title:	
Force units:	Length units:

Choose one of the following:

	1. One top load only
	Top load on the pile:
	2. Load-settlement curve
	Number of points on load-settlement curve:

Choose one of the following:

	1. Negative friction, no bitumen coating
	Number of increments on the pile:
	2. Negative friction, varying bitumen coating length
	Number of increments on the pile:
	Shear strength of bitumen:
	Maximum number of iterations to find the depth of bitumen coating:
	Tolerance for convergence of neutral point:
	3. Negative friction, constant bitumen coating length
	Number of increments on the pile:
	Shear strength of bitumen:
	Depth of bitumen coating:
	4. Positive friction, no bitumen coating
	Number of increments on the pile:
	5. Positive friction, constant bitumen coating length
	Number of increments on the pile:
	Shear strength of bitumen:
	Depth of bitumen coating:

Multiplier for negative friction:	
Multiplier for positive friction:	

PILE DATA

Cross sectional area:
Area of pile point:
Perimeter:
Embedded length:
Modulus:

MAXIMUM SHAFT RESISTANCE DATA

No.	Depth	Maximum Shaft Resist.	No.	Depth	Maximum Shaft Resist.	No.	Depth	Maximum Shaft Resist.
1			16			31		
2			17			32		
3			18			33		
4			19			34		
5			20			35		
6			21			36		
7			22			37		
8			23			38		
9			24			39		
10			25			40		
11			26			41		
12			27			42		
13			28			43		
14			29			44		
15			30			45		

SOIL SETTLEMENT DATA

No.	Depth	Soil Settlement	No.	Depth	Soil Settlement	No.	Depth	Soil Settlement
1			11			21		
2			12			22		
3			13			23		
4			14			24		
5			15			25		
6			16			26		
7			17			27		
8			18			28		
9			19			29		
10			20			30		

BEARING SOIL DATA

Young's modulus of bearing soil:
Poisson's ratio of bearing soil:
Ultimate bearing capacity of bearing soil:

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Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board

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2101 Constitution Avenue, N.W.
Washington, D.C. 20418

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